1/16/01

I SOIL COMPOSITION, ETC (4 pages + 3 sheets)

1. Background

1.1 Soil Types

1.2 Particle Size & Shape

2. Soil Composition

2.1 Silicate Frameworks

2.2 Silicate Sheets = Clay Minerals

2.3 Predominate Clay Minerals

3. Electrical Nature of Platy Clay Particles

3.1 Shape & Charges

3.2 Diffuse Double Layer

4 Three Common Clay Minerals

4.1 Na Kaolinite

4.2 Na Illite

4.3 Na Montmorillonite

Sheets A, B & C = Soil Composition & Clay Mineralogy

II CLAY-WATER FORCES (8 pages + 2 sheets)

1. Water Vapor Sorption

1.1 Water Content vs Relative Humidity (RH)

1.2 Capillary Pressure vs Relative Humidity

1.3 Mechanisms of Water Vapor Adsorption

1.4 Measurement of Water Content

1.5 Tensile Strength of Water

II Cont.

2. Soil Suction

2.1 Overview

2.2 Components of Soil Suction

2.3 Mechanisms Causing Matrix Suction

2.4 Direct Lab Measurements of Soil Suction

2.5 Soil Suction Measurement Techniques

3. Nature of Adsorbed Water

3.1 Total vs Pressure Head & Attraction Pressure

3.2 Physical Properties of Adsorbed Water

Sheets A & B = Soil suction measurement techniques

III INTERPARTICLE FORCES: Components & Interaction (4 pages & 3 sheets)

1. Components of Effective Stress

1.1 Physico-Chemical Effective Stress Eqn

1.2 Discussion

2. Particle Interaction

2.1 Energy Diagrams

2.2 Energy Diagram for Hypothetical Contact

2.3 Source of True Cohesion

Sheets A & B = Long range DL forces ; C = hypothetical energy diagram

IV STRENGTH GENERATION IN SOILS (2 pages + 1 sheet)

1. Frictional Resistance

1) Terzaghi-Bowden-Tabor Adhesion Theory

2) Granular Soils

3) Cohesive soils

2. Cohesive Resistance

Sheet A: friction of Quartz

V MECHANISMS CONTROLLING COMPRESSIBILITY OF CLAYS (5 pages)

1. Background

1) Definition

2) Importance & definition of initial fabric — Particle orientation / " distribution

3) Two models of clay compressibility (2 extremes)

- Mechanical = physical interaction with $\sigma' = \bar{F} \cdot a_c$ (all contacts)
- Physico-Chemical = " " " $\sigma' = (R - A)$ (all double layer)

2. Components of Volume Change (Table V)

1) Elastic deformation
2) Change in closest spacing
3) Particle reorientation
4) Crushing

3. Examples of Factors Affecting Initial Fabric

VI SOIL STRUCTURE: EFFECTS OF CLAY TYPE AND ENVIRONMENTAL FACTORS (6 pages + Summary & 8 sheets)

1. Smectite

 1.1 Na Montmorillonite
 1.2 Ca "
 } 1-D & CIUC data

 1.3 CIDC(L) Data with $\sigma_3' = 0$

 1.4 Summary

2. Kaolinite
 2.1 Schematic
 2.2 1-D Compression
 2.3 Strength Data
 2.4 Summary

3. Illite
 3.1 Deposition: Fresh vs Sea Water for Natural Clay
 3.2 1-D Compression: Fractionated Illite
 3.3 Strength Data

4. Summary (Still being prepared)

Sheets: M1-4 for montmorillonite; K1-3 for kaolinite; I1-3 for illite

VII CLASSIFICATION TESTS & RADIOGRAPHY

1) Specific gravity
2) Grain size distribution
3) Atterberg limits
4) Summary plots
5) Plasticity chart
6) Correlations
7) Radiography

I SOIL COMPOSITION, ETC

1.361 References

1. BACKGROUND

II 1-1 & 4

1.1 Soil Types

- Inorganic – Gravel (G), Sand (S) — Granular; Silt (M), Clay (C) — Cohesive
- Organic – Peat, Muck

How distinguish?
1) G vs S
2) gran. vs coh.
3) M vs C

1.2 Δ(Particle Size & Shape) → Δ Soil Types

II 1-1.2

- Decreasing particle size → decreasing k
  (increasing SSA) → increasing max u_c (capillary pressure)
  → " importance surface forces
- Spheres to platy shaped → larger differences in fabric
- Both result from Δ mineral composition

2. SOIL COMPOSITION

II 1-2

2.1 Silicate Frameworks (Sheet A)

1) Quartz Si_4O_8 2) K Feldspar $KAlSi_3O_8$
3) Plagioclase $NaAlSi_3O_8$, $CaAl_2Si_2O_8$ 4) Calcite. 5) Dolomite

} Relative abundance in sand & silt size

Why weather resistant?

2.2 Silicate Sheets = Clay Minerals (Sheets A, B & C)

1) Basic building blocks = $Si_4O_{10}^{-4}$ S   GorB $Al_4(OH)_{12}$ – OHs, $Mg_6(OH)_{12}$ – OHs

2) 2 sheets/layer (7Å) → Kaolinite, Halloysite =

3) 3 sheets/layer (10Å) → Mica, Illite, Montmorillonite, etc =

---

*(A) = Composition of Soil (B) = Data on Common Clay Minerals (C) = "Clay Mineralogy"

1.361 References

2.3 Predominate Clay Minerals (Very general)

1) Cold climates → marine clays from glaciation with illite & chlorite (BBC, quick clays of Canada & Scandanavia)

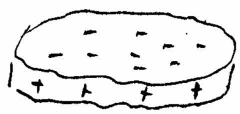
2) Moderate to arid climates → expansive clays with smectite (evap. > rainfall) (S. Africa, S. mid-west US)

3) Wet tropical → residual soils with kaolin, halloysite, Al/Fe oxides (red) (Caribbean islands)

3. ELECTRICAL NATURE OF PLATY CLAY PARTICLES — II 2-1 & 2

3.1 Shape & Charges — II 2-1.2

1) Why negative face charge?



Surface charge density $\sigma_0 \approx \frac{CEC}{SSA} \approx 0.2 \pm 0.1\ C/m^2$
(1-2 charges / 100 Å^2)

Effect of environment on σ_0 ?

2) Edge charge & effect of pH: incr. pH → ________
↑ Usually ___

3) Overall charge = ? Electrophoresis =
Electroosmosis =

pH = $-\log_{10}$ (H^+ conc) : < 7 = acidic (High H^+)
> 7 = basic (high OH^-)

3.2 Diffuse Double Layer (1 Å = 10^{-4} µm) — II 2-2.2

1) Definition

2) Debye thickness (t_D) = distance* between parallel plate condensers having the same σ_0 & electric potential (V) — II 2-2.2 (7a)

$$t_D(\text{Å}) = \frac{0.0199}{v}\sqrt{\frac{D \cdot T}{C_0}}$$

(for $v^+ = v^-$)

v = ion valence T = temp. (°K)
D = dielectric constant = 80 H_2O
20 alcohol
C_0 = conc. cations = conc. anions (for $v^+ = v^-$) in BULK pore water (M = moles/Liter)

* Beyond Stern layer (≈ 5 Å); distance to $c = c_0$ approx. 2-3 × t_D

1.361 Refer.

(3.2 Cont.)

D	$C_0(M)$	$t_D(\text{Å})$ at 20°C (293°K) $\nu=1$	$\nu=2$
80 (H_2O)	5×10^{-5}	430	215
	10^{-4}	305	152
	10^{-3}	96	48
	10^{-2}	30	15
	10^{-1}	10	5
20 (alcohol)	10^{-4}	152	76
	10^{-2}	15	7.5

4. THREE COMMON CLAY MINERALS (sheet C)

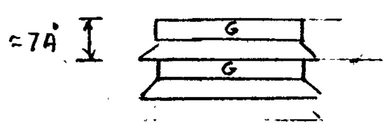
II 1 - 2.6, 2.7

II 2 - 3.2

4.1 Na Kaolinite $(Al)_4[Si]_4O_{10}(OH)_8$

1) Structure

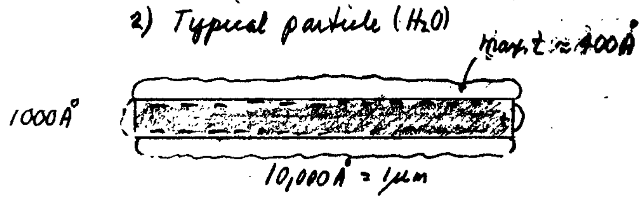
Halloysite =



σ_0 due to:

Bonding due to:

2) Typical particle (H_2O)

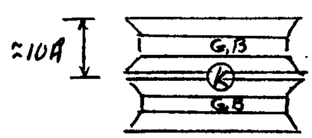


- 140 layers/particle
- SSA ≥ ≈ 10 m²/g
- $\sigma_0 \approx 0.3\ C/m^2$
- EDGE effects

4.2 Na Illite $K(Al,Mg,Fe)_{4or6}[Al,Si]_8O_{20}(OH)_4$

1) Structure

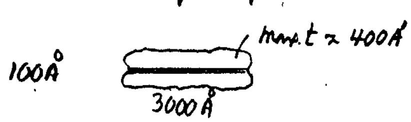
Mica =



σ_0 due to:

Bonding due to:

2) Typical particle (H_2O)

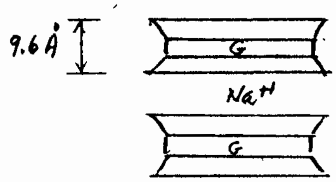


- 10 layers/particle
- SSA ≈ 80 m²/g
- $\sigma_0 \approx 0.3\ C/m^2$
- ∴ EDGE + DL effects

Dia $H_2O \approx 3\text{Å} = 3\times10^{-4}\mu m = 0.3\mu m$

4.3 Na Montmorillonite $(Al_{3.3}\,Mg_{0.7})\,[Si]_8\,O_{20}\,(OH)_4$

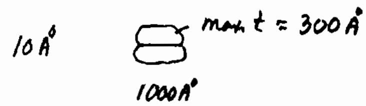
1) Structure



σ_0 due to:

Bonding due to:

2) Typical particle (H_2O)



- 1 layer / particle
- SSA ≈ 800 m^2/g
- $\sigma_0 \approx 0.1\ C/m^2$
- DL effects

NOTE: Part V will discuss how Δ pore fluid composition changes engr. properties of these 3 clay minerals (K, I & M)

1.322 Part AI

1.361-1.366 Part II-1

Data on Common Clay Minerals

1.322 Soil Technology

Dept. of Civil Engineering
M.I.T.

CCL & RTM
9/65
9/95

Mineral	Unit Cell Formula	Structural Symbol	Isomorphous Substitution	Charge Density (1) (meq/100g)	Interlayer Bonding	Typical Particle Shape	Typical Particle Size	CEC (2) ($\frac{meq}{100g}$)	SSA (3) ($\frac{m^2}{g}$)	σ (4) ($\frac{charge}{100Å^2}$)	Var. d(001)
Kaolinite	$(Al)_4[Si]_4O_{10}(OH)_8$	G G	Al for Si Mg for Al (~1 in 400)	~2	Secondary valence + H-bonding Strong	Hexagonal sheets	$d=3\text{-}10\mu$ $t=\frac{1}{3}\text{-}\frac{1}{10}d$	~3	10-15	1.2	No
Hydrated Halloysite Dehydrated	$(Al)_4[Si]_4O_{10}(OH)_8 \cdot nH_2O$ n=4 n=0	H_2O G G	As above	~8	As above except weak if larger spacing	Hollow tubes	$OD=.07\mu$ $ID=.04\mu$ $l \sim .5\mu$	~12	30-50	1.8	Yes No
Muscovite (Mica)	$K_2(Al)_4[Al_2Si_6]O_{20}(OH)_4$	K G G	Al for Si (~1 in 4) maybe Mg, Fe for Al	250	K-bonding + sec. vol. Very strong	Platy	Very large	3-10	1-10	~2.2	No
Vermiculite	$(Mg,Al,Fe)_{6\ or\ 4}-[Si,Al]_8O_{20}(OH)_4 \cdot nH_2O$	H_2O Mg, Ca G or B G or B	Mainly Al for Si (~1 in 8) Also Mg, Fe for Al Al, Fe for Mg	150±20	Weak primary val. (Ca, Mg) + sec. vol. Weak	Sheets	Variable	150 ±20	500-700 when expanded	2.2	Yes
Illite (Hydrous mica)	$K(Al,Mg,Fe)_{4\ or\ 6}-[Si,Al]_8O_{20}(OH)_4$	K G or B G or B	Mainly Al for Si (1 in 6-8) Also Mg, Fe for Al Al, Fe for Mg	~150	K-bonding + sec. vol. Fairly strong	Flakes	$d=.1\text{-}2\mu$ $t\leq\frac{1}{10}d$	25 ±5	80-100	1.5	No
Sodium Montmorillonite	$(Al_{3.3}Mg_{0.7})[Si]_8O_{20}(OH)_4$	G G	Mainly Mg for Al (~1 in 6)	100	Weak secondary valence	Sheets	$d=.1\text{-}1\mu$ $t\leq\frac{1}{100}d$	95 ±10	700-800	0.75	Yes
Chlorite	$(Mg,Al,Fe)_{4\ or\ 6}[Si,Al]_8-O_{20}(OH)_4\cdot(Mg,Al,Fe)_6(OH)_{12}$	G or B B+ G or B	Al for Si Fe, Mg for Al Fe, Al for Mg in B^+	200-250	Primary valence via B^+ sheet Very strong	Platy	Variable	2-40	5-30	—	No

(1) Charge density = $\frac{\text{charge/formula} \times 10^5}{\text{formula weight}}$

Mineral	Formula Wgt.
Kaolinite	517
Talc	750
Pyrophyllite	710

(2) CEC = cation exchange capacity
(3) SSA = specific surface area
(4) σ = surface charge density

- Octahedral & Tetrahedral Sheets
- Unit Layer = 2 or 3 sheets
- Particle = Σ Layers

B

Composition of Soil

1.322 Soil Technology
Dept. of Civil Engr. - M.I.T.

CCL & RTM 9/65

CCL 9/95

- **SOIL**
  - **ORGANIC**
  - **INORGANIC**
    - **AMORPHOUS**
      - Opal
      - Hydrous oxides of Si, Fe, Al
    - **CRYSTALLINE**
      - **CARBONATES**
        - (4) Calcite - $CaCO_3$
        - (5) Dolomite - $CaMg(CO_3)_2$
      - **OXIDES**
        - (14) Rutile - TiO_2
        - (15) Magnetite - Fe_3O_4
      - **HYDROUS OXIDES**
        - Goethite
        - Gibbsite - $Al_4(OH)_{12}$
        - Brucite - $Mg_6(OH)_{12}$
      - **PHOSPHATES**
        - (20) Apatite - $Ca_5(F,Cl,OH)(PO_4)_3$
      - **SILICATES**
        - **INDEPENDENT TETRAHEDRONS**
          - (11) Olivine - $(Mg,Fe)_2SiO_4$
          - (12) Sillimanite - Al_2SiO_5
          - (13) Zircon - $ZrSiO_4$
          - (17) Staurolite
          - (18) Garnet
          - (19) Sphene - $CaTiSiO_5$
        - **DOUBLE TETRAHEDRONS**
          - Lawsonite
        - **RINGS**
          - (16) Tourmaline
        - **CHAINS**
          - Pyroxene (single)
            - (10) Augite
          - Amphibole (double)
            - (9) Hornblende
          - Palygorskite (Attapulgite)
        - **SHEETS**
          - Two Sheet
            - Serpentine - $Mg_6Si_4O_{10}(OH)_8$
            - Kaolinite - $Al_4Si_4O_{10}(OH)_8$
            - Halloysite -
          - Three Sheet
            - Talc - $Mg_6Si_8O_{20}(OH)_4$
            - Pyrophyllite - $Al_4Si_8O_{20}(OH)_4$
            - (6) Muscovite - $K_2Al_4[Al_2Si_6]O_{20}(OH)_4$
            - (7) Biotite
            - Vermiculite
            - Illite - $K(Al,Mg,Fe)_{4 \text{ or } 6}[Si,Al]_8O_{20}(OH)_4$
            - Montmorillonite - $(Al_{3.3}Mg_{0.7})[Si]_8O_{20}(OH)_4$ ← → Smectite
            - (8) Chlorite
        - **FRAMEWORKS**
          - (1) Quartz - Si_4O_8
          - (2) K Feldspar - $KAlSi_3O_8$
          - (3) Plagioclase - $NaAlSi_3O_8 \rightarrow CaAl_2Si_2O_8$

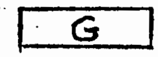
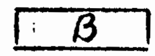
(3) Numbers refer to order of abundance in sands and silts (Clays & Clay Technology, Bulletin 169, 1955)

A

CCL 9/15/83 9/86
1.322 2/89 2/96

CLAY MINERALOGY

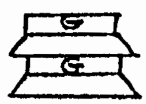
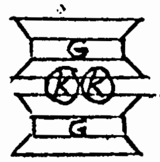
"BUILDING BLOCKS" = SHEETS COMPOSING THE LAYERS

	· Silica Tatrohedra	$Si_4O_{10}^{-4}$		} sheets
Hydrous Oxides	· Gibbsite	$Al_4(OH)_{12}$ – OH's		
	· Brucite	$Mg_6(OH)_{12}$ – OH's		

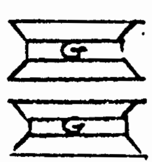
Which sheets combined in layer + nature of "glue" holding layers together to form particles → different clay minerals

CLAY MINERALS OF PRIME INTEREST

($1\,Å = 10^{-8}cm = 10^{-10}m = 10^{-4}\mu m$)

Name	Symbol	Glue Between Layers	Remarks
(2S) Kaolinite		Hydrogen, van der Waals } strong	· Large hexagonal · $t \approx 1000\,Å$ · $SSA \geq 10\,m^2/g$
(2S) Halloysite	Above w/ H_2O between some layers.	Much weaker	· Hollow tubes → low γ_d · Drying → loss of bonded H_2O
(3S) Muscovite		Very strong K	· Very large particles · Reference for unit layer = 3 sheets

{ Isomorphous substitution: Cations of lower valance substituted during formation → net negative charge within layers. }

(3S) Illite	As above but: · G or B · Less I.S.	Weaker K	· Flaky particles · $t \approx 100\,Å$ · $SSA \approx 80\,m^2/g$ · Marine clays
(3S) Smectite = Montmorillonite		If Na – then no bonding (Bantonite)	· Flaky particles · $t \to 10\,Å$ (1 layer) · $SSA \to 800\,m^2/g$ · Axle grease

(Problem soil → expansive clays, very low residual ϕ', etc.)

+Sheets A&B

II CLAY-WATER FORCES

(1.361 Refer.)

1. WATER VAPOR SORPTION ("Adsorbed" Water) (II 2-2.1)

1.1 Water Content vs. Relative Humidity (RH)

1) Relative humidity, RH(%)

$$RH(\%) = 100 \times \frac{P_w}{P_s}$$

P_w = partial pressure of (pore) water vapor

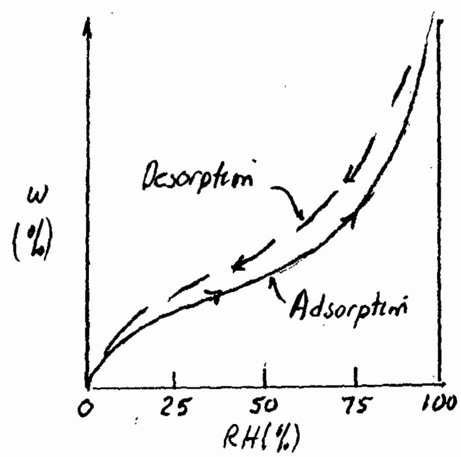
P_s = saturation pressure of pure H_2O (flat surface) at same temp.

[$\log P_s(kPa) \approx -0.174 + 0.0265\,T(°C)$; $T = 20 \pm 15°C$]

{ Actual $T =$ 10 20 30°C
$P_s =$ 1.23 2.34 4.245 kPa }

Table 2.7 Fredlund & Rahardjo (1993) Soil Mech. for Unsaturated Soils, Wiley textbook

2) Adsorption - desorption curves (starting from oven dry soil)



3) Values of water content

a) RH = 50% → 1-2 molecular thicknesses of water : $t \approx 5Å$ $w_{50}(\%) = 0.05 \times SSA(m^2/g)$

$\phi_{H_2O} = 3Å$

{ $t(Å) = 100\,w(\%)/SSA(m^2/g)$ }

• Empirical data from Ladd / Lambe (1961)

$w_{50}(\%) \approx (0.2 \pm 0.05)\,I_p(\%)$

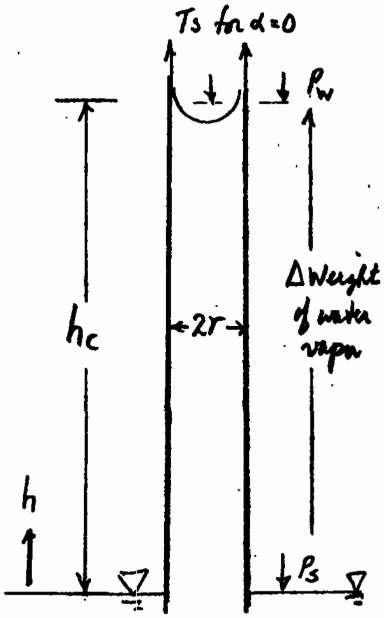
b) RH ≈ 99% → $t = 10\text{-}15Å \approx$ 3-5 molecular thicknesses

Often referred to as thickness of tightly bound "adsorbed" water

(1.361 Refs.)
(II 2-2.1; IV 1-2)

1.2 Capillary Pressure (u_c) vs. Relative Humidity

1) Theoretical relationship



$pV = nR_gT$
+
$\frac{dP}{dh} = -\rho_{wv} \cdot g$
↑ mass density of water vapor

• 1.361 Eqn. $u_c = h_c \cdot \gamma_w = \frac{2T_s}{r}$, T_s = surface tension of H_2O = 72.75 N·m/m² at 20°C (energy/unit area)

$u_c = u_a - u_w$

• Revised eqn. to include reduced vapor pressure of curved miniscus & wgt water vapor

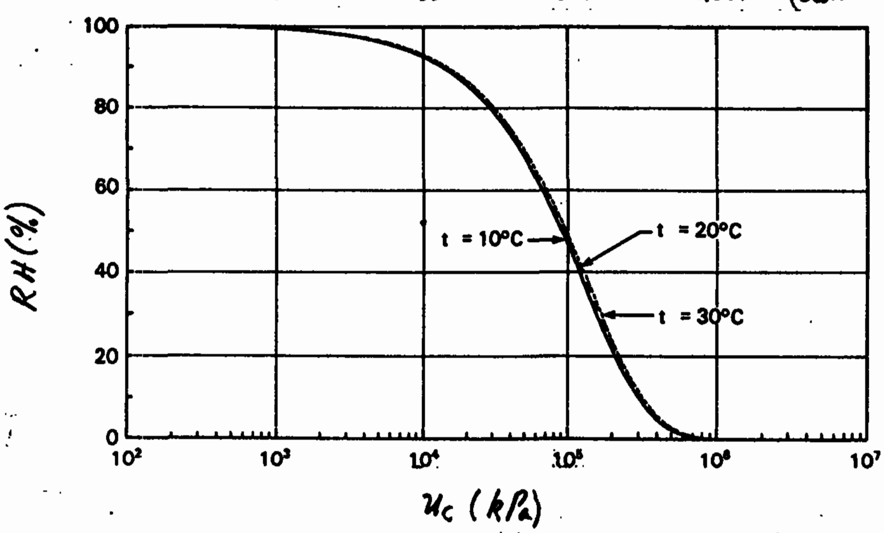
$$\underbrace{h_c \cdot \gamma_w}_{\text{Force} \downarrow} = \underbrace{\frac{2T_s}{r} + P_s - P_w}_{\text{Force} \uparrow} \quad \Big\}\ \text{At bottom of tube}$$

• $u_c = h_c \gamma_w = -\frac{\rho_w R_g T}{M} \ln\left(\frac{RH}{100}\right)$

ρ_w = 998 kg/m³
R_g = 8.314 J/mol·°K
T = °K
M = 18.015 g/mol
(J = N·m)

• For T=20°C $u_c\ (kPa) = -1.35 \times 10^5 \ln\left(\frac{RH}{100}\right)$
$u_c\ (bar) = -1350 \ln\left(\frac{RH}{100}\right)$

≈ 3	4	5	6	7	8	pF
1	10	100	1000	10,000	(bar)	



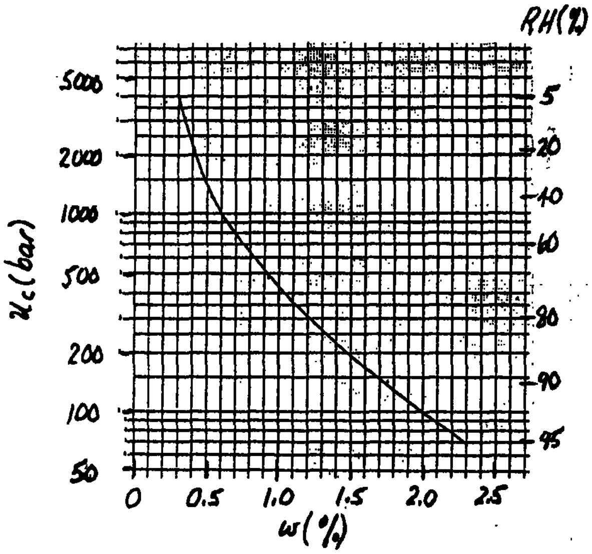
$pF = \log(h_c \cdot cm)$

h_c = 1000 cm = 10 m
→ pF = 3.00 ≈ 1 atm
≈ 1 bar = 100 kPa

• 1 bar = 100 kPa ÷ γ_w = 9.81 kN/m³ = 10.2 m (= 1020 cm)
• 1 atm = 1.013 bar

RH(%) =	99.9	99.5	99	95	90
u_c(bar) =	1.35	6.77	13.5⁵	69	142
pF =	3.1⁵		4.1⁵		5.1⁵

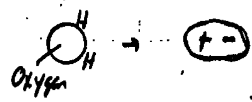
(1.2 Cont) 2) Water content vs capillary pressure (1.361 Ref)



Computed from w vs RH data on Peerless (Li) Kaolinite with
CEC = 2.7 meq./100g (SSA = 9 m²/g for σ_0 = 0.3)
Adsorption curve for RH = 0 to 95%
{ W vs RH data by R.T. Martin, 1958, Proc. 5th Nat. Conf. Clays & Clay Mineralogy NAS/NRC Publ. 566, p 23-38 }

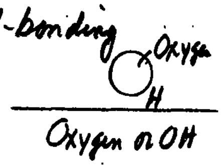
1.3 <u>Mechanisms of Water Vapor Adsorption</u> (CCL + Mitchell (1993), Chap6)

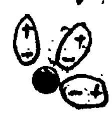
Note: H_2O molecule = dipole

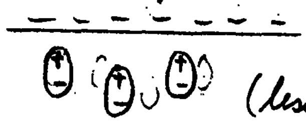


Starting from ovendry soil

<u>Mechanism</u>	<u>Remarks</u> (II 2-2, p6)
1) H-bonding	Very important for 1st layer
2) Cation hydration	Very important ; Hydrated dia. of cations ≈ 15 ± 5 Å
3) Orientation of H_2O dipole in electric field	Uncertain importance
4) van der Waals attraction	" "







(less mobile → less free energy)

Cation	K^{+1}	Na^{+1}	Ca^{+2}	Mg^{+2}	Li^{+1}
Hydrated Dia (Å)	9±2.5	13.5±2	19	21.5	17.5±2.5

(JKM 1993, p122)

(1.361 Ref)

1.4 Measurement of Water Content

1) ASTM 2216 $T = 110 \pm 5°C$ really doesn't remove all H_2O; need more like 200°C
Is that of practical significance? ____

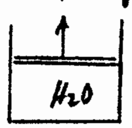
2) Don't leave soil in humid environment before weighing after remove from oven

1.5 Tensile Strength of Water [Ridley & Burland 1993, geot 43(2)]

(Can $u_c \to$ 10's to 100's atm. in cohesive soils? ____)

1) Tabor (1979): theoretical tensile strength, $u_w = -5000$ atm.
However, very small amount of dissolved gas → cavitation at
$u_w >>$ theoretical value ($u_c << 5000$ atm). Typical lab test → ? ____

2) Temperley & Chambers (1946 Proc. Royal Soc., London)



Carefully degassed, smooth-walled chamber → $u_w \approx -500$ atm

3) Conclusion: answer to ( ) is YES, e.g. drying of initially sat. clay to the shrinkage limit
(see p8)

2. SOIL SUCTION (S) [Also see 1.361 Section 2.6 of Part IV-1]

2.1 Overview

- Used as a measure of the "free energy" of the bulk pore water in soil*
(at constant elevation) relative to pure water at atm. pressure
at same temperature when this free energy is less than zero.

