II C: STRESS SYSTEM: Experimental Techniques & Results (Cohesive Soils)

CCL 4/9/01 Sorry that I did not have time to rewrite these notes.

STRESS SYSTEM: Experimental Techniques & Results

(For saturated clays; granular soils later)

1. INTRODUCTION

1.1 Definition

Stress system = Direction of σ_1 wrt vertical (δ angle)
→ anisotropic behavior

\+ Effect of σ_2 à la $b = \dfrac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3}$

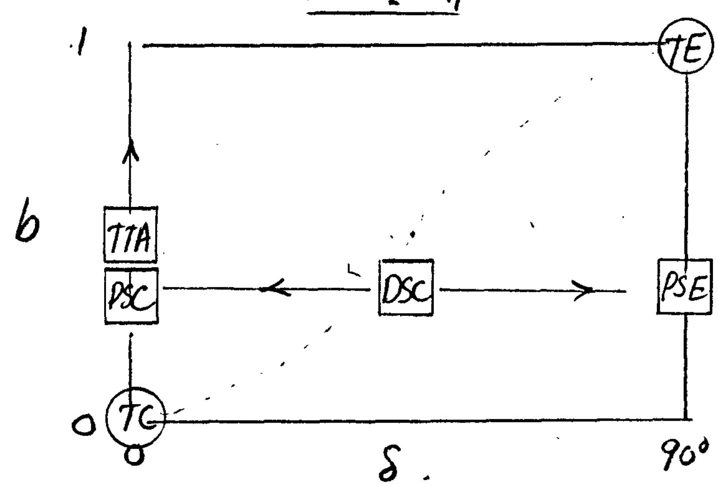
1.2 Objectives

- How and why does SS affect behavior?
- How to measure experimentally – in situ – lab
- Magnitude of effects
  - When δ & b important?
  - Effect soil type & OCR
- s_u = function 1) Initial $\bar{\sigma}$ ($\bar{\sigma}_{vc}$, K_c)
  2) $\Delta\bar{\sigma}$ (Δq, A_f)
  3) Envelope ($\bar{c}$, $\bar{\phi}$)

1.3 Overview of Experimental Capabilities

Doesn't include Cavity Expansion = SBPT ($\sigma_2 = \sigma_v$)

For $\sigma_2 = \sigma_h$



à la JTG (1982)

Note: other test devices to be added

2. TYPES OF ANISOTROPY (Tokyo 2.2.2, S.F. 2.4, 1.605 Chap.5) +TL 4 & 5

2.1 Initial Anisotropy of Clay with 1-D History

(Deposition & straining)

5 Elastic Parameters: E_v, E_h, μ_{vh}, μ_{hh}, G_{vh}

(1) Inherent (due to depositional & consolidation history)

- {Transversely Isotropic / Cross-anisotropy} → varying, $\bar{c}$, $\bar{\phi}$, A, G, etc.

a) "Structural" due to preferred "soil structure" (fabric + forces) } "micro-~~fabric~~ level"

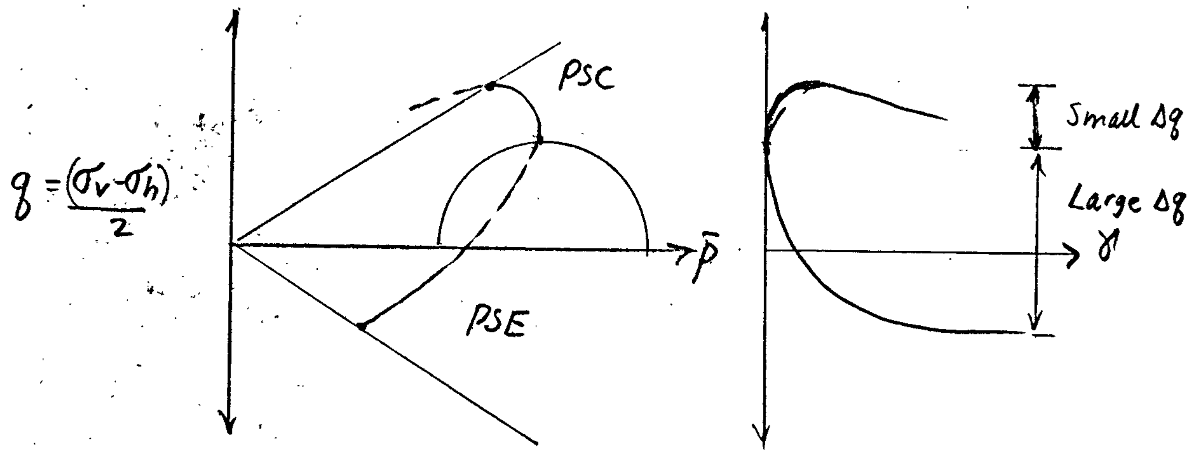
b) "Material" } "macro-~~fabric~~ level"

e.g. varved clay

fissures, bedding planes

(2) Initial Shear Stress (whenever $K_0 \neq 1$)

- Hansen & Gibson (1949) (Tokyo p 437)
- CK_0UPS C/E



$$\frac{q_f(C)}{\bar{\sigma}_{vc}} = \frac{[K_c + (1-K_c)A_f] \sin\bar{\phi}}{1 + (2A_f - 1)\sin\bar{\phi}}, \quad A = \frac{\Delta u - \Delta\sigma_h}{\Delta\sigma_v - \Delta\sigma_h} = \Delta\sigma_3$$

$$\frac{q_f(E)}{\bar{\sigma}_{vc}} = \frac{[1 - (1-K_c)A_f] \sin\bar{\phi}}{1 + (2A_f - 1)\sin\bar{\phi}}, \quad A = \frac{\Delta u - \Delta\sigma_v}{\Delta\sigma_h - \Delta\sigma_v} = \Delta\sigma_3$$

A wrt applied stresses

- Can produce su anisotropy w/o any inherent anisotropy (i.e. for same K_c, A_f & $\sin\bar{\phi}$)

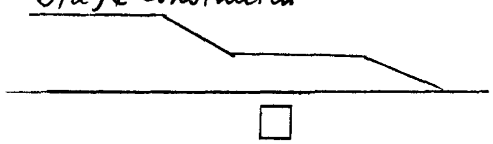
(3) Combined = Inherent + $K_0 \neq 1$

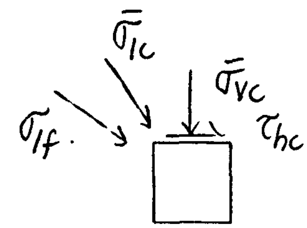
2.2 Other Types of Anisotropy

1) Prestraining isotropic soil → subsequent anisotropic behavior à la Arthur et al tests on sand (INDUCED)

2) Evolving (TL Fig. 12)

Stage Construction





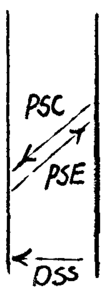
∴ Δ shape of yield surface (Treated in Section 7.4)

3. USE OF UU TYPE TESTS TO MEASURE ANISOTROPY

3.1 In Situ

1) FV with varying shapes (Tokyo 4.2.4)

- Disturbance + Progressive failure + Unknown stresses → unreliable results

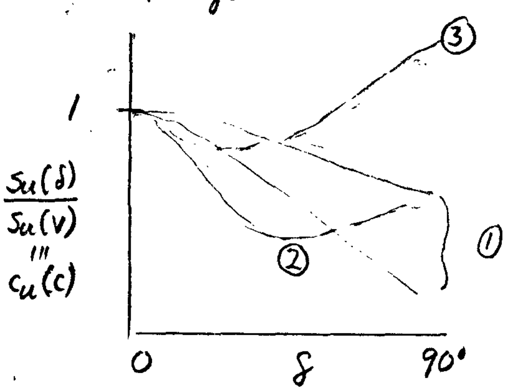


2) NGI special in situ DS device (Table 11.2 of CCL, 1971)

(Manglerud Quick clay $I_p = 8\%$ $S_t \geq 100$) $c_u/\bar{\sigma}_{v0} = 0.31$ C, 0.12 E

3.2 Lab UUC Cut at Varying δ

Tokyo F21



① Homogeneous sedimentary; inc. St → more effect

② Varved clay, Su(DSS) min.

③ Stiff fissured

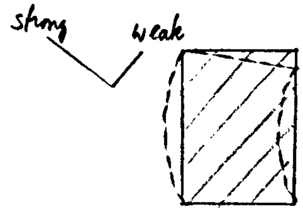
Problems with UUC(δ)

1) Neglects initial stress shear component ($K_c = 1$ vs K_0)

2) Sample disturbance ... (E) (C) $K_s = S_u(H)/S_u(V)$

	UUC(δ)	CK_0UPS
Portsmouth	0.75	0.44
BBC	0.8	0.56
CVVC	0.6 (separation of vane)	0.9

3) Bending & shear at ends à la Saada et al (1970, 1977)



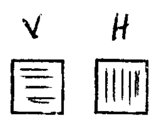
Conclusion: Need CK_0U Type testing

4. TEST VARIABLES FOR CU TESTING

4.1 Stress level $\bar{\sigma}_{vc}$ vs $\bar{\sigma}_{vo}$ & $\bar{\sigma}_{vm} \equiv \bar{\sigma}_p$ SHANSEP vs RECOMP

4.2 K_c + stress path → K_c (Covered Part IIB)

4.3 Sample orientation



4.4 σ_{1f} direction = δ angle

4.5 σ_2 magn. = b value

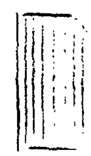
(Note: Really need to specify σ_2 direction, e.g. PSE vs SBPT)
SBPT = Cavity Expansion

5. EXPERIMENTAL CAPABILITIES

Tokyo 4.1.1
SF 2.4.3
1.605 Chap 5

5.1 Triaxial

- $CK_0UC/E \rightarrow \delta = 0/90°$ but $b = 0 \rightarrow 1$
- Use of TC/TE on "horizontal" sample



  - On b vs δ plot
  - Problems: Wrong σ_{hc}; " σ_{hf}

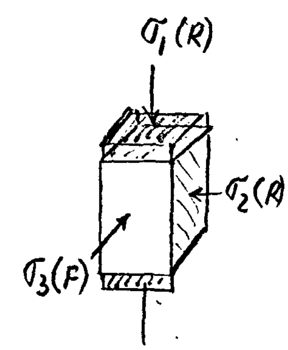
5.2 Plane Strain   Campanella & Vaid (1974)

- $PSC/E \rightarrow \delta = 0, 90°$ with "constant" b
- Correct but limited capability

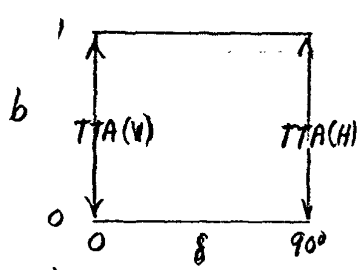
5.3 True Triaxial Apparatus (TTA)

1) Boundary conditions

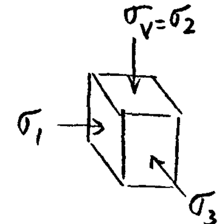
Cube {
- Flexible (Rubber Bags) Scott UCL (MIT)
- Rigid - Cambridge Univ.
- Mixed Lade (UCLA)



2) What can do in b-δ plot



+ Cavity Expansion (SBPT)

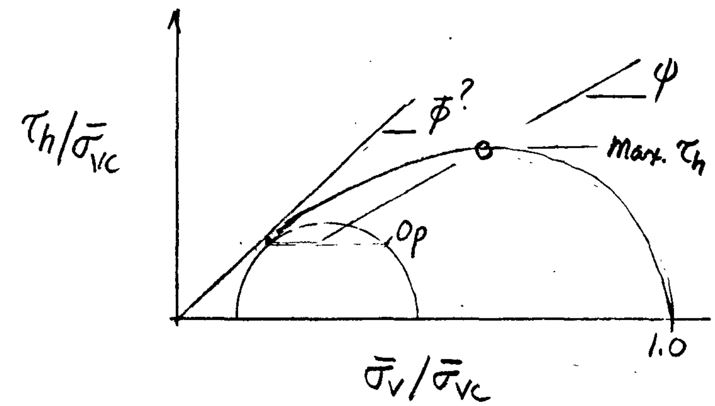


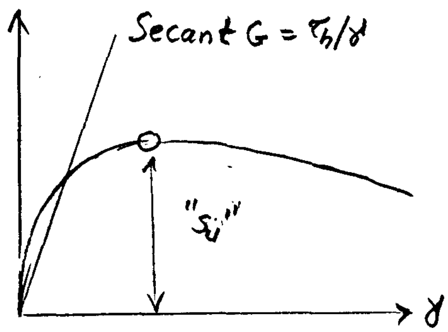
3) Conclusion → Mainly useful for studying b

NOTE: Very little CK_0U data available from TTA

5.4 Direct Simple Shear (Geonor) = DSS

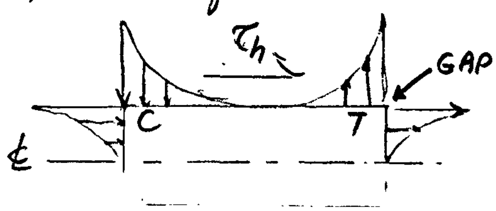
(1) "Std" Test on OCR=1 Clays. (Vary $\bar{\sigma}_v \rightarrow \Delta H = \Delta V = 0$)





(2) Problems

a) Non-uniform stresses



Vucetic & Lacasse (1982) JGE No.

Ladd & Edgers (1972)

Saada et al (1981) + NGI rebuttal

"worst than DS"

Elastic vs plastic

De Groot et al. (1994) p6a

Tests on rubber

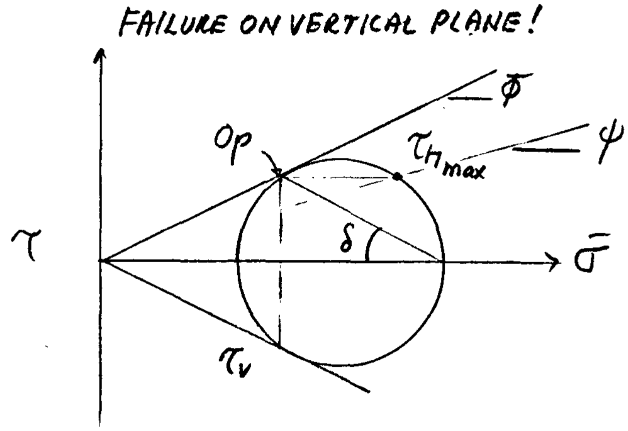
b) Indeterminate state of stress

- δ = ?
- N.G. $\bar{c}, \bar{\phi}, A$ ..
- CCL opinion $G = E_u/3$

$\tau_{ff} \le \tau_h \le q_f$, $\delta = 40 \pm 10°$

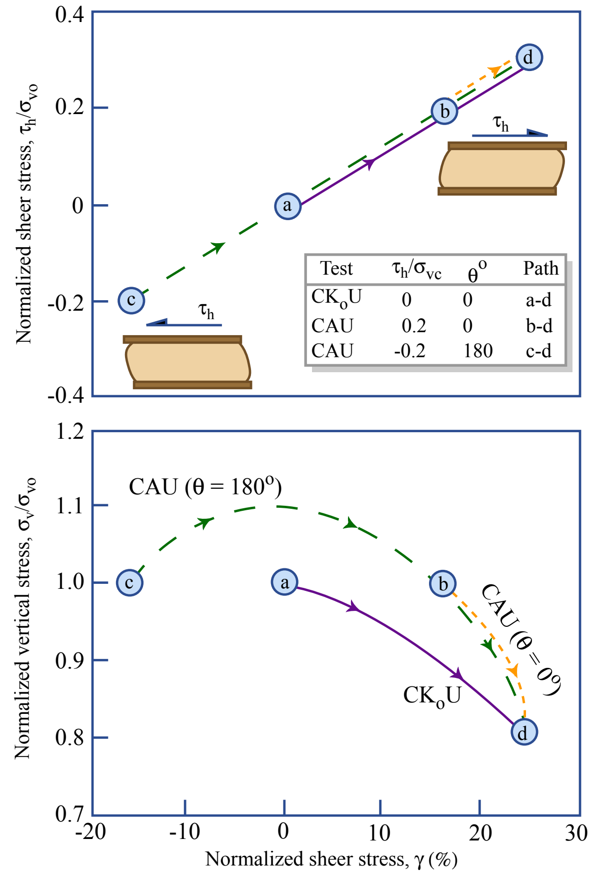
c) Randolph & Wroth (1981) interpretation

FAILURE ON VERTICAL PLANE!



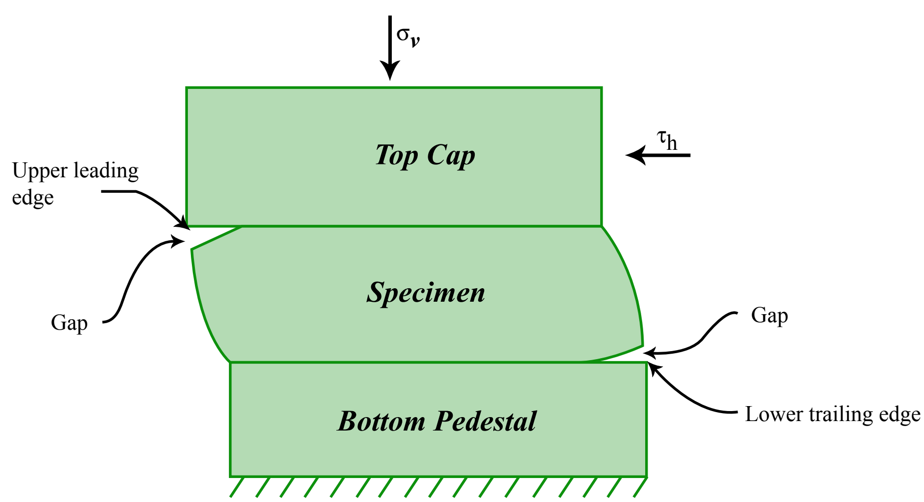
$$\tan \psi = \frac{\sin \bar{\phi} \cos \bar{\phi}}{(1 + \sin^2 \bar{\phi})}$$

τ_v for pile capacity



Summary of effect of shear strain on variation of normalized vertical stress for DSS tests on rubber (Idealization of data presented in Figs - 6 & 7)

Figure by MIT OCW.



Schematic of deformed shape of rubber specimen under constant height direct simple sheer conditions (γ > 10%)

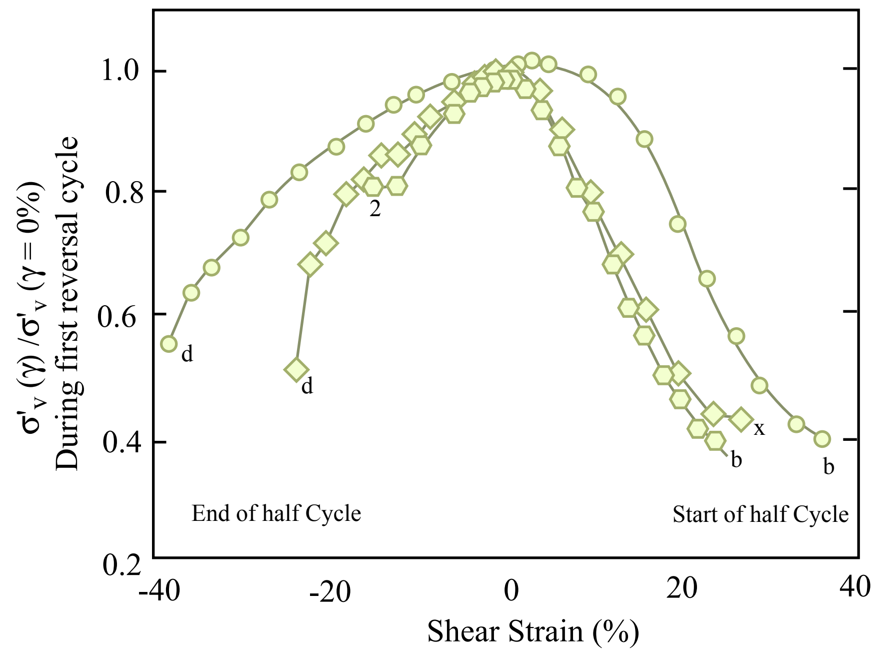
Figure by MIT OCW.