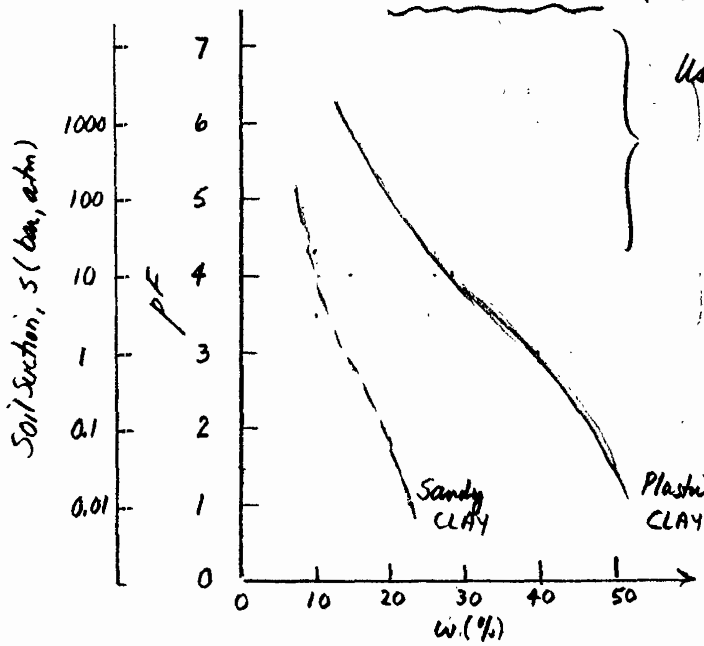
- Free energy can be expressed in terms of: potential - J/kg
pressure - kPa } most common
head - m }

* Questions: (1) Restricted to soil with $\sigma - u_a = 0$?
(2) Physical meaning when low % sat. → no bulk water?

CCL tentative definition
S = free energy of pure H_2O in contact with soil having $\sigma - u_a = 0$

2.1 Cont

Soil suction curves (location = f wetting vs drying)



Usually indirect, e.g. relative humidity (psychrometer) or filter paper

(CCL believes that most data are not very reliable)

Various direct measurements,
e.g. • suction plate ($u_a = 0$)
• pressure plate ($u_c = u_a - u_w$)
• tensiometer = piezometer with v fine porous stone à la Section 2.4

Note: Most reported data are for drying; will show wetting/drying cycles when cover compacted soils. Also usually use θ_V = volumetric water content = V_w/V (p8)

2.2 Components of Soil Suction (Aitchison & Richards, 1965) (1.361 II 2, p8)

1) Total suction (s) = matric suction (s_m) + solute suction (s_s)

2) Solute suction (s_s) = osmotic pressure due to salt concentration in bulk pore water of soil

s_s (atm) ≈ 24 (salt conc., M = moles/l) for both cations & anions) for $T \approx 20°C$

(Sea water ≈ 35 g/l → ≈1.1M → ≈ 26 atm!)

matric = matrix

3) Matric suction (s_m) = ($u_a - u_w$), where pore water in measurement system has same salt conc. as bulk pore water in soil

NOTE: In CCL opinion, matric suction denotes s_m for soil having $\sigma - u_a = 0$. (at least via techniques used in lab to measure s_m). We should discuss difference between s_m & σ'

2.3 Mechanisms Causing Matric Suction

1) Those for water vapor sorption in Section 1.3 (i.e., for "adsorbed" H_2O)

need $2d > 10$-$15Å$

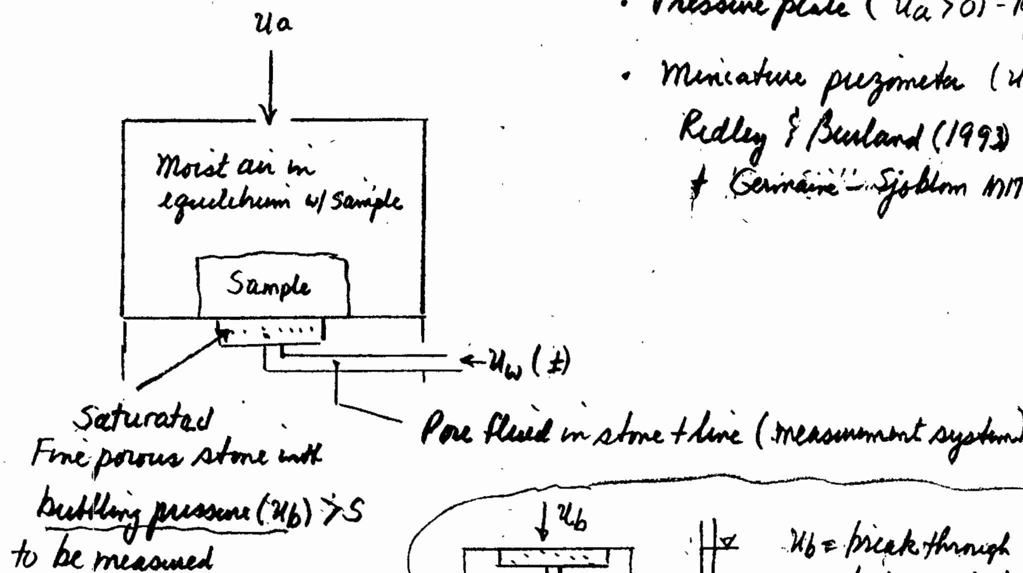
2) Osmotic pressure due to depressed double layer, i.e., need apply neg. u_w to prevent soil from sucking in more water ($\neq$ solute suction)

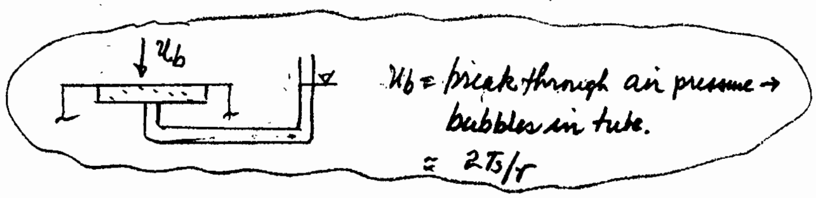
3) Elastic deformation of platy particles (mechanical) drying → bending, wetting → unbending

4) Net contact stress ($\bar{\sigma}''_{ac}$) > 0

2.4 Direct Lab Measurements of Soil Suction (Sheet A)

1) Experimental set-up for: . Suction plate ($u_a = 0$) "axis translation"
- Pressure plate ($u_a > 0$) - Requires continuous air voids
- Miniature piezometer ($u_a = 0$) à la Ridley & Burland (1993) with $u_b = 15$ bar & ~~Germaine~~ Sjoblom MIT research





2) Test conditions & measured soil suction $= (u_a - u_w)$

Sample	Case	System Pore Fluid	Does Stone Act as Semi-Perm. Membrane?	Meas. Soil Suction	Remarks
Clay with salt in bulk pore fluid	1	Same as soil pore fluid	Both Yes & No, i.e. makes no difference	Matric	—
"	2a	Pure H_2O	Yes	Total	**
	2b	"	No	Matric	**

** Whether or not 15 bar stone will ~~exhibit~~ a significant "osmotic efficiency" (i.e., act as partial semi-permeable membrane) is somewhat controversial. For example, discussion[(1)] to Ridley & Burland concludes that 15 bar stone approaches total suction, whereas authors disagree. Kurt Sjoblom's (9/00) data indicate that measure matric suction

(1) Geotechnique 1994, 44(3), 551-556

2.5 Soil Suction Measurement Techniques

1) See Sheet A for overview & schematic of direct measurements

2) " " B for info. on filter paper technique *

3) See Stannard [1992: ASTM, GTJ, 15(1)] for SOA paper on field measurement technique

* Note that calibration curve becomes very flat (hence very imprecise) at low suctions. Also exact details of "contact" vs "noncontact" very important, plus may have different calibration curves for these 2 conditions (R&B, 1994 closure)

3. NATURE OF ADSORBED WATER

3.1 Total vs. Pressure Head and Attraction Pressure

1) Components of total head (total free energy)

$$\text{Total head } h = h_t = h_e + \underset{\substack{\| \\ -\frac{s_m}{\gamma_w}}}{h_p} + \overset{=\frac{s_s}{\gamma_w}}{h_s} + \overset{=\frac{1}{2}\frac{V^2}{g}}{h_v} + h_{pc}$$

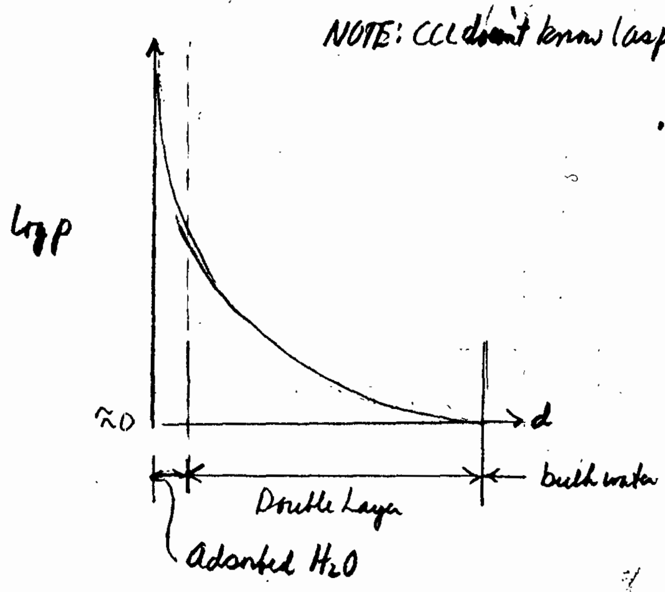
h_{pc} = physico-chemical within double layer (DL)

$h_p = -\frac{s_m}{\gamma_w}$ for bulk water beyond DL at $h_e = h_s = h_v = 0$

(JKM 1993 Section 9.5)

2) Attraction pressure (p) vs distance (d) from particle surface

NOTE: CCL doesn't know (as per writing) shape of curve



- Discussion of actual pressure in H_2O → can't have very high s_m, but still have large +p in adsorbed water

- Values of p within adsorbed H_2O

  $d = 10\text{-}15\,Å \rightarrow$ 10's atm

  $d = 3\text{-}6\,Å \rightarrow$ 100's-1000's atm.

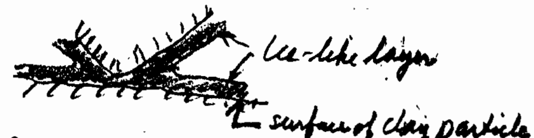
3) Discussion: what is meaning of h_t, h_p, s_m etc. when low % sat. → no bulk water?

(1.361 Ref)

(II 2, p6)

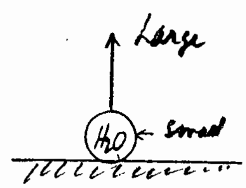
3.2 Physical Properties of Adsorbed Water

1) Is "ice-like"



- Very high viscosity
- Causes cohesion & creep (e.g., Terzaghi, Bjerrum 1973)
- Don't get mineral-mineral contacts (or greatly inhibits) in clay

2) Like 2-D liquid (R.T. Martin)

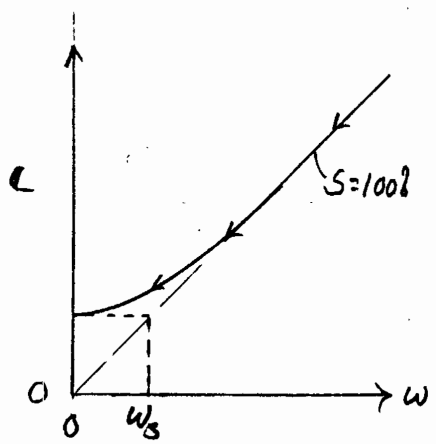


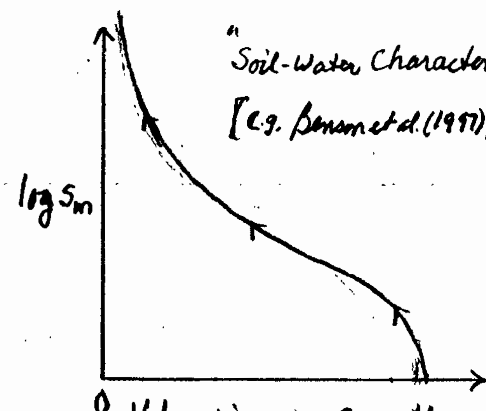
- Like ball bearings on magnet
- Does not cause strength, but does inhibit min.-min. contact

3) Discussion -

- Story of discussion at 1961 ICSMFE, Paris (Geuze of Holland: "adsorbed H_2O → clay strength → creep." TWL reply =

- Predicted unconfined s_u of initially saturated clay during drying.





"Soil-Water Characteristic Curve"
[e.g. Benson et al. (1997), JGGE, 123(11), 1060-]

Volumetric w.c. $= \Theta_w = \dfrac{V_w}{V} = \dfrac{SV_v}{V} = S \cdot n = \dfrac{Se}{1+e} = \dfrac{G_s w}{1+G_s w/S}$

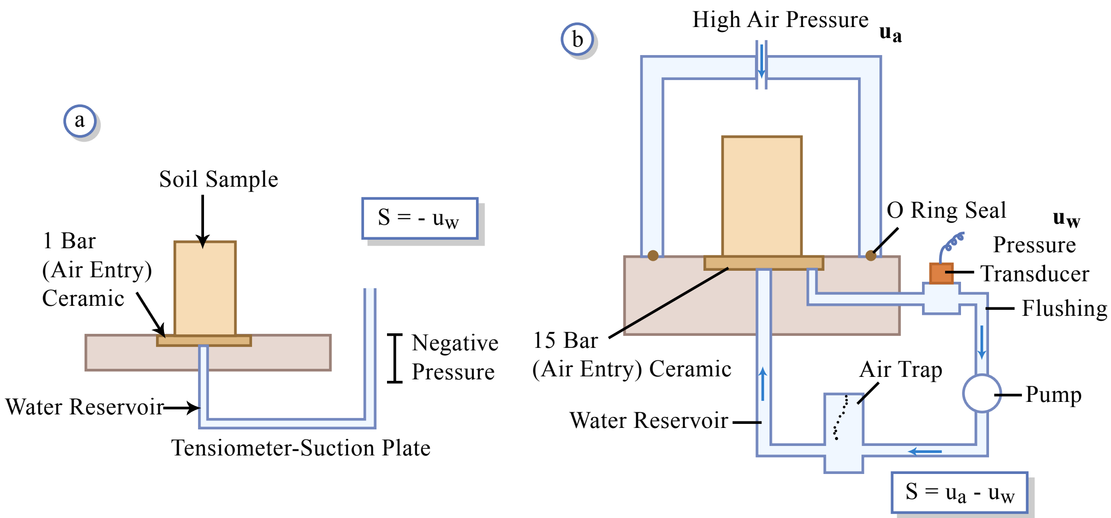
- From 1.361
  - Strength of oven-dried BBC & what happened when put in water
  - Part IV-1, p5 → $\sigma'_{nt} \approx 70$ bar at shrinkage limit of BBC

Adapted from RIDLEY AND BURLAND (1993)

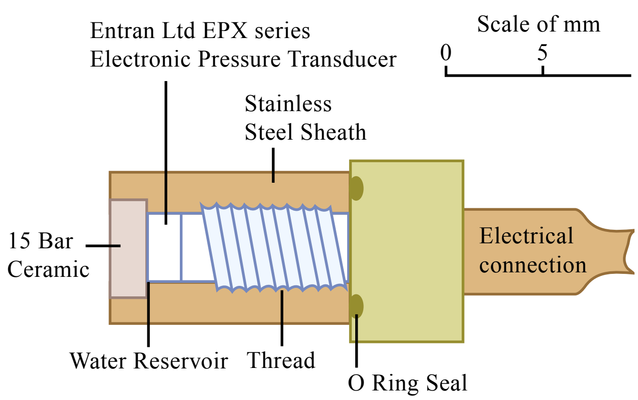
Suction Measurement Techniques

	Suction Value#	Principal Usage	Direct/ Indirect	Range: kPa	Equilibrium Time
Vacuum Desiccator	Total	Lab.	Indirect	10^3-10^6	Months
Psychrometer	Total	Field	Indirect	300-7000	Months
Filter Paper	Total	Field	Indirect	1000-30000	Weeks
	Matrix	Lab.	Indirect	30-30000	1 week
Porous Block	Matrix	Field	Indirect	30-3000	Weeks
Thermal Block	Matrix	Field	Indirect	0-175	Days
Suction Plate	Matrix	Lab.	Direct	0-90	Hours
Tensiometer	Matrix	Field	Direct	0-90	Hours
Pressure Plate	Matrix	Lab.	Direct	0-5000	Hours
Osmotic Tensiometer	Matrix	Field	Direct	0-1500	Days

As defined by Aitchison and Richards (1965).



Direct Measurement of Soil Suction: a) Tensiometer; b) Pressure Plate Apparatus



The New Instrument

Figure by MIT OCW.

Adapted from:

Geotechnique (1993) 43(2), 321-324

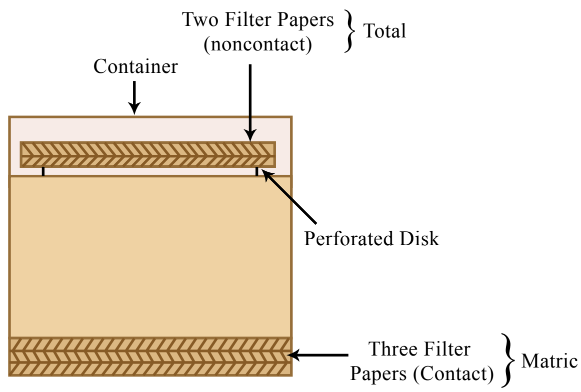
Also see discussion by Marinho & Chandler (1994) & closure by Ridley & Burland - Geot 44(3), 551-556

CCL 2/6/96
1/31/99

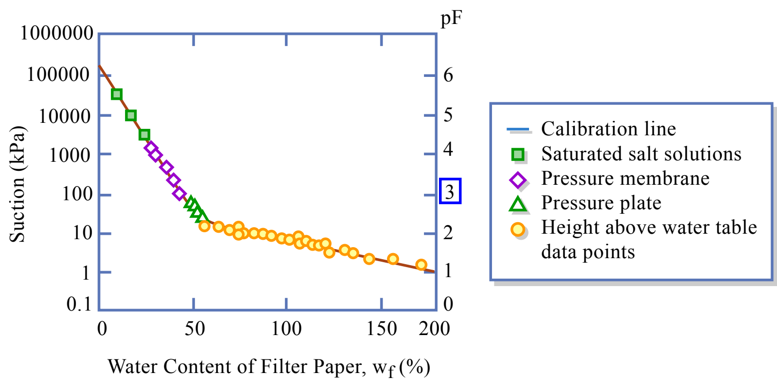
1.322 Part A-II

Adapted from Fredlund & Rahardjo (1993) Soil Mechanics for Unsaturated Soils, John Wiley & Sons

Filter paper technique

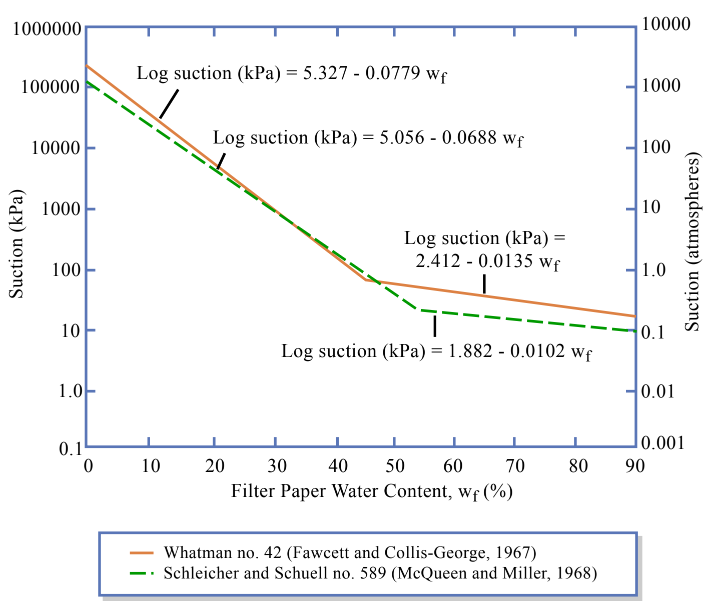


Contact and noncontact filter paper methods for measuring matric and total suction, respectively.



A typical calibration curve showing measured filter paper water contents for applied suctions.

Figure by MIT OCW.



Calibration Curves for Two Types of Filter Paper

Figure by MIT OCW.

B

+Sheets A, B & C

III INTERPARTICLE FORCES: Components and Interaction

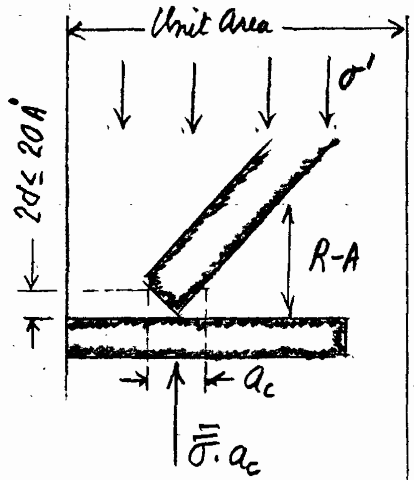
(1.361 Reference)

1 COMPONENTS OF EFFECTIVE STRESS

(II 2-25)

1.1 Physico-Chemical Effective Stress Eqn (Ladd 1961)

1) Objective - general idea of how σ' is transmitted between particles in cohesive soil.



2) Assume "contact" forces act when $2d \leq 20\text{Å}$ (rather arbitrary) over area ratio a_c & long range double layer type forces act at $2d > 20\text{Å}$

3) Eqn: σ' = net contact stress + net long range stress

$$= \bar{\bar{\sigma}} \cdot a_c \quad + \quad R - A$$

$$= (\bar{\bar{\sigma}}_r - \bar{\bar{\sigma}}_a)\, a_c \quad + \quad R - A$$

4) Long range stresses ($2d > 20\text{Å}$)

- Double layer (osmotic) repulsion $R = f(P_r)$ - Sheets A & B
- Long range van der Waals attraction $A = f(P_a)$

$$P_a \text{(parallel particles)} = \frac{A''}{48\pi}\left(\frac{1}{d^3} + \frac{1}{(d+\delta)^3} - \frac{2}{(d+\frac{\delta}{2})^3}\right)$$

A'' = Hamaker constant $\approx 2 \pm 0.5 \times 10^{-20}$ J (JKM '93, p 124)
δ = particle thickness
d = half spacing between particles

$\therefore A \propto 1/d^3$, increases w/ δ & thought to be indep. of pore fluid

5) Contact stresses ($2d < 20$ Å)

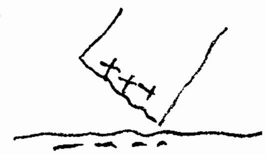
Repulsion $\bar{\bar{\sigma}}_r$ = displacement of "adsorbed" water (pore fluid when not H_2O)

+ Born repulsion of mineral contact

(+ edge to face repulsion for neg. edge charge)

Attractive $\bar{\bar{\sigma}}_a$ = short range van der Waals

+ edge-to face electrostatic attraction (for +edge & neg. face)



+ primary valence bonding of mineral contact (ionic & covalent)

+ cementation (like carbonates, iron oxides..)

} put together

1.2 Discussion

1) Components for granular soils

• Is there a DL?   • $\sigma' = \bar{\bar{\sigma}} \cdot a_c$ (10,000s atm)

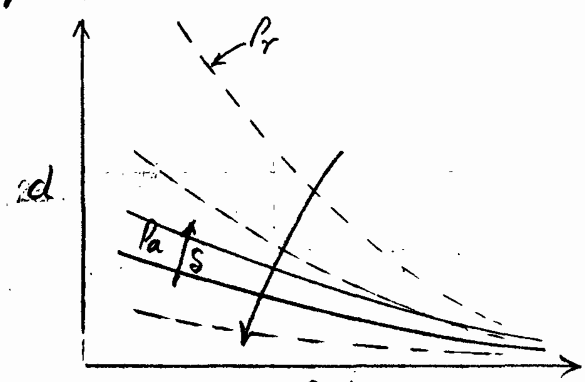
2) Effects of pore fluid on R-A for cohesive soils with "high" SSA

Decreasing $R = f(P_r)$ for

(1) EC valence ν

(2) Bulk c_0

(3) Dielectric const. D



• For clay in sea water (35 g/l ≈ 1.1M → c_0 = 0.6M): $R \gtrless A$?

• For clay in alcohol (D = 20 vs 80 for H_2O): $R \gtrless A$?

3) Effects of pore fluid on $\bar{\bar{\sigma}} \cdot a_c$ for cohesive soils

What trends for

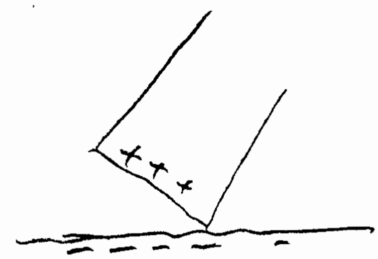
(1) Incr. pH

(2) Incr. c_0

(3) V. high anion valence

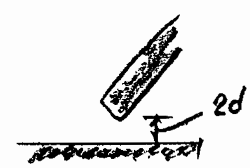
(4) Decr. D

$\bar{\bar{\sigma}}_r$	$\bar{\bar{\sigma}}_a$
Adsorbed H_2O (Fluid)	vdWaals
Born	E/F +/-
	prim. val. bonding

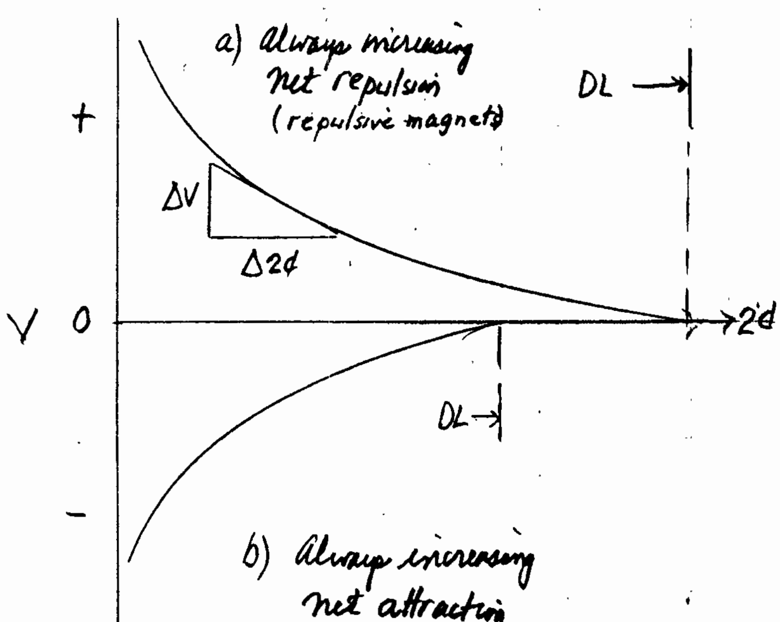


2. PARTICLE INTERACTION

2.1 Energy Diagrams



V = energy/unit area to change minimum distance (2d) between two particles



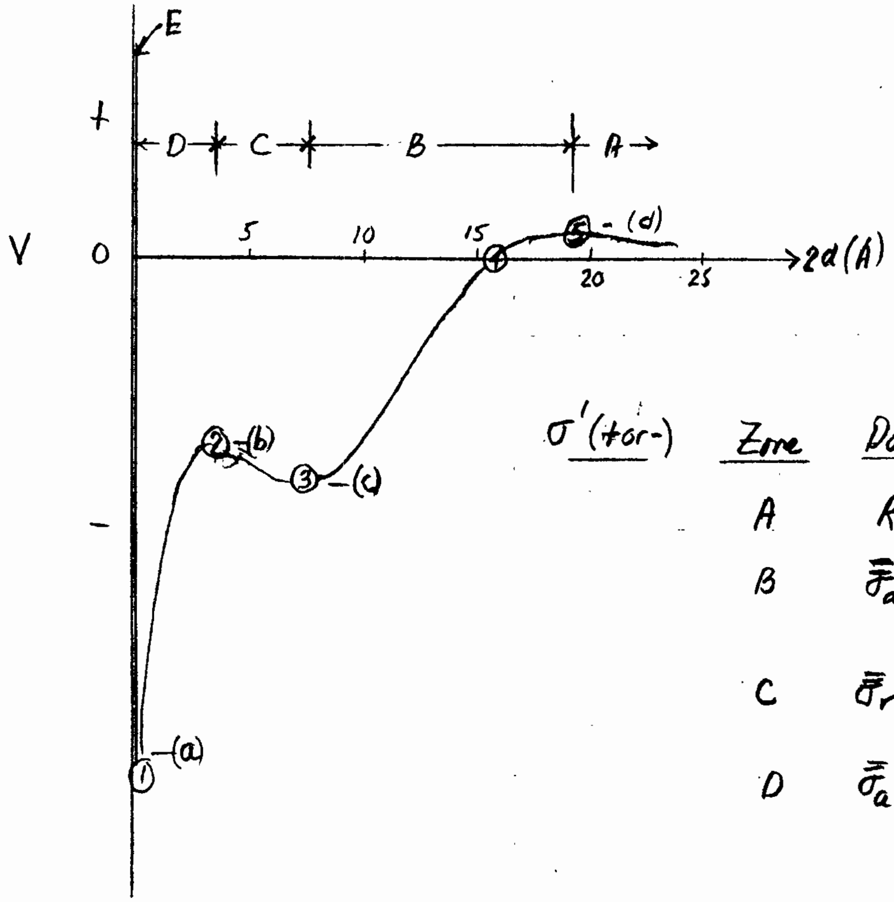
Net stress = $-\Delta V / \Delta 2d$

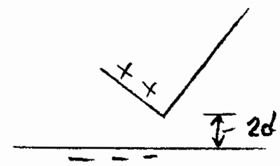
Need to apply energy to DECREASE spacing

Need to apply energy to INCREASE spacing

2.2 Energy Diagram for Hypothecial Contact (Sheet C)

1) Overview for illitic clay at low salt conc.





↑ Energy into form contact

↓ Energy in to break contact

σ' (+or−)	Zone	Dominant Component
	A	R = osmotic repulsion
	B	$\bar{\bar{\sigma}}_a$ due to electrostatic + to − attraction & van der Waals
	C	$\bar{\bar{\sigma}}_r$ due to adsorbed H_2O (fluid)
	D	$\bar{\bar{\sigma}}_a$ due to v.d. Waals & then primary valence
	E	$\bar{\bar{\sigma}}_r$ due to Born repulsion

2) Significance of numbered pts

② & ⑤ Metastable

$\sigma' =$, but slight $\Delta V \rightarrow$

① & ③ Energy sink

$\sigma' =$, but need significant $+\Delta V \rightarrow$

2.3 Source of True Cohesion

Must put energy into system to increase spacing

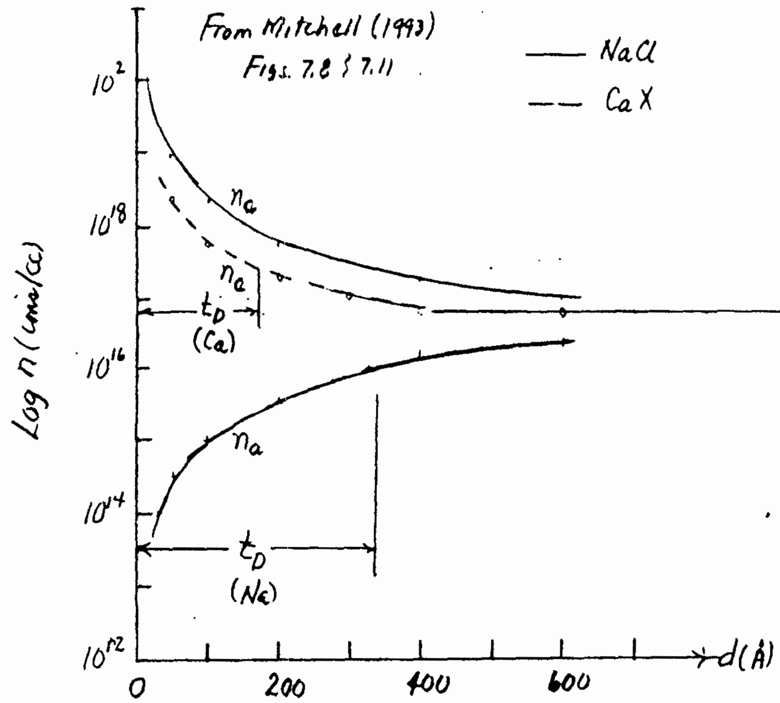
① → ② $\Delta V = b - a$

③ → ⑤ $\Delta V = d - c$

9/97

Supplement on Double Layer Repulsion

Single Double Layer: Ion conc vs. distance — Na & Ca Montmorillonite

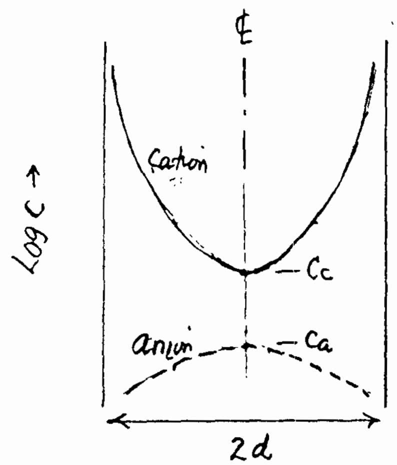


t_D = Debye thickness
= 333 Å Na
= 167 Å Ca

$C_o = 0.83 \times 10^{-4}$ M
$\times 6 \times 10^{23} \times \frac{1}{1000}$
$= 5 \times 10^{16}$

M = moles/l
= 6×10^{23} anions & cations

Interacting Double Layer (valence $v = v_c = v_a$; C_o = bulk concentration, M = moles/l of anions + cations)



$$P_r = R_g T\,(\underbrace{C_c + C_a}_{\text{Mid-plane}} - \underbrace{2C_o}_{\text{Bulk}})$$

$T = 273 + °C$

$R_g = 8.314 \frac{J}{mol \cdot °K}$

C = M, moles/l

$$\boxed{P_r(\text{bar}) = 24.37\,(C_c + C_a - 2C_o) \quad \text{at } 20°C,\ D = 80 \text{ for } H_2O}$$

C_o (M)	v	2d (Å)	P_r (bar ≈ atm)	
10^{-3}	1	200	0.2	$C_o \to 0.06$ g/l NaCl
		100	0.8	
		50	3.3	
		25	12	
10^{-3}	2	100	0.2	≈ ½ spacing
		50	0.8	
		25	3.3	
0.1	1	50	0.6	Large decr. at larger spacings
		25	7	
	2	25	0.6	

Fig 2-1
1.361 Part II-2

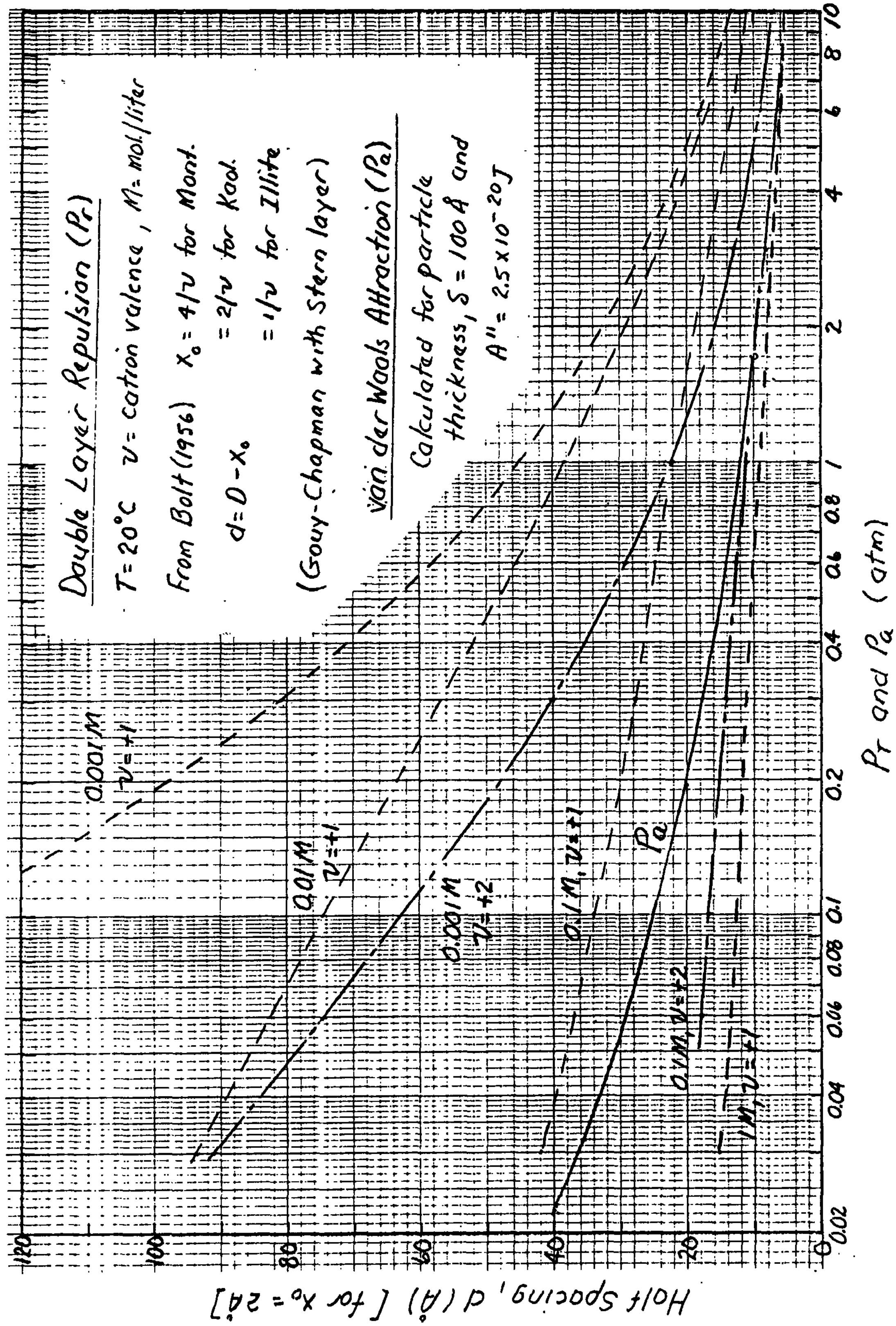


Fig. II2-1 Double Layer (Osmotic) Repulsion and van der Waals Attraction vs. Half Spacing for Parallel Infinite Plates (after Ladd, 1961 ScD Thesis)

CCL 9/12/95

(B)

CCL 2/11/96
2/99

1.322 Part A-III

Adapted from (Ladd & Kinner 1967)

Hypothetical Relationship of Energy vs. Interparticle Spacing
(Illitic clay at low salt concentration)

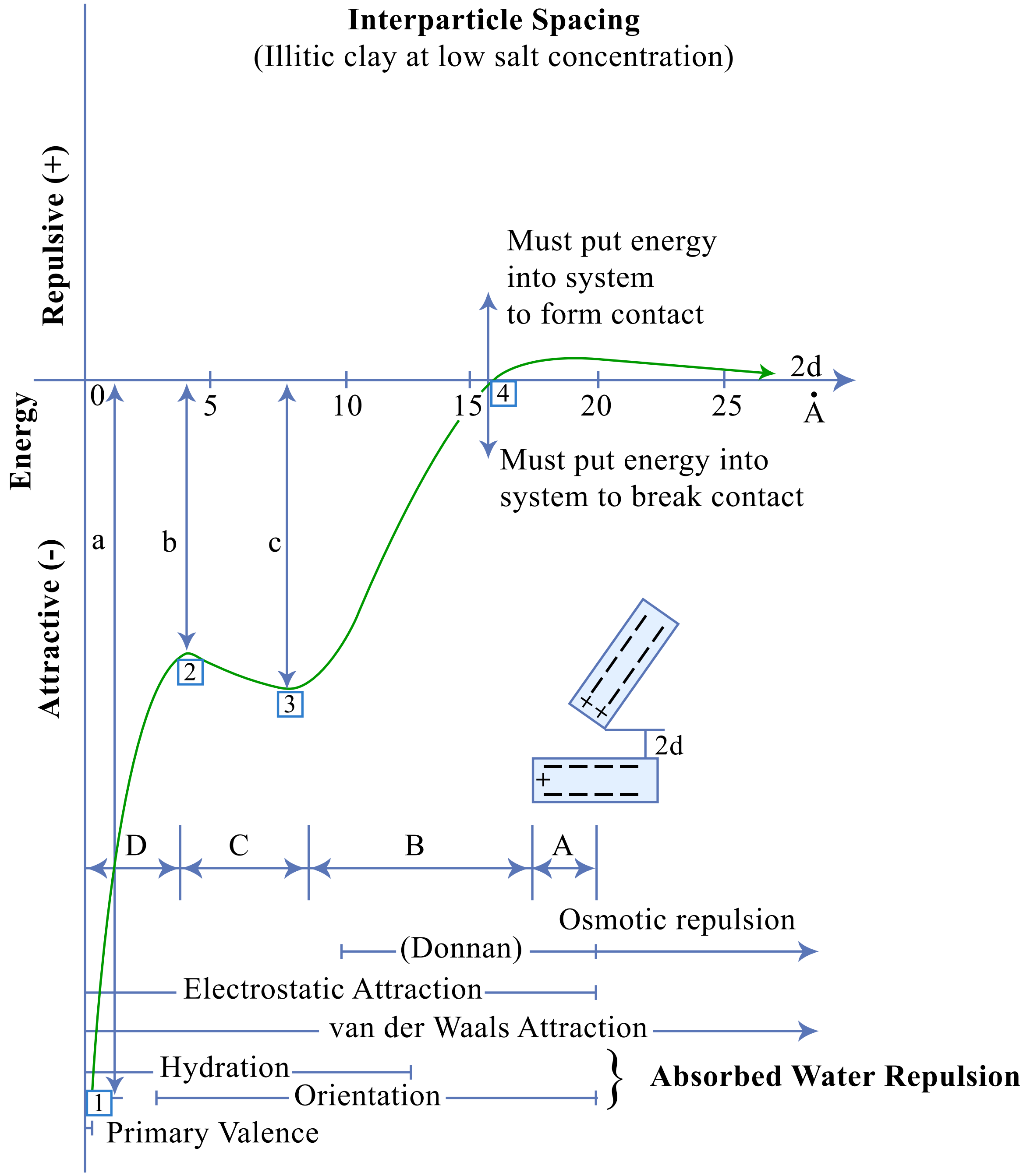


Figure by MIT OCW.

$$\sigma' = (\bar{\sigma}_r - \bar{\sigma}_a)a_c + (R-A)$$

A.W., Born — vdw, electrostatic, primary valence.

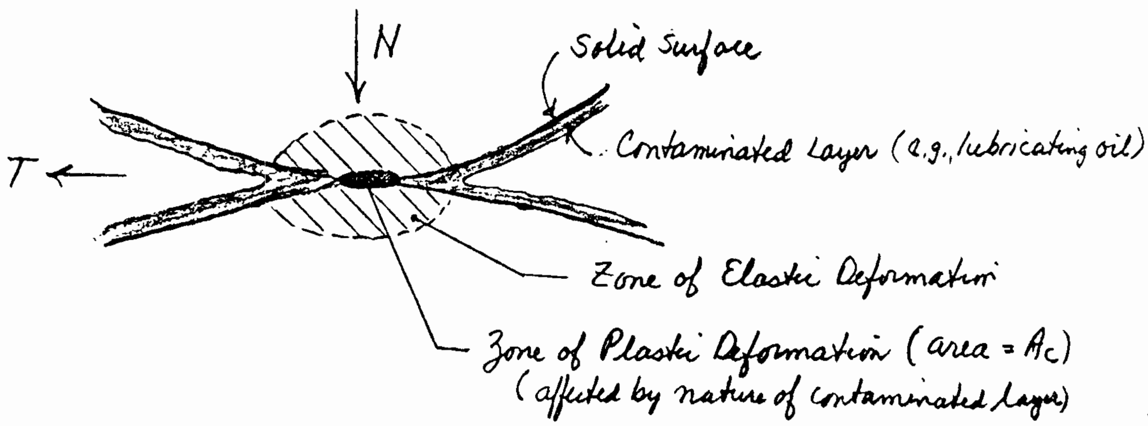
C

IV STRENGTH GENERATION IN SOILS (= 1.361 Part II 2, Sect. 2.6)

1. FRICTIONAL RESISTANCE

1) Terzaghi - Bowden - Tabor <u>Adhesion</u> theory (developed for metals)
(1940s)

All surfaces are rough at microscopic scale. Therefore get contacts at asperities



- Normal force = $N = A_c \cdot \bar{\bar{\sigma}}_y$, where $\bar{\bar{\sigma}}_y$ = yield stress
  Shear force = $T = A_c \cdot \bar{\bar{\tau}}$, where $\bar{\bar{\tau}}$ = shear strength due to primary valence bonding

- Increasing $N \rightarrow$ increasing $A_c \rightarrow$ increasing T
  Decreasing $N \rightarrow$ decreasing A_c due to elastic rebound $\rightarrow$ decreasing T
  $\Big\}$ Constant coef. of friction $= T/N = \bar{\bar{\tau}} / \bar{\bar{\sigma}}_y = \tan \phi'_\mu$

- Tests on Quartz by Bromwell (1966) (Sheet A)
  Ultrasmooth surfaces, $\phi'_\mu = 10 - 35°$ is function of surface contamination
  Regular, rough surfaces, $\phi'_\mu = 25 \pm 5°$ independent of contamination

2) Granular Soils

$$\sigma' = \bar{\bar{\sigma}} \cdot a_c, \text{ where } \bar{\bar{\sigma}} \approx 10{,}000 \text{ atm at typical stress levels}$$

(For $\sigma' = 1$ atm, $a_c = 0.01\%$)

3) Cohesive Soils

a) Are there mineral to mineral contacts in clays at typical σ' levels (say $\sigma' \geq 1$ atm)?

- Ladd (1961) back calculated likely values of contact shear stresses $\rightarrow \bar{\bar{\tau}} \approx$ 100's of atm. $\therefore$ must have primary valence bonding at min.-min. contacts

- Mitchell (1993 book), but based on research in 1960's using rate process theory $\rightarrow$ activation energy of bonding

Calorie × 4.2 = J = N·m

Material	Activation Energy (kcal/mol)	
Water	4-5	
Ice	10-15	
Metals & Concrete	≥ 50	
Soil	30 ± 5	Sands & clays, both wet & dry!

b) Conclusion: clays develop a frictional resistance (ϕ') due to primary valence bonding at contacts. However, get wide variation in ϕ' due to wide variation in $\bar{\bar{\sigma}} \cdot a_c / \sigma'$ ratio (Part A V)
i.e. surface forces affect ratio and ability to reform broken contacts

2. COHESIVE RESISTANCE (True Cohesion)

1) Very controversial since difficult to measure or even define

2) However one can list potential sources of true cohesion

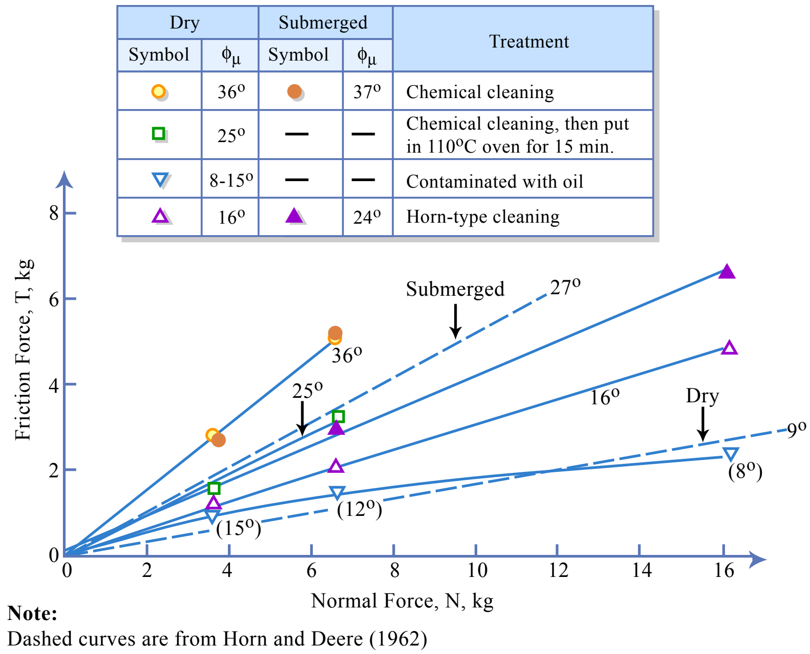
a) Cementation due to carbonates, Fe/Al oxides, amorphous silica

- Difficult to quantify, but certainly occurs $\rightarrow$ brittle behavior
  Calcareous sands, calcareous clay shales, Champlain clays,
  (Will give examples in Parts C & D)

b) When physico-chemical $\bar{\bar{\sigma}}_a > \bar{\bar{\sigma}}_r$, so that added energy required to break contacts during shearing (Energy diagrams Part A III)

c) Adsorbed water ?? Some still promote "ice-like" behavior

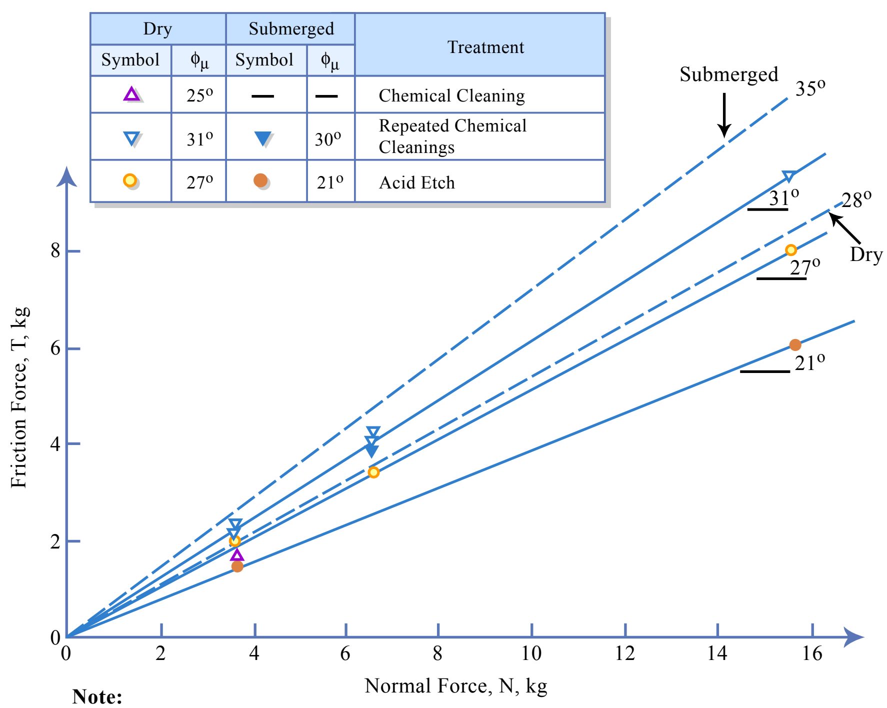
NOTE: Unconfined shear strength of ovendried remolded clay can be high. Can't be due to adsorbed water; must be caused by $\bar{\bar{\sigma}}_a > \bar{\bar{\sigma}}_r$



Note:
Dashed curves are from Horn and Deere (1962)

Results of Atmospheric Friction Tests on Smooth Quartz Surface

Figure by MIT OCW.



Note:
Dashed curves are from Horn and Deere (1962)
Geot. 12(4), 319-335

Results of Atmospheric Friction Tests on Rough Quartz Surfaces

Figure by MIT OCW.

Adapted from Bonaparte & Mitchell (1981) UC Berkeley NASA Contractor Report 3365

Bromwell (15) = PhD thesis by L.G. Bromwell (1966) on "Friction of Quartz in a High Vacuum" to predict behavior of lunar soils
(CCL's 1st PhD student)

A

CCL 2/9/99
2/4/01

1.322 Part B

A. Material Initially Distributed

1) "Simple Clay" Notes = CCL (1964) Res. Report R64-17

2) 1st set of HO Notes = Cover Sheet + p1-8 (attached)

3) Home Problem No. 1 & Solution (Self graded)

B. Approximate Class Schedule*

Class No.	Coverage & Remarks
B 1 / B 2	• N/C Simple Clay via class discussion (p 1-4b), including use of Principle II to predict $w_c = f(K_c)$ & CAUC ESP (Fig II-12, 13); prediction of UUC test from 1 CIOC test (also see 1.361 II 2-3.7) • You need to study Chap I & II of SC Notes & do most of HP #1
B 3 / B 4	• OC Simple Clay via class discussion (p5-8), including Hvorslev parameters → State Boundary Surface (not in SC Notes) and triaxial extension. • You need to study Chap III - VI • Distribute MCC Notes & HP #2 (due for class #5)
B 5 / B 6	• Cover MCC Notes, mostly via "lectures" • Distribute HP #3 • Distribute C-I Notes • Comments on MIT-E3 (no HO notes)
B 7	• Either complete Part B or start Part C

* 1st Class = 1st or 2nd hr, Tues. 2/20/01 (Mon. class → Tues due to Monday holiday)

No. 5505 Engineer's Computation Pad

Handout on Basic Strength Principles & "Simple Clay"

BASIC STRENGTH PRINCIPLES & STRESS-STRAIN-STRENGTH BEHAVIOR OF "SIMPLE CLAY"

INTRODUCTION

1.1 Types of Shear Tests (Restricted now to TC & TE)

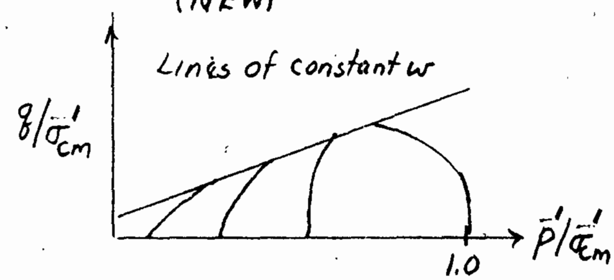
- CD
- CU
- UU

CD, CU } CI, CK_0 .... D/U C/E L/U

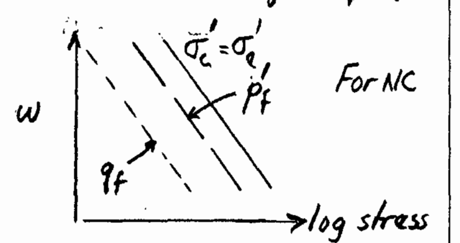
1.2 3 Basic Principles (For given b & δ, i.e. treat TC & TE separately)

Principle	Limitation	Independent Of
I Unique Failure Envelope e.g. $q_f = a' + \bar{p}'_f \tan\alpha'$ ($\tan\alpha' = \sin\phi'$)	NC vs OC	Drainage (CD, CU & UU) TSP = L vs U
II Unique $w - q - \bar{p}'$ ($q = 0 \rightarrow q_f$)	NC vs OC $\Delta q \geq 0$	Same as above
III Unique $w_f - q_f - \bar{p}'_f$ à la Hvorslev Parameters (NEW)	None	Same as above PLUS both NC & OC!