Adapted from:

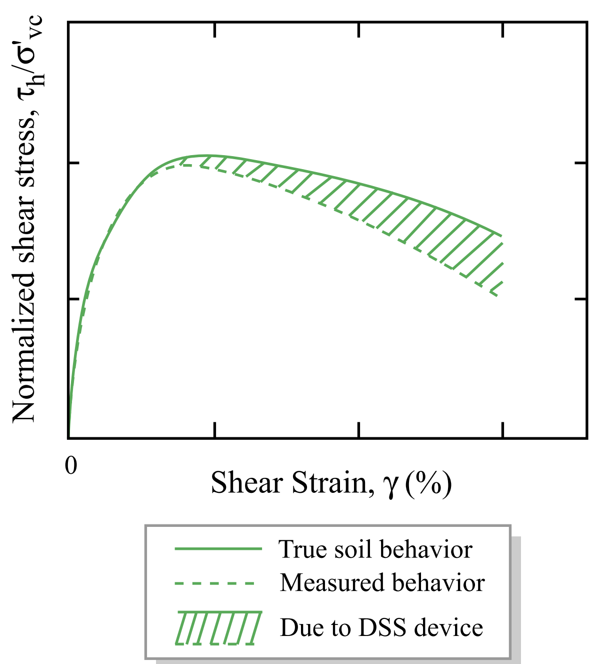
1994 JGE, ASCE 120(5)

EFFECT OF NONUNIFORM STRESSES ON MEASURED DSS STRESS-STRAIN BEHAVIOR

By Don J. DeGroot,[1] Associate Member, ASCE, John T. Germaine,[2] Member, ASCE, and Charles C. Ladd,[3] Fellow, ASCE

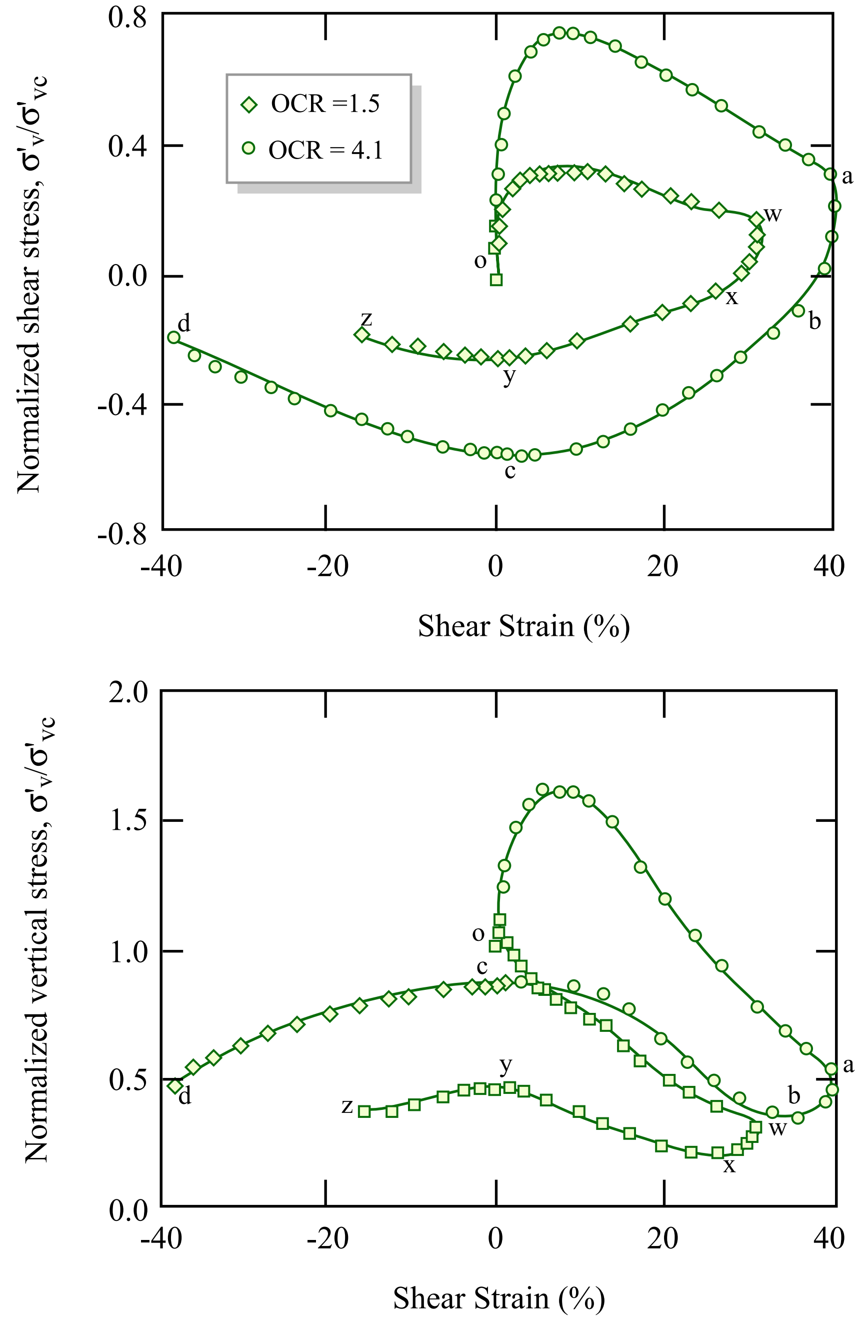


Vertical stress during first reversal stage (Normalized by σ'_v at $\gamma = 0\%$) versus shear for undrained cyclic geonor CK_oUDSS test on BBC and SFBM



Schematic of hypothesis showing influence of DSS apparatus on behavior of OCR = 1 specimen in CK_oUDSS test.

Figure by MIT OCW.

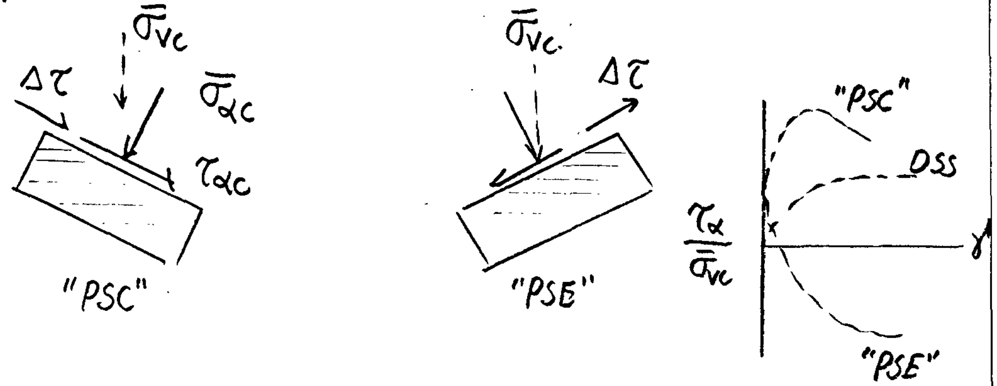


Normalized results of cyclic Geonor CKoUDSS test on SFBM with σ'$_{max}$ = 501 k Pa: (a) Shear stress-strain curve; and (b) Vertical effective stress versus shear strain

Figure by MIT OCW.

(3) DSS-1 p7a ψ vs $s_u(DSS)/\bar{\sigma}_{vc}$

Soydemir (1976) (4) Special DSS on inclined samples - Add field case
Bjerrum Memorial Vol.



(5) Geonor vs Marshall Silva Device
↳ higher s_u 15±5%

(6) Cambridge SSA

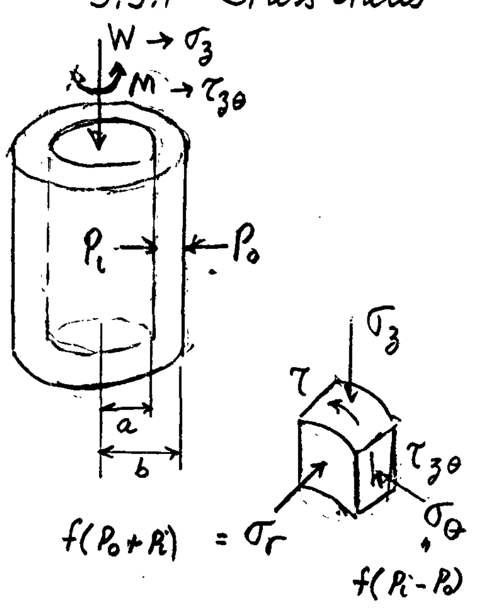
(7) CCL opinion of DSS (SHANSEP testing)
- Reasonable s_u for stability analyses & easier/cheaper CK_0UC/E
- Reasonable E_u & hyperbolic parameters for FEECON
- Excessive strain softening at large strains, p6b

DSS-2 p7b

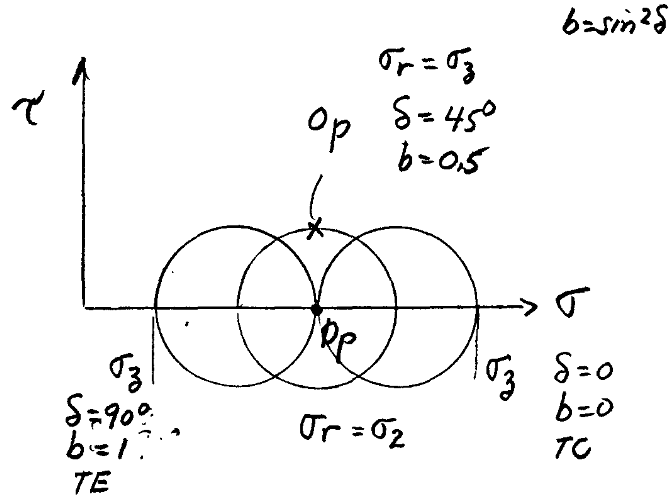
5.5 Torsional Shear Hollow Cylinder (TSHC)

SF 2.4.3

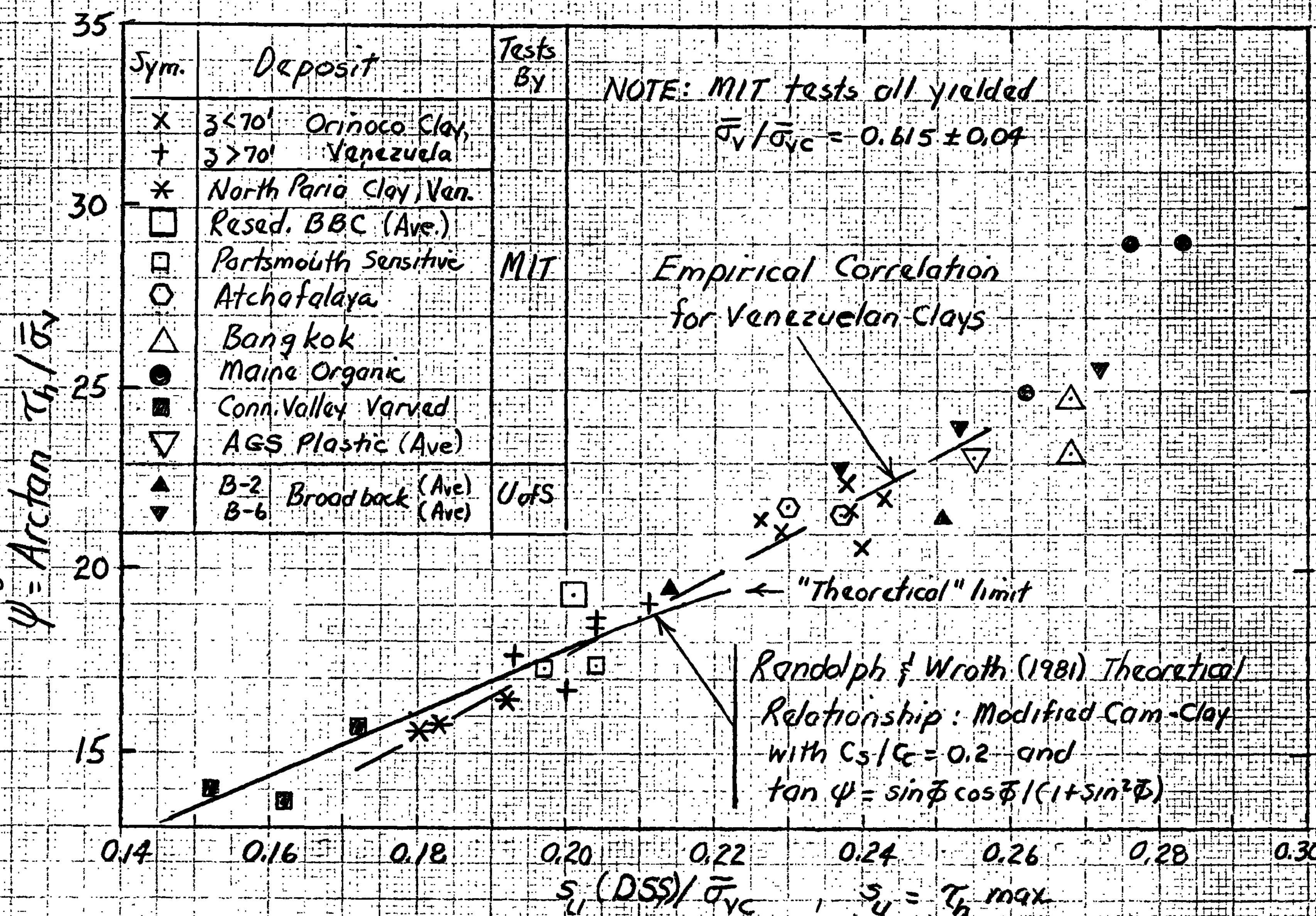
5.5.1 Stress States



(a) Saada et al $P_i = P_o = \sigma_r$ → $b = \sin^2\delta$



CCL 4/22/82
1.322



CK₀UDSS Test Results on Normally Consolidated Clays

IIC

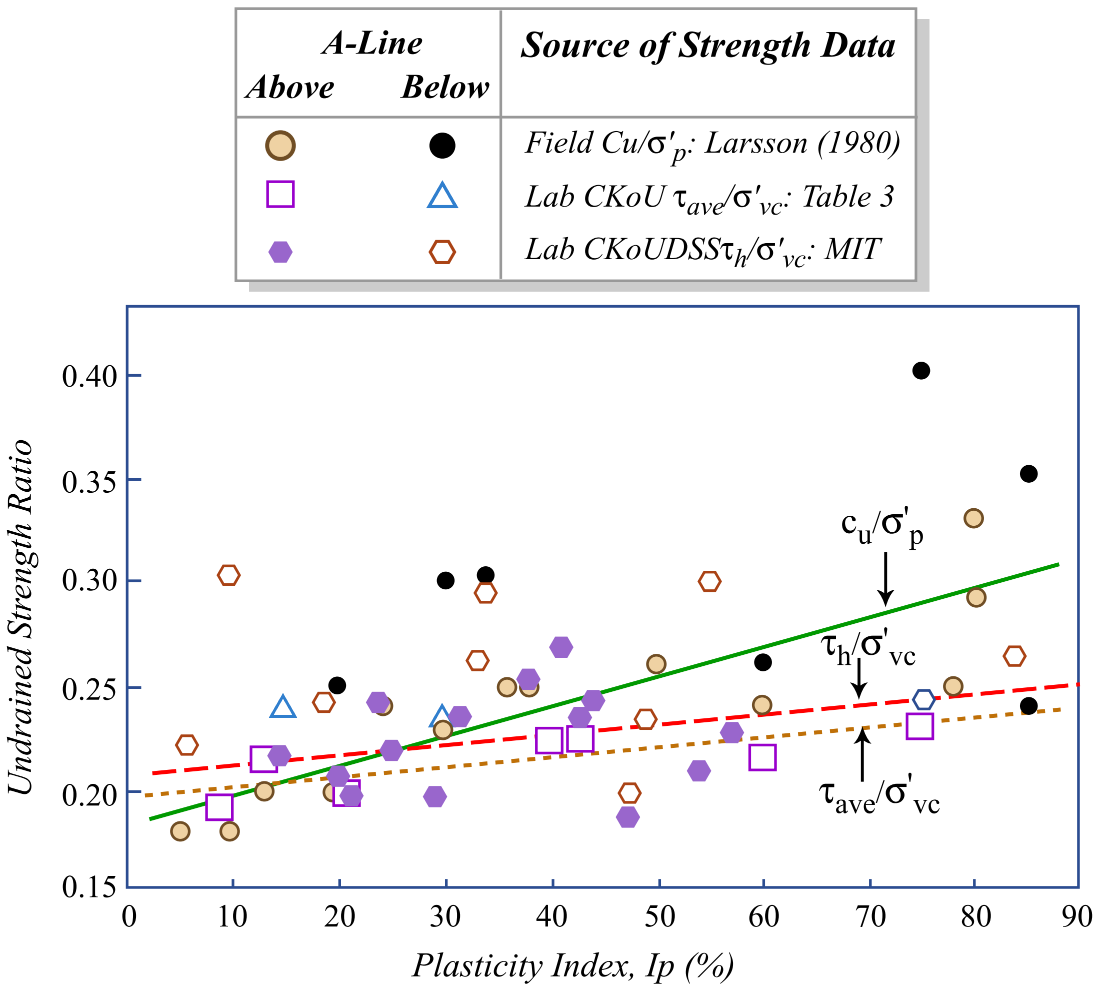
DSS-1

p7a

CCL 4/88 1.322

IIC

DSS-2
p76

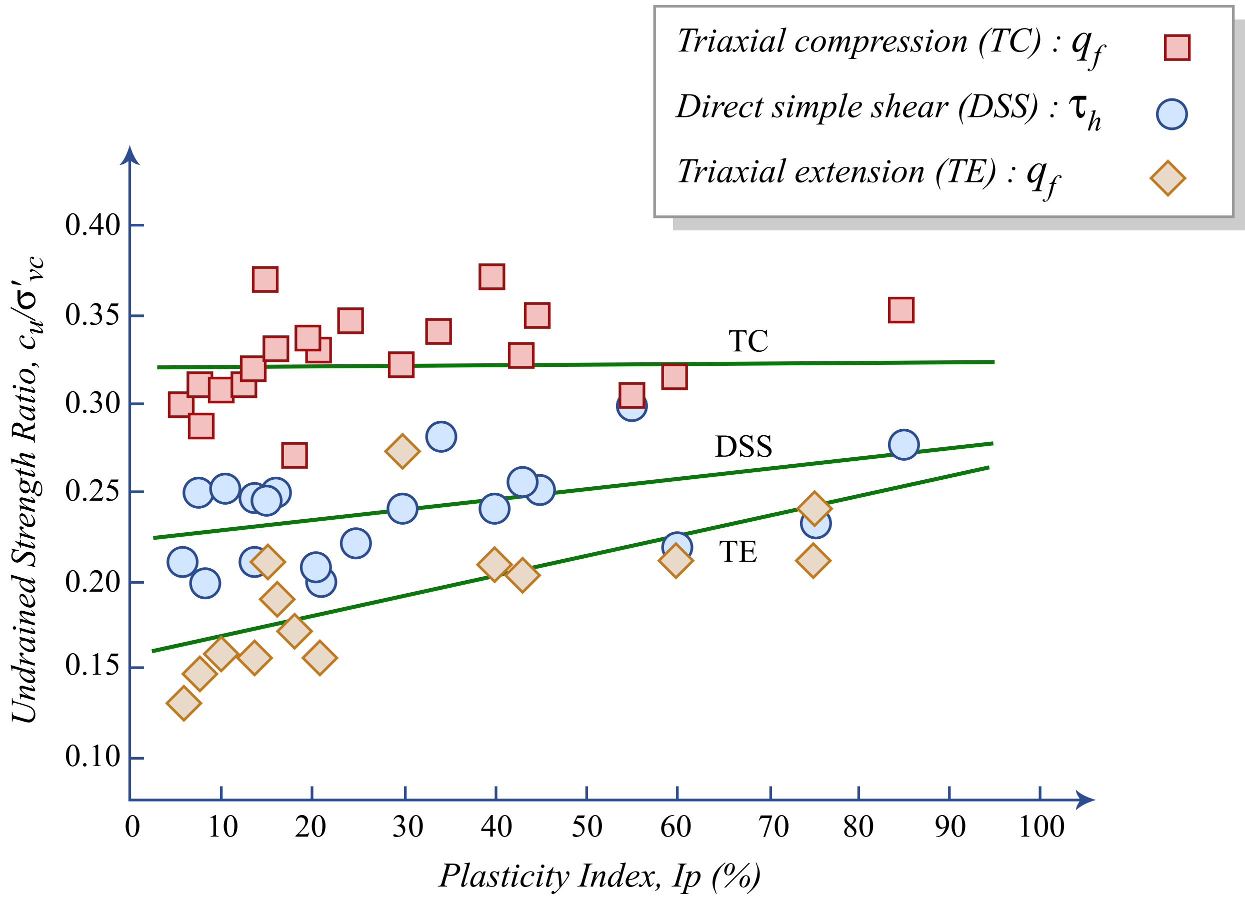


Comparison of field and laboratory undrained strength ratios for non-varved sedimentary soils (OCR = 1 laboratory CK_0U testing)

Note : Linear Regression lines for clay data

Figure by MIT OCW.

Cu 4/99 1,322 · IIC DSS-3 p7c



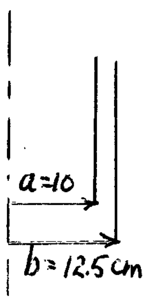
Undrained strength anosotropy from CK_0U tests on normally consolidated clays and silts.

Figure by MIT OCW.