(NEW)



Corollary (1.361) → unique w_f-log stress



Note: Will lead to concept of "State Boundary Surface" (p7)

1.3 Why Study These 3 Strength Principles?

1) Reasonable approximation for many insensitive clays

2) Frequently assumed/used in practice

3) Framework for more complex behavior

4) Background for discussion of "generalized Soil Model" à la MCC = Modified Cam-Clay

1.4 Simple Clay

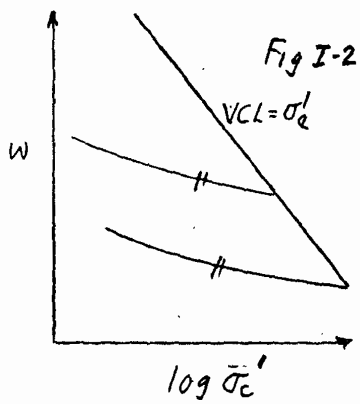


Fig I-2

1) Developed as teaching aid for home problems with clay having perfect "normalized behavior"

2) Behavior reasonably typical of insensitive plastic clays (for $K_c = 1$ consolidation)

3) Not intended for direct use in practice)

1.5 Variables Considered by Simple Clay

1) Drainage (CD → CU → UU)

2) OCR

3) TSP, e.g. L vs U

4) $K_c = \bar{\sigma}'_{hc}/\bar{\sigma}'_{vc}$

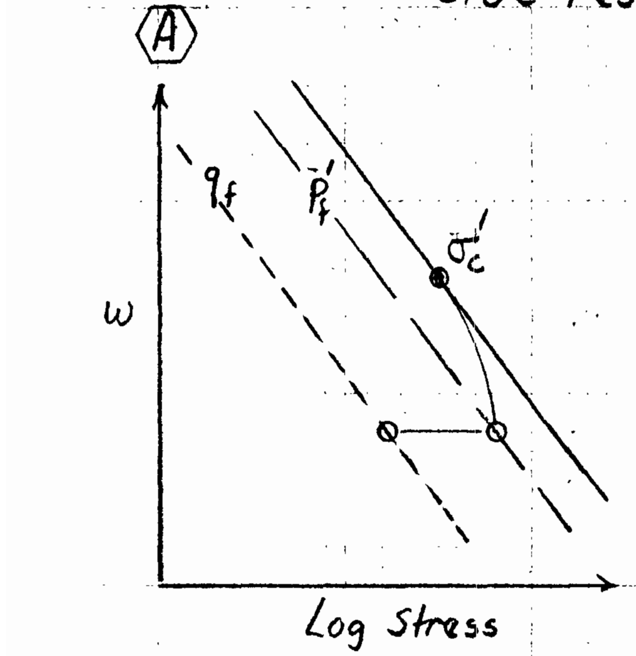
5) σ_2, e.g. TC vs TE

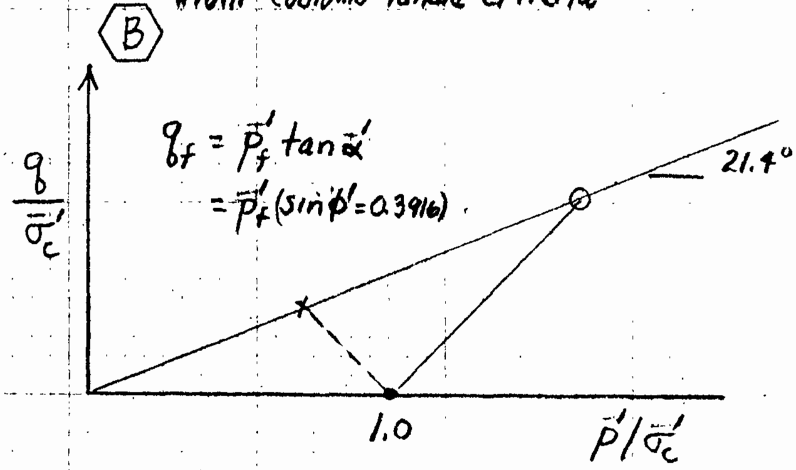
NOTE: Sheets 3 & 4 = start of OCR=1 Simple Clay behavior

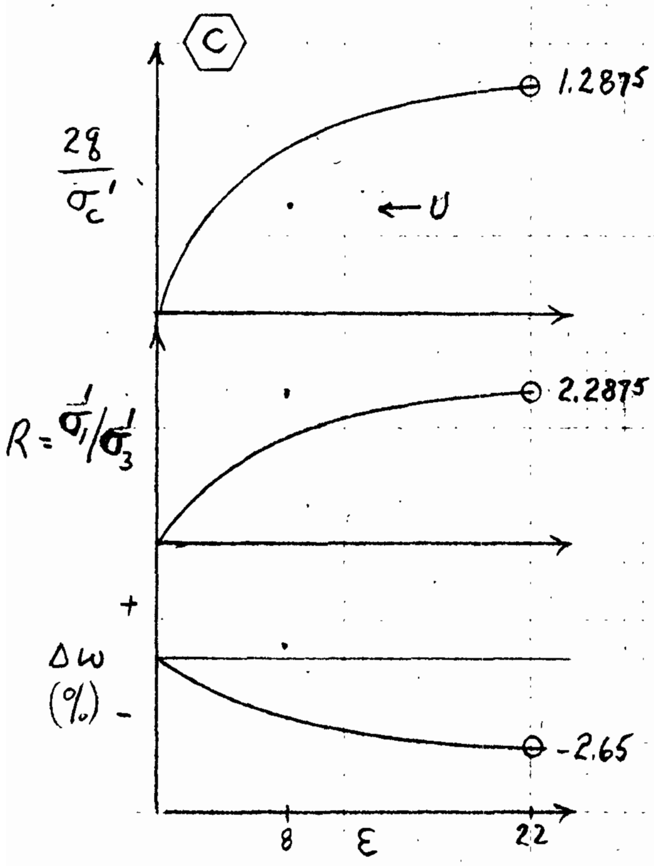
CIUC Tests N.C. Simple Clay

○—— L
×- - - U

Mohr-Coulomb failure criteria





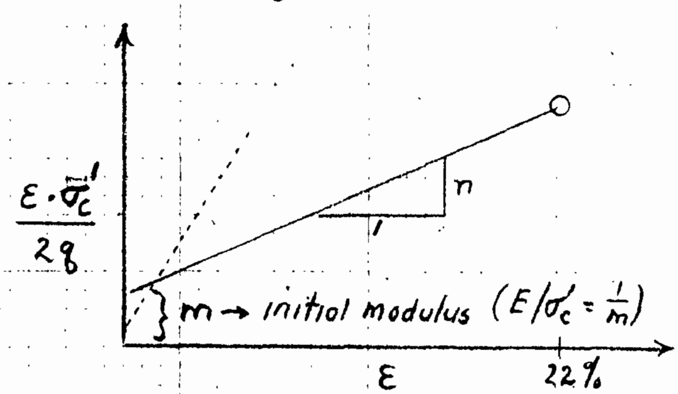


- Perfect Normalized Behavior
- Why different σ-ε for U?

L $+\Delta q \rightarrow$
$+\Delta p' \rightarrow$

U $+\Delta q \rightarrow$
$-\Delta p' \rightarrow$

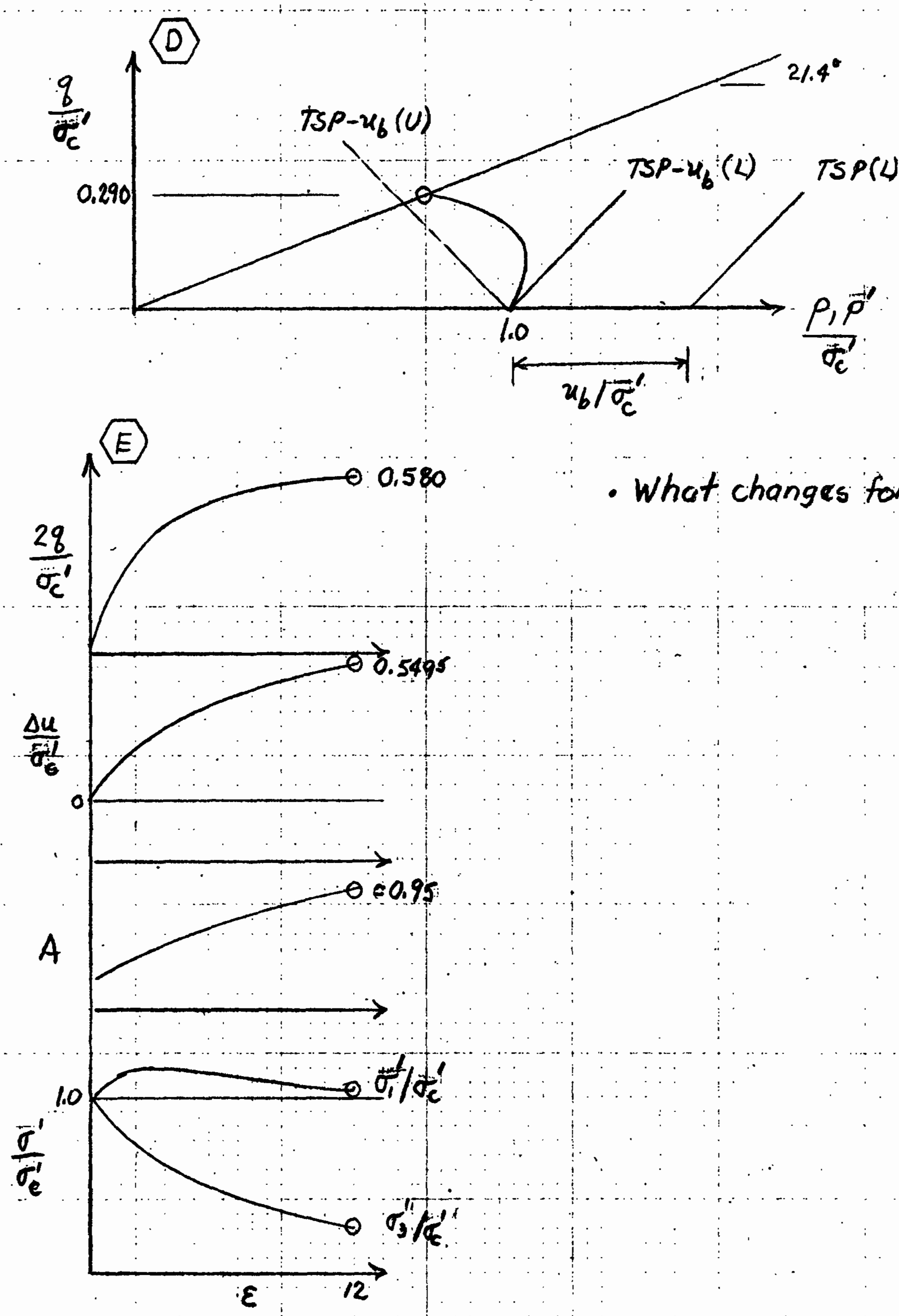
$$\frac{2q}{\bar{\sigma}_c'} = \frac{\varepsilon}{m + n\varepsilon} \quad \text{(Hyperbolic)}$$



$\overline{CIUC}$ Tests N.C. Simple Clay

See ⟨A⟩ for w-log stress

o—— L { S=100% }
x - - - U { B=1.00 }

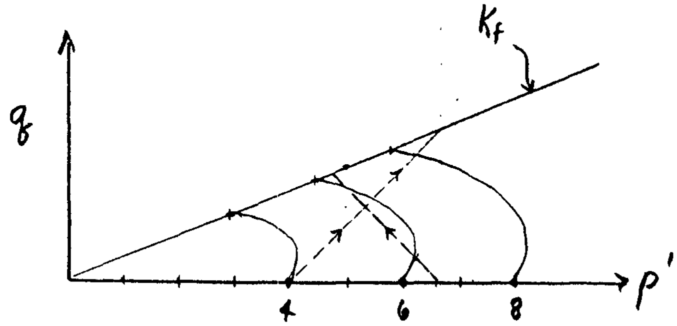


• What changes for U ?

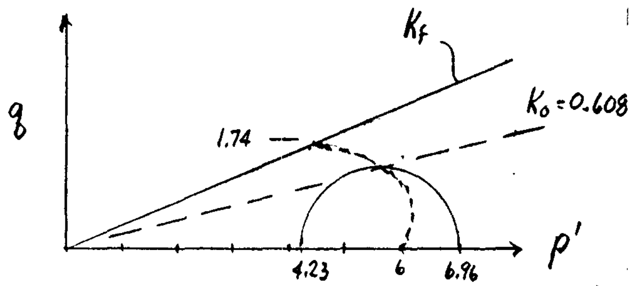
Principle II : Unique $w - q - p'$ for $\Delta q > 0$

Defined by ________ for NC SC

a) Fig II-12: Prediction of Δw for CIDC (L) & (U) tests



b) Fig II-13: Prediction of ESP, q_f/σ'_{vc}, etc. for CK_oUC test



- $\sigma'_{vc} / \underset{\sigma''_e}{\text{Equiv. } \sigma'_c} = \frac{6.96}{6.00} = 1.16$
- $q_f/\sigma'_{vc} = \frac{0.29}{1.16} = \frac{1.74}{6.96} = 0.25$
- $A_f =$

c) Prediction of Parameters from CIUC Stress-Strain Data Given σ'_{vc} & σ'_{hc} $(K_c < 1)$

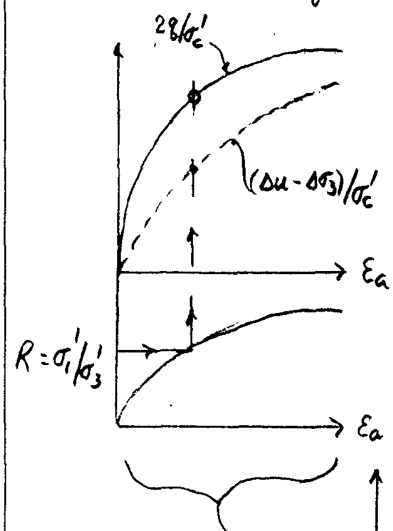
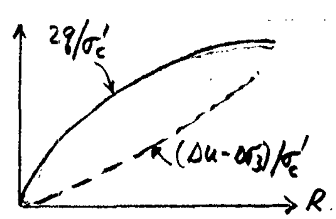


Fig II-8a
(HP #1)



1) Compute $R = \sigma'_v/\sigma'_h = 1/K_c$ & $2q_c = (\sigma'_{vc} - \sigma'_{hc})$
2) Scale $2q/\sigma'_c$ at R
3) Equivalent $\sigma'_c = \sigma'_e = \frac{2q_c}{(2q/\sigma'_c)} \rightarrow$ value of w_c
4) For CAUC, $q_f = (0.29)(\sigma'_e) \rightarrow q_f/\sigma'_{vc}$
5) " " , also can get $\Delta u_f/\sigma'_c \rightarrow A_f$, etc.

Three Factors Controlling $s_u = q_f$

1)

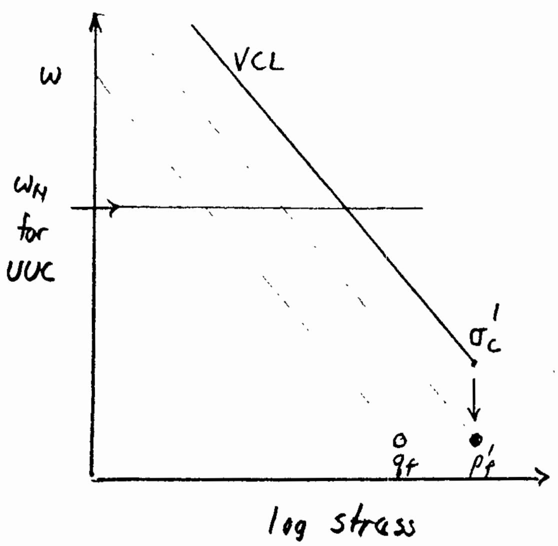
2)

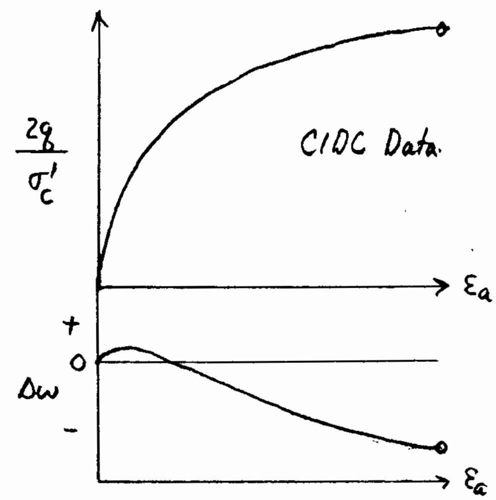
3)

CAUC : $K_c = \bar{\sigma}'_{hc}/\sigma'_{vc} \gtrless 1$

$$\frac{q_f}{\sigma'_{vc}} = \frac{(c'\cos\phi')/\sigma'_{vc} + [K_c + (1-K_c)A_f]\sin\phi'}{1 + (2A_f - 1)\sin\phi'}$$

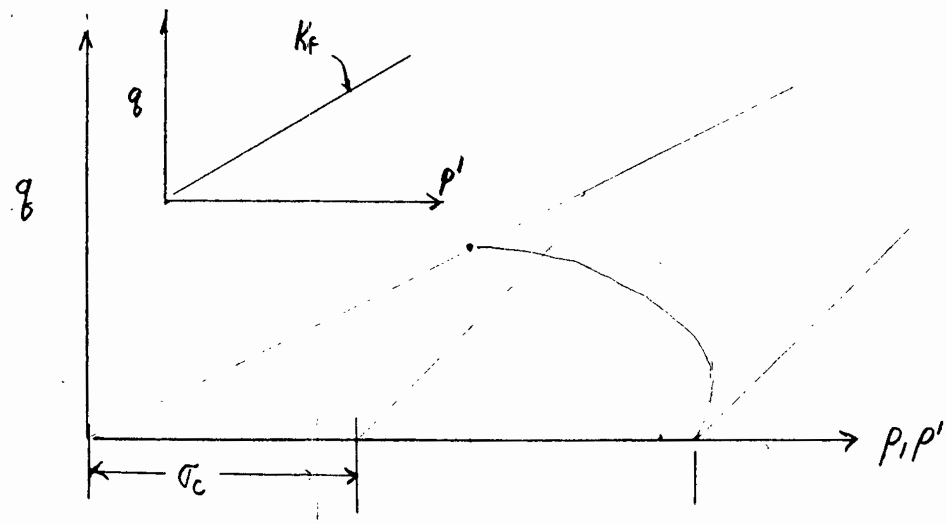
Prediction of UUC Test from CIDC ($p' = \sigma'_c$) Data on NC Clay ($K_c = 1$, no disturbance)





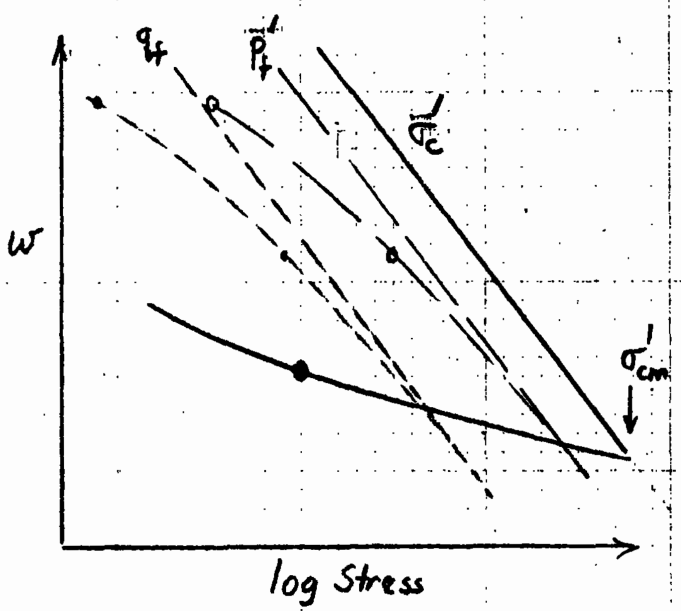
1) σ'_0 & u_0 2) $s_u = q_f$ & p'_f 3) u_f 4) Δu_f 5) A_f

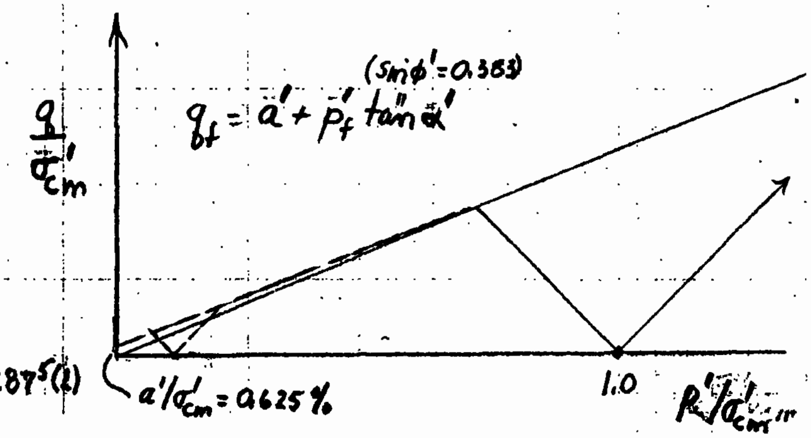
6) ESP 7) q vs ε_a



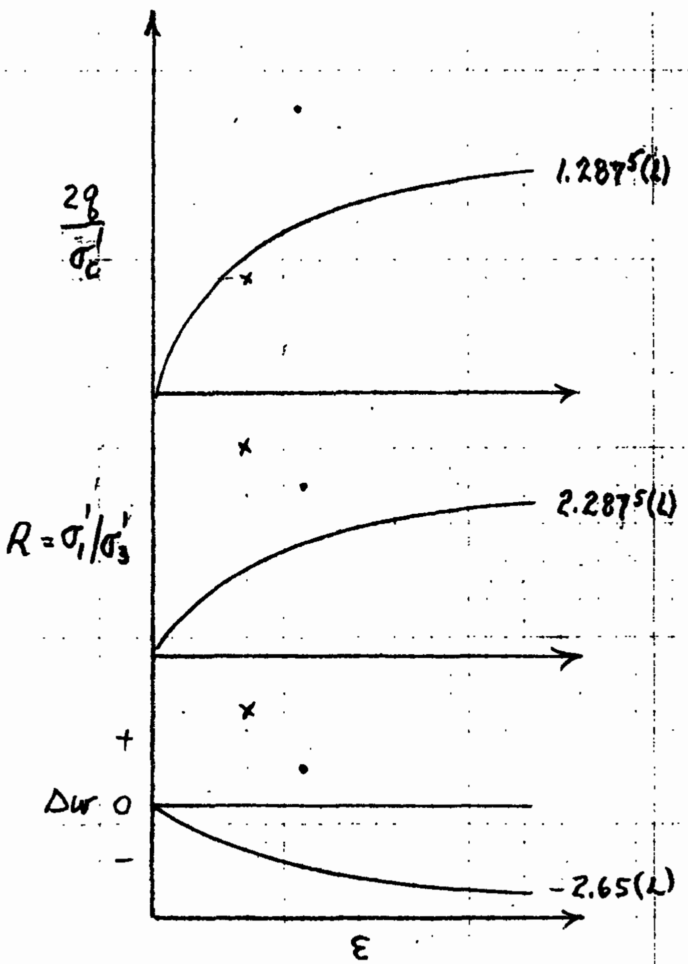
CIDC Tests - Effect of OCR

(Fig. III-5)





(Fig. III-2)

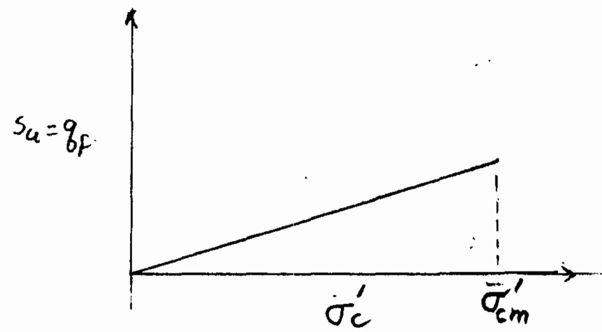


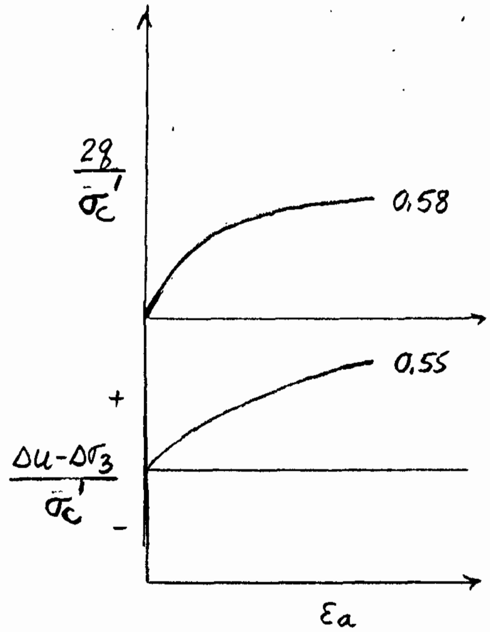
Increasing OCR →

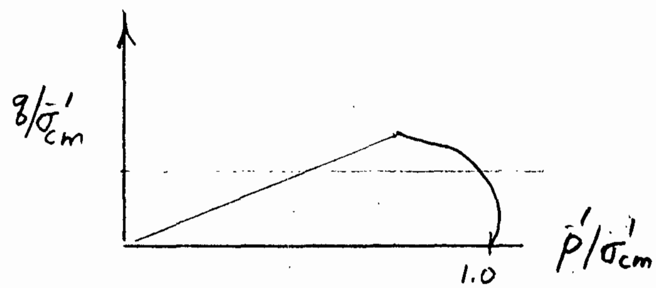
- $q_f/\bar{\sigma}'_c$
- R_f
- Δw
- Post-peak q-ε_a

Causes of Dilation (Discussion)

CIUC Tests - Effects of OCR





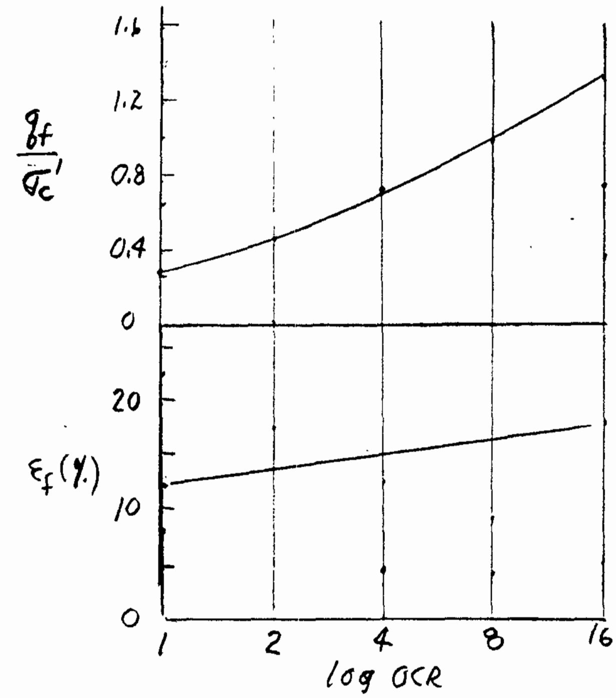


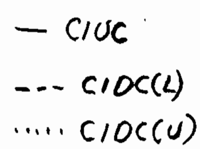
Effect of incr. OCR on:

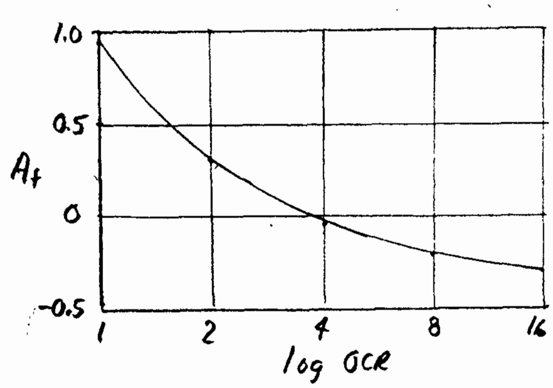
- $s_u/\bar{\sigma}_c'$
- ε_f
- A_f

What is main reason for OCR → incr. $s_u/\bar{\sigma}_c'$?

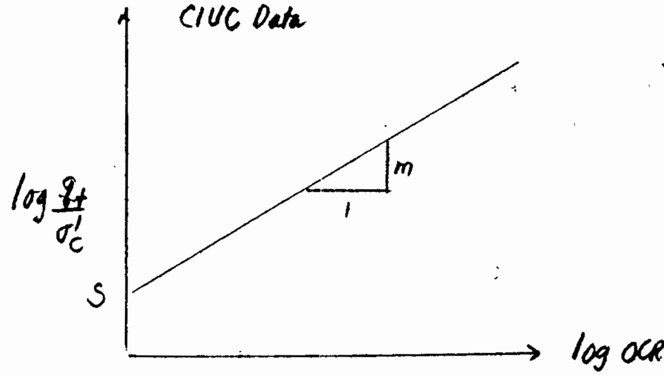
Summary (Fig III-17, 18, 19)







SHANSEP Eq. $q_f/\sigma'_s = S(OCR)^m$



—— SHANSEP Eq → $S = 0.31$ & $m = 0.54$ (LR OCR = 1 → 16)

- - - Simple Clay "data"

$S = 0.29$, $m \approx 0.7 \to 0.58$ with increasing OCR

$\left(m = \frac{\log(q_f/\sigma'_c/0.29)}{\log OCR} \right)$

∴ Should revise SC data → Constant m

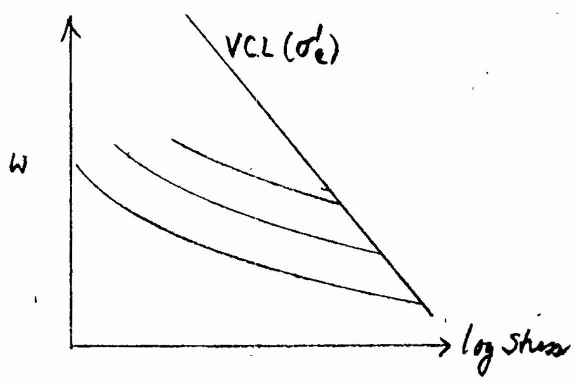
Hvorslev Parameters

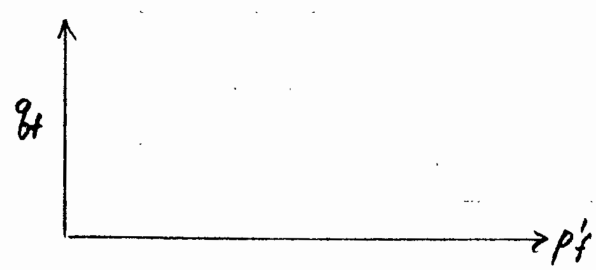
1) Hvorslev's contributions → $\tau_{ff} = K\sigma'_e + \sigma'_{ff} \tan\phi'_e$

2) Golden Rule

3) Revised determination / presentation for CIUC/CIDC data

$q_f = \beta\sigma'_e + p'_f\,(\tan\alpha'_e = \sin\phi'_e)$ where $\beta\sigma'_e = a'_e(w_f)$





4) Table with data from CIUC and/or CIDC at varying OCR

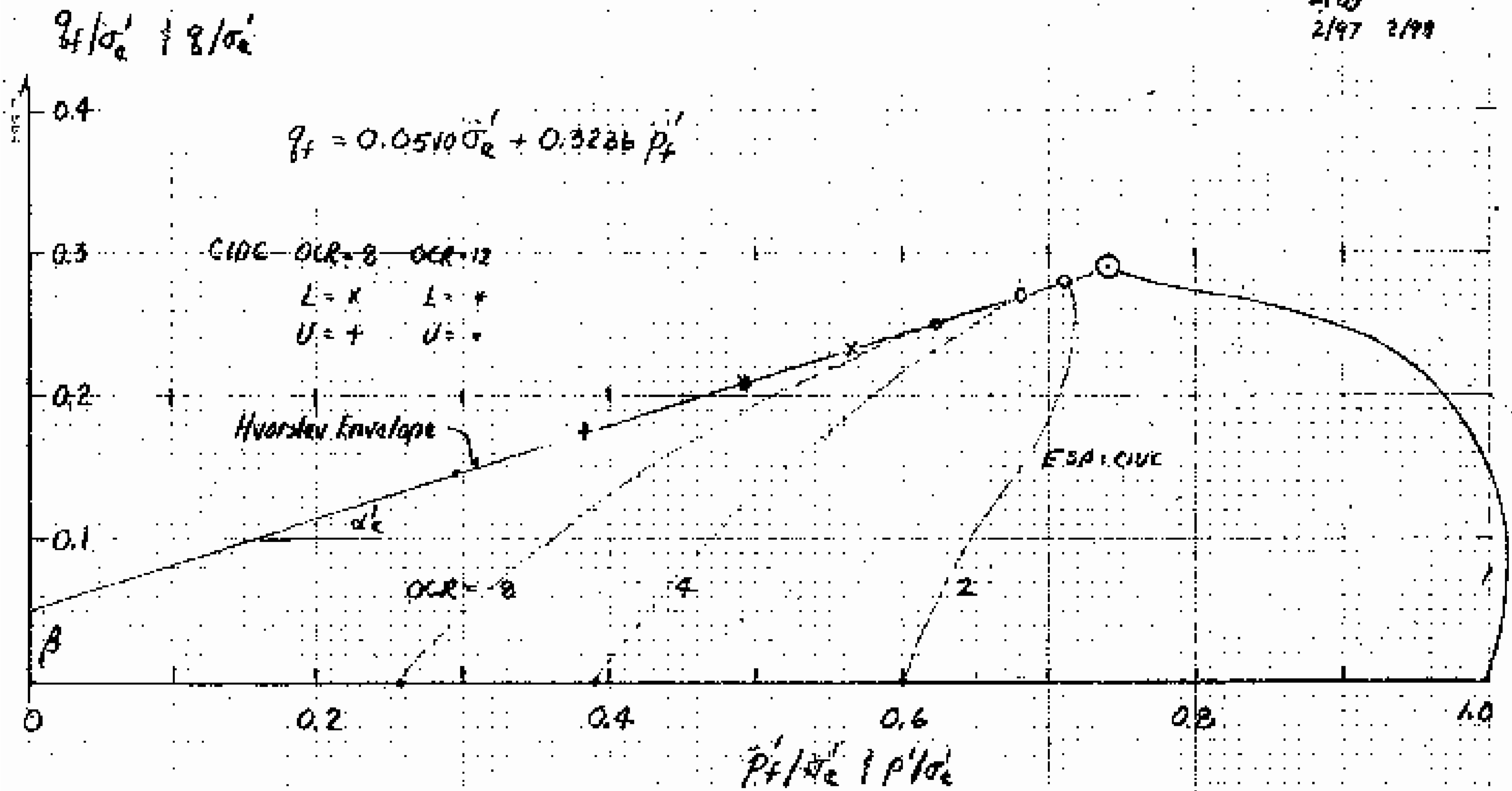
w_f	q_f	p'_f	σ'_e	q_f/σ'_e	p'_f/σ'_e

5) Hvorslev Envelope (p?)

6) <u>State Boundary Surface</u> (Hvorslev + Roscoe = NC CIUC ESP)

7) Discussion

1.322 Part B
CCL 2/85 2/01
2/89
2/97 2/98



State Boundary Surface for Simple Clay

7B

COMMENTS ON SIMPLE CLAY TE vs TC

1) Generalized Pore Pressures - Henkel (1960) p53-54 incorrect

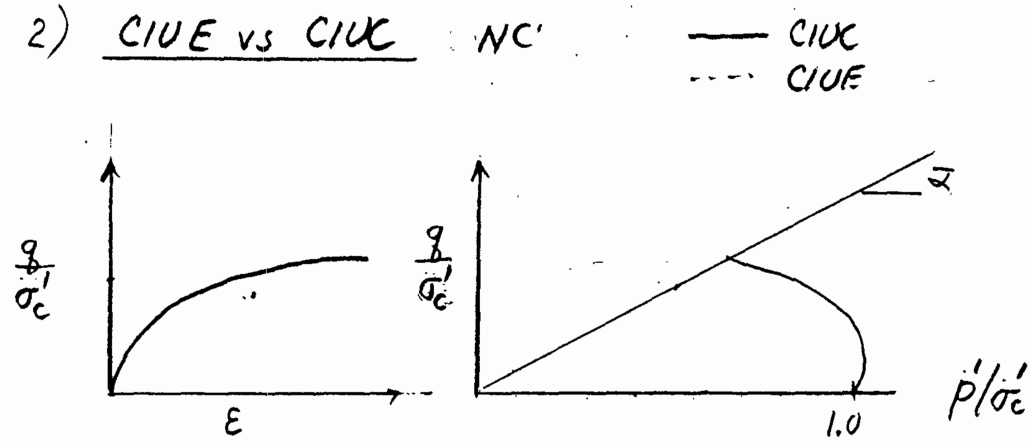
$$\Delta u = \Delta \sigma_{oct} + a\,\Delta \tau_{oct} \quad , \quad \Delta \tau_{oct} = \left(\frac{1}{3}\right)\sqrt{(\Delta\sigma_1-\Delta\sigma_2)^2+(\Delta\sigma_1-\Delta\sigma_3)^2+(\Delta\sigma_2-\Delta\sigma_3)^2}$$

For $b = 0 \rightarrow A = \frac{a\sqrt{2}}{3} + 1/3$

$0.5 \quad A = \text{''} + \frac{1}{2}$

$1.0 \quad A = \text{''} + 2/3$

2) CIUE vs CIUC NC'



- Change in s_u/σ'_c ? Why?
- Comparison MCC

3) Effect of OCR (III-19)

- $s_u(OC)/s_u(NC)$ vs OCR - How compare to CIUC?

4) What happens to Principle II relationships?

5) What happens to OC ESE and Hvorslev Parameters for SC?

(Actually don't know for real clays)

1 INTRODUCTION

Objectives of "Generalized" Model

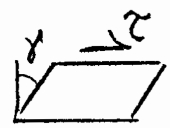
1) Mathematical model (set of constitutive eqn) to represent stress-strain-strength behavior in reasonable fashion for various stress paths & stress systems (b & δ) for BOTH drained & undrained conditions
2) Reasonable number input parameters that have physical significance & can be measured!

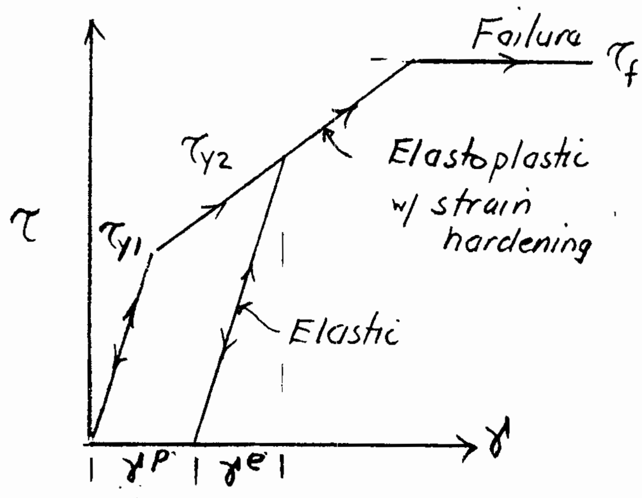
(Simplest "model" = linear, elastic, isotropic → 2 parameters)

2. PLASTICITY THEORY (Elasto-Plastic Model)

2.1 Yielding vs Failure

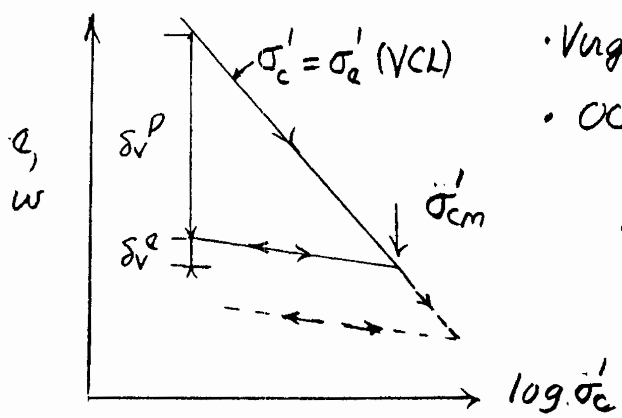
a) Simple shear





- Yielding = transition from elastic (all recoverable) to plastic (irrecoverable) strains
- Plastic strains → increase in τ_y via "hardening law"
- Failure - i.e. Mohr-Coulomb (continued deformation at constant stress)

b) Isotropic consolidation test



- Virgin → plastic (δ_v^p) + some elastic strains (δ_v^e)
- OC → only elastic, δ_v^e
- Simple "isotropic" hardening law with $\sigma_y = \sigma'_{cm}$

NOTE: Continuous yielding on VCL = Virgin Compr. Line

2.2 <u>State Variables</u> (SV)

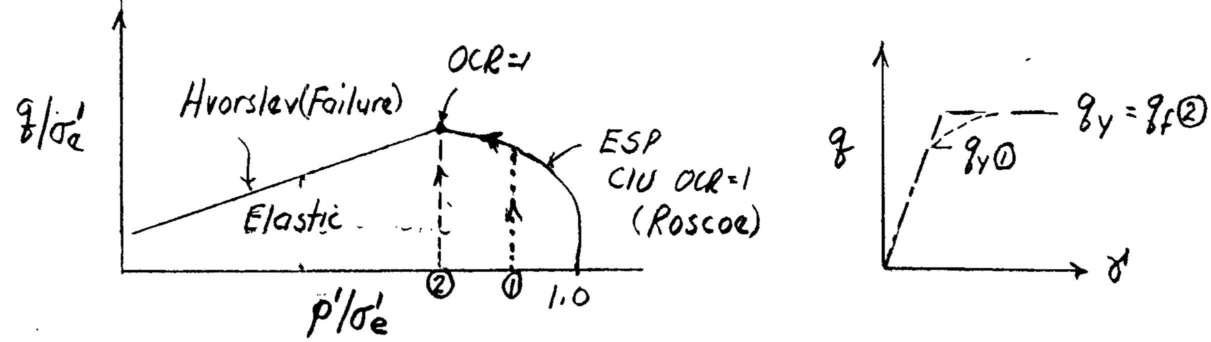
Define "state" of soil . Simple Clay → w - q - p' $= f(\sigma'_e)$
MCC → e - $\bar{q}$ - $\bar{p}$

2.3 <u>State Boundary Surface & Yield Surface (Locus-Envelope)</u>

- Consider CIU PSC with elastic $A_e = 0.5$ ($b = 0.5$)

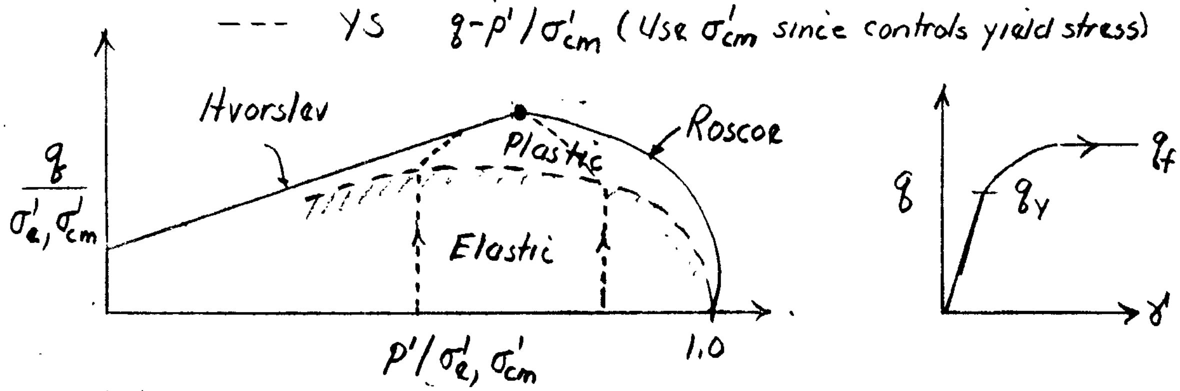
(a) <u>Simplified Behavior</u> <u>SBS = YS</u> → Elastic behavior inside SBS = YS

① Yielding before failure
② Simultaneous yielding & failure



(b) <u>More Realistic Behavior</u> : Yielding before hit SBS

—— SBS q-p'/σ'_e
- - - YS q-p'/σ'_{cm} (Use σ'_{cm} since controls yield stress)



2.4 <u>Flow Rule</u> (Set of eqn. = Plastic Potential) δ_v^p & $\delta\gamma^p$

- Gives direction & magnitude of plastic strain <u>increments</u> when SV hits YS, i.e. during yielding
- "Associated": same eqn. plastic potential & yield surface

Plastic Potential = surface that is ⊥ to plastic strain increments (Sheet A)

2.5 <u>Hardening Law</u>

- Governs changes in shape & location of YS resulting from plastic deformations

3. MODIFIED CAM-CLAY (MCC)

3.1 Background

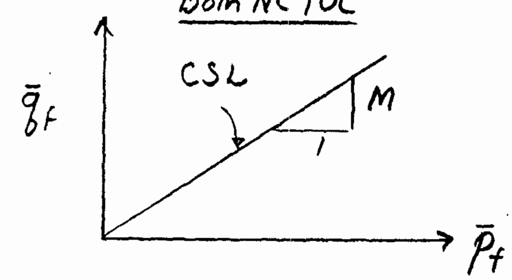
- First generalized model; developed at Cambridge Univ (Roscoe*, Schofield, Wroth, Poorooshasb, Burland*) in 1960's ±
- Intended to model results from CIUC & CIDC tests on remolded clay, especially at low OCR. But can predict Plane strain, CK_0UC/D, etc. ISOTROPIC

3.2 State Variables

$e \qquad \bar{q}' = (\sigma_1 - \sigma_3)$ ← For TX test** $\qquad \bar{p}' = \sigma'_{oct} = \frac{1}{3}(\sigma'_1 + \sigma'_2 + \sigma'_3)$

3.3 Failure Law (Surface = Envelope)

Both NC & OC



Extended von Mises

$\bar{q}_f = \bar{p}_f M$

at Critical State Line (CSL) →

unique $e_f - \bar{q}_f - \bar{p}_f$

Sheet B
Not realistic

$b = 0\ (TC) \quad M = 6\sin\phi'/(3 - \sin\phi') \quad$ or $\sin\phi' = 3M/(6+M)$

$b = 0.5 \quad M = \sqrt{3}\sin\phi'$

$b = 1.0\ (TE) \quad M = 6\sin\phi'/(3+\sin\phi') \quad$ or $\sin\phi' = 3M/(6-M)$

→ For $M = 1.2 \quad \phi' = 30°$ TC increasing to $\phi' = 48.6°$ TE

$M > 1.5 \quad > 36.9 \quad \rightarrow \phi'_{TE} > 90°$

- Note: As applied in practice, usually use ϕ' triaxial compression to compute M

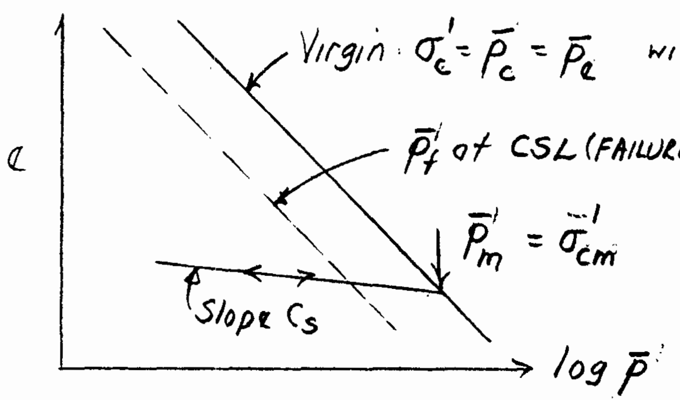
** For $0 < b < 1.0 \quad \bar{q} \rightarrow q^* = \frac{1}{\sqrt{2}}\sqrt{(\sigma_1-\sigma_2)^2 + (\sigma_1-\sigma_3)^2 + (\sigma_2-\sigma_3)^2}$ } Sheet B

Extended von Mises: $q^*/\sigma'_{oct} = M$ (Bishop, 1971)

* Roscoe, K.H & Burland, J.B. (1968), "On the Generalized Stress-Strain Behaviour of 'Wet' Clay", in Engineering Plasticity, Cambridge Univ. Press, pp. 535–609. (MCC Input Parameters: ϕ'_{TC} + isotropic e vs. $\log\bar{p}$ + ν')

3.4 Compressibility & Hardening Law - $K_c = 1.0$

{ MCC actually uses e vs. $\ln \bar{p} \rightarrow$
$\lambda = -de/d\ln \bar{p}_c = C_c/2.3$ & $\kappa = C_s/2.3$ }



with $C_c = -de/d\log \bar{p}_c$

• $m = (1 - C_s/C_c)$

$\left\{ \therefore \dfrac{\bar{p}_f}{\bar{p}_e} = (0.5)^m \right\}$ later

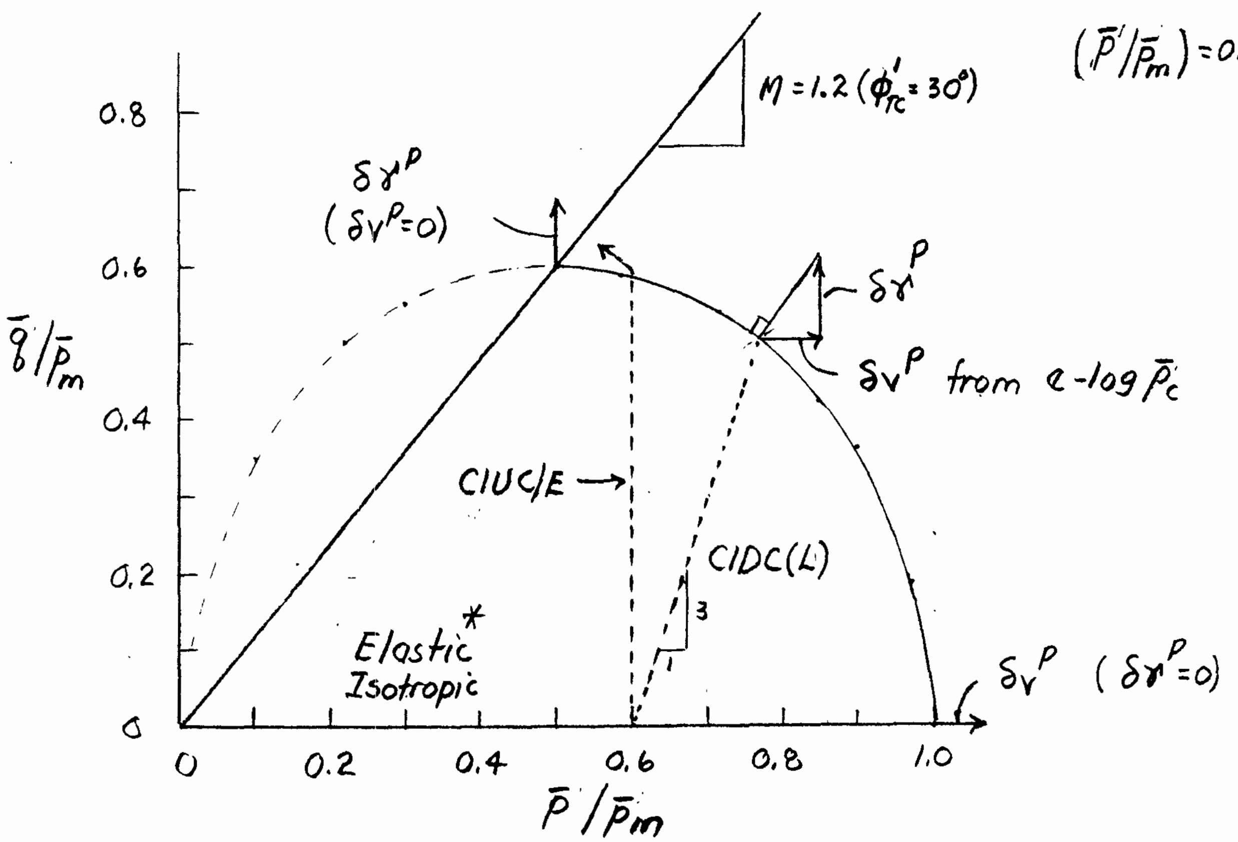
• $\underline{\text{Isotropic}}$ $\underbrace{\text{hardening law}}_{\text{size of YS}}$ with end of YS at $\bar{p}_m'$

3.5 Yield Surface & Associated Flow Rule

• Eqn for Yield surface is ellipse & also equals "plastic potential" → associated flow rule (equals "normality")

$$\left\{ \frac{\bar{p}}{\bar{p}_m} = \left(\frac{M^2}{M^2 + R^2} \right) \right\} \quad R = \bar{q}/\bar{p}$$

At $R = M$ (failure) $(\bar{p}'/\bar{p}_m) = 0.5$



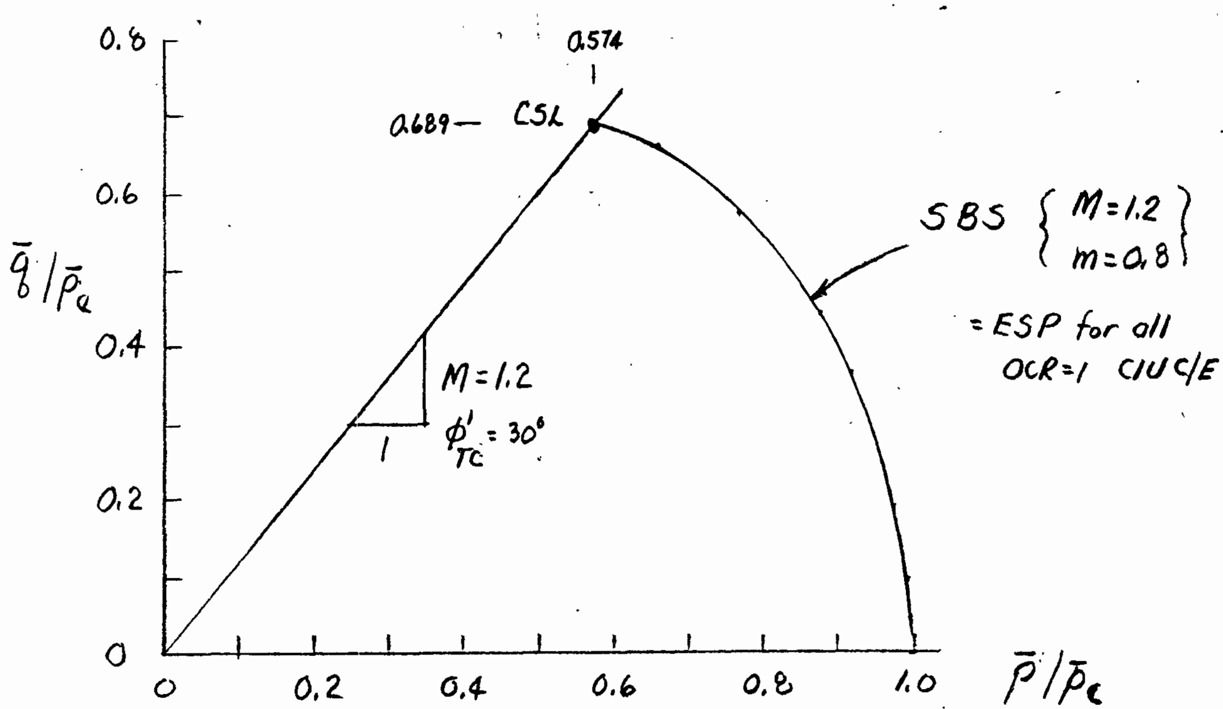
* Need input E' or ν' or G etc (Already have $K' = \delta\bar{p}/\delta v^e$ from C_s line)
↑ usually assumed