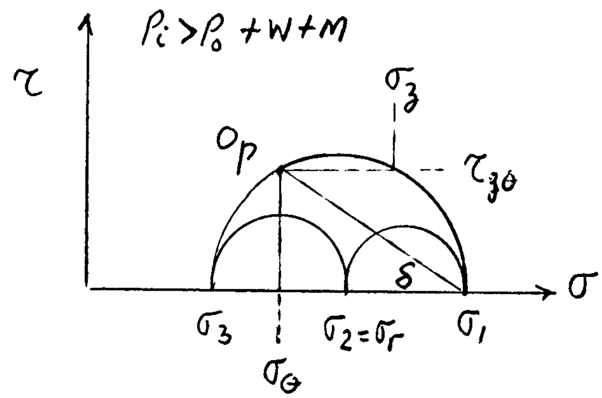
5.5.1 (a) Continued

- Where test plots on b vs δ ($b = \sin^2\delta$)
- Comments on SF Fig. 19 (p8a)
  - Variation in ϕ' :- Expected for $b = 0 \to 1$
  - " " c_u/σ'_c : Differs from normal trends
  - Scatter : alot

(b) Imperial College Hight et al (1983 Geot. #4)



- $H = 25cm$ $OD = 25cm$ $t = 2.5cm$ Measure strains in Central portion
- CU & CD tests on sat. sand
- Apparently limited to $P_o/P_i = 1.2 - 0.9$ (with $\delta \le 45°$)
- $P_i > P_o$ to left of $b = \sin^2\delta$ line
  $P_i < P_o$ " right " " "



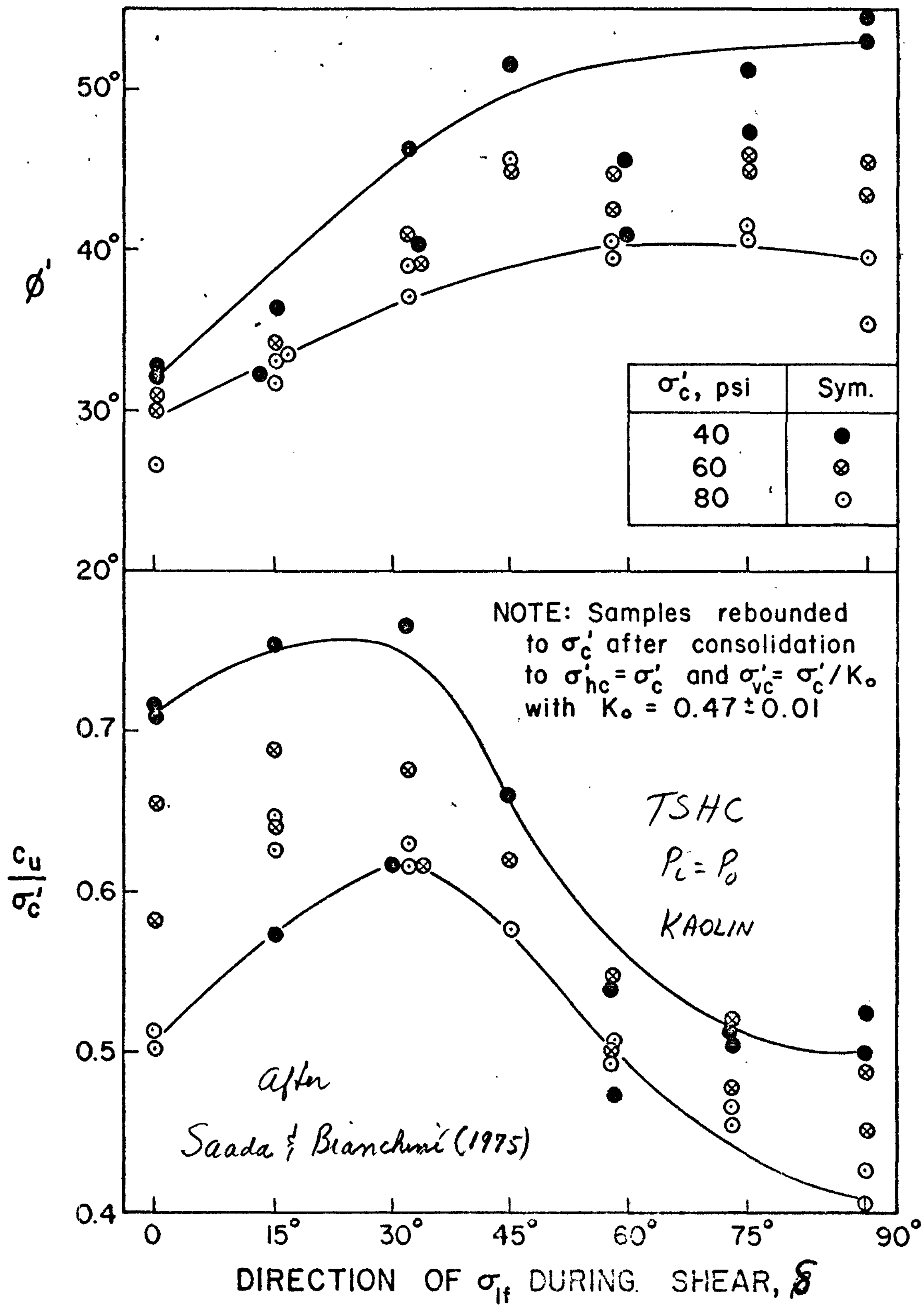
$$\sigma_\theta - \sigma_r = r\frac{d\sigma_r}{dr}$$

$$\sigma_r = \frac{(P_o b + P_i a)}{(b+a)}$$

$$\sigma_\theta = \frac{(P_o b - P_i a)}{(b-a)}$$

$$\tau = \frac{3M}{2\pi(b^3 - a^3)}$$

Advantages	Disadvantages
Most versatile of any device	Very complex & costly
Data from CIU tests on sand look excellent	Non-uniform stresses with $P_i \ne P_o$
(Fig. 20 SF - cover later under sand anisotropy)	→ End effects
	Problems w/ testing clays
	→ Need to measure strains internally

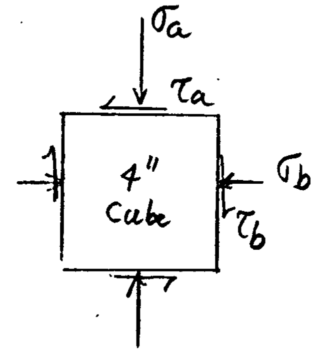


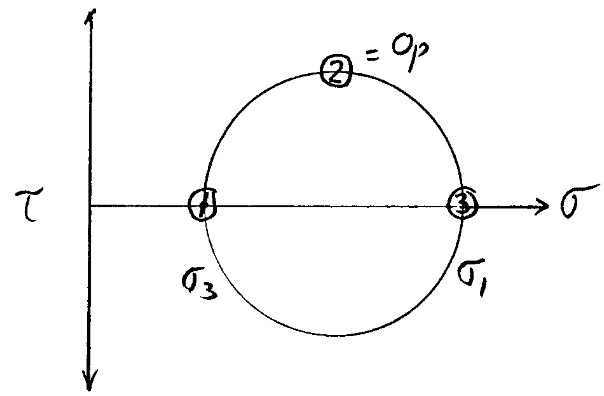
NOTE: Same basic material as Fig. 19 of S.F.

5.6 Directional Shear Cell (DSC) – Only plane strain

5.6.1 Principle (Developed by Arthur et al @ UCL)

Fig. 17 SF





- Pressure bags + shear sheets → any σ_1 angle

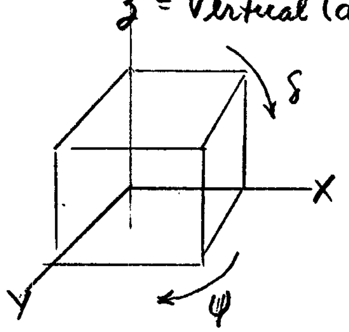
① $\sigma_a > \sigma_b$, $\tau = 0$

② $\sigma_a = \sigma_b$, $\tau \neq 0$

③ $\sigma_b > \sigma_a$, $\tau = 0$

5.6.2 Sample Orientation

z = Vertical (deposition)



(Can't plot on b–δ diagram)

a) Shear in x-y plane (no inherent: ψ)
  - Proof testing • SBPT = Cavity Expansion
  - Strain induced anisotropy

b) Shear in x-z plane (Inherent: δ)
  - Measure inherent + initial shear stress anisotropy
  - Where falls b–δ plot

5.6.3 Misc

- Radiography / photography → strain distribution* + $\Delta\sigma_1$ vs $\Delta\varepsilon_1$ directions
- ULC sand testing
- MIT clay testing (JTG '82 ScD) (T.H. Seah, '90 ScD)
- Limited to low stresses (τ < 50 kPa; MIT version)

* Optical Comparator → displacements ± 2μm

6. INFLUENCE OF K_c AND b

6.1 Influence of K_c

4/13/98 Replaced by 6.1-1 → 6.1-5 (after p10)

6.2 Influence of b

(1) General considerations of increasing b

- Effect on Δu
- " " $\bar{\phi}$

Mohr Coulomb →
MCC →

Matsuoka (1974)
$I_1 \cdot I_2 / I_3 = \text{constant}$

Lade & Duncan (1975)
$I_1^3 / I_3 = \text{constant}$

(2) CIU TTA N.C. Grundite Lade & Mucante (1977) (1978)

- Handout (p10e)
- As b increases 0→1

s_u : increasing & then decreasing
ε_f : decreasing & then constant
$\bar{\phi}$: increasing & then decreasing
A_f : constant & then increasing

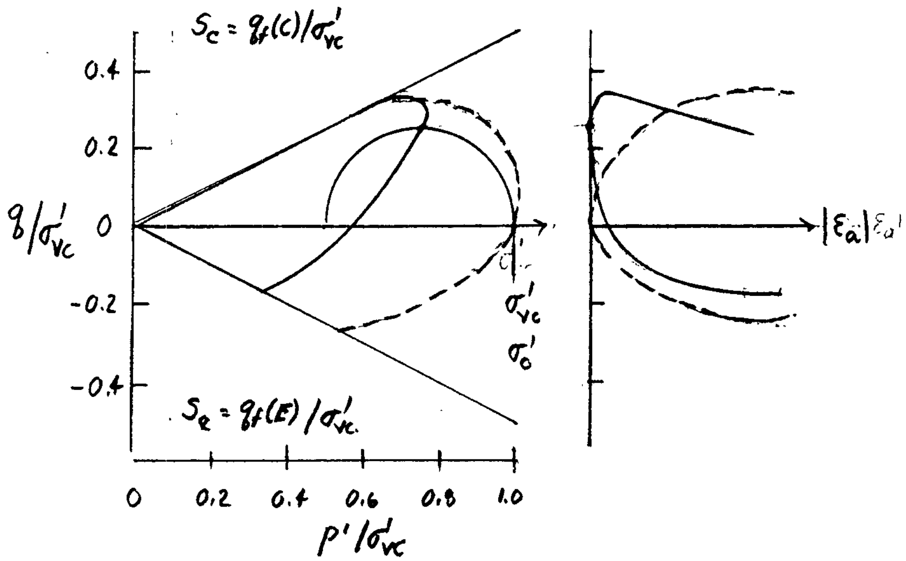
PS vs TC →
Inc. s_u
" $\bar{\phi}$
Dec. ε_f

6. INFLUENCE OF K_c AND b

$K_c = \sigma'_{hc}/\sigma'_{vc}$; $b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3}$

6.1 Influence of K_c (OCR=1)

6.1.1 CAU vs CIU : General Trends (Ladd 1965; Ladd & Varallyay 1965)



Going from CIU to CAU

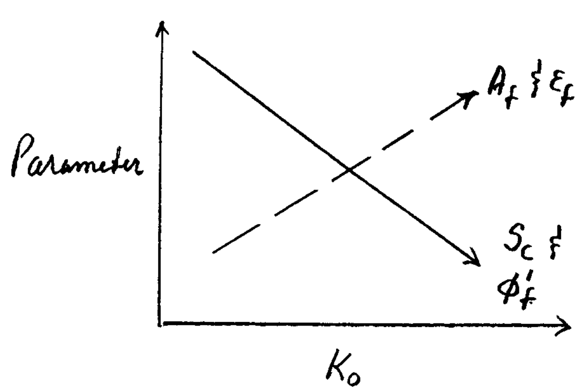
- Approx. same S_c ± 10-15%
- Large decr. in ε_f
- Often incr. in strain softening

- Always expect decrease in S_e since starting from lower p'_c/σ'_{vc}, plus larger Δq_f

6.1.2 Influence of K_0 on CK_0UC Behavior

1) Before ≈1990, I had expected little effect given trends in 6.1.1, plus $q_f(C)/\sigma'_{vc}$ vs I_p ≈ 0.33 ± 0.02 from CK_0UC testing (Fig. 15, CCL '91)
But NOT TRUE

2) Data on natural BBC (See Kc1 & Kc2 for actual data)



Increasing K_0 → decreasing S_c* due to:

- lower ϕ'_f
- higher A_f

Also increasing ε_f

* Rel K_0 = 0.51 → 0.61 leads S_c = 0.30 → 0.26
(+20%) (-13%)

6.1.2 Cont

3) Collective data (Sergio Covarrubias 1994) from NC CK_oUC Tests

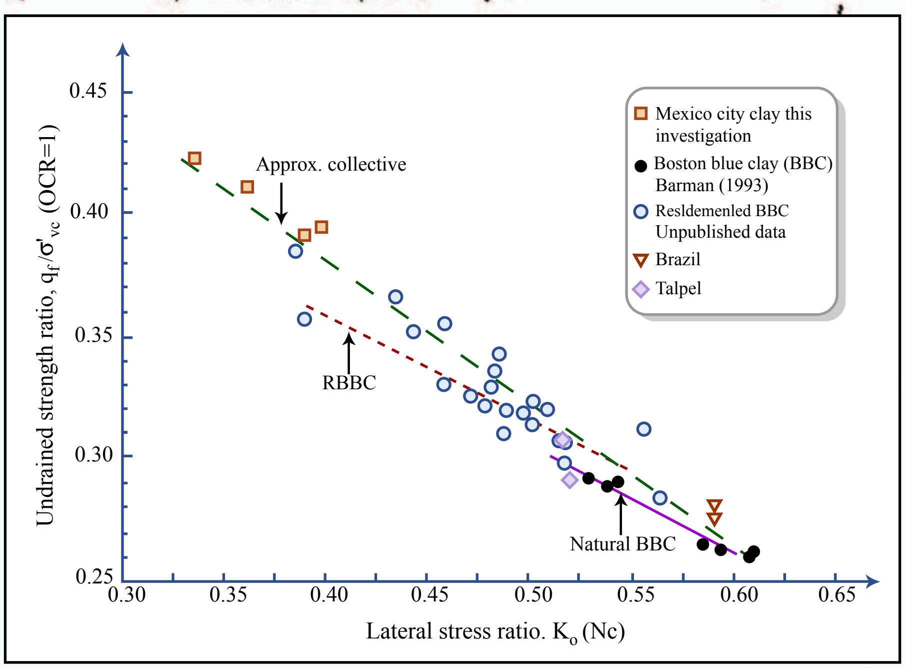


Figure by MIT OCW.

- Although collective data on wide range of soils → good correlation ($S_c \approx 0.62 - 0.60 K_o$), individual clays have different trends
- MCC is very plastic, but with high diatom content → high ϕ' → low K_o → high S_c

6.1.3 Influence of K_o on CK_oUE Behavior

1) Should expect increasing K_o → increase in S_e since:
   - starting from higher p'_c/σ'_{vc}
   - Smaller Δq_f [i.e., $q_o - q_f(E)$]

2) Only available data (below) supports this expectation

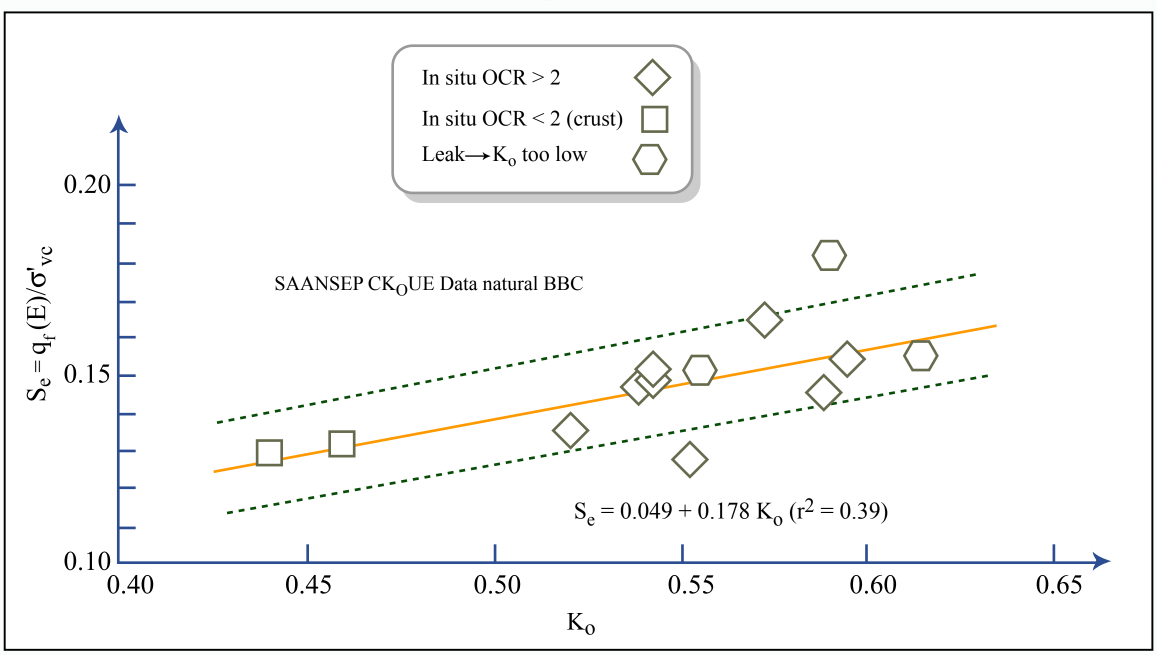


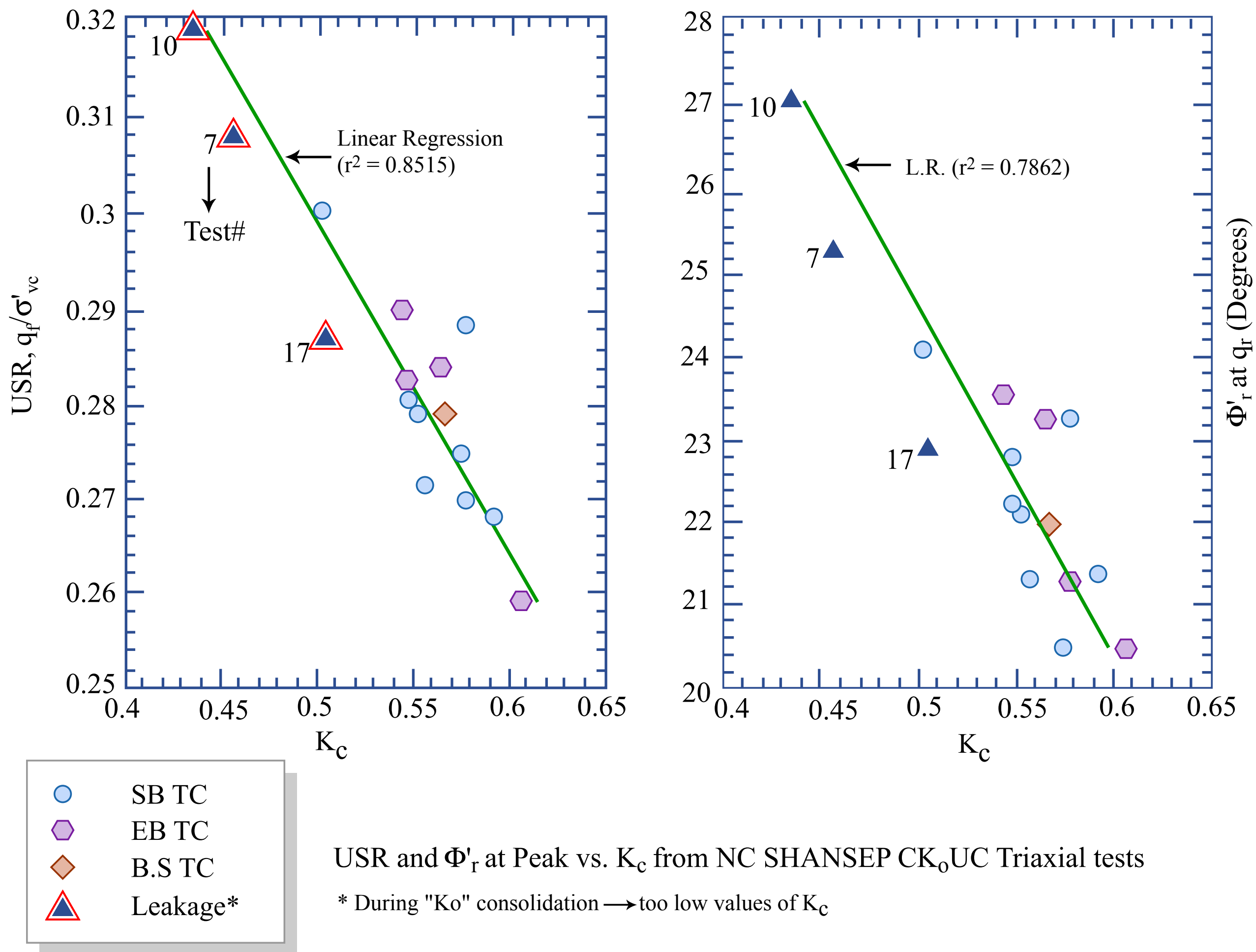
Figure by MIT OCW.

6.1.4 Conclusions of Influence of K_o on CK_oU Behavior

1) For TC, increasing $K_o \rightarrow$ significant reduction in $S_C = q_f(C)/\sigma'_{vc}$ due to lower ϕ'_f and higher A_f. Was not expected, but abt data

2) For TE, increasing $K_o \rightarrow$ significant increase in $S_E = q_f(E)/\sigma'_{vc}$. To be expected, but limited data to support.

3) Therefore using K_c = in situ K_o for Recompression CK_oU tests may be important for reliable values of q_f/σ'_{vc}



USR and Φ'_r at Peak vs. K_c from NC SHANSEP CK_0UC Triaxial tests

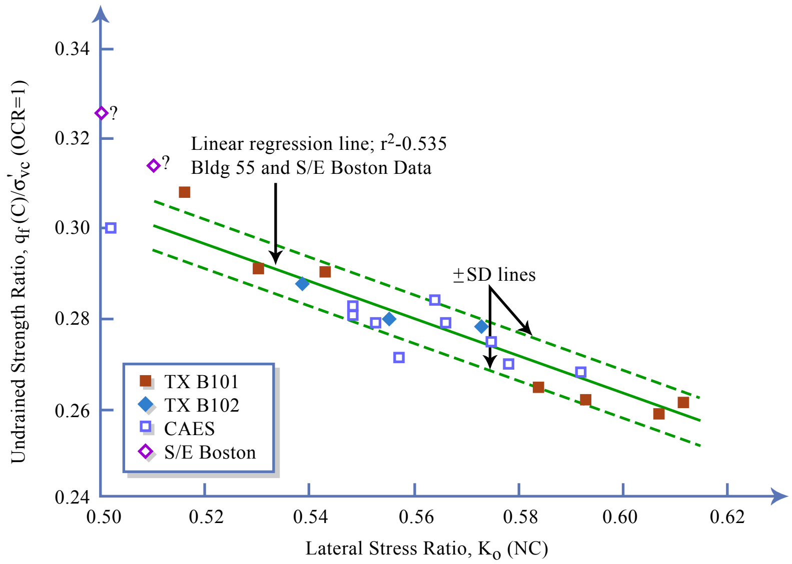
* During "Ko" consolidation ⟶ too low values of K_c

Figure by MIT OCW.