3.6 SBS = ESP for CIUC/E OCR=1 (Triaxial Tests)

(Follows from yield surface + flowrule + hardening law & failure law)

· Eqn for SBS $\dfrac{\bar{p}}{\bar{p}_e} = \left(\dfrac{M^2}{M^2+R^2}\right)^m$ $m = 1 - C_s/C_c$, $R = \bar{q}/\bar{p}$

· For $R = M$, i.e. at failure, $\bar{p}_f/\bar{p}_e = (0.5)^m \rightarrow$ CSL in 3.4



· SBS = ESP for OCR=1 CIUC/E + governs Δe for CIDC/E

3.7 Predicted CIU q_f/σ_c' for OCR=1

· $q_f = \frac{1}{2}\bar{q}_f$; $\bar{q}_f = \bar{p}_f M$; $\bar{p}_f = \bar{p}_e(0.5)^m = \sigma_c'(0.5)^m$

$\therefore q_f/\sigma_c' = \dfrac{M}{2}(0.5)^m$ for both CIUC & CIUE

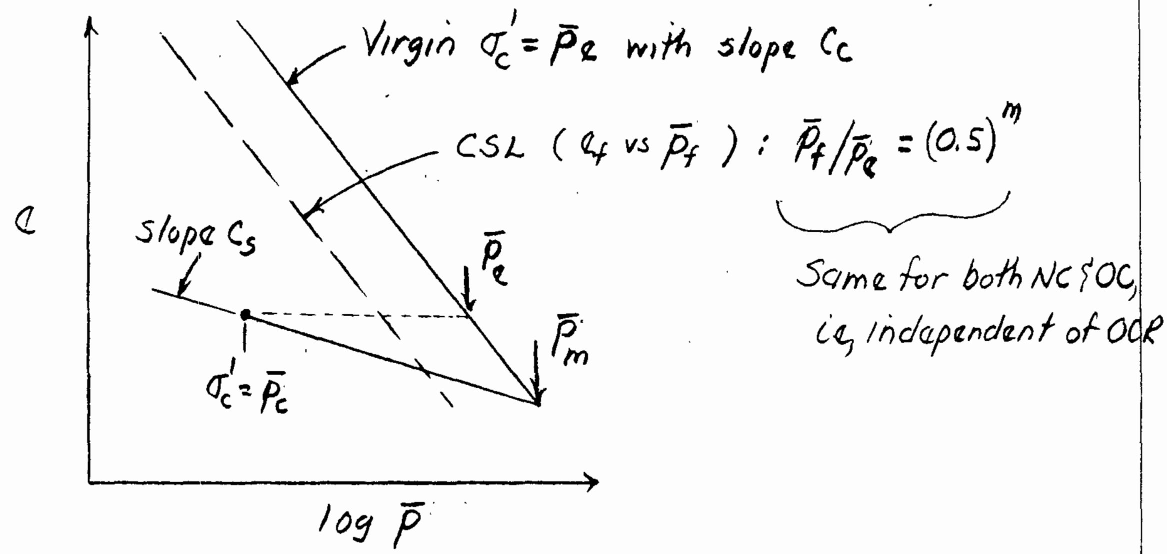
Examples for m=0.8

$\phi'(TC)$	M	q_f/σ_c'
25	0.984	0.283
30	1.200	0.345
35	1.418	0.407

See Sheet A for MIT ESP (q vs $p' \div \sigma_c'$)

3.8 Predicted CIU q_f/σ_c' vs OCR

(a) Graphical model



(b) Derivation of eqn.

(1) From 3.7 $\quad q_f/\bar{p}_e = \frac{M}{2}(0.5)^m$

(2) $\underbrace{C_s \log \frac{\bar{p}_m}{\sigma_c'}}_{\text{(swelling)}} = \underbrace{C_c \log \frac{\bar{p}_m}{\bar{p}_e}}_{\text{(virgin)}} \rightarrow \frac{C_s}{C_c} \log \overbrace{\frac{\bar{p}_m}{\sigma_c'}}^{= OCR} = \log \frac{\bar{p}_m}{\bar{p}_e}$

$\log \frac{\bar{p}_m}{\bar{p}_e} = \log \frac{\bar{p}_m}{\sigma_c'} - \log \frac{\bar{p}_e}{\sigma_c'} \rightarrow \log OCR \underbrace{\left(1 - \frac{C_s}{C_c}\right)}_{=m} = \log \frac{\bar{p}_e}{\sigma_c'}$

$\therefore \frac{\bar{p}_e}{\sigma_c'} = (OCR)^m$

(3) Multiply (1) by (2) $\rightarrow \frac{q_f}{\bar{p}_e} \times \frac{\bar{p}_e}{\sigma_c'} = \frac{M}{2}(0.5)^m (OCR)^m$

$$\rightarrow \boxed{q_f/\sigma_c' = \underbrace{\frac{M}{2}(0.5)^m}_{\text{Value at OCR=1 (= S in SHANSEP Eqn)}} (OCR)^m}$$

NOTES: (1) Similar form to 1.361, $\frac{s_u}{\sigma_c'} = S\,(OCR)^m$

(2) Tends to overpredict effect of OCR

(3) $s_u(TE) = s_u(TC)$ not realistic

3.9 Predicted ESP for CIU Tests at OCR ≥ 1

(Use Cambridge $\bar{q}$ vs $\bar{p}$ → TC = TE)

(1) At OCR = 2.0: Vertical ESP → simultaneous yielding and failure

(2) At OCR < 2.0

See Fig. 1 (Sheet C) for example with $\sigma_c' = 4$, $\sigma_{cm}' = 6$; OCR = 1.5

- Get vertical ESP until intersect YS for $\bar{p}_m = 6$
- Then follows ESP on end of SBS for $\bar{p}_e$ having same e_f $[\bar{p}_e = \bar{p}_c(OCR)^m]$ = NC CIU ESP with $\sigma_c' = \bar{p}_e$. Continued shearing → increasing $\bar{p}_m$

(3) At OCR > 2.0

See Fig. 2 (Sheet D) for ESP normalized to $\bar{p}_e$ at OCR = 1.0, 1.5, 2 ···· 10

(i.e., treat as results for tests with same e_f, but varying $\bar{p}_m$)

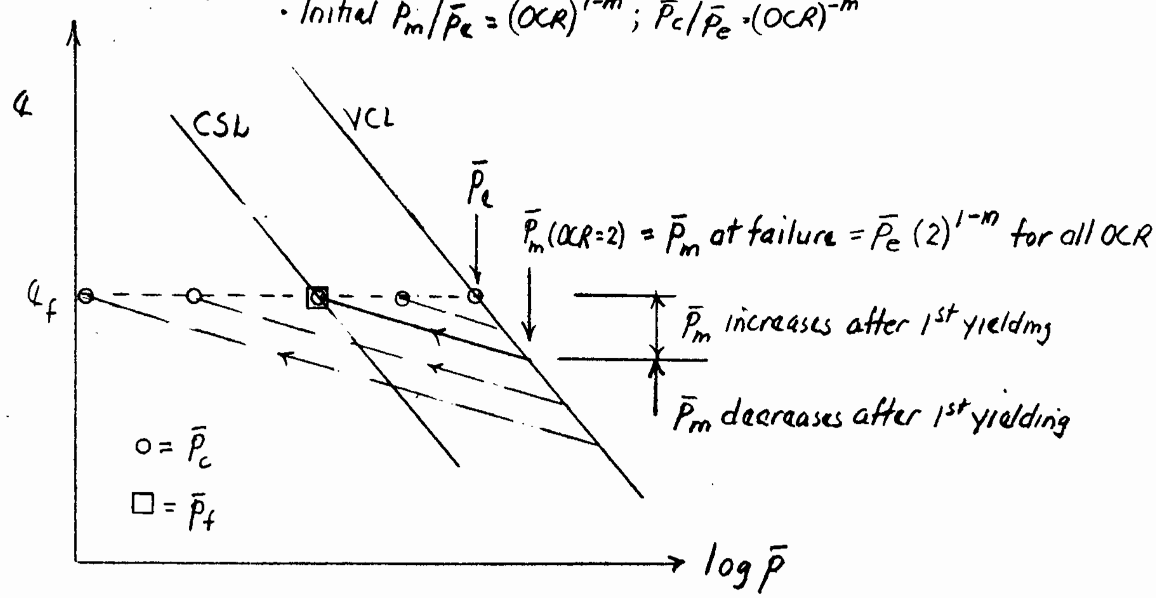
Eqn hold at all OCRs

- At consolidation: $\bar{p}_c/\bar{p}_e = 1/(OCR)^m = (OCR)^{-m}$
- At 1st yield: $\bar{p}_y = \bar{p}_c$ since vertical ESP ; $\bar{q}_y/\bar{p}_e = \dfrac{M\sqrt{OCR-1}}{(OCR)^m}$
- Thereafter follows SBS → $\dfrac{\bar{p}}{\bar{p}_e} = \left(\dfrac{M^2}{M^2+R^2}\right)^m$, where $R = \bar{q}/\bar{p}$
- At failure (CSL), $\bar{p}_f/\bar{p}_e = (0.5)^m \rightarrow \bar{p}_f/\underbrace{\bar{p}_{1st\,yield}}_{=\bar{p}_c} = (\tfrac{1}{2}OCR)^m$

continued shearing → decreasing $\bar{p}_m$

(4) Summary

- Initial $\bar{p}_m/\bar{p}_e = (OCR)^{1-m}$; $\bar{p}_c/\bar{p}_e = (OCR)^{-m}$



3.10 Summary of Input Parameters for MCC

(1) Compressibility ($K_c = 1$)

- NC e vs $\log \sigma'_c = \log \bar{p}_e \rightarrow C_c = -de/d\log\sigma'_c$ ($\lambda = -de/d\ln \bar{p}_e$)
- Swelling = recompression line $\rightarrow C_s$ ($\kappa = C_s/2.3$) & $m = (1 - C_s/C_c)$

(2) Failure envelope for NC clay, e.g. ϕ'_{TC}

- $M = \bar{q}_f/\bar{p}_f = f(\phi') = (6\sin\phi')/(3-\sin\phi')$ for $\phi' = \phi'_{TC}$

(3) One elastic parameter to combine with (1) to give elastic stress-strain behavior before yielding

$$\left\{ K' = \frac{\delta\bar{p}}{\delta v} = \frac{E'}{3(1-2\nu')} = \frac{2G(1+\nu')}{3(1-2\nu')} \right\}$$

usually selected

3.11 Summary of Relationships for CIUC/E Tests

(1) At Consolidation: $\bar{p}_c/\bar{p}_m = 1/OCR$; $\bar{p}_m/\bar{p}_e = (OCR)^{1-m}$; $\bar{p}_c/\bar{p}_e = (OCR)^{-m}$

(2) At 1st (initial) Yielding $[\bar{p}_Y = \bar{p}_c = \sigma'_c = \bar{p}_m/OCR = \bar{p}_e(OCR)^{-m}]$

$$\therefore \frac{\bar{q}_Y}{\bar{p}_c} = M\sqrt{OCR-1} \; ; \; \frac{\bar{q}_Y}{\bar{p}_m} = \frac{M\sqrt{OCR-1}}{OCR} \; ; \; \frac{\bar{q}_Y}{\bar{p}_e} = \frac{M\sqrt{OCR-1}}{(OCR)^m}$$

(3) At Failure $[\bar{q}_f = \bar{p}_f M \; ; \; \bar{p}_f/\bar{p}_e = (0.5)^m]$

- $\bar{q}_f/\sigma'_c = \underbrace{\frac{M}{2}(0.5)^m}_{S}(OCR)^m$
- $\dfrac{\bar{p}_f}{\sigma'_c} = \left(\dfrac{OCR}{2}\right)^m$
  - $\rightarrow$ OCR < 2 : decreasing $\bar{p}$ & increasing $\bar{p}_m$ of YS
  - $\rightarrow$ OCR > 2 : increasing $\bar{p}$ & decreasing $\bar{p}_m$ of YS
- $\dfrac{\bar{p}_f}{\bar{p}_m} = (0.5)^m (OCR)^{m-1}$

4. COMPARISON OF MCC WITH SIMPLE CLAY (CIUC)

4.1 s_u/σ'_c vs. OCR

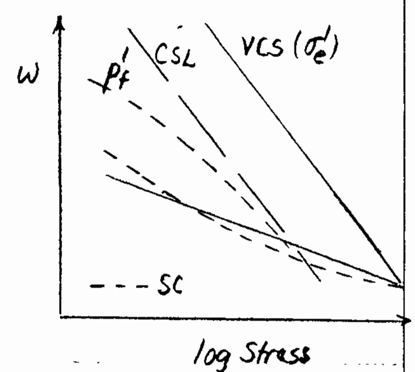
(1) Selected parameters

- $\sin\phi'_{TC} = 0.3916 \rightarrow M = 0.9008 \simeq 0.90$
- $m = (1 - C_s/C_c) \rightarrow 0.65 \simeq$ ave. value at OCR = 8

OCR	$(1 - C_s/C_c)$
2	0.73^5
4	0.68
8	0.65
16	0.63

(2) Resultant ratios

OCR	q_f/σ'_c MCC	SC	$\frac{MCC}{SC}$
1	0.287	0.290	0.99 } very
4	0.706	0.696	1.01^5 } close
12	1.442	1.166	1.24



Note: $m = 0.7 \rightarrow q_f/\sigma'_c = 1.58$ at OCR = 12 (+35%)

(3) Discussion $\rightarrow$ why MCC overpredicts s_u at high OCR

4.2 Stress-Strain and ESP at OCR ≥ 1

(1) See Fig 3 (Sheet E) for comparison. Stress-strain curves required computer program done by M. Kavvadas who developed MIT-E1 (for anisotropic NC clay)

(2) At OCR = 1; MCC $\rightarrow$ lower ESP, but stiffer response

" " = 2, MCC $\rightarrow$ linear ESP, same s_u and much stiffer, linear q vs ε_a

(3) At OCR = 8, MCC $\rightarrow$ ESP that goes far above Hvorslev envelope (with $A = A_e = 1/3$) before yielding; p' then increases significantly to reach "failure" at CSL

(4) Questions

- Are CSL and Hvorslev concepts incompatible?
- Can difference be attributed to experimental errors?

5. DISCUSSION

5.1 Comments on MCC

(1) Represented very significant advance (in late 1960s) that wasn't appreciated by geotechnical engr. profession for > 10-15 yr.

(2) Highly innovative and elegant for its simplicity (but need computer program to obtain stress-strain curves). Formed the basis for Critical State Soil Mechanics (CSSM) that is still widely taught at some major universities (esp. in England) and assumed in practice (e.g., unique CSL).

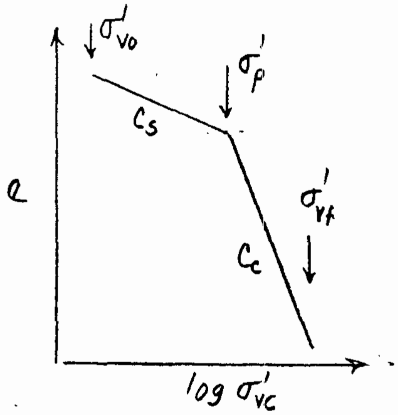
e.g., Wood, D.M. (1990). Soil Behavior and Critical State Soil Mechanics, Cambridge Univ. Press, 462p

(3) Most popular & widely used clay model in finite element computer codes (e.g., ABAQUS,)
but using ϕ' = constant rather than M = constant

(4) However, has severe limitations when applied to natural OC clays
- Elastic behavior at OCR > 1
- Too high s_u at OCR > ≈ 2
- No s_u anisotropy

5.2 MCC 1-D Finite Element Consolidation Analyses

(1) Objective is to have correct e vs $\log \sigma'_{vc}$ for K_0 loading for each layer.



(2) How MCC operates: $\nu' = 0.3,\ M = 1.2,\ m = 0.8$

- $K_0(OC) = \dfrac{\nu'}{1-\nu'}$ $\rightarrow K_0 = 0.43$
- $K_0(NC) = f(\nu', M\ \&\ m)$ $\rightarrow K_0 = 0.65$

$\therefore$ OC clay will yield at $\sigma'_{vy} < \sigma'_p$

(3) Therefore need to select ν', M & m $\rightarrow$ same K_0 for OC & NC clay.
Will cover under Part C

5.3 MIT-E3 Model of Clay Behavior

(1) Formulation described by Whittle & Kavvadas (1994), ASCE JGE, 120(1), 173-198

(2) Three major components

1. Elasto-plastic model for NC clay that incorporates anisotropy and strain-softening
   - See Fig. 2, Sheet F1, for rotated yield surface = bounding surface
   - " Fig. 4, " , for NC CK_0UPSC/E

2. Eqn. to describe small strain nonlinearity and hysteretic response in unloading/reloading
   - See Fig 1a, Sheet F1, for hysteresis in e vs log σ_c'
   - " Fig. 4, " , for nonlinear small strain behavior

3. Bounding surface plasticity for irrecoverable, anisotropic and path-dependent behavior of OC clays
   - See Fig 1b, Sheet F1, for plastic strain (Δ^P) for reloading to VCL
   - " Fig. 2, " " , bounding surface plasticity
   - " Fig. 4, " " , for anisotropy of OC clay

(3) Input parameters : Table 2, Sheet F2 → 15 parameters

a) 1-D consolidation data with measurement of K_0 → 7 parameters

b) CK_0UC at OCR = 1 & 2, CK_0UE at OCR = 1 → 6 parameters

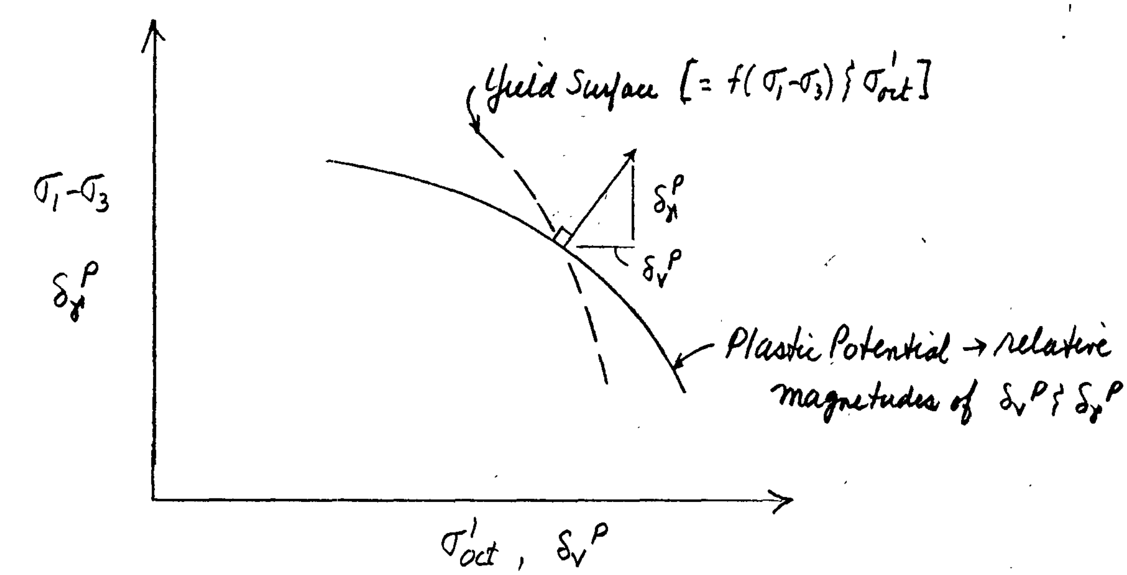
c) Resonant column or in situ shear wave velocity → 1 parameter (G_{max})

d) Special test to measure "evolving" anisotropy → 1 parameter (ψ_0) (rotation of bounding surface)

(4) See Tables 1 & 2 for values of parameters for three clays

Plastic Potential

Note: Associated flow rule = normality if Plastic Potential = Yield Surface

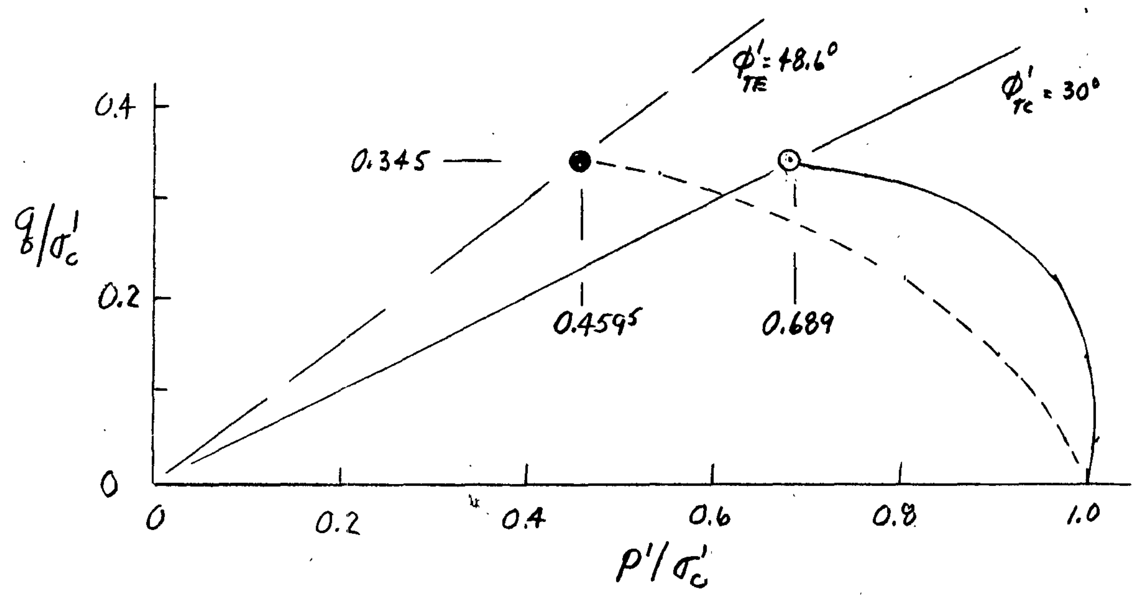


MCC CIUC/E OCR=1 (M=1.2 & m=0.8)

Note: Using MIT q vs p' → $q_f/\sigma'_c = 0.344^6 = 0.345$

—o— TC : $(\Delta u - \Delta\sigma_3)/\sigma'_c = 0.655$ → $A_f = 0.95$

- - ● - - TE : " $= 0.885$ → " $= 1.28^5$



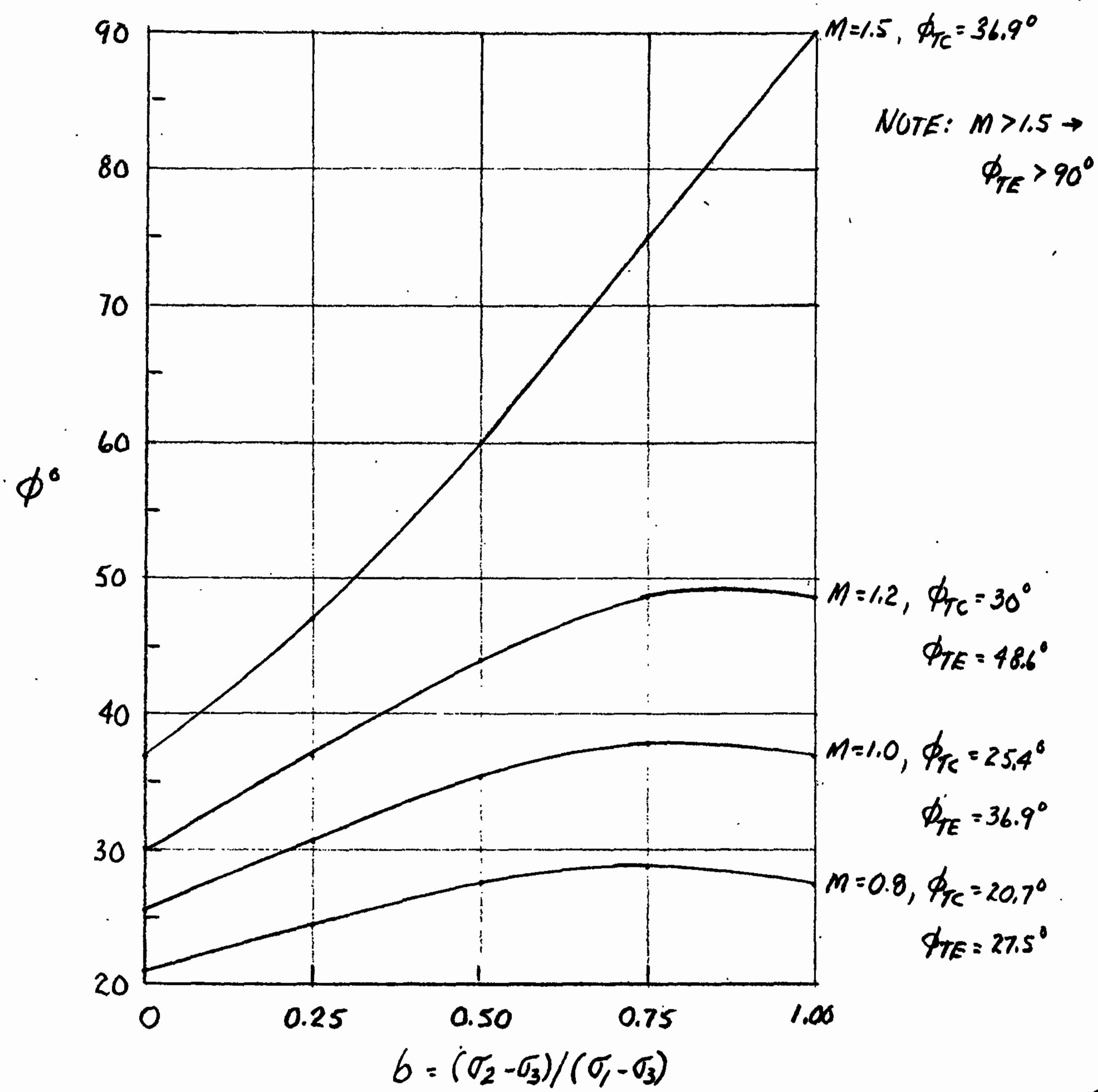
A

Extended von Mises Failure Criterion

From Bishop (1971) Roscoe Memorial Volume:

- $\frac{q^*}{\sigma_{oct}} = M$ with $q^* = \frac{1}{\sqrt{2}}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_1-\sigma_3)^2+(\sigma_2-\sigma_3)^2}$