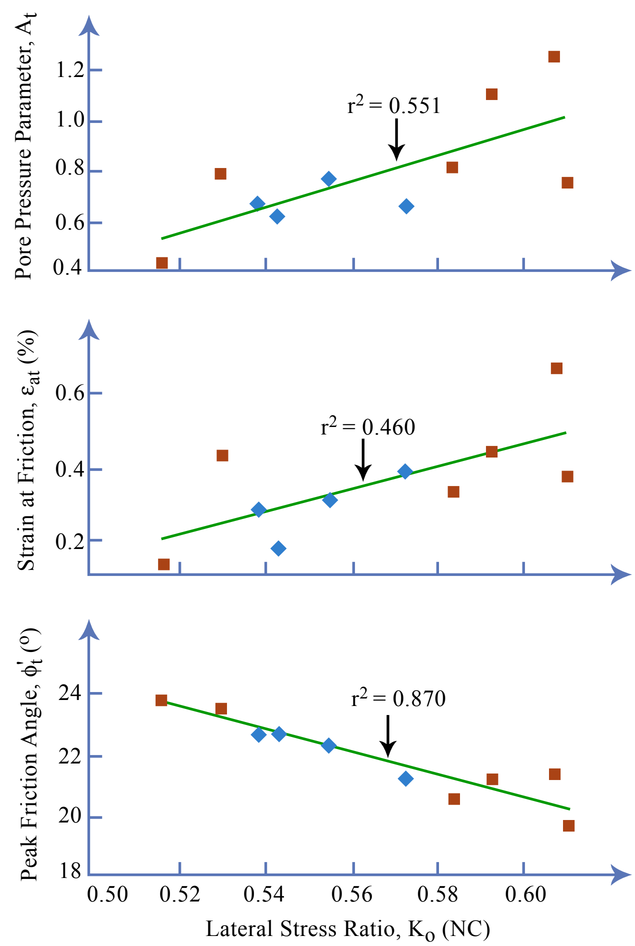
Adapted From de La Beaumelle (1991) SM thesis : HSA STP CA/T Project

- 1st CK_0U data from MIT's automated triaxial system developed for CA/T STP on natural BBC
- One of TX cells had a small leak (→ increased "measured" $\Delta\varepsilon_{vol}$) → reduced σ'_{hc} → values of K_0 that were too low. (tests 7, 10 & 17 above)
- But leakage rate too small to affect undrained shearing
- $S_c = 0.475 - 0.350 K_0$ ($r^2 = 0.85$) where $S_c = q_f(C)/\sigma'_{vc}$

Kc1



NC Undrained Strength Ratio vs. Lateral Stress Ratio for SHANSEP CK_0 UC Triaxial Tests: Collective Data



Lateral Stress Ratio vs. Pore Pressure Parameter, Strain at Failure and Peak Friction Angle: Building 68

Figures by MIT OCW.

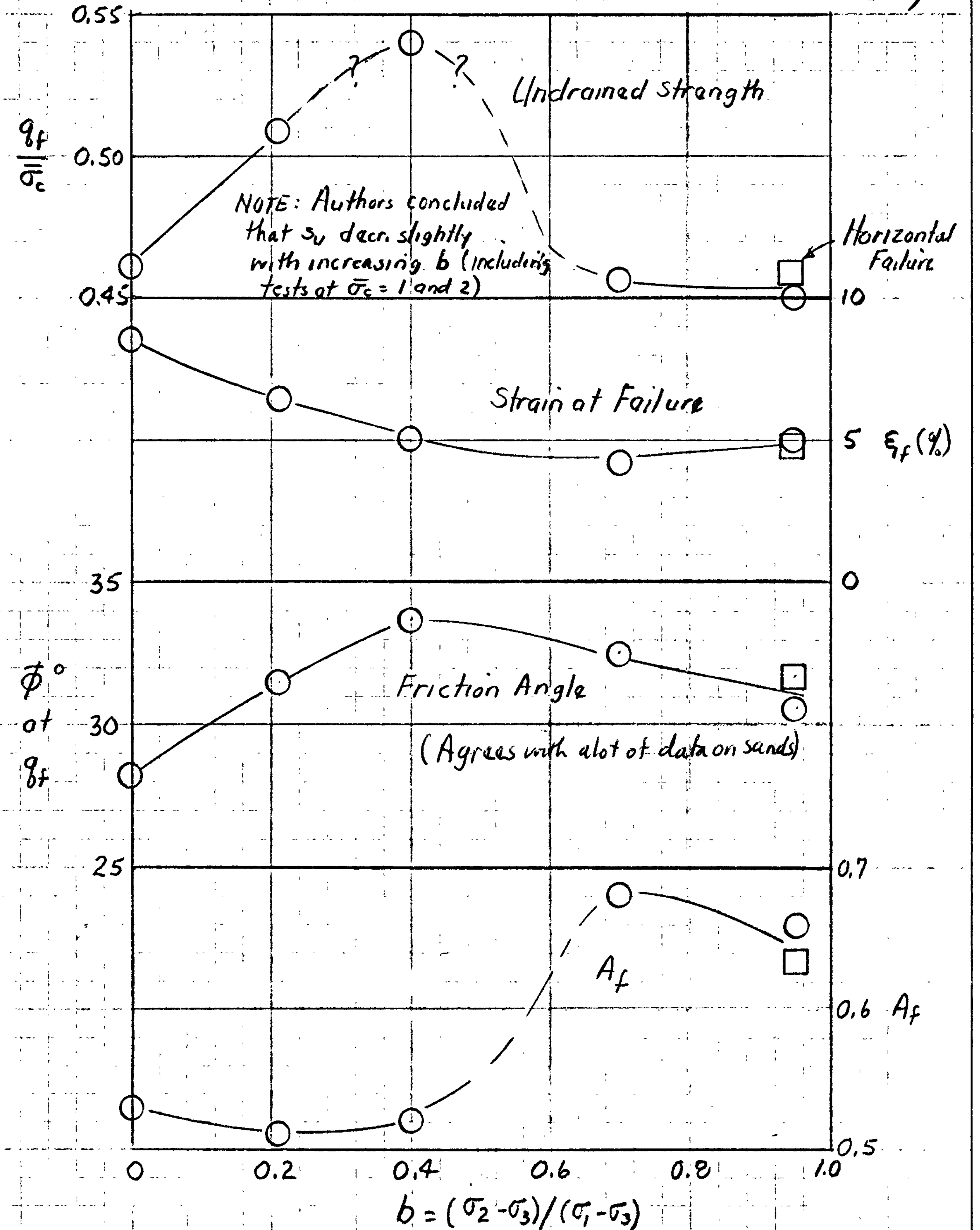
CK_0UC Data on Natural BBC (Berman 1993)

- TX B101 & B102 = MIT Bldg 68 (Biology)
- S/E Boston = CAF STP by MIT: (excluding 3 tests with data)

K_0 2

$\overline{CIU}$ True Triaxial on Remolded Grundite (w_L=54.%, PI=31.1%)

$\overline{\sigma}_c$ = 1.5 kg/cm² (Data from Lade & Musante, 1978 JGED GT2)



(3) Comparison PS vs Triaxial CK_0U Data (Table 1 Tokyo) p439

a) PSC vs TC 10 clays mostly NC

- q_f + 8±5%
- $\bar{\phi}_u$ + 2±2°
- Maybe increased strain softening

NOTE: TC $\gamma = 1.5\varepsilon$
PS $\gamma = 2\varepsilon$

b) PSE vs TE 4 NC clays

- q_f = +20-25%

Conclusion: TX → conservative s_u for PS problems, but need more data.

7. INFLUENCE OF ROTATION OF PRINCIPAL STRESSES

(CK_0U on low OCR clays mostly)

7.1 General Expectations ($K_0 < 1$)

With increasing δ

- Increasing Δq → incr. Δu & hence reduced $\bar{p}_f$ à la Hansen & Gibson } Effect of initial shear stress, τ_0
- Inherent anisotropy :- structure more resistant in vertical direction

7.2 Available Test Data

1) DSC BBC @ OCR = 4 & 1

2) PSC/TC $\delta = 0$
DSS $\delta = ?$
PSE/TE $\delta = 90°$

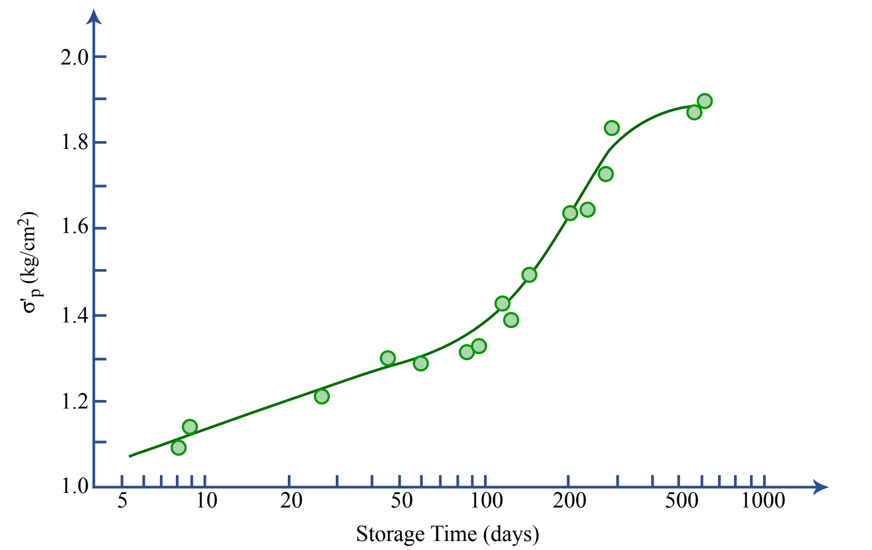
} → $K_s = s_u(H)/s_u(V) = s_u(E)/s_u(C)$

- Problem w/ TE/TE is?

7.3 Results from DSC Tests on RBBC

7.3.1 Data at "OCR=4" (Germaine 1982, O'Neill 1985)

1) Clay was thixotropic; therefore normalize to σ'_p

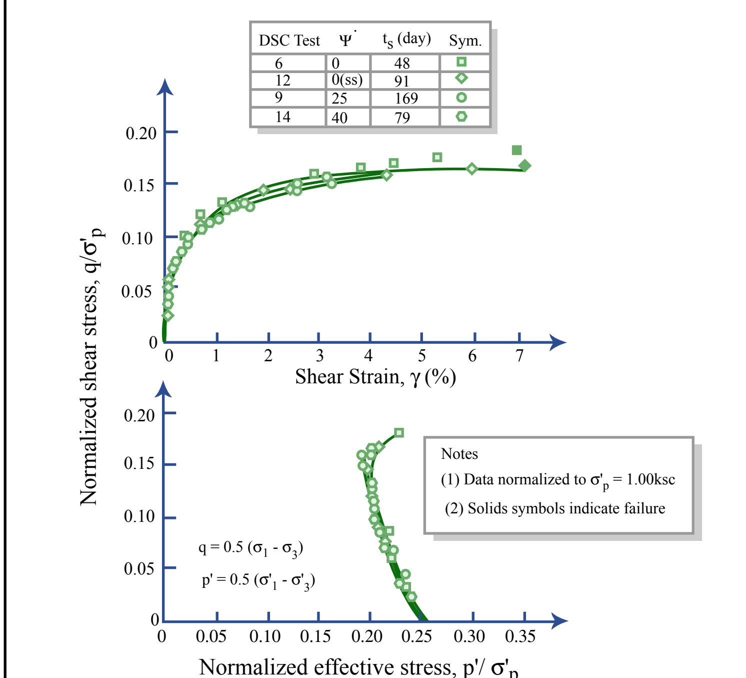


Effect of thixotropy on preconsolidation pressure of resedimented BBC (O' Neill, 1985)

Figure by MIT OCW.

Batch $\sigma'_{vm} = 1$ ksc, plus one cycle secondary compression → $\sigma'_p \approx 1.1$ ksc

2) Proof Testing, i.e., do pressure bags and shear sheets → same results?



Normalized stress-strain and effective stress paths from CU DSC proof tests on BBC at OCR =4 (Corrected data from O'Neill, 1985).

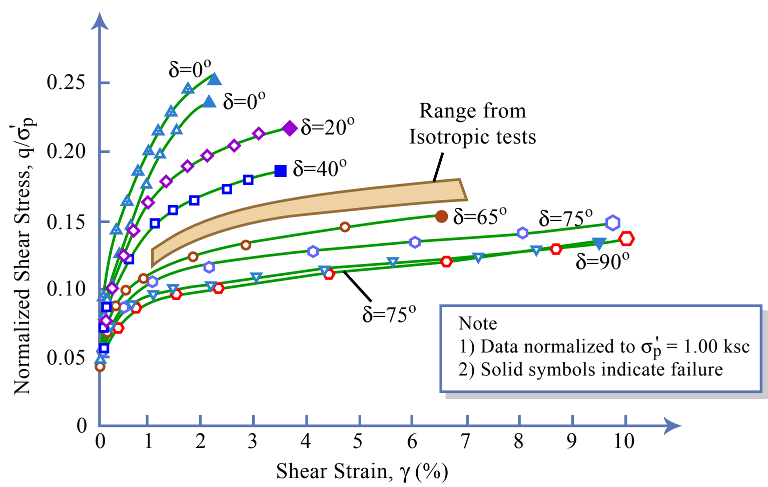
Figure by MIT OCW.

Results for shearing in X-Y plane

- $\psi = 0$, only $\sigma_a > \sigma_b$
- $\psi > 0$, $\sigma_a > \sigma_b$ + $\tau_a = -\tau_b$
- $\psi = 40°$, almost only $\tau_a = -\tau_b$

($\psi = 45°$ → only $\tau_a = -\tau_b$)

3) Effect of δ: Since $K_0 = 1$, all inherent anisotropy

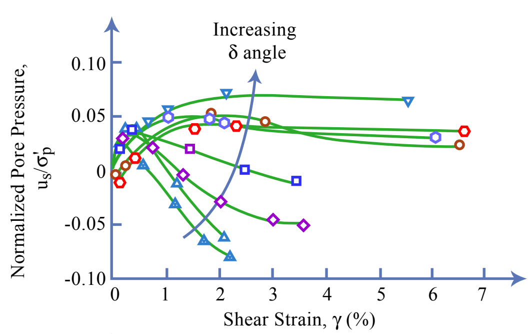


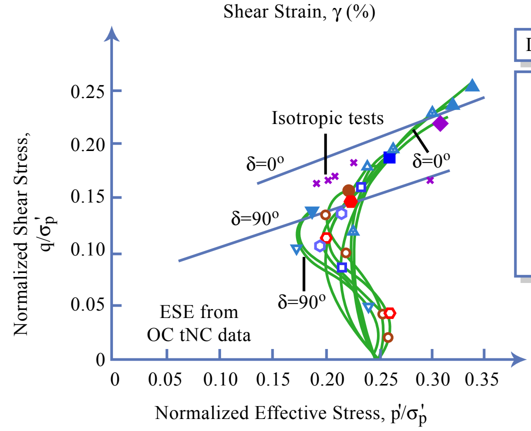
DSC Test	t_s(days)	δ^o	Symbol
1	29	0	△
3	60	0(SS)	△
15	92	20	◇
13	39	40	□
8	157	65	○
11	71	75	⬡
16	105	75	⬡
2	44	90	▽

Normalized Shear Stress vs. Strain from CU DSC Anisotropic Tests on BBC at OCR = 4.

Increasing $\delta \rightarrow$

- Decreasing q_y = yield stress
- Decreasing $q_f = s_u$
- Increasing γ_f
- Change in shape of q-γ curves
  - low δ perfectly + strain softening after peak
  - High $\delta \rightarrow$ strain hardening after initial yielding





DSC Test	δ^o	Symbol
1	0	△
3	0(SS)	△
15	20	◇
13	40	□
8	65	○
11	75	⬡
16	75	⬡
2	90	▽

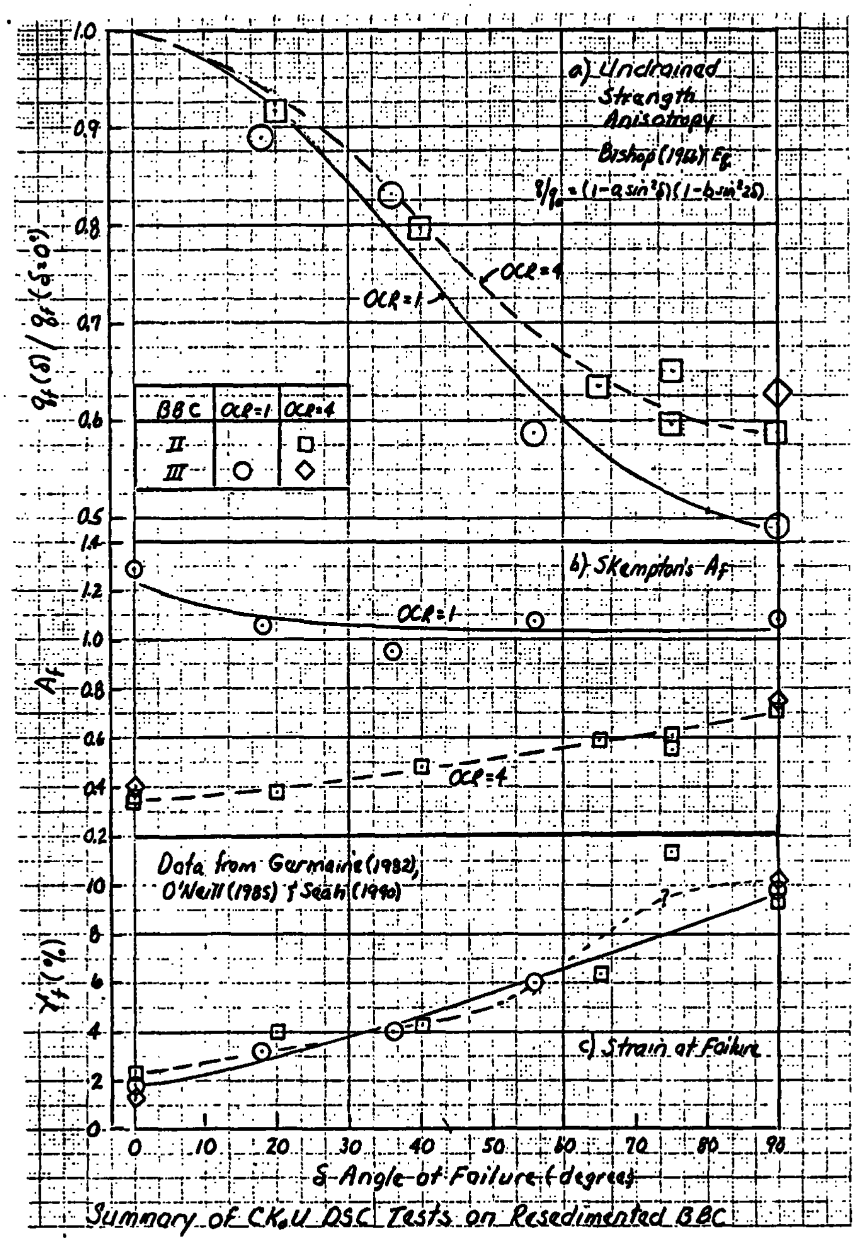
Normalized Pore Pressure and Effective Stress Paths for CU DSC Anistropic Tests on BBC at OCR = 4.

Decrease in s_u due to:

- Increasing u_s, i.e. lower p'_f
- Plus lower ESE (see p 7.3-4)

Figures by MIT OCW.

7.3.2 Collective DSC Data at OCR = 1 & 4 (OCR=1 data from Seah 1990)



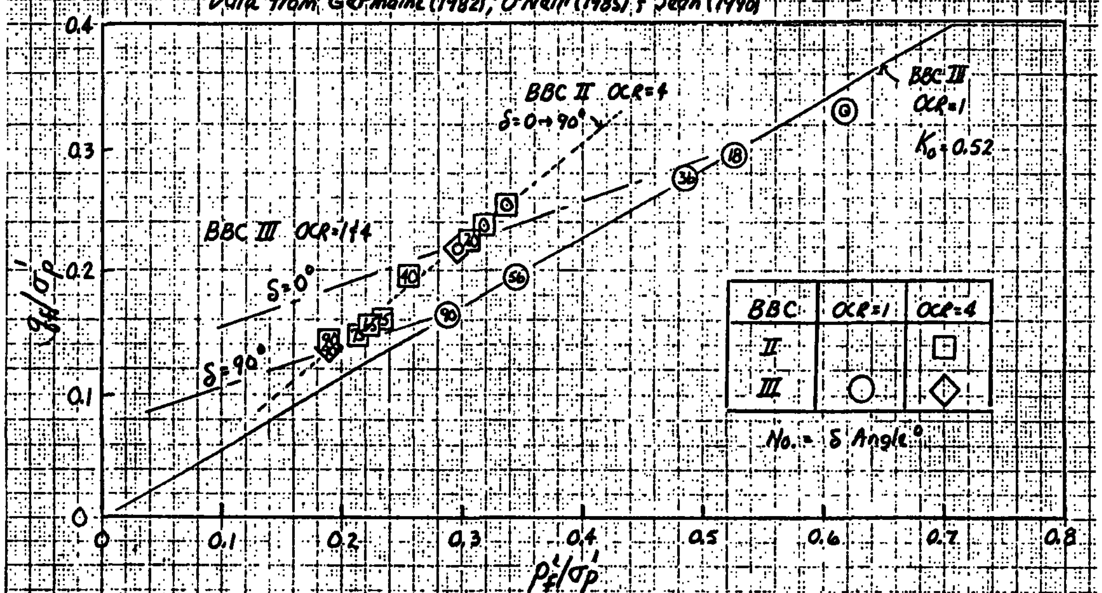
1) Trends in $s_u(\delta)$.
   - Similar shapes w/ OCR=1 → more anisotropy since includes both inherent and initial shear stress ($q_o > 0$)

2) Both show similar increase in γ_f with increasing δ.