- Defining $b = \frac{\sigma_2-\sigma_3}{\sigma_1-\sigma_3} \rightarrow \sin\phi = \frac{3M}{M(1-2b)+6\sqrt{1-b+b^2}}$



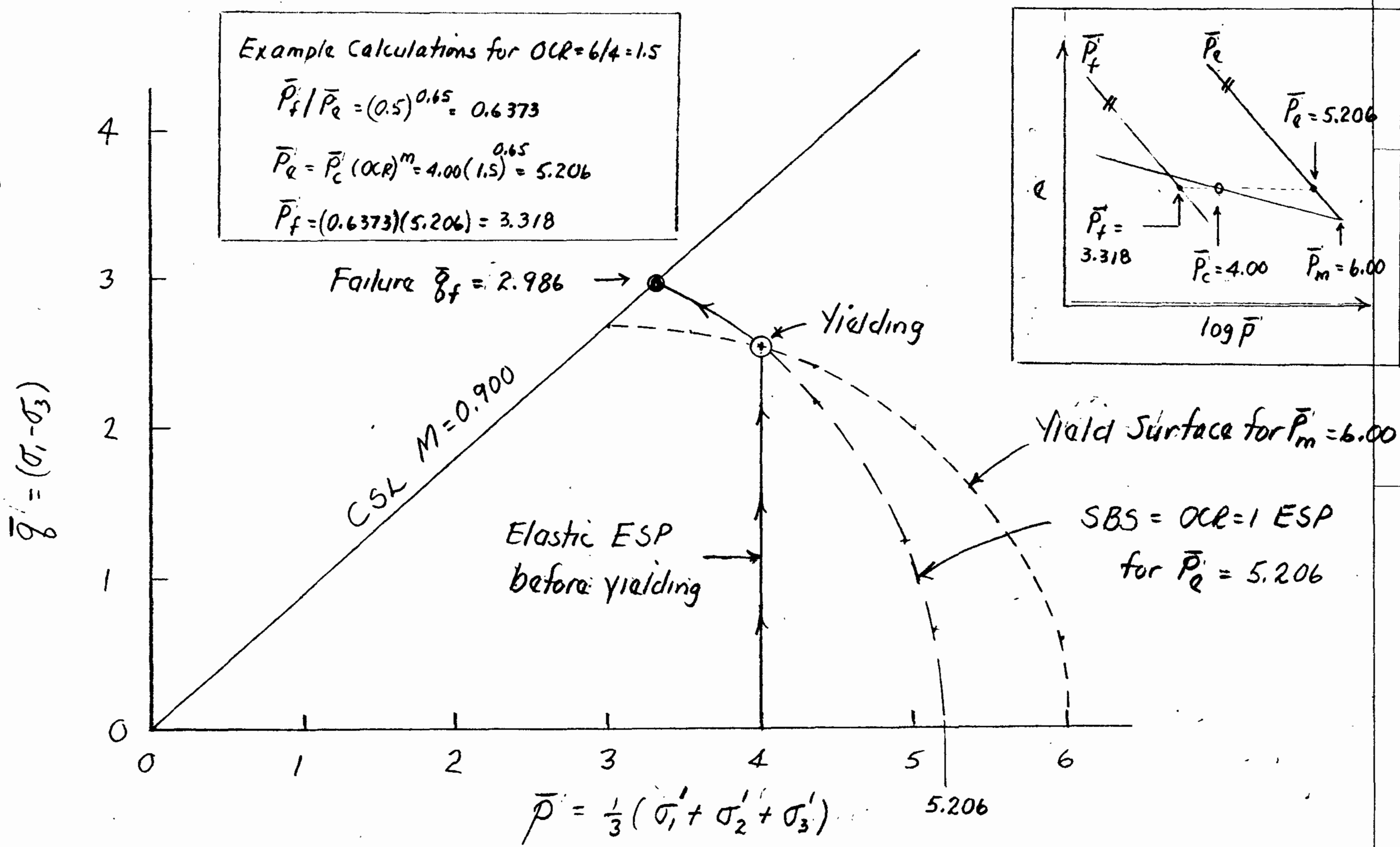
B

CCL 2/22/87 2/90

1.322 2/96

Fig. 1 ESP for CIUC/E OCR = 1.5 ($\bar{\sigma}'_c = 4.00$ & $\sigma'_{cm} = 6.00$)

Predicted via MCC for $M = 0.900$ and $m = 0.650$



C

CCL 2/25/93 1.322
2/24/96

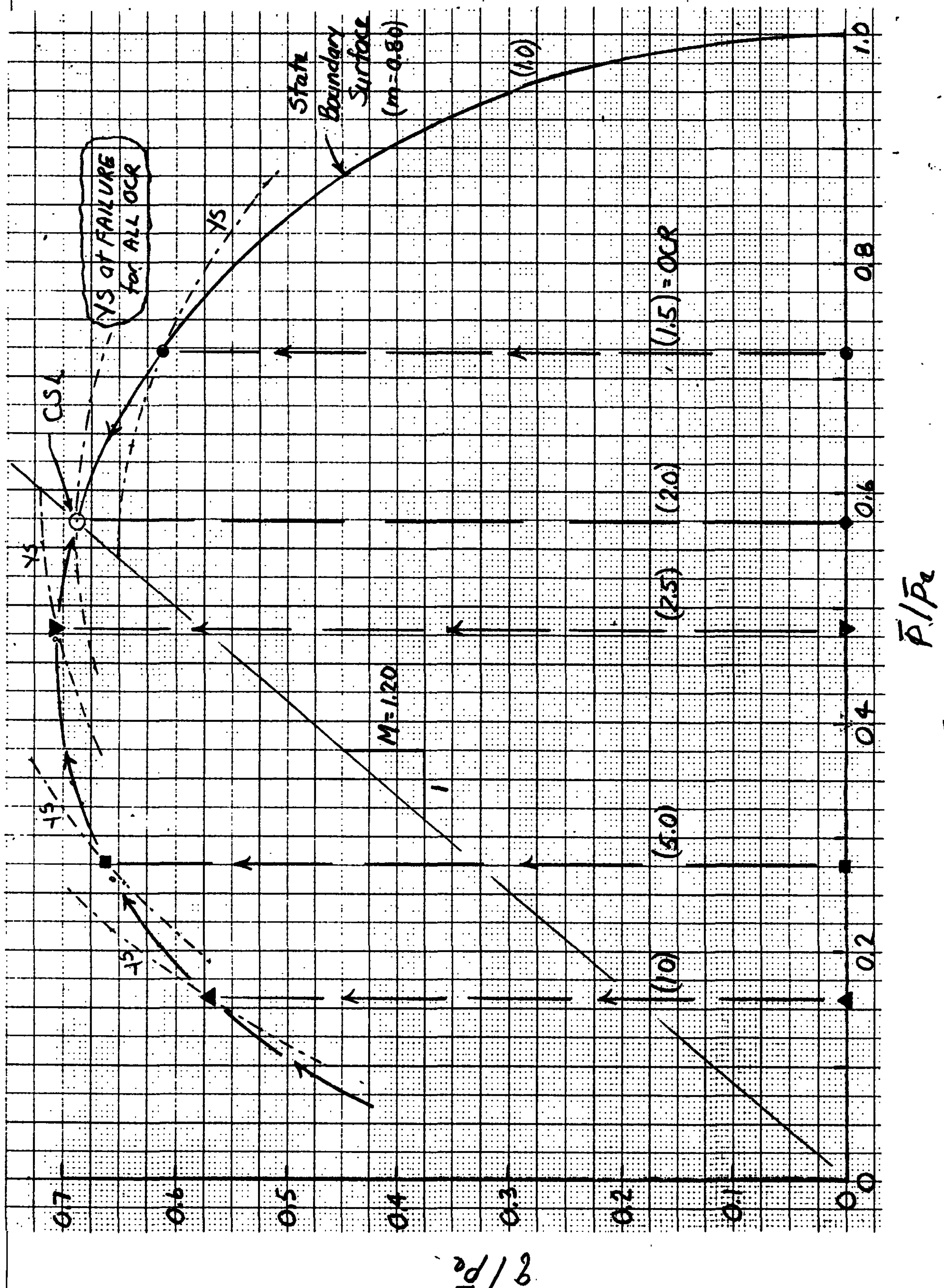
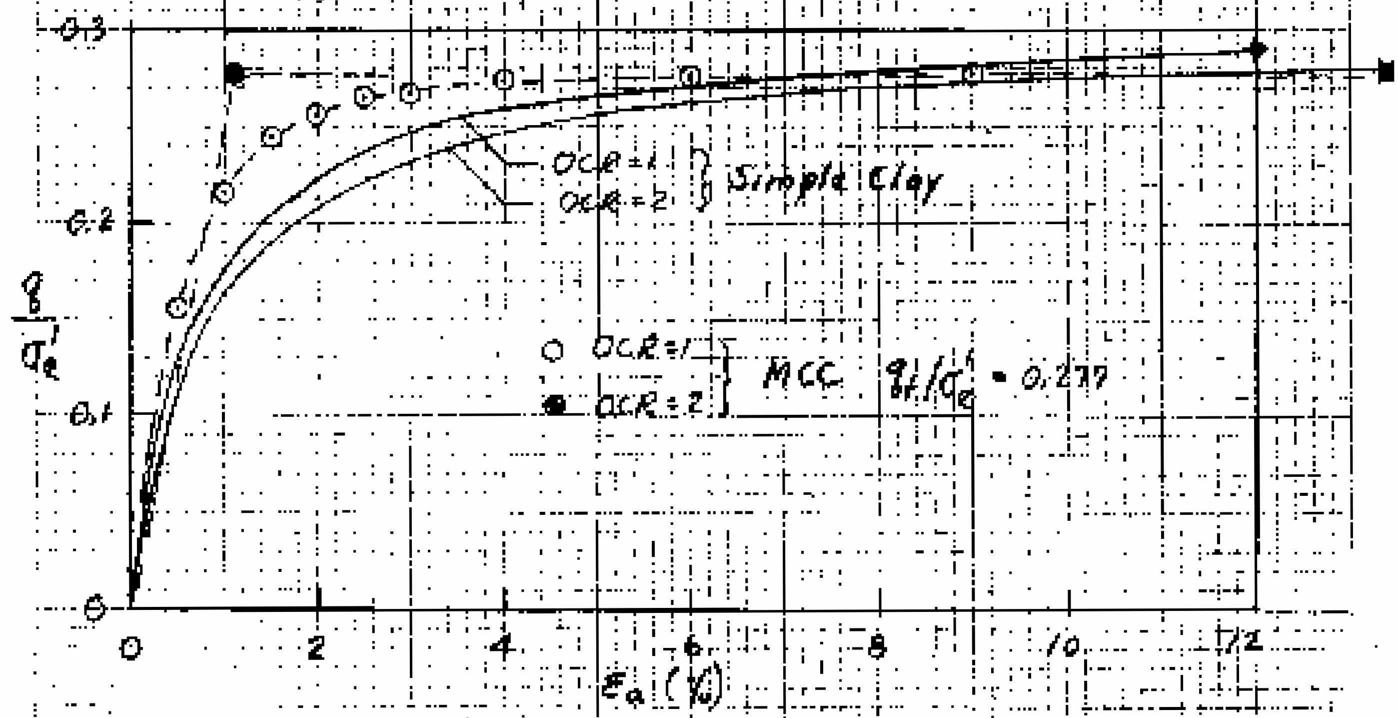


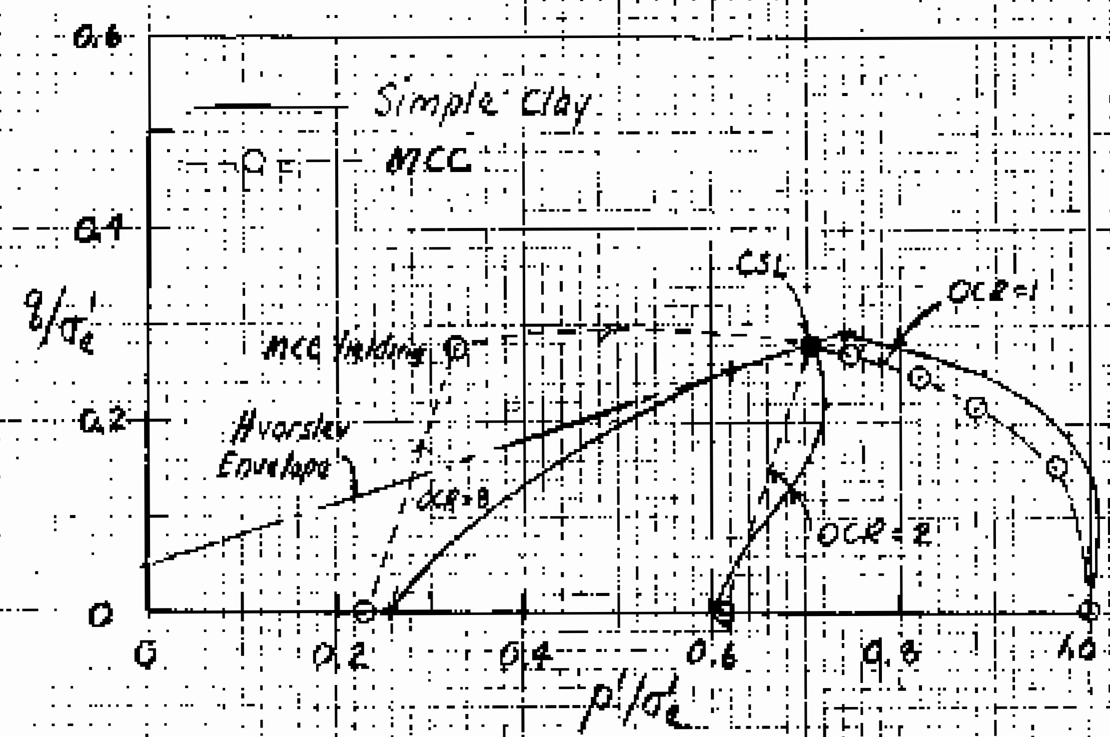
Fig.2 ESP for CIUC/E: After Initial Yielding $\frac{\bar{p}}{\bar{p}_c} = \left(\frac{M^2}{M^2+R^2}\right)^m$

D

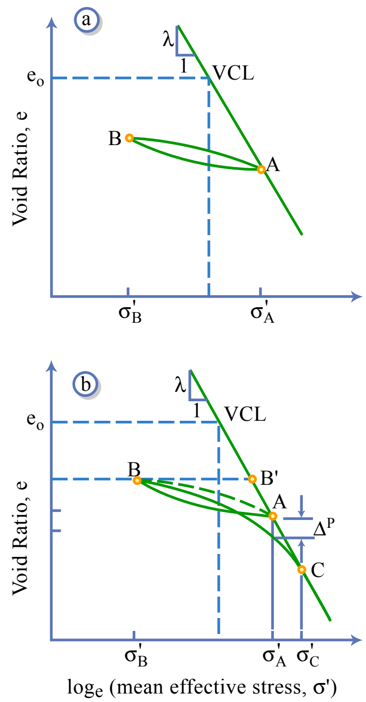
CCL 3/19/81 2/87 2/88 2/96 2/98

Fig. 3. Comparison of MCC vs Simple Clay CIUC

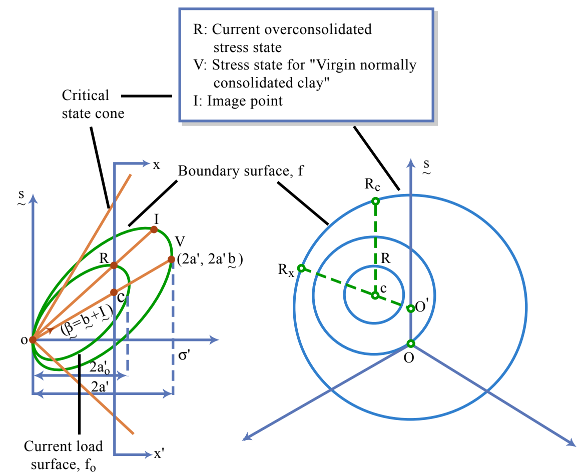




Adapted from Whittle & Kavvadas (1994)

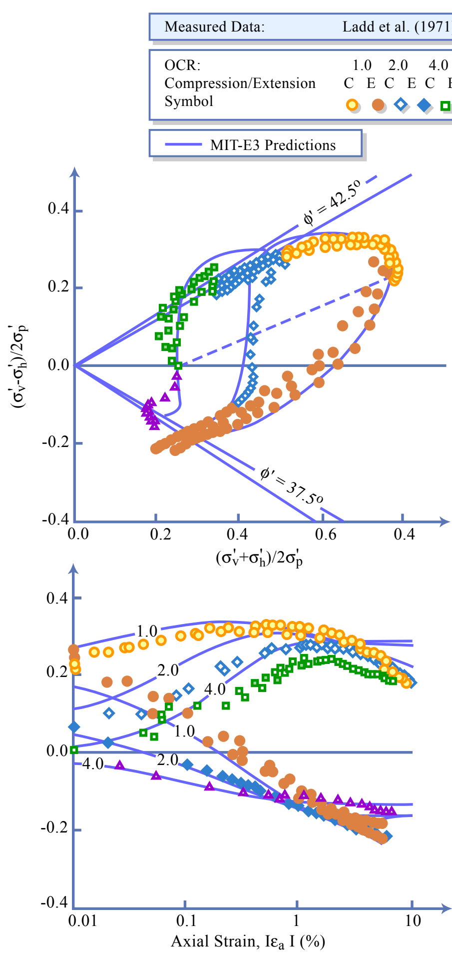


Conceptual Model of Unload-Reload Used by MIT-E3 for Hydrostatic Compression:
(a) Perfect Hysteresis; and (b) Hysteresis and Bounding Surface Plasticity.



Yield, Failure, and Load Surfaces used in MIT-E3 Model

Adapted from Whittle et al. (1994) ASCE JGE 120(1)



Evaluation of MIT-E3 Predictions of Undrained Plane Strain Shear Tests on K_o-Consolidated BBC

Figures by MIT OCW.

Whittle (1993) Geotechnique 43(2), 289-313.

Table 1. Average index properties of three clays

Property	Boston blue clay	Empire clay	London clay
w_L: %	42	76	75
w_P: %	21	26	28
I_P: %	21	50	47
I_L: %	95	36	5

Table 2. Input parameters for the MIT-E3 model

Test type	Parameter/symbol	Physical contribution/meaning	Boston blue clay	Empire clay	London clay
One-dimensional consolidation (oedometer CRS, etc.)	e_0	Void ratio at reference stress on virgin consolidation line	1·12	1·26	1·21
	λ	Compressibility of virgin normally consolidated clay	0·184	0·274	0·172
	C	Non-linear volumetric swelling behaviour	22·0	24·0	65·0
	n		1·60	1·75	1·50
	h	Irrecoverable plastic strain	0·2	0·2	0·1
K_0-oedometer or K_0-triaxial	K_{0NC}	K_0 for virgin normally consolidated clay	0·48	0·62	0·62
	$2G/K$	Ratio of elastic shear to bulk modulus (Poisson's ratio for initial unload)	1·05	0·86	0·99
Undrained triaxial shear tests: degrees OCR = 1; CK_0UC OCR = 1; CK_0UE OCR = 2; CK_0UC	ϕ'_{TC}	Critical state friction angles in triaxial compression and extension (large strain failure criterion)	33·4°	23·6°	22·5°
	ϕ'_{TE}		45·9°†	21·6°	22·5°
	c	Undrained shear strength (geometry of bounding surface	0·86	0·75	0·80
	s_t	Amount of post-peak strain softening in undrained triaxial compression	4·5	3·0	3·9
	ω	Non-linearity at small strains in undrained shear	0·07	0·20	0·20
	γ	Shear induced pore pressure for OC clay	0·5	0·5	0·5
Resonant column*	κ_0	Small strain compressibility at load reversal	0·001	0·0035	0·001
Drained triaxial	Ψ_0	Rate of evolution of anisotropy (rotation of bounding surface)	100·0	100·0	100·0

* Alternatively use field data from cross-hole shear wave velocity type tests.
† Recent data (Germaine, 1989) suggest $\phi'_{TE} \approx 40°$.

CONSOLIDATION BEHAVIOR OF SATURATED SOILS

Part I INTRODUCTION

NOTE: Will consider in situ testing during term in order to estimate following properties

	EPC	SBPT	DMT	FVT	CPTU
K_o	✓	✓	✓		
Stress History			✓	✓	✓
s_u		✓	✓	✓	✓
c_h					✓

CONSOLIDATION BEHAVIOR OF SATURATED SOILS

1. BACKGROUND

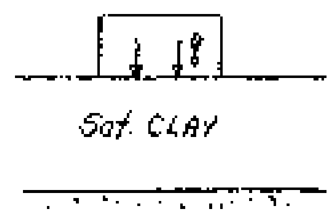
1.1 Difference Between:

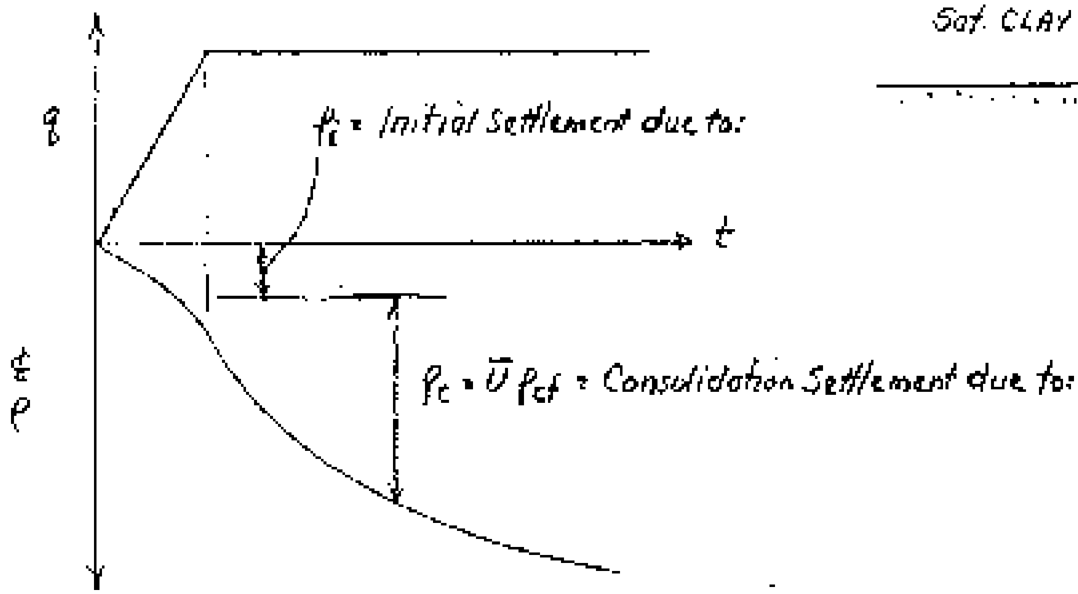
- Compaction

- Consolidation

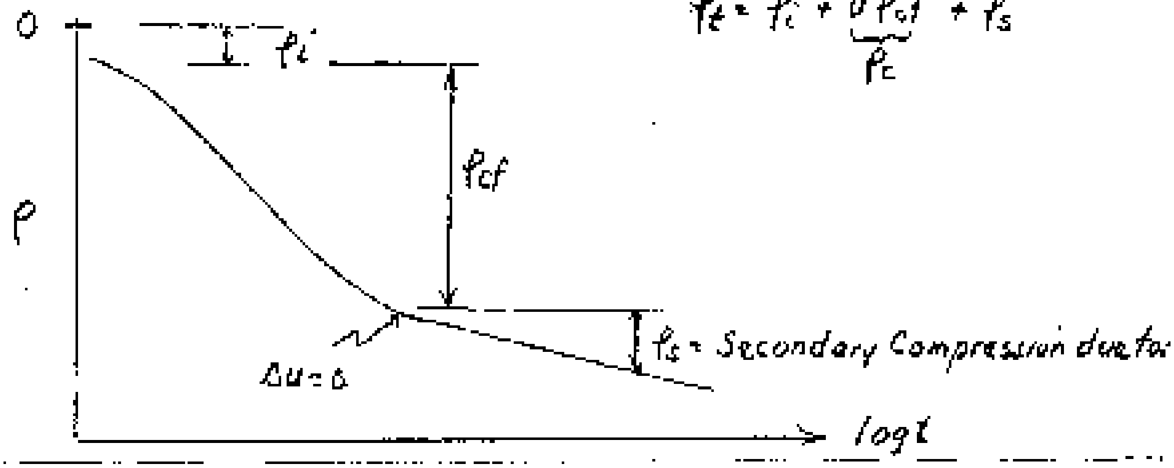
- Drained Shear

1.2 Types of Settlement





$$\rho_t = \rho_i + \underbrace{\bar{U}\rho_{cf}}_{\rho_c} + \rho_s$$



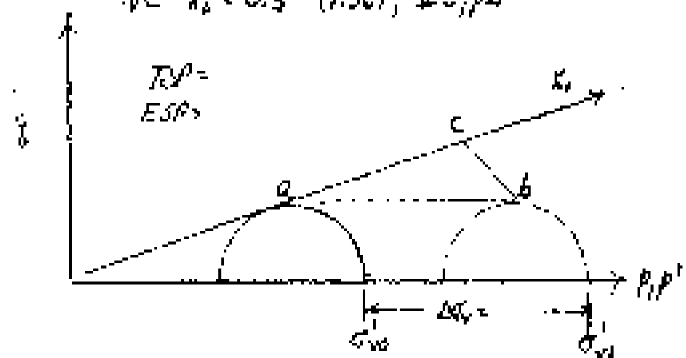
2. COEFFICIENT OF EARTH PRESSURE AT REST (K_0): BEHAVIORAL TRENDS

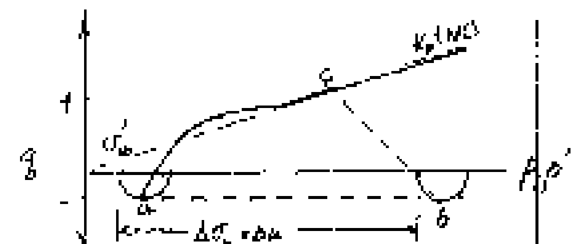
2.1 Relevance - Importance

- Lab reconsolidation CK_0UC → in situ σ-ε properties
- Stresses on underground structures, e.g. retaining walls, tunnels, etc.
- Predictions of deformations due loading/unloading
  & especially "local yielding" $f = (1-K_0)/(2q_f/\sigma'_{v0})$

2.2 Stress Paths: 1-D Consolidation

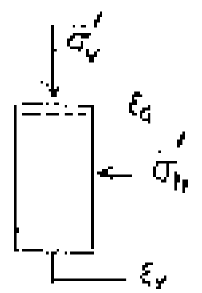
N.C. $K_0 = 0.5$ (1.361, #5, p2) O.C. $K_0 = 2.0$

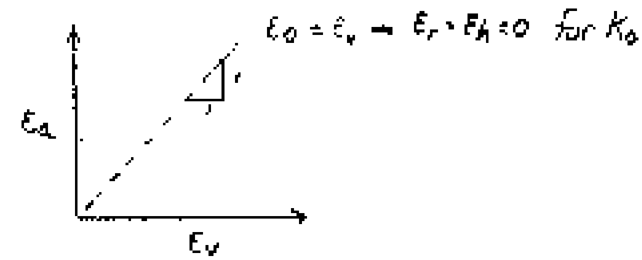




2.3 Lab Measurements of K_0

1) Triaxial: Stress Path Cell (p2a for data from MIT automated CK_0-TX)





2) Instrumented Oedometer

- Square with pressure transducer (R.S. Ladd, 1965)

- Circular with fluid chamber

Brooker & Ireland, 1965 } U of I
Hendron, Ph.D. }
R.J. Martin - MIT
Mesri et al. 1993 U of I (p2b)