3) For OCR=1, decreasing s_u mainly due to increasing Δq_f with increasing δ, i.e., approximately constant ϕ'_f & A_f

7.3.2 Continued

Data from Germaine (1982), O'Neill (1985) & Seah (1990)

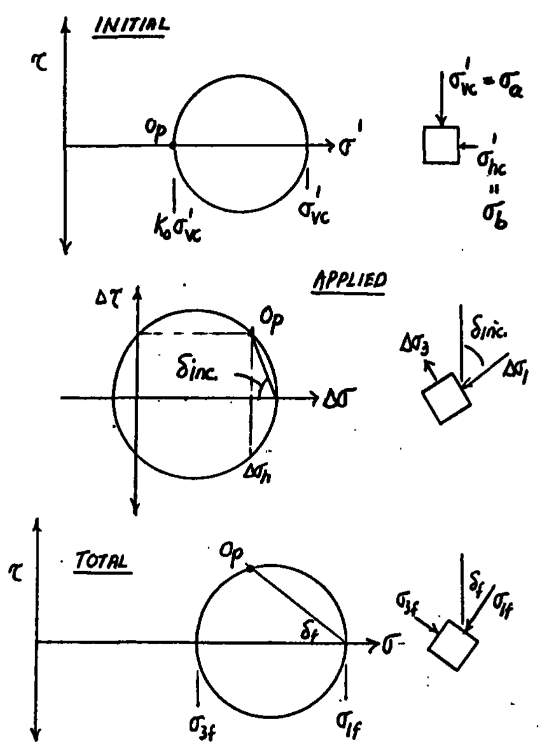


Normalized Shear Stress - Effective Stress at Failure: CK_0U DSC Resed BBC

- Approximately constant $\phi_f' \approx 34°$ for OCR=1 tests, but decreasing ESE with increasing δ for OC tests

7.3.3 DSC Data at OCR=1 : Comparison of Measured vs MIT-E3 Predicted

CK_0U DSC BBC OCR=1 (Seah, 1990)
{ $q=0.5(\sigma_1-\sigma_3)$, $p'=0.5(\sigma_1'+\sigma_3')$ }



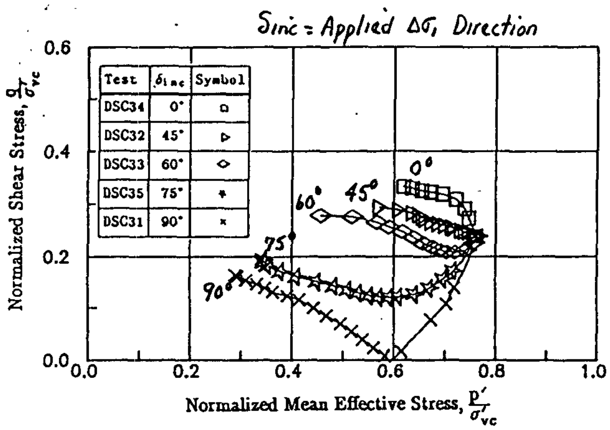
1) Experimental procedures very complex
- 1st had to K_0 consolidate to OCR=1 using silk between rubber membrane & shear sheets to reduce side friction
- Then had to remove silk in order that shear sheets could apply $\tau_a = -\tau_b = \Delta\tau$ to sides of test cube

- $\delta_{inc.} = 0$: $+\Delta\sigma_a$ & $\Delta\tau = 0$
- " $< 45°$: $+\Delta\sigma_a$ & $+\Delta\tau$
- " $= 45°$: $\Delta\sigma = 0$ & $+\Delta\tau$
- " $> 45°$: $+\Delta\sigma_b$ & $+\Delta\tau$
- " $= 90°$: $+\Delta\sigma_b$ & $\Delta\tau = 0$

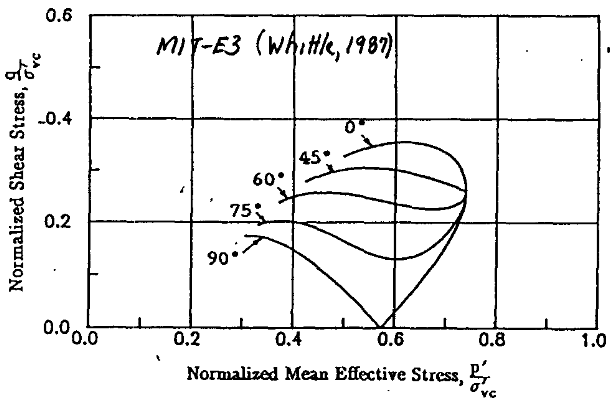
2) Predictions by Whittle (1987 ScD thesis) made *before* tests were run (Type A)

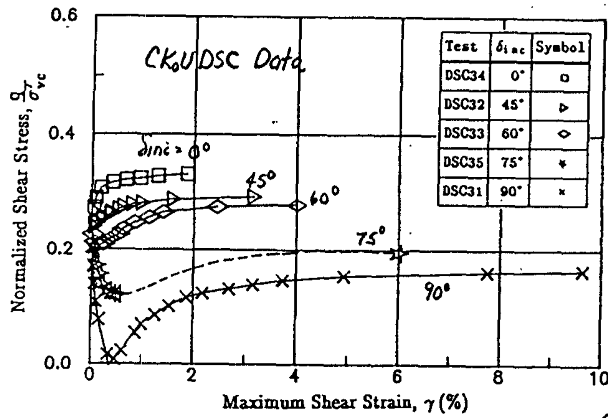
(a) Comparison of ESP

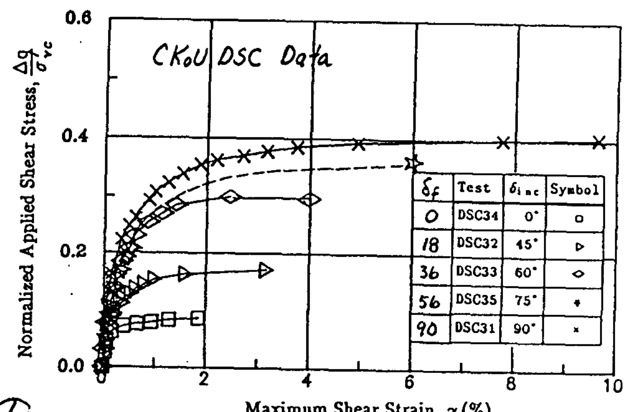
(b) Comparison of shear strain γ vs q and applied Δq
[$q = \frac{1}{2}(\sigma_1 - \sigma_3)$]



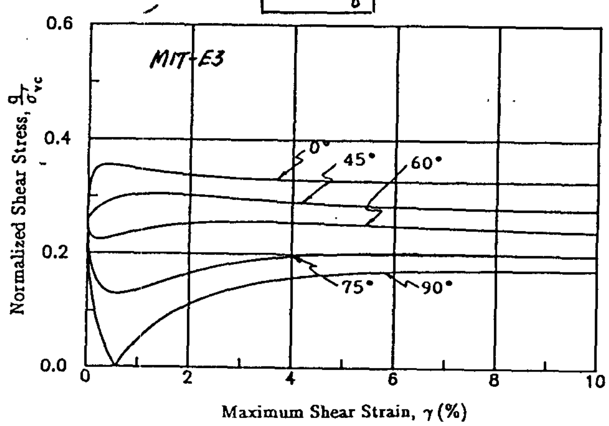
(a)

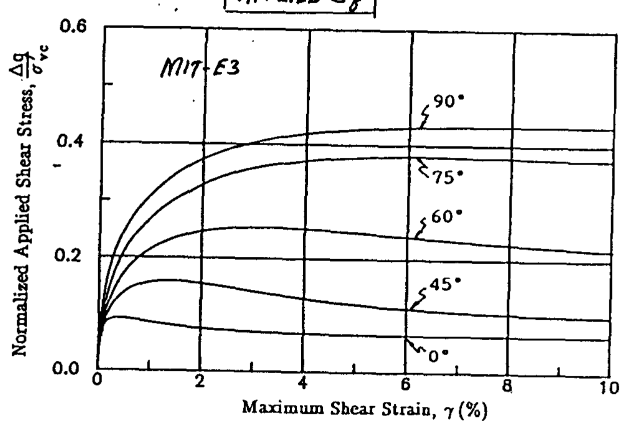






TOTAL q (b) APPLIED Δq

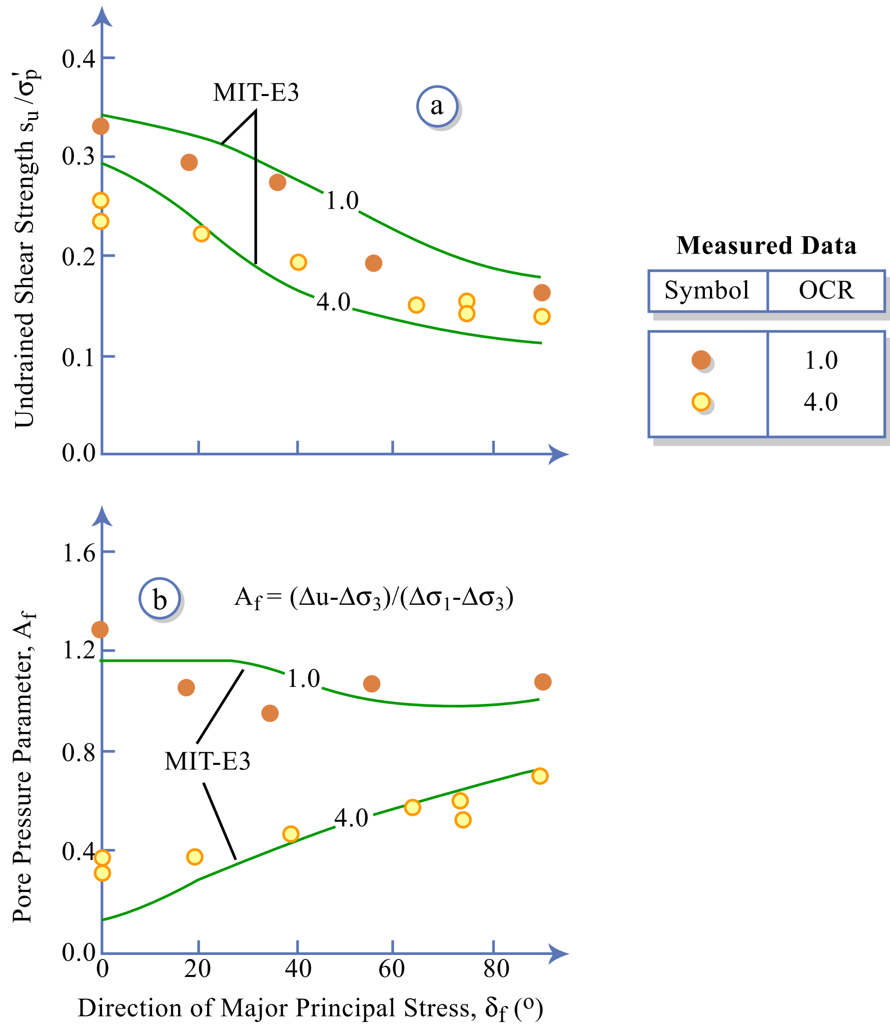




Adapted from:

Whittle, DeGroot, Ladd & Seah (1994) ASCE JGE 120(1)

(c) Comparison of s_u/σ'_p & A_f vs δ for OCR = 1 & 4



Evaluation of MIT-E3 Predictions of Peak Strength Conditions in DSC Tests: (a) Undrained Shear Strength; and (b) Pore Pressure Parameter, A_f

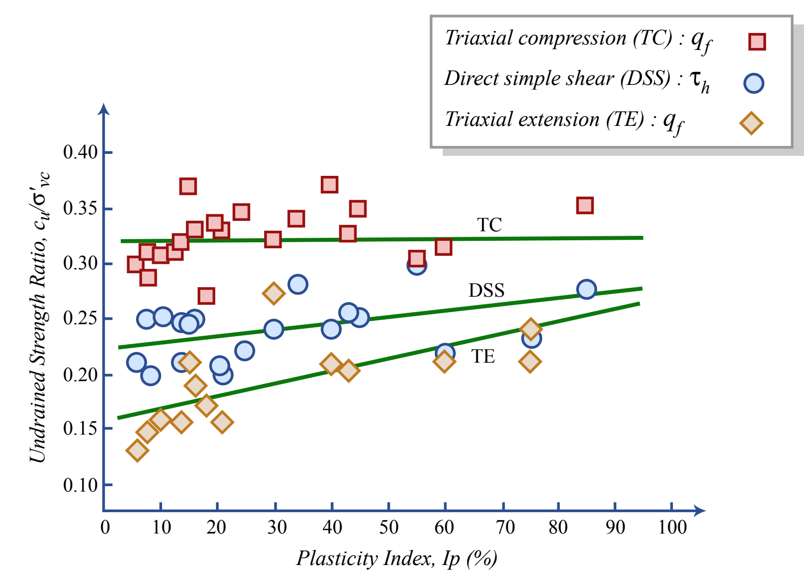
Figure by MIT OCW.

3) Conclusions:

- CK_0UDSC data on RBBC are only complete definition of s_u anisotropy for plane strain shearing of any clay, let alone at OCR = 1 & 4
- MIT-E3 does an excellent job of modeling this anisotropy
- In contrast, MCC predicts constant s_u/σ'_p independent of δ

7.4 General Trends in Undrained Strength Anisotropy from CK_oU PS, TX and DSS Testing

7.4.1 S_u Anisotropy for NC Clays & Silts (Non-Varved)

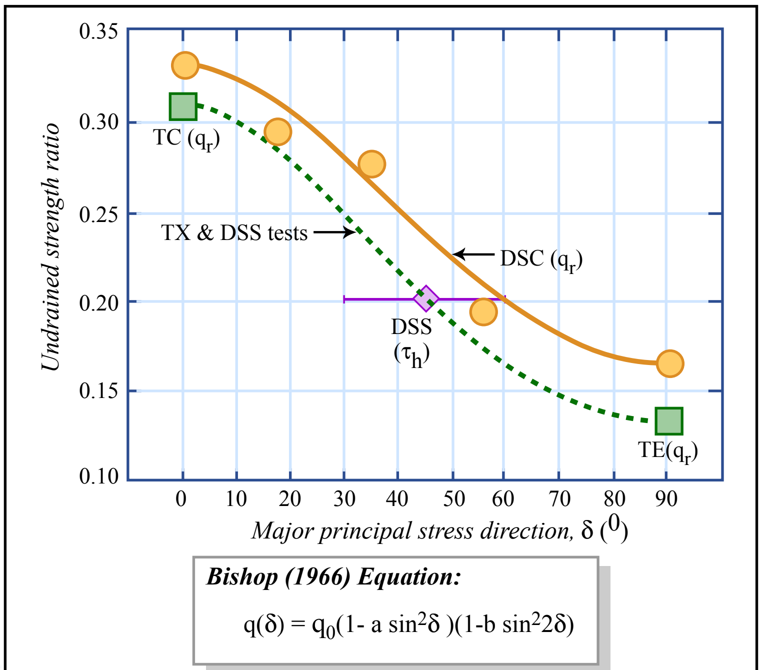


Undrained strength anosotropy from CK_OU tests on normally consolidated clays and silts.

Figure by MIT OCW.

1) Trends with Plasticity (Fig 15)

- TC > DSS > TE
- Low I_p → more anisotropy, i.e. lower $K_s = q_f(E)/q_f(C)$
- NOTE: TX K_s should be lower than PS K_s à la Section 6.2 & Table I, Tokyo '77 (p 438)



Undrained strength Ratio vs. Stress direction.

Figure by MIT OCW.

Trends with δ (Fig. 3)

- CCL would expect trends similar to DSC data on RBBC
- Bishop (1966)

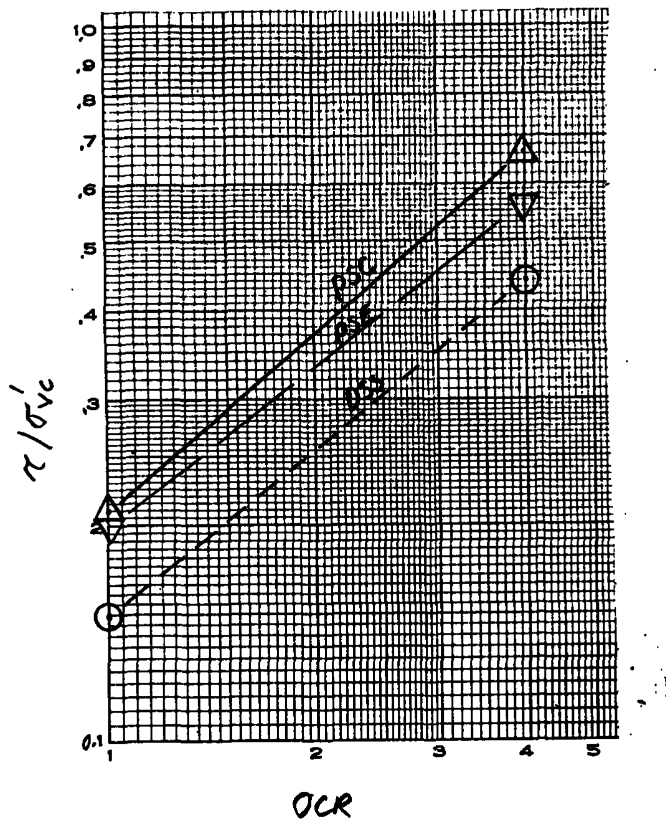
  $q(\delta) = q_0(1-a\sin^2\delta)(1-b\sin^2 2\delta)$

  $a = 1 - K_s$

  b = need to assume value for $q(\delta)$ to back calculate b

(Ladd 1994: 13th ICSMFE)

7.4.2 s_u Anisotropy of Varved Clays (Sambhandharaksa, ScD thesis MIT 1977)



- Varved clays are unusual since $CK_0USS \rightarrow$ lowest s_u/σ'_{vc}, i.e., below compression & extension
- In addition $S_d \doteq$ NC τ_h/σ'_{vc} DSS is extremely low
- Fig. at left from Table 2 (CCL '91) where $\tau = q\cos\phi'$ (or τ_h) tested for strain compatibility

7.4.3 s_u Anisotropy of OC Clays

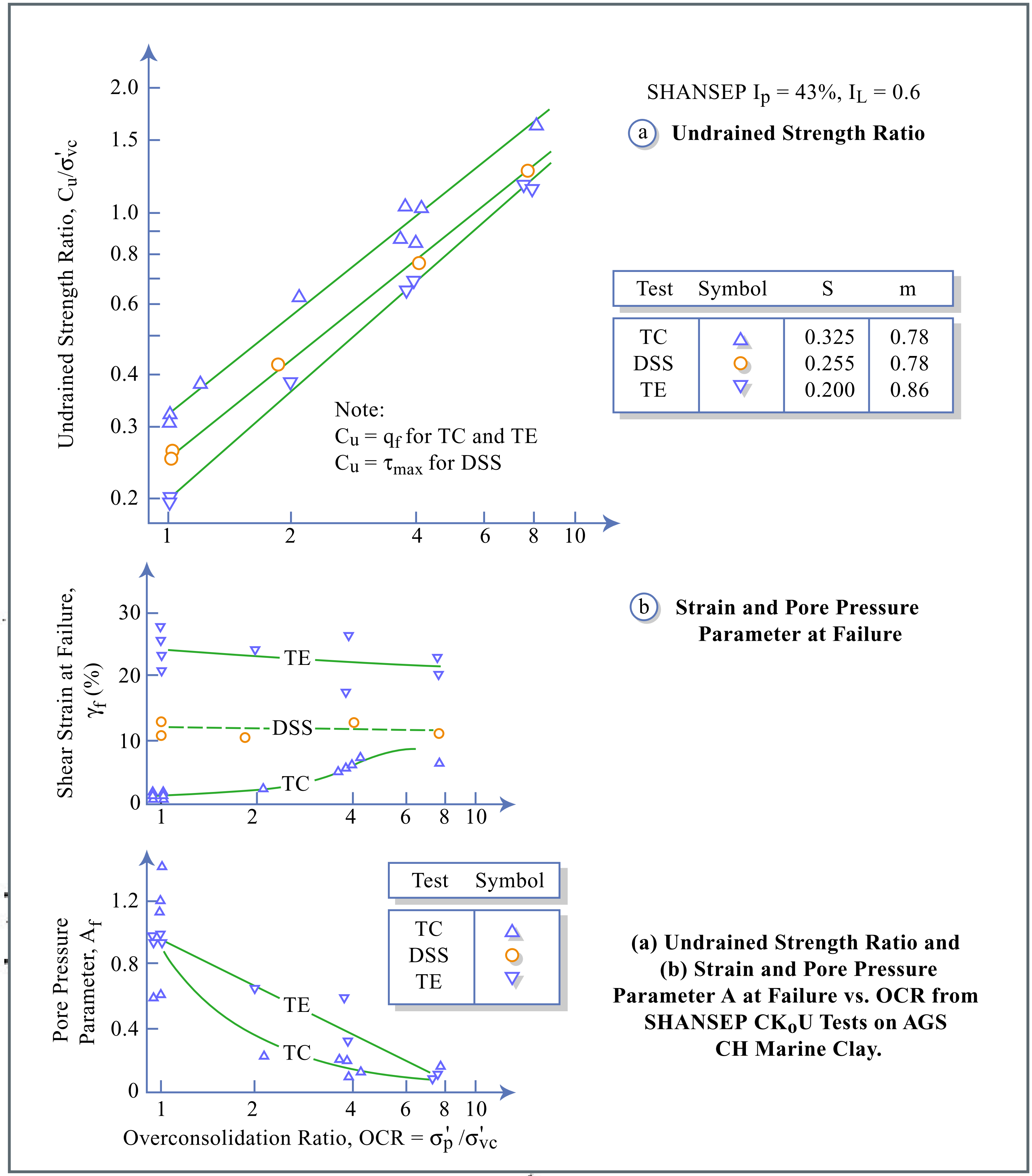
1) See p 7.4-3 for CK_0U TC, DSS & TE data vs OCR

Fig. 6: SHANSEP testing on CH clay
Fig. 7: Recompression testing on sensitive cemented CL clay
} note difference in γ_f trends

2) See p 7.4-4 for $\log K_s$ vs $\log$ OCR on several clays: $\left\{ K_s = \frac{s_e}{s_c}(OCR)^{(m_e - m_c)} \right\}$

- Increasing OCR → less anisotropy (except for B6 clay). Should expect since incr. OCR → incr. K_0 → smaller q_0 → less effect of "initial shear stress" anisotropy
- Note that Recomp. → less s_u anisotropy (higher K_s) than SHANSEP for natural BBC. CCL thinks this may be generally true

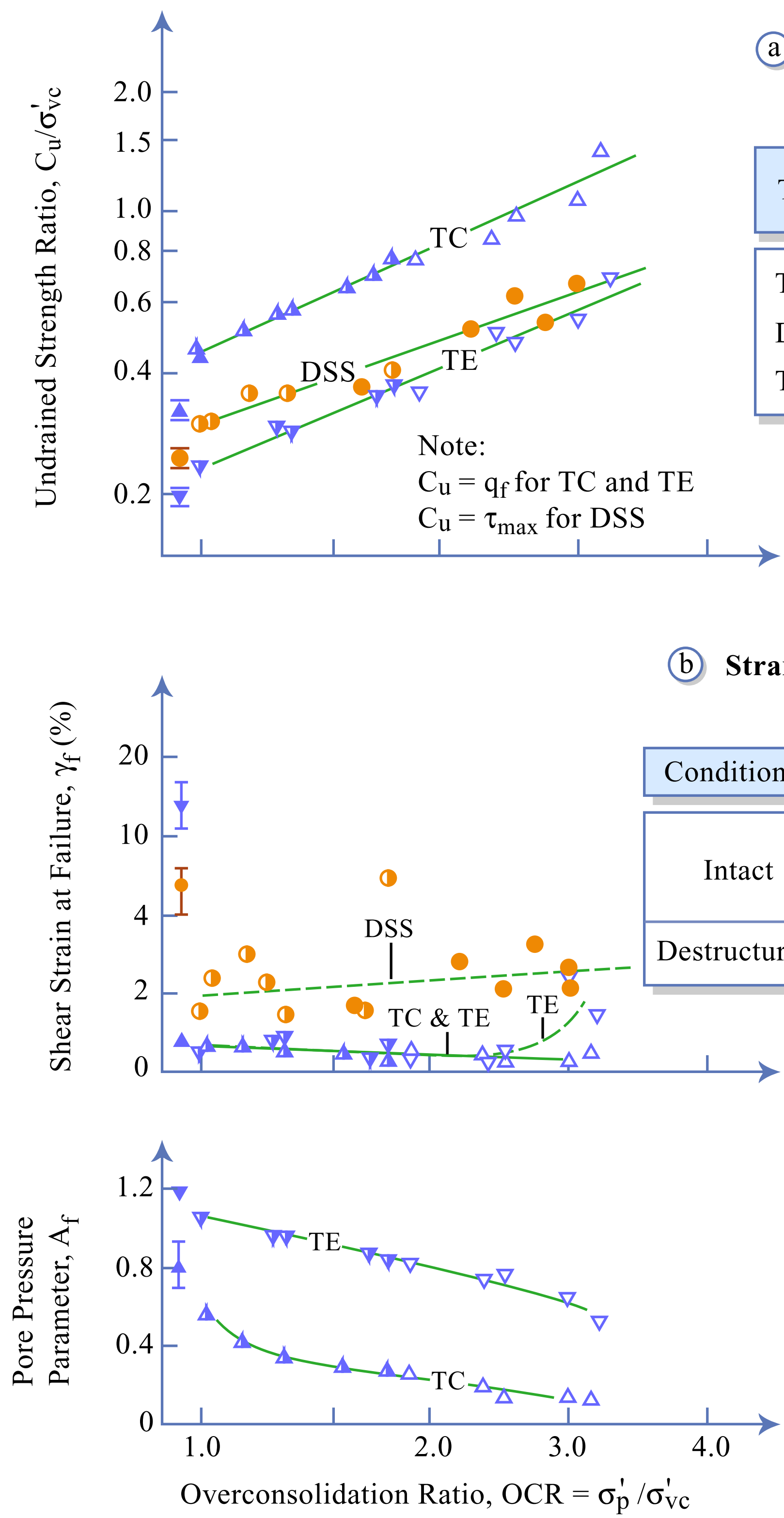
CCL 4/12/87 1.322 IIC 7.4-3
4/26/97



(a) Undrained Strength Ratio and (b) Strain and Pore Pressure Parameter A at Failure vs. OCR from SHANSEP CK_0U Tests on AGS CH Marine Clay.

Figure by MIT OCW.

Adapted from Jamiolkowski et al (1985)



Test	Intact S	Intact m	Destru. S
TC	0.45	0.865	0.335
DSS	0.29	0.695	0.25
TE	0.235	0.82	0.20

Condition	OCR	TC	DSS	TE
Intact	In situ	△	○	▽
	≥	▲ (half-filled)	◐	▼ (half-filled)
Destructured	1	▲	●	▼

(a) Undrained Strength Ratio and (b) Strain and Pore Pressure Parameter A at Failure vs. OCR from CK_0U Tests Run on Intact and Destructured James Bay B-6 Marine Clay.

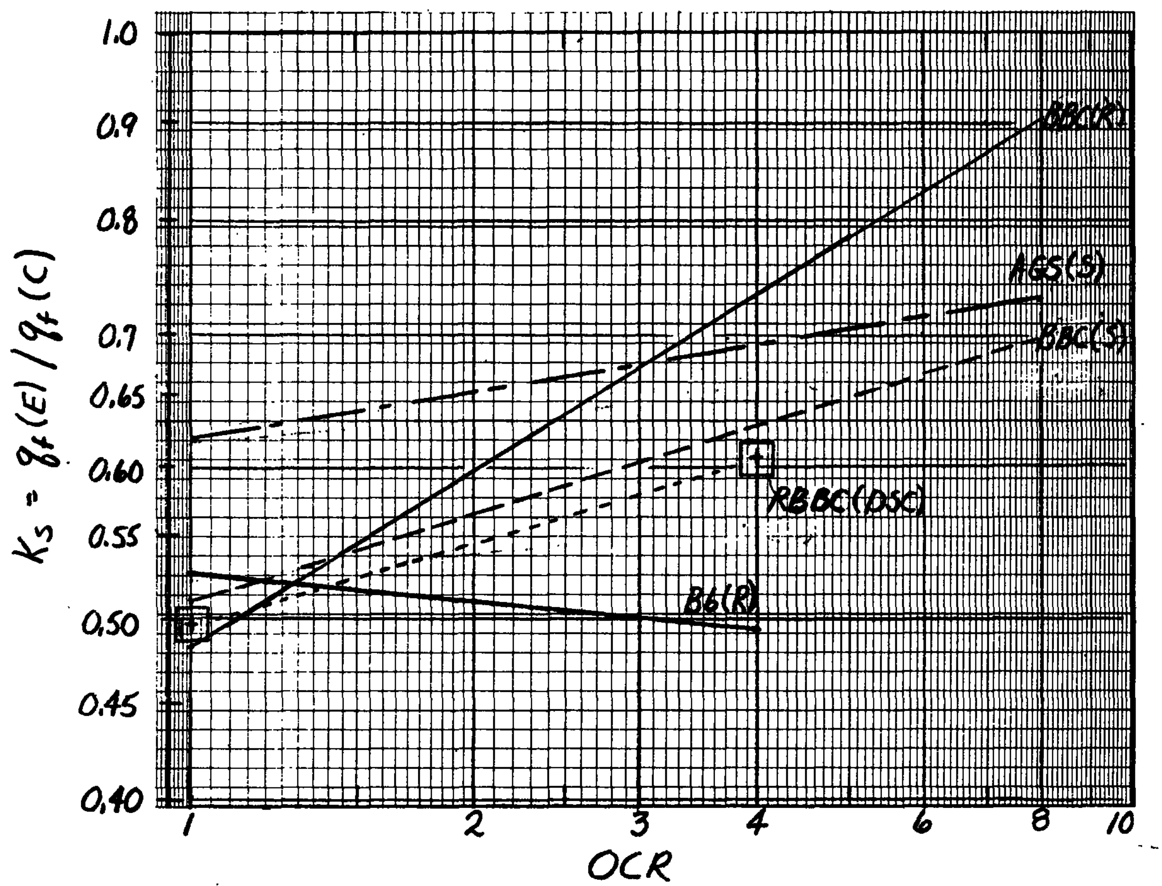
Figure by MIT OCW.

Adapted from Jamiolkowski et al (1985)

II C

Label	Clay	Program	Reference
B6(R)	B-6 James Bay	CK_0-TX R	IIC p.iia Fig.16 (Ladd 1991)
AGS(S)	AGS CH	CK_0-TX S	
BBC(R)	Natural BBC	CK_0-TX R	IIB, BBC-3,4
BBC(S)	"	CK_0-TX S	
⊞	Resed. BBC	CK_0-DSC R	II Section 7.3

R = Recompression S = SHANSEP



Variation in Undrained Strength Anisotropy with OCR

CCL 4/22/92 1.322 4/08/01

(4/13/99 NO Section 7.5)

7.6 Example of Evolving Anisotropy (Insert bottom p13)

1) Background:

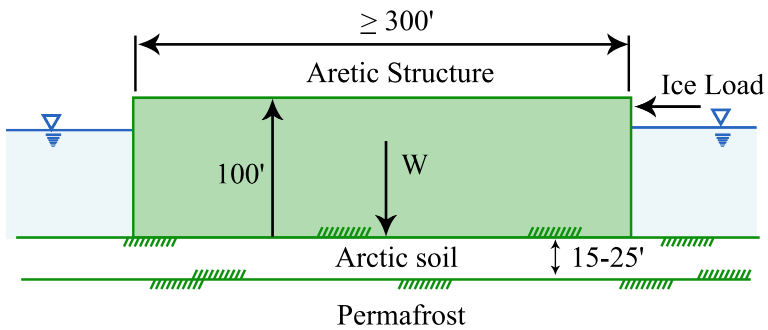
- DeGroot (1989) doctoral thesis to simulate stress conditions within the foundation soil for an Arctic offshore gravity platform
- MDSS = Multi-directional Direct Simple Shear apparatus. Same dimensions as Geonor DSS, but can apply two different horizontal shear stresses

2) Results

- MDSS-2 Schematic of problem
- " -3 " " MDSS
- " -4 Peak strength vs direction of cu loading
- " -5 Typical stress-strain data in direction of cu loading
- " -6 Comparison with MIT-E3 predictions

DeGroot, Ladd & Germaine (1996) "Undrained multidirectional direct simple shear behavior of cohesive soils" JGE, ASCE 122(2), 99-108

CCL 4/22/92 1.322 IIC MDSS-2 B8

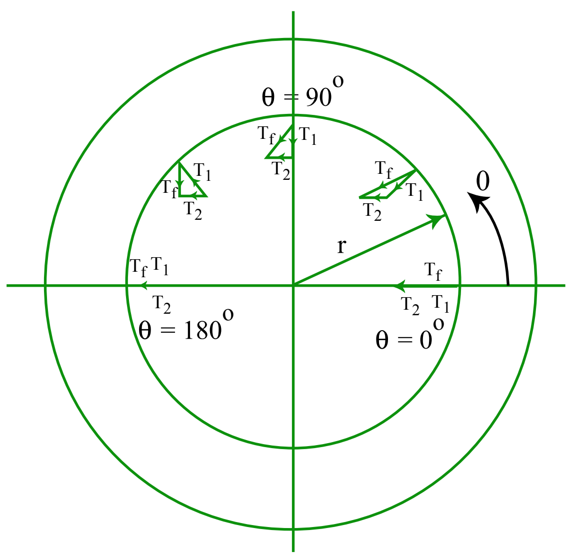


Shear stresses on soil at structure Interface (Top of foundation soil)

T_1: Weight structure → Consolidation shear stress

T_2: Ice load → Undrained shear stress

T_f: Final = f(r, θ)

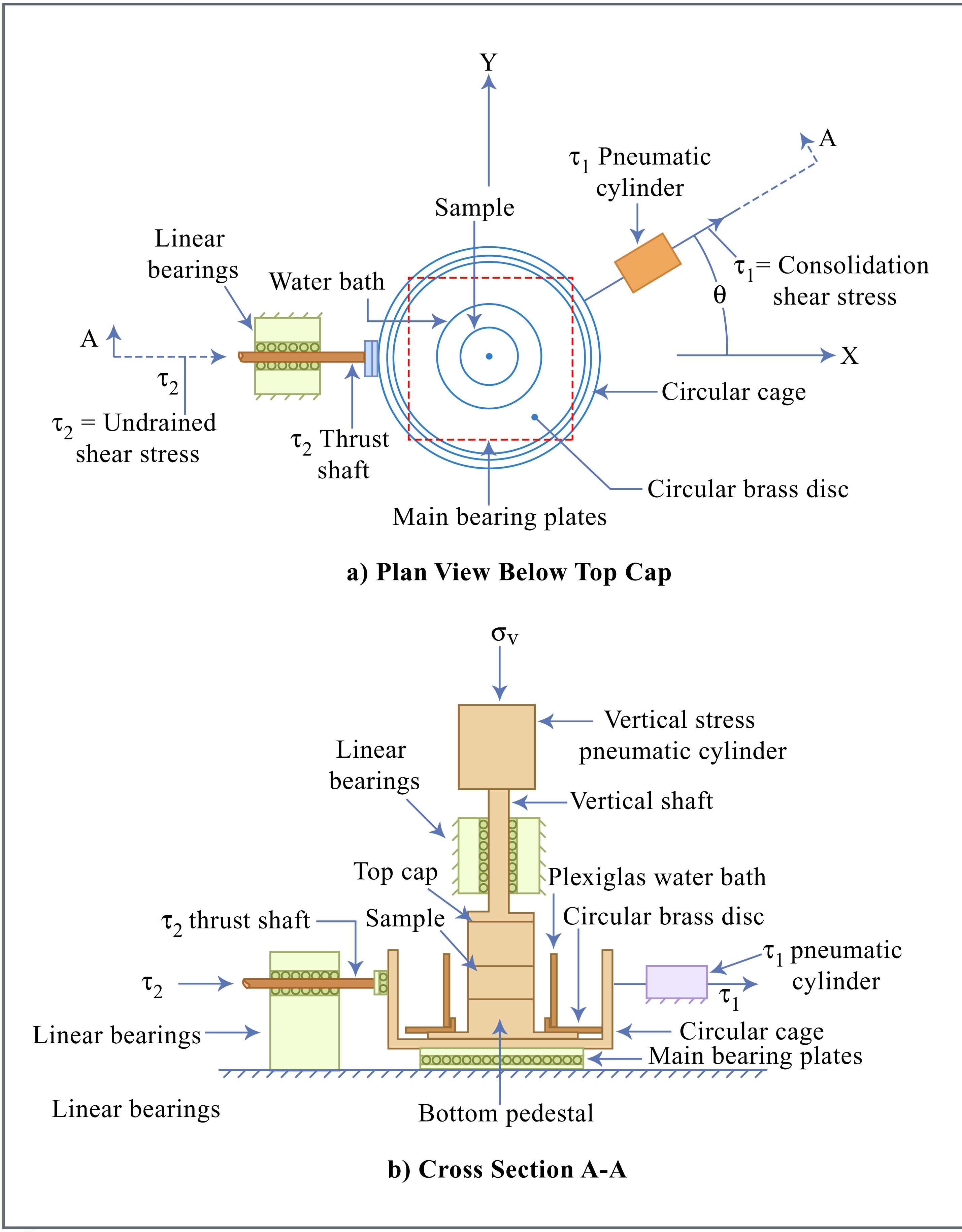


Shear stresses on soil at structure Interfae due to gravity and ice loading.

Figure by MIT OCW.

10/91 IIC MOSS-3 B11

CCL 4/22/92 1.322



a) Plan View Below Top Cap

b) Cross Section A-A

Figure by MIT OCW.

Figure 2 (from DeGroot, 1989)

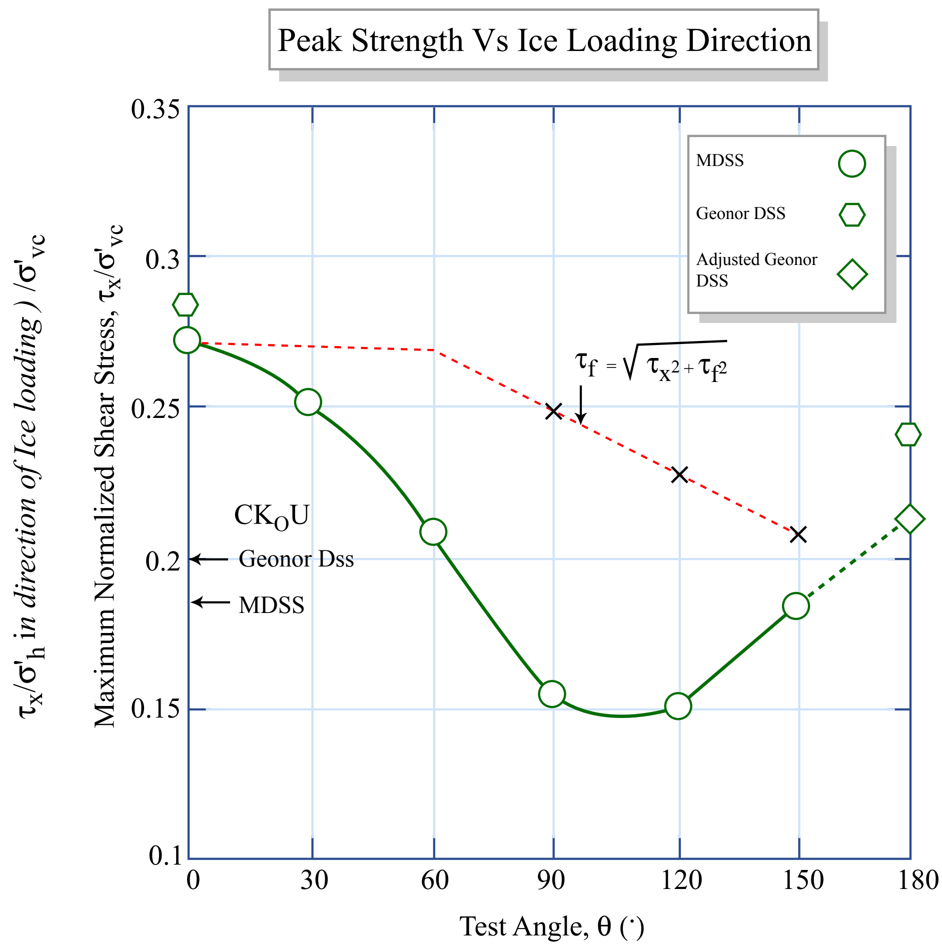
10/89 CCL 4/22/92
6/90 1.322
10/91

IIC.
249

MDSS-4

:B12

BBC OCR=1 $\tau_{hc}/\sigma'_{vc} = 0.20$



Maximum normalized shear stress τ_x/σ'_{vc} versus test angle q for CAUMDSS ***and Geonor CAUDSS tests on BBC. (Degroot, 1989***)

Figure by MIT OCW.

Adapted from:

(De groot, 1989)

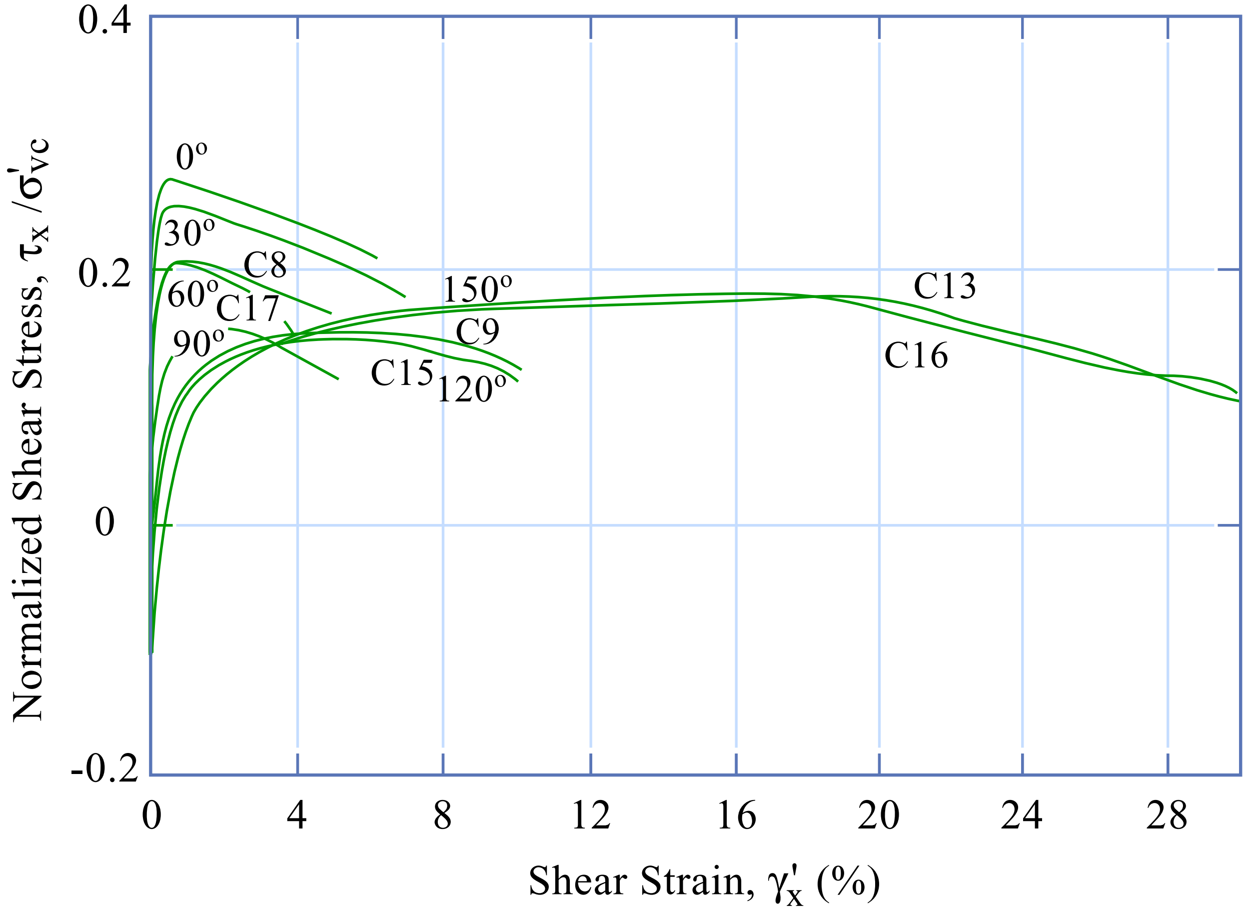
CCL 10/89
6/90
CCL 4/22/92
1.322

IIC

MDSS-5

B13

Stress vs. Strain as f(θ)



Shear Stress-Strain Curves for CAUMDSS Tests on BBC.

Image by MIT OCW.

Adapted from Degroot, 1989)

Low θ → Brittle Behavior
- High Peak Strength
- Low Strain at Failure
- Pronounced Strain Softening

High θ → Ductile Behavior
- Low Peak Strength
- Large Strain at Failure

CCL 4/22/92 1,322

IIC

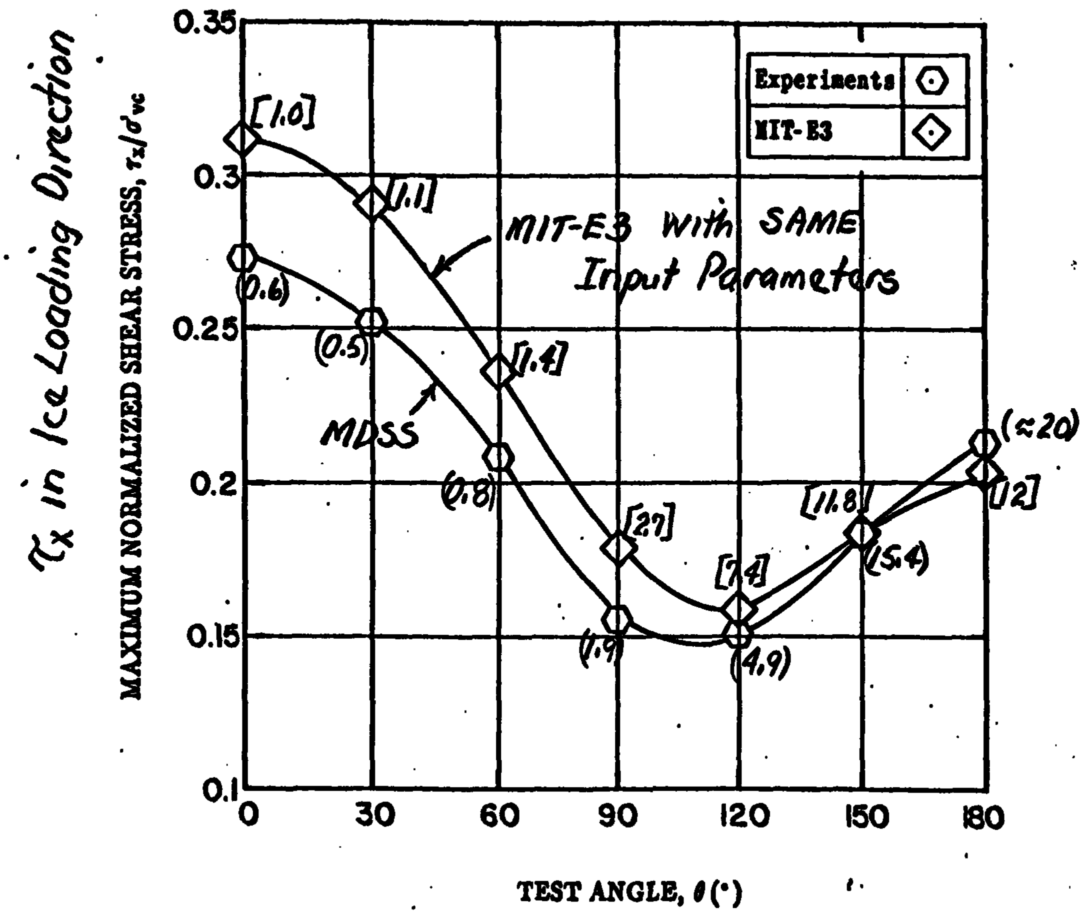
MDSS-6

CCL 10/89 10/91
6/90
10/90

B15

OCR=1 BBC $\tau_{hc}/\sigma'_{vc} = 0.20$

Peak Strength Comparison



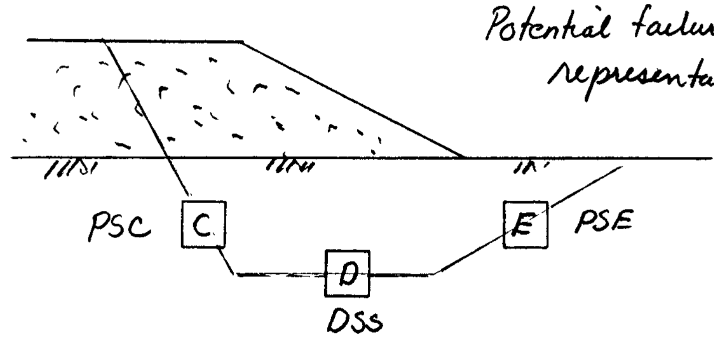
( ) = % Shear Strain in ice loading direction

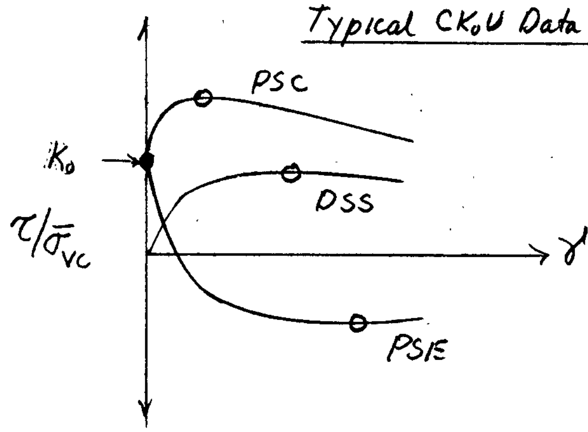
Figure 6.13: Measured and Predicted Maximum Shear Resistance τ_x/σ'_{vc} Versus Test Angle θ for CAUMDSS Behavior of BBC With $\tau_{hc}/\sigma'_{vc} = 0.2$.

(De Groot, 1989)

8. PROGRESSIVE FAILURE

8.1 Definition of Problem





- Anisotropy → varying peak s_u at different strains
- PSC loses resistance before mobilize peak s_u in DSS & especially PSE for strain softening clays

Conclusion: Can't mobilize peak strengths due to progressive failure if have strain softening

8.2 Strain Compatibility Technique

Koutsoftas & Ladd (1985)
Ladd (1991) Sect. 4.9

1) Semi-rational procedure to select design strengths considering progressive failure

2) Basic assumptions:

   a) Define $s_u = \tau$ on shear plane at failure
      $\tau = q \cos\bar{\phi}$ Triaxial & PS ; $\tau = \tau_h$ in DSS
      } For circular arc & wedge analyses (Not conventional q_{ult})

   ✱ b) Uniform shear strain (γ) along failure surface at moment before gross displacements → failure

3) Application - See SC-2 for AGS OCR = 1 & 4 (SHANSEP)
or Fig.17, p575 of Ladd (1991) SC-3 for B2 OCR = 1 & 2.1 (RECOMP.)

a) Plot τ (or $\tau/\bar{\sigma}_{vc}$) vs γ ($= 1.5\,\varepsilon$ for Triaxial; $= 2.0\,\varepsilon$ for PS)

b) Plot $\tau_{ave} = \frac{1}{3}(\tau_c + \tau_d + \tau_e)$

- At given OCR, max. resistance at max τ_{ave}
- If fdn. clay has variable OCR, need judgement to select design $\gamma_f \rightarrow \tau_{ave}$
- Also want γ_f leading reasonable anisotropic strengths, i.e. values of $\tau_c \sim \tau_d \sim \tau_e$

c) For circular arc with "isotropic" strengths, use τ_{ave}
" wedge analyses, can use τ_c, τ_d & τ_e

8.3 AGS Case History (K&L, 1985) - Handout

1) Background

- Breakwater for floating nuclear power plant with 3 stage construction (Fig. 1, 2)
- Initial in situ OCR ~ 4.2 ± 0.9 (Fig. 2)

2) Application strain compatibility technique (Fig. 7) = SC-2 at OCR = 1 & 4 + $\tau/\sigma'_{vc} = S\,(\sigma'_p/\sigma'_{vc})^m$ at $\gamma_f = 8\%$

Mode of Failure		S	m
PSC	τ_c	0.265	0.79
DSS	τ_d	0.25	0.77
PSE	τ_e	0.16	0.88
Ave.	τ_{ave}	0.225	0.81

3) Resultant c_u profiles for initial in situ condition
(Fig 8 = SC-4)

- $\tau_{ave} / c_u(FV) = 0.725 \pm 0.015$ SD vs Bjerrum (1972)
  $\mu = 0.84$ for $I_p = 43\%$ & $\mu_{PS} = 0.76$ after (1)
  Consideration of end effects à la Azzouz et al. (1983) ASCE JGE 109(5)
  - Conclusions wrt Bjerrum μ : Unsafe for PS failure (x 1.16) ; ok for typical 3-D failure (x 1.05)
- Comments on c_u(UUC) data ($\dot{\varepsilon} = 10\%/hr$)
  - Increased scatter vs $c_u(FV)$ & $\tau_{ave} \pm 1SD$: Expected
  - Mean UUC vs $c_u(FV)$: incr. more rapidly w/ depth - expect opposite
    vs τ_{ave} : 30% unsafe
    vs τ_c : larger - probably due to higher $\dot{\varepsilon}$
  - Conclusions.

4) Results for Stage 3 Stability (Fig. 1)

Method of Analysis	c_u Profile	F
a) Wedge via M-P	SHANSEP τ_c, τ_d, τ_e	1.27
b) Same	q_f from UUC & CIUC ($q_f/\sigma'_c = 0.33$)	1.45
c) Wedge via USCE (in upper CL clay)	Same	1.29 *

Conclusions - Wrong c_u + wrong analysis → correct F due to compensating errors

* Would get lower FS if used QRS envelope → lower $\bar{\phi}_{cu} = 0.26$

(1) $F(3\text{-}D)/F(2\text{-}D) = 1.11 \pm 0.06$ SD for 18 case histories (circular arc analyses of embankment failures) $\approx [1 + 0.7(\frac{D}{L})]$

8.4 Application to Several Clays

1) See SC-1, -1a: for results that apply to PS failures for OCR = 1 (SC-1a plots normalized τ_{ave} & K_s, plus τ_c & τ_a vs I_p)

2) Based on these and some other data, ~~typical~~ effect of progressive failure on design c_u is:
τ_{ave} as above & τ_p = ave. of <u>peak</u> τ values

Design γ_f = 5-10% { N.C. – $\tau_{ave}/\tau_p \approx 0.9 \pm 0.03$
OC – ≈ 0.95 for low S_t (e.g. BBC & AGS) }

Design γ_f = 2% OC – ≈ 0.85 for very high S_t like James Bay.

Note: $q_f(TC)/\tau_{ave} = 1.4? \pm 0.08$
$\tau_h(DSS)/\tau_{ave} = 1.07 \pm 0.07$ (w/o CVVC)

9. CONSIDERATION OF ANISOTROPY IN USA (Undr. Str. Anal.)

9.1 Bearing Capacity (PS)

1) Davis & Christian (1971)

$$\Delta q_{ult} = \tfrac{1}{2}\left[s_u(V) + s_u(H)\right] N_c' \quad , \quad N_c' = f\left(\frac{b}{a} = \frac{s_u(45)}{\sqrt{s_u(V)\cdot s_u(H)}}\right)$$

$$= s_u(V)\left[\tfrac{1}{2}(1+K_s)\right] N_c'$$

N_c' = 5.14 for b/a = 1.0
= 4.00 for b/a = 0
= 5.0 ± 0.14 for typical b/a = 0.9 ± 0.1

2) Definition s_u = ◯

3) Should apply strain compatibility to PS CK_oU for PS problems

4) If use $CK_oUC/E \rightarrow s_u(V)$ & K_s for PS problem

Peak
- TC/PSC = 0.92 ± 0.05
- TE/PSE = 0.82 ± 0.02

} → × 0.87 ≈ effects strain compatibility (1/1.11 = 0.90)

5) Kinner & Ladd (1973) model footing tests on BBC at OCR = 1, 2 & 4 (Table 11-4 of Strength Notes ≈ SC-5)

- Using peak q_f from CK_0U PSC/E → predicted / measured q_{ult} ≈ 1.0
- Explanation: Compensating Errors: increased q_f due to faster $\dot{\varepsilon}$ offset strain compatibility

6) Other procedures to get $s_u = c$ for q_{ult}

- q_f(UUC) DEPENDS ON COMPENSATING ERRORS ($\dot{\varepsilon} + \delta$ vs disturbance)
- q_f (CIUC) ALWAYS UNSAFE
- $\mu\, s_u$(FV)
  - For circular arc neglecting end effects → unsafe (x 1.11)
  - τ_{ff} vs q_f → too low (x $\cos\bar{\phi}$ ≈ 0.87)
  - ∴ Compensating errors

9.2 Circular Arc Stability Analyses Using "Isotropic" strengths

1) Above comments / conclusions apply but now presumably want τ_{ff} vs q_f + end effects

2) Comparison of c_u(DSS) vs τ_{ave} from SC

From SC-1 $c_u(DSS)/\tau_{ave} = 1.07 \pm 0.07$ (w/o CVVC)

∴ Slightly unsafe for plane strain failures

But for typical failures with 3-D effects, on average is slightly conservative since $\frac{F(3\text{-}D)}{F(2\text{-}D)} = 1.11 \pm 0.0650$

3) Level C analysis using empirical correlations to estimate S & m as f(soil type) à la Section 5.4 of CCL (1991)

e.g. CL-CH	S = 0.22	m = 0.8
OH-MH	= 0.25	= 0.8
CVVC	= 0.16	= 0.75

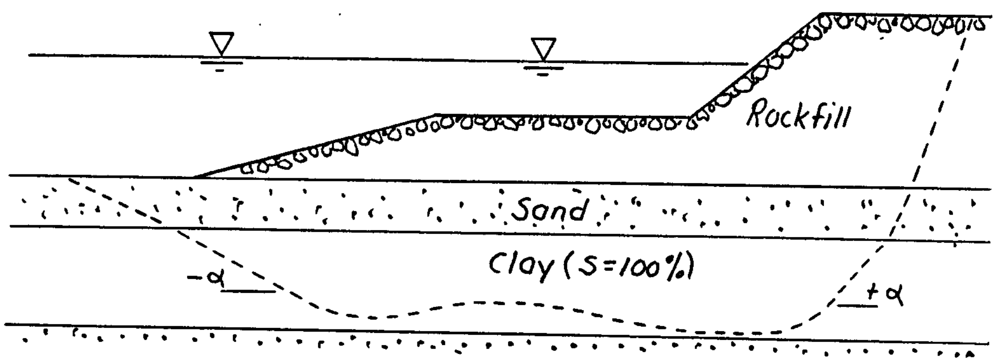
9.3 Non-Circular Analyses using Anisotropic Strengths (e.g. UTEXAS3)

(Note: p19-21 from Ladd (1994) Panel Discussion 13th ICSMFE, New Delhi)

CCL 12/26/93 6

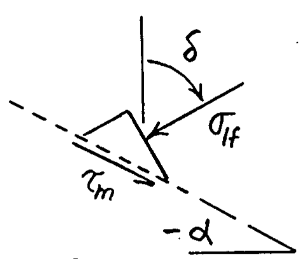
STABILITY ANALYSIS OF EMBANKMENT

- "Total stress" analysis → "$\phi = 0$", $c = c_u$
- Critical shear surface from UTEXAS3 search (Spencer)
- Required input: $c_u = f(\alpha)$



CCL 12/26/93 7

TWO MAJOR QUESTIONS (Mohr-Coulomb Failure Criteria)



$\tau_m = \frac{c_u}{F}$

1) How to define c_u?

$c_u = q_f \cos\phi \quad [q_f = 0.5(\sigma_1 - \sigma_3)_f]$

2) Relationship between α and δ?

$\alpha = \theta - \delta$

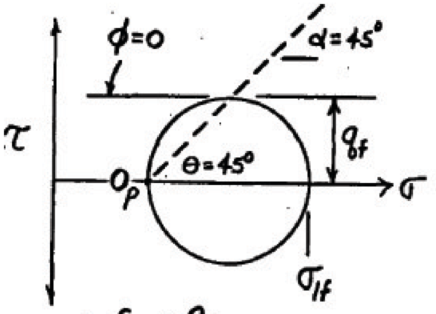
$\theta = 45 + \phi/2$ = angle between failure plane and σ_{1f} plane

SHOULD ONE USE: total stress $\phi = 0$ OR effective stress ϕ'?

CCL 12/26/93 8

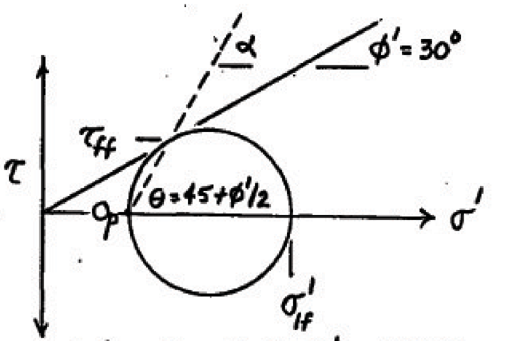
APPLICATION OF TWO HYPOTHESES (For $\delta = 0°$)

Using $\phi = 0$



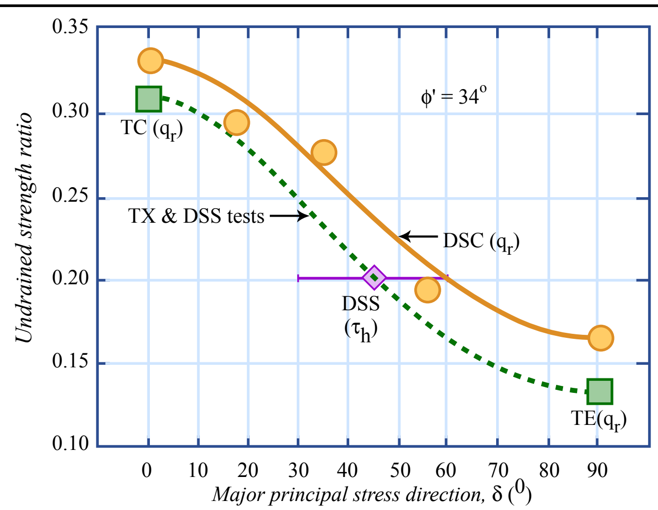
- $c_u = q_f$
- $\theta = 45°$
- $\alpha = 45° - \delta$

Using $\phi = \phi'$ (Critical = Actual Shear Surface)



- $c_u = \tau_{ff} = q_f \cos\phi' \rightarrow 0.87 q_f$
- $\theta = 45 + \phi'/2 \rightarrow 60°$
- $\alpha = \theta - \delta \rightarrow 60° - \delta$

CCL 12/26/93



Bishop (1966) Equation:

$$q(\delta) = q_0(1 - a\sin^2\delta)(1 - b\sin^2 2\delta)$$

Undrained strength Ratio vs.Stress direction.

Figure by MIT OCW.

9

APPLY TWO HYPOTHESES TO MEASURED BBC DATA

Using $\phi = 0 \rightarrow$

$c_u = q_f$ & $\alpha = 45° - \delta$

Using $\phi' = 34° \rightarrow$

$c_u = 0.83 q_f$

$\alpha = 62° - \delta$

ANISOTROPIC c_u/σ'_{vc} RATIOS FOR STABILITY ANALYSES

CCL 12/26/93

Line	Fig. 3 Data	$c_u =$	$\alpha =$
A	DSC	q_f	$45 - \delta$
B	TX & DSS	q_f	$45 - \delta$
C	DSC	$q_f \cos\phi'$	$\theta - \delta$
		$\phi' = 34^o$	$\theta = 62^o$

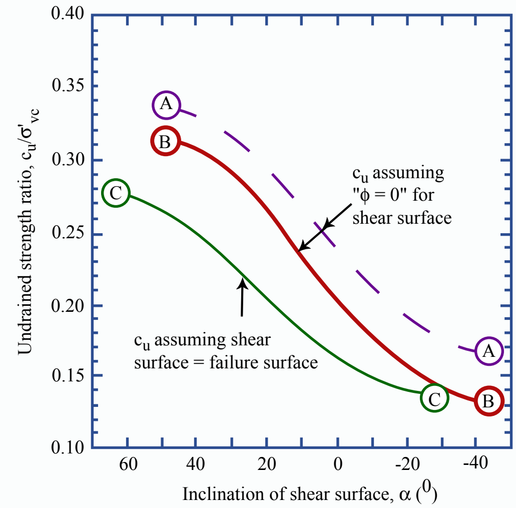


Figure by MIT OCW.

CCL 12/26/93

CONCLUSIONS

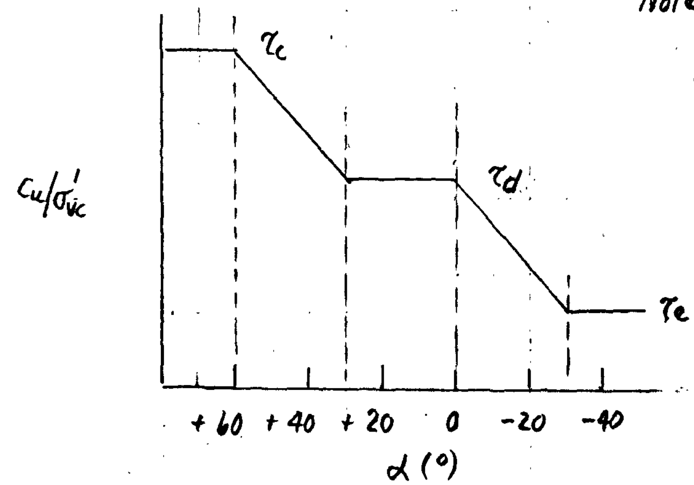
1) Run lab CK_oU tests with varying δ to measure anisotropy
   - Apply corrections to TC/TE data
   - Assume $\delta = 45 \pm 15°$ for DSS

2) If the PREDICTED critical shear surface from a sophisticated search routine is close to the most likely ACTUAL failure surface, then

   - Assuming $\phi = 0 \rightarrow c_u = q_f = 0.5(\sigma_1 - \sigma_3)_f$ and $\alpha = 45° - \delta$ } Probably UNSAFE

   - Assuming $\phi = \phi' \rightarrow c_u = \tau_{ff} = q_f \cos\phi'$ and $\alpha = (45 + \phi'/2) - \delta$ } Recommended

Simplied Approach Given Uncertainty in δ / α for DSS tests



Note: Drawn for $\alpha = 60° - \delta$ ($\phi' = 30°$)

Replacing actual variation with stepped linear

CCL 4/88 1.322
4/98

IIC

SC-1

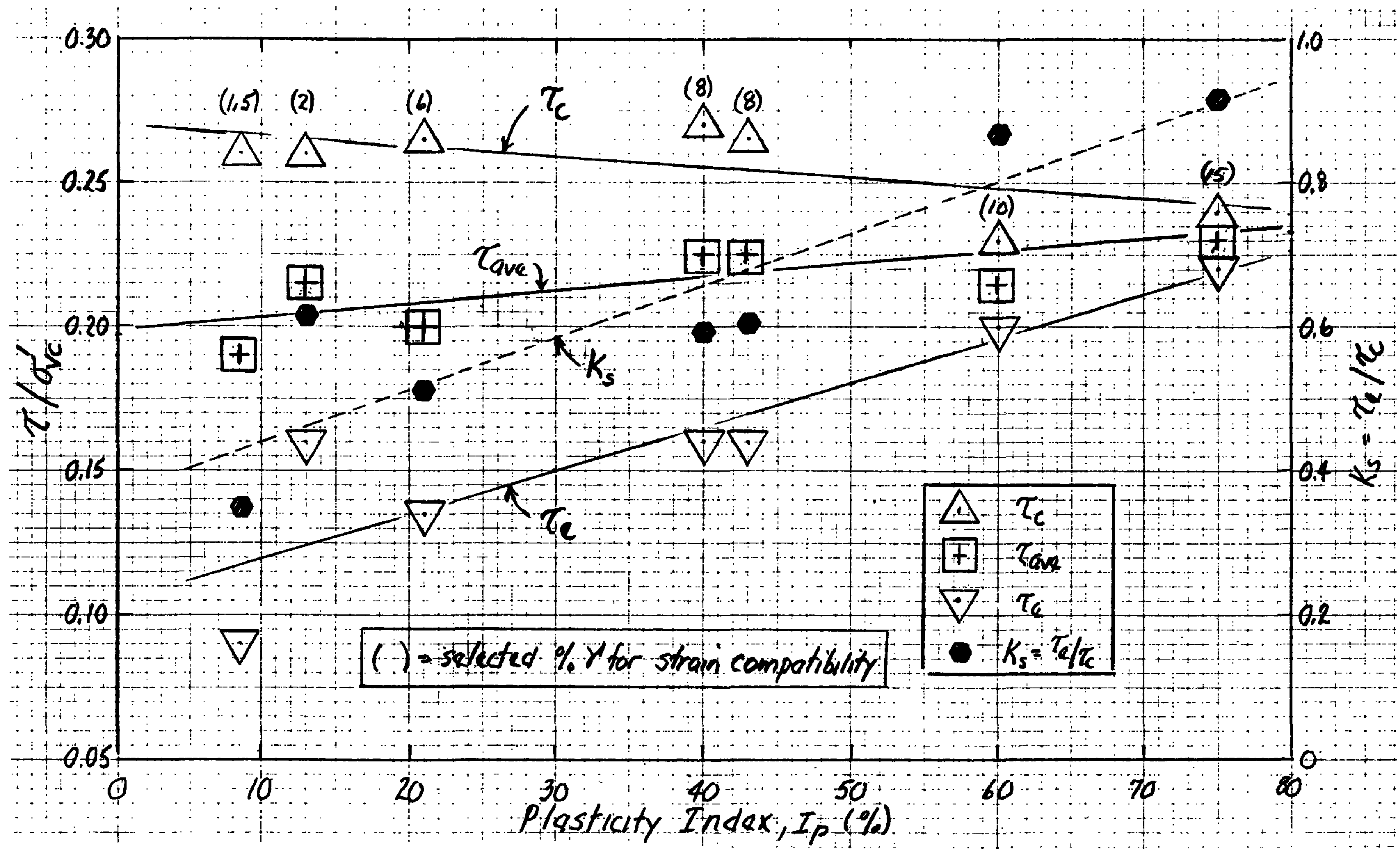
TABLE 3. - Normally Consolidated Undrained Strength Ratios From CK_oU Compression, Direct Simple Shear and Extension Tests Treated For Strain Compatibility

(from Ladd Terzaghi Lecture = CCL 1991)

No.	Soil	Index Properties			Peak c_u/σ'_{vc}		Strain Compatibility c_u/σ'_{vc}					C/E Testing[b]	Ref.
		USC	I_p	I_L	q_f(TC)	τ_h(DSS)	γ^a	τ_c	τ_d	τ_e	τ_{ave}		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	B2 Marine Clay	CL	8.5%	2.6	0.31	0.23	1.5%	0.26	0.22	0.09	0.19	TX	( )
2	B6 Marine Clay	CL	13%	1.9	0.33	0.24	2%	0.26	0.225	0.16	0.215	TX	( )
3	Resedimented BBC	CL	21%	1.0	0.33	0.20	6%	0.265	0.20	0.135	0.20	PS	MIT
4	Conn. Valley Varved Clay	CL CH	12% 39%	-	0.25	0.16	6%	0.21	0.15	0.20	0.185	PS	( )
5	Great Salt Lake Clay	CH	40%	1.1	0.37	0.24	8%	0.27	0.24	0.16	0.225	TX	MIT
6	AGS Marine Clay	CH	43%	0.6	0.325	0.255	8%	0.265	0.25	0.16	0.225	PS	( )
7	Omaha, NE Clay	CH	60%	0.7	0.315	0.22	10%	0.23	0.21	0.20	0.215	TX[c]	MIT
8	Arctic Silt A	ML	15%	0.3	0.37	0.245	12%	0.305	0.24	0.18	0.24	TX	MIT
9	Arctic Silt B	MH	30%	0.7	0.32	0.24	12%	0.27	0.24	0.20	0.235	TX	MIT
10	EABPL Clay	CH	75%	0.85	0.24	0.235	15%	0.24	0.23	0.22	0.23	PS/TX[d]	MIT

a Design shear strain selected for strain compatibility.
b TX = triaxial and PS = plane strain
c Triaxial τ_c increased by 5%.
d Approximate mean of plane strain and triaxial data.

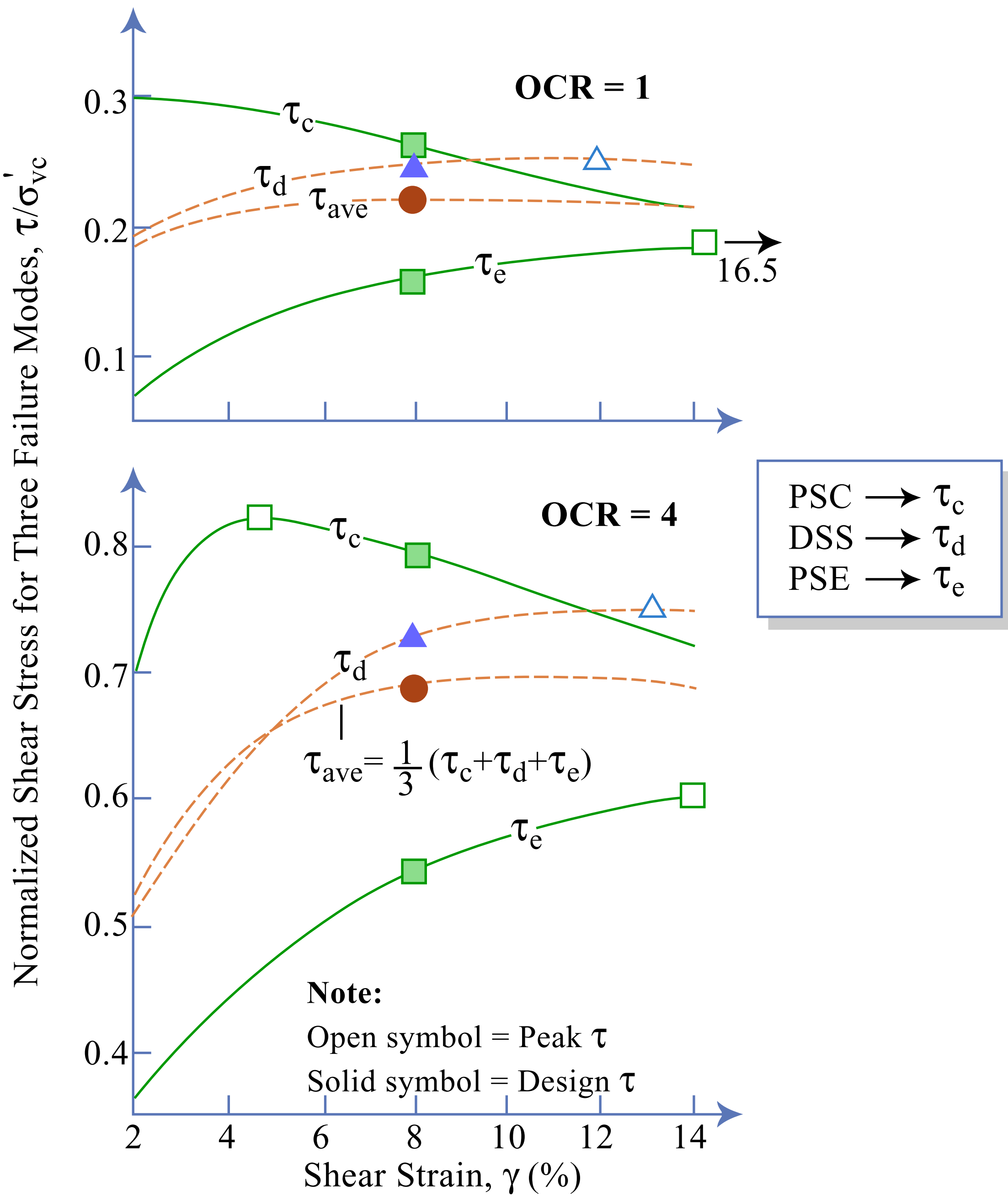
CCL 8/10/90 4/27/97 4/98



Undrained Shear Strength Ratios vs. Plasticity Index for CL and CH Clays Treated for Strain Compatibility (Data from Table 4, Ladd 1991)

IIc

1.322 CCL 4/85 AGS Plastic Clay SC-2



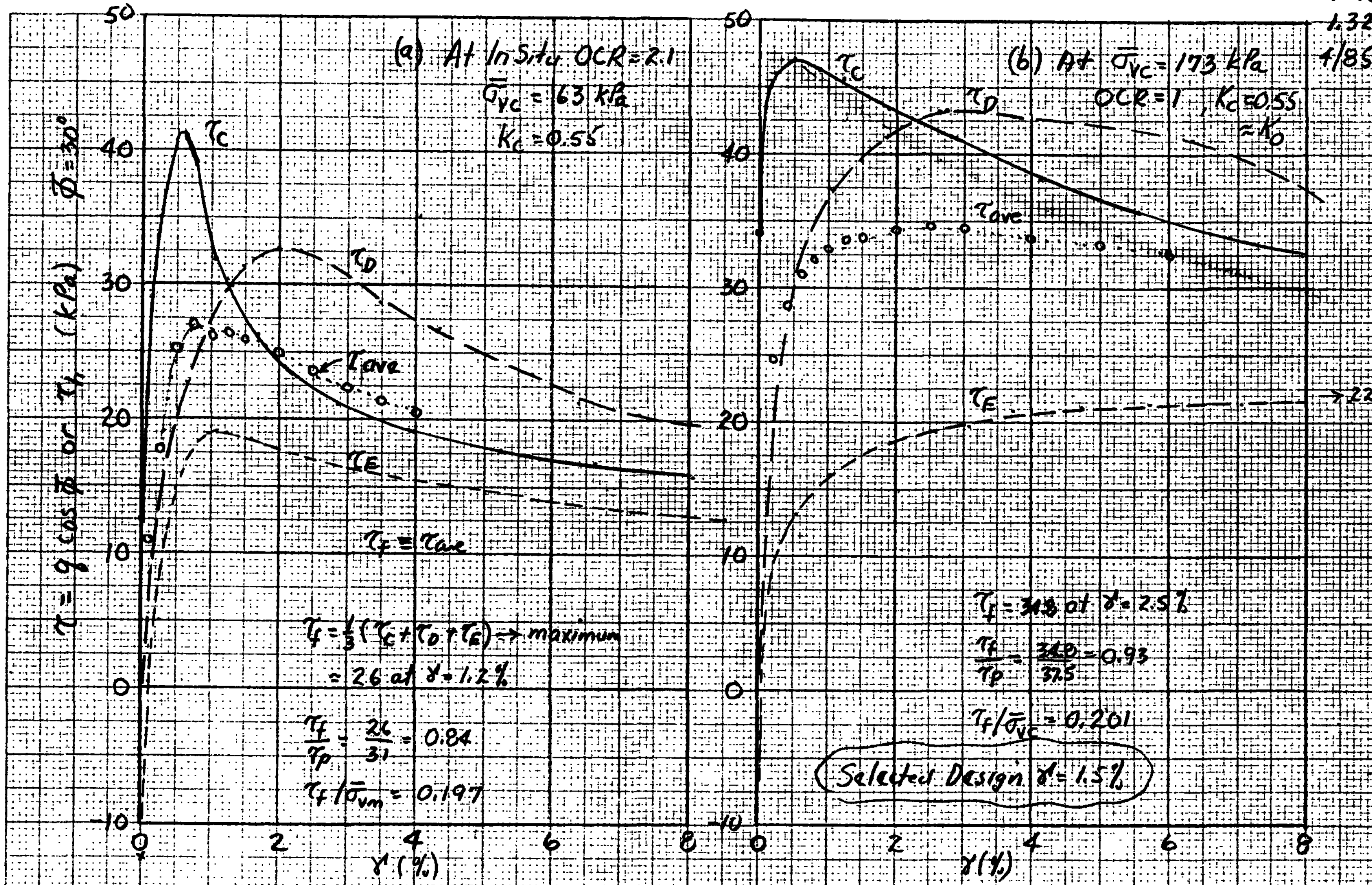
Normalized Stress-Strain Data used for the Strain Compatibility Technique.

Figure by MIT OCW.

Adapted from Koutsoftas & Ladd (1985)

CCL 4/25/82
1.322
4/85

IIC

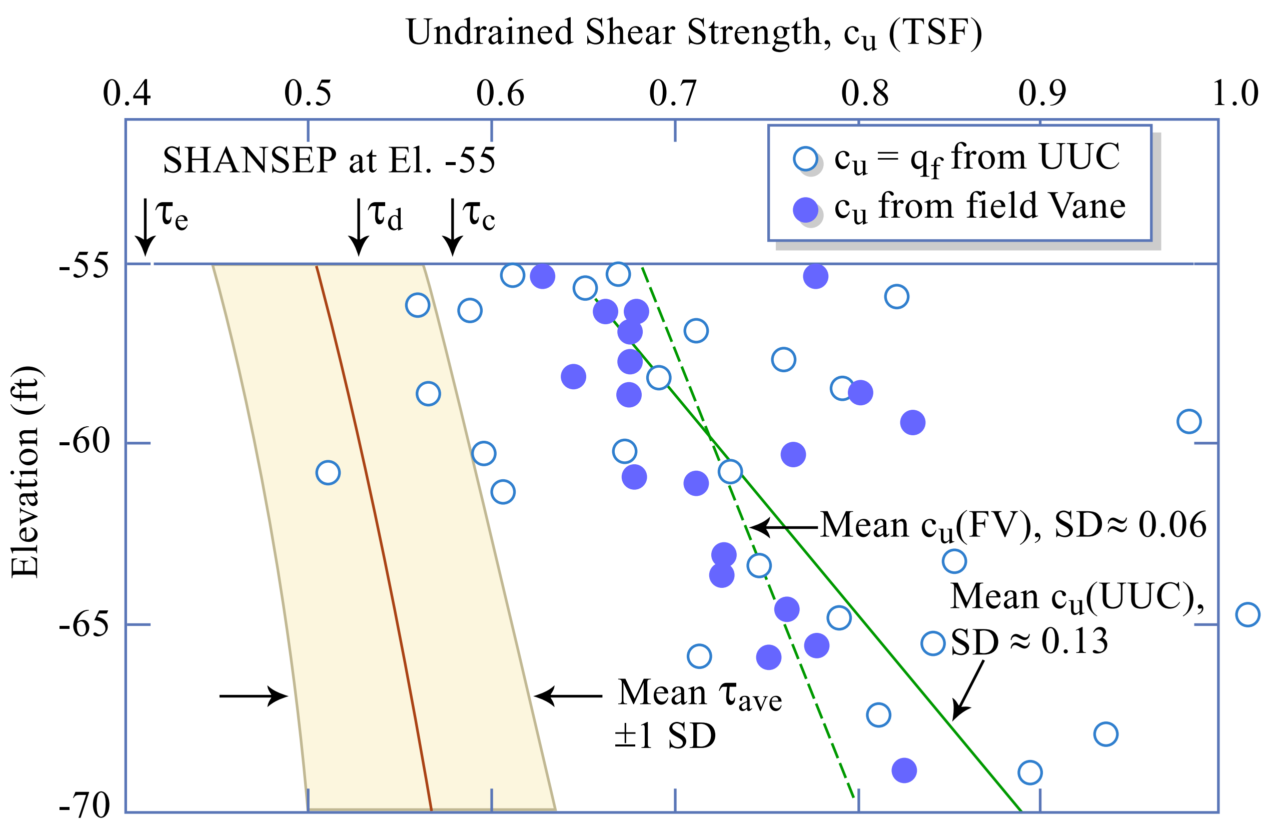


Application of Strain Compatibility Method to CK_0UC, DSS & E Tests
SEBJ B-2 z = 10.4 m $\bar{\sigma}_{vo}$ = 63, $\bar{\sigma}_{vm}$ = 132, I_p = 7.4%, I_L = 3.4

IIC

1.322 CCL 4/85

SC-4



AGS Case History

Comparison of Initial Undrained Shear Strength Profiles
(1 ft = 0.305 m; 1 tsf = 95.8 kPa)

Figure by MIT OCW.

Adapted from Koutsoftas & Ladd (1985)

PREDICTED VS MEASURED ULTIMATE BEARING CAPACITY OF STRIP FOOTING ON BOSTON BLUE CLAY

(from Kinner & Ladd, 1970; Ladd, et al., 1971; & Ladd and Edgers, 1971)

Undrained Shear Strength Determined From	OCR $\bar{\sigma}_{vm}/\bar{\sigma}_{vc}$	Undrained Strength Ratio $s_u(ave)/\bar{\sigma}_{vc}$	$s_u(V)/\bar{\sigma}_{vc}$	$s_u(H)/\bar{\sigma}_{vc}$	Ultimate Bearing Capacity $q_{ult}/\bar{\sigma}_{vc}$ Predicted (1)	Predicted/Measured % (2)
A $\overline{CK_oU}$ (3) Plane Strain Active & Passive	1	0.26[5]	0.34	0.19	1.36	101.5
	2	0.47	0.57	0.37	2.41	99.5
	4	0.81	0.95	0.67	4.15	99
B $\overline{CK_oU}$ (3) Plane Strain Active	1	0.34	0.34	—	1.75	130
	2	0.57	0.57	—	2.93	121
	4	0.95	0.95	—	4.88	116
C $\overline{CIU}$ (3) Triaxial Compression	1	0.32[5]	0.32[5]	—	1.67	125
	2	0.55[5]	0.55[5]	—	2.85	118
	4	0.90	0.90	—	4.62	110
D $\overline{CK_oU}$ (4) Direct-Simple Shear	1	0.20	—	—	1.03	77
	2	0.37	—	—	1.90	78.5
	4	0.61	—	—	3.14	75
E $\overline{UU}$ (3) Triaxial Compression (D'Appolonia, 1968)	1	0.18	0.18		0.92[5]	69
	2	0.36	0.36		1.85	76.5
	4	0.60	0.60		3.08	73.5

(1) Predicted $q_{ult} = N_c\, s_u(ave)$ with $N_c = 5.14$ (Davis & Christian, 1971)

(2) Measured at $\rho/B = 0.1$ with $\bar{\sigma}_{vm} = 3.4$ kg/cm²

(3) $s_u = q_f = \frac{1}{2}(\sigma_1 - \sigma_3)_f$

(4) $s_u = \tau_h$ maximum

Ladd (1971)

Table 11-4