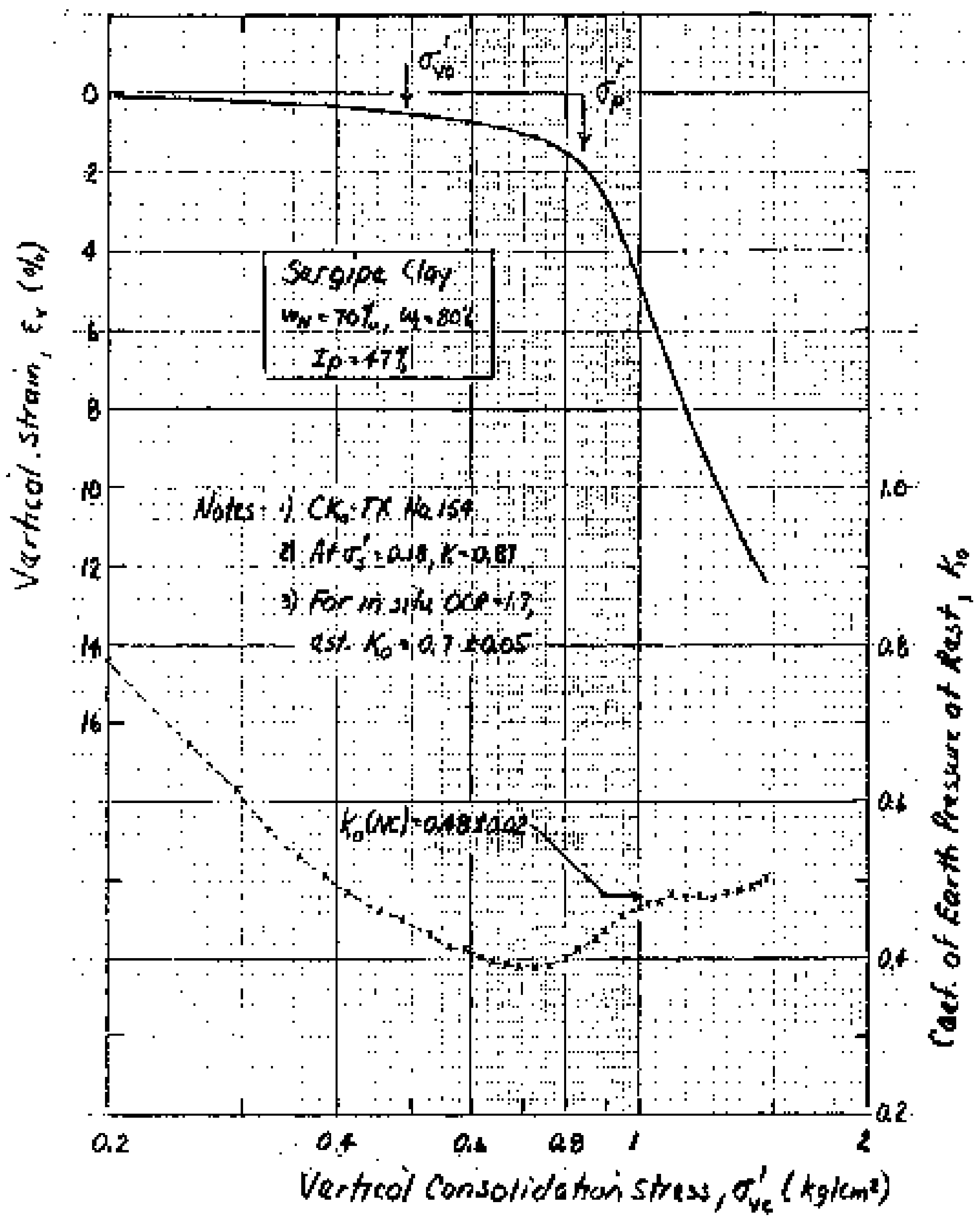
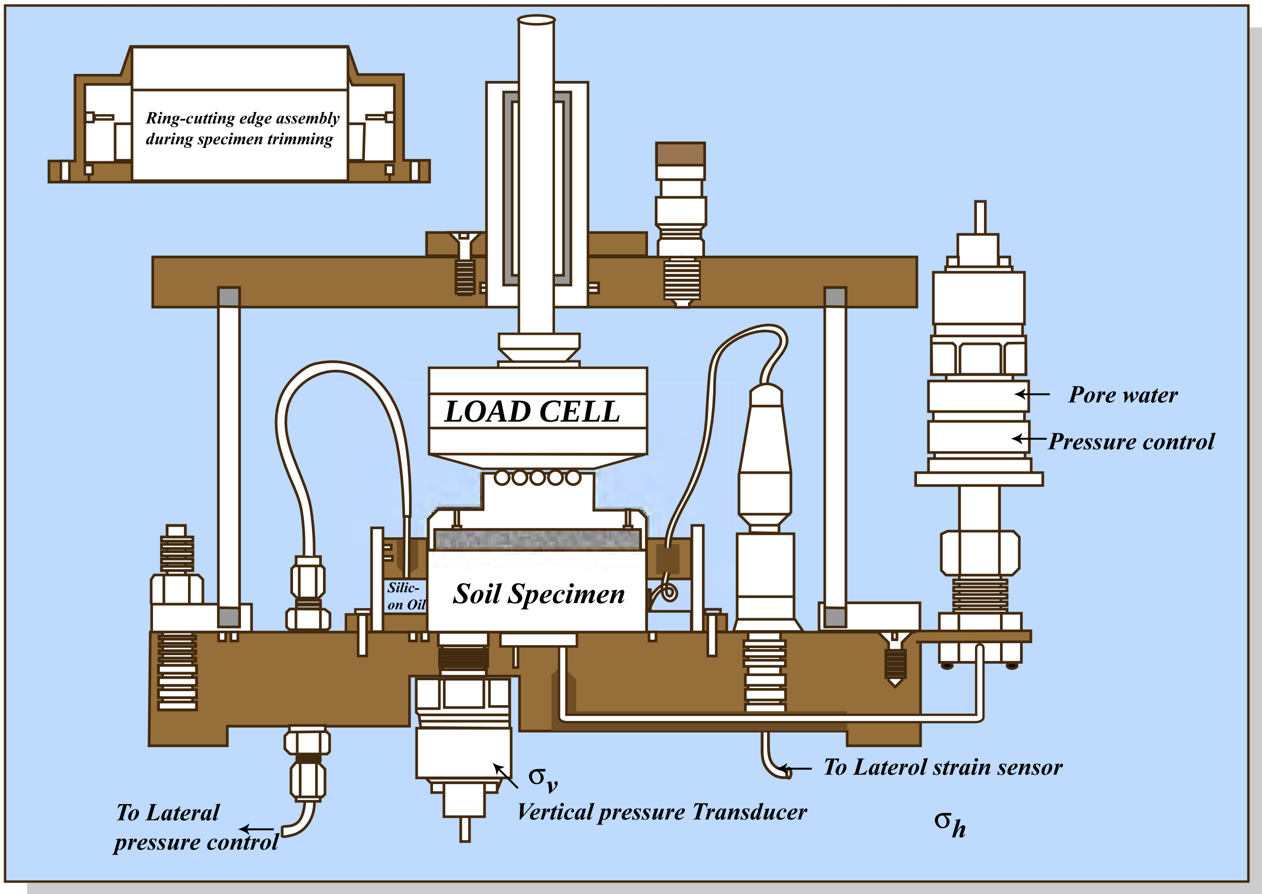
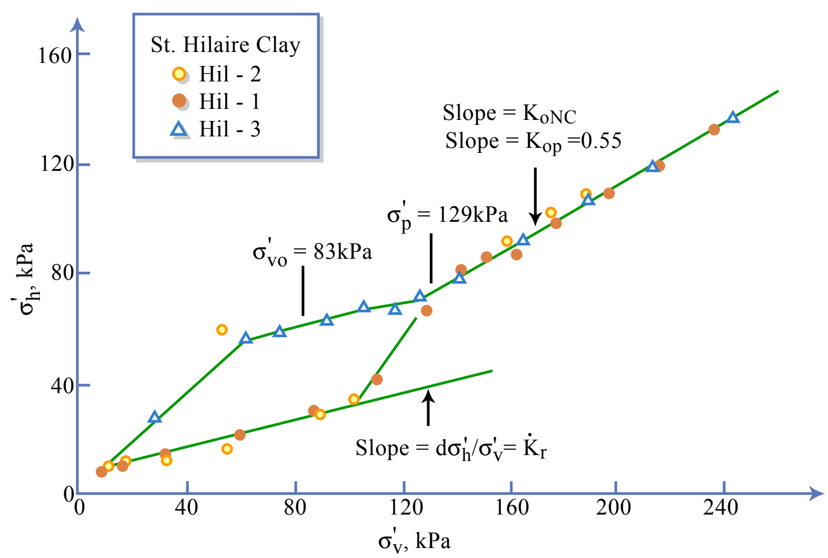


Fig. Consolidation Data From MIT Automated Stress Path Triaxial Apparatus During 1-D Compression Of Undisturbed Soft Clay

Adapted from: Mesri, G. & Hayat, T.M. (1993). "The coefficient of earth pressure at rest", CGJ, 30(4), 647-666



Special oedometer for measurement of horizontal pressure, together with measurement of vertical pressure at top and bottom and pore-water pressure at bottom



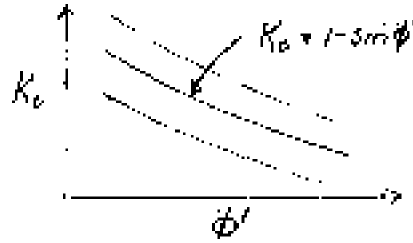
Specimens HIL-1 and HIL-2 subjected to laterally constrained (LC) compression from σ'_s, and specimen HIL-3 subjected to LC compression after consolidation under equal all around pressure to point a.

Figures by MIT OCW.

3/5/95

2.4 Normally Consolidated K_0

1) Jaky (1944) Empirical correlation: $K_0 = 1 - \sin\phi'$



ϕ'	K_0
20	0.66
30	0.50
40	0.36

NOTE: Elastic Theory

$K_0 = \frac{\nu'}{1-\nu'}$ $\quad \nu' = 1/3 \rightarrow K_0 = 0.5$

2) Tokyo SOA (p4) + Mesri & Hayat, 1993 (p4a)

- Sands Fig 14 $K_0 = 0.4 \pm 0.1$ $1-\sin\phi'$ not so good
  M & H, 93 $= 0.5 \pm 0.1$ $1-\sin\phi_{cv}$ is good
- Clays Fig. 30 $K_0 \approx 1 - \sin\phi'$ with SD ± 0.05, quite good
  $\approx 0.45 - 0.7$

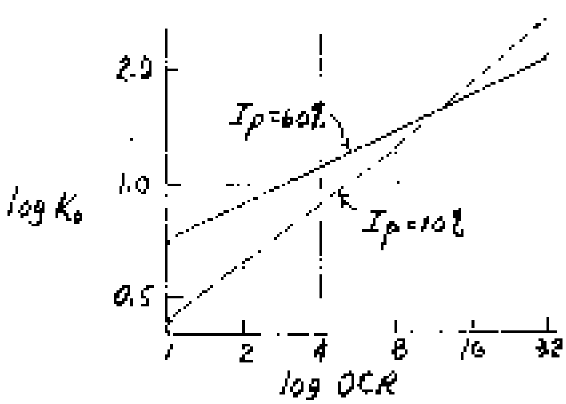
3) Mayne & Kulhawy (1982) JGED GT6

- Sands (n = 90) $K_0 = 1 - 0.988 \sin\phi'$ ($r^2 = 0.39$)
- Clays (n = 81) $K_0 = 1 - 0.987 \sin\phi'$ ($r^2 = 0.73$)

2.5 Overconsolidated K_0

1) General trends: Clays · UNLOADING

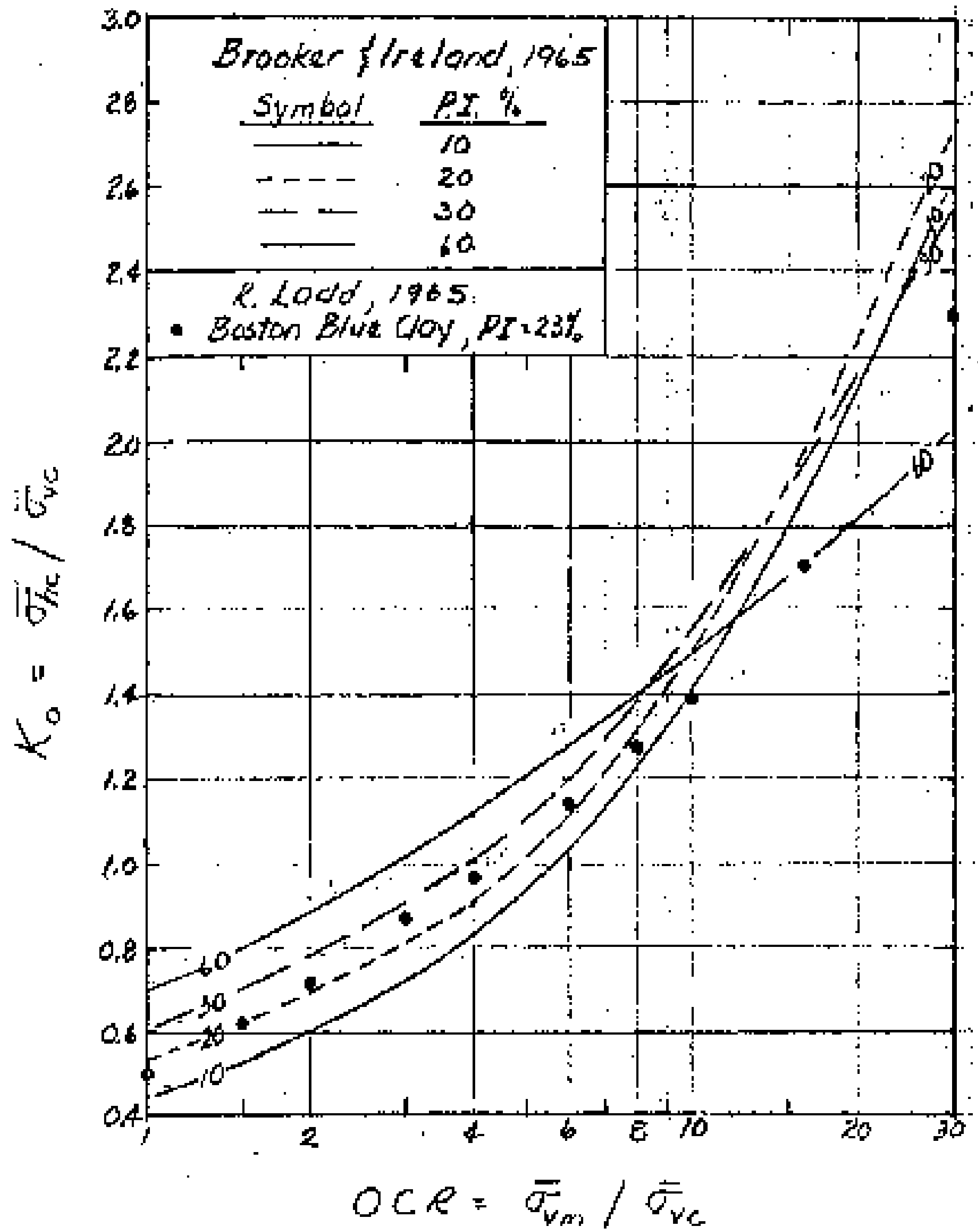
- Fig. 1-1, p3a K_0 vs log OCR Brooker & Ireland (1965) Remolded clays
- Convert to $\log K_0$ vs log OCR



$K_0 = K_{0_{NC}} (OCR)^n$

- n decreases with incr. I_p
  à la Fig. 32 (p4) Tokyo SOA

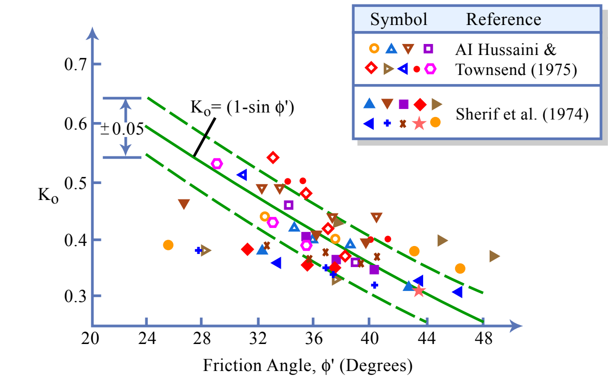
  $n = 0.4 \pm 0.05$



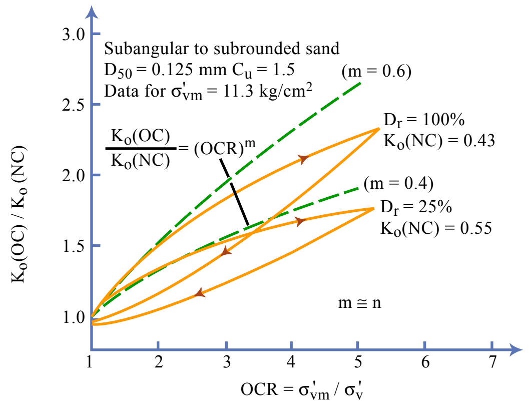
Note: Brooker & Ireland data redrawn from their Figure 11

From Ladd (1973) "Notes"

K_o VERSUS OCR FOR SOILS OF VARYING PLASTICITY



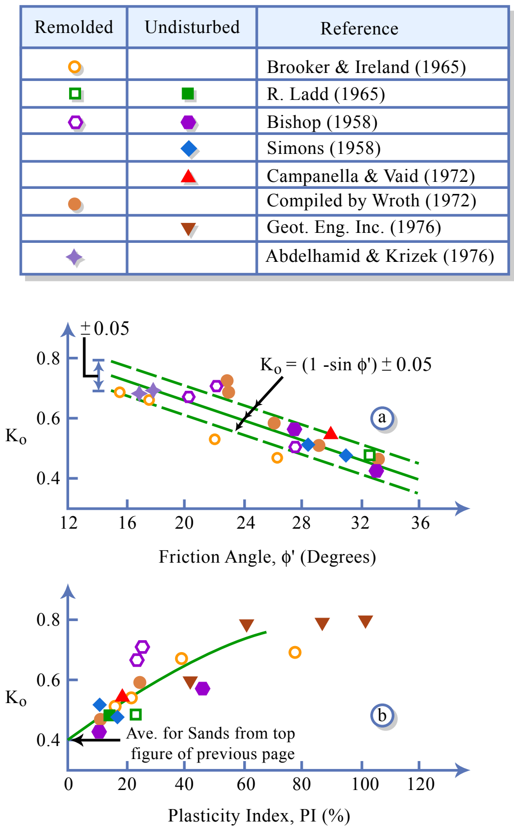
Relationship between K_o and Friction Angle for Normally Consolidated Sands.



Effect of Overconsolidation Ratio on K_o of Reid-Bedford Sand.

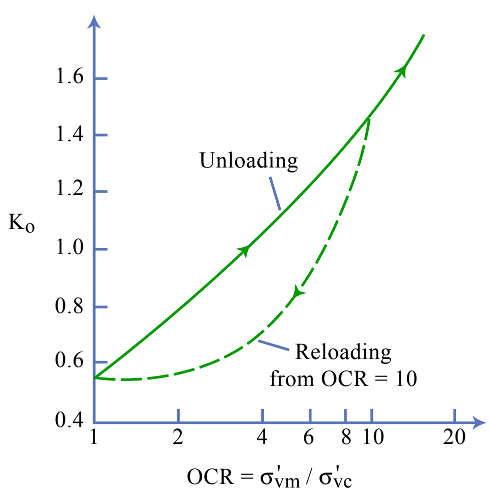
Figure by MIT OCW.

Adapted from: Al-Hussaini and Townsend, 1975.



K_o of Normally Consolidated Clays vs. Friction Angle (a) and Plasticity Index (b).

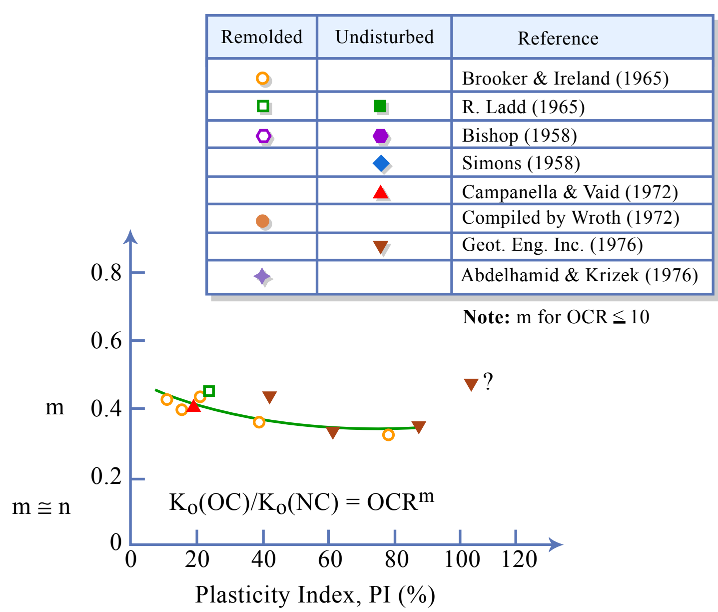
Figure by MIT OCW.



K_o vs. OCR for Haney Sensitive Clay During Unloading and Reloading.

Figure by MIT OCW.

Adapted from: Campanella and Vaid, 1972.



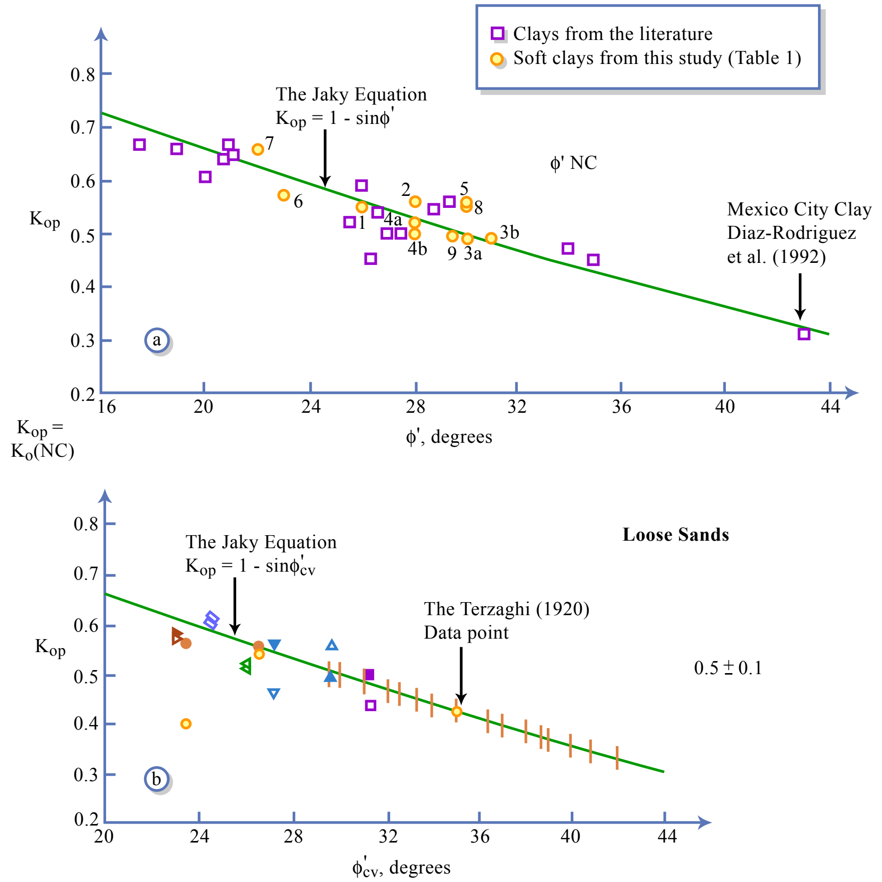
Coefficient m Relating K_o and OCR vs. Plasticity Index.

Figure by MIT OCW.

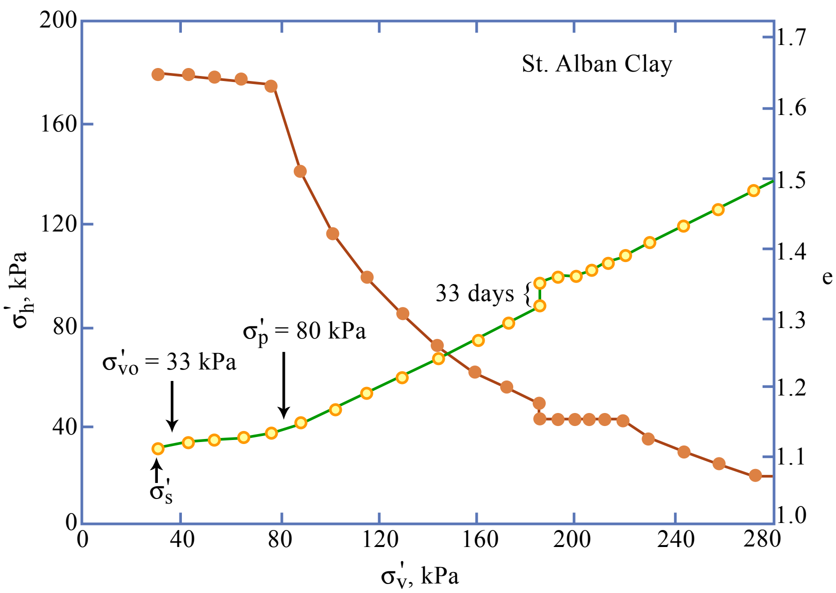
Adapted from: Ladd, et al. (1977) Tokyo SOA

CCL 3/5/95
1.322
654

p4a



(a) Data on K_{op} of clays. (b) Data on K_{op} of loose sands for which $\phi' = \phi'_{cv}$. Vertical lines indicate values of ϕ'_{cv} for granular soils assembled by Stroud (1988). Open symbols indicate K_{op} determined during increase in σ'_v; closed symbols indicate K_{op} determined during decrease in σ'_v at $\sigma'_h/\sigma'_v = 1$.



Laterally constrained secondary compression and associated increase in σ'_h in triaxial cell.

Figures by MIT OCW.

Adapted from: Mesri Hayat CAN. GEOTECH. J. VOL. 30, 1993

2) Mayne & Kulhawy (1982) : Unloading

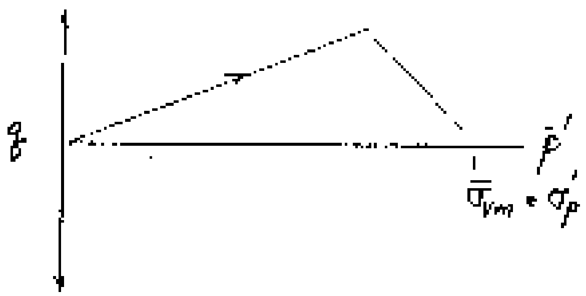
Mesri & Hayat (1993)
$n = 1 - K_{oNC}$

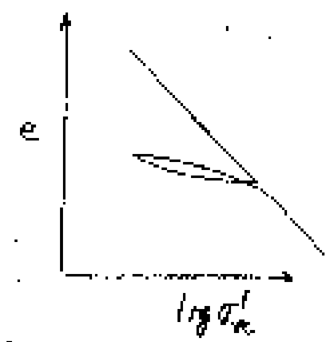
- Clays (n = 82): $n = 0.018 + 0.974 \sin\phi'$, ($r^2 = 0.45$)
- Sands (n = 107): $n = 0.929 - 0.852\, K_{oNC}$, ($r^2 = 0.58$)

$\qquad\qquad \approx 0.077 + 0.850 \sin\phi'$

$\therefore\ n \approx \sin\phi' \longrightarrow \quad K_o \approx (1-\sin\phi')(OCR)^{\sin\phi'}$ Loading/Unloading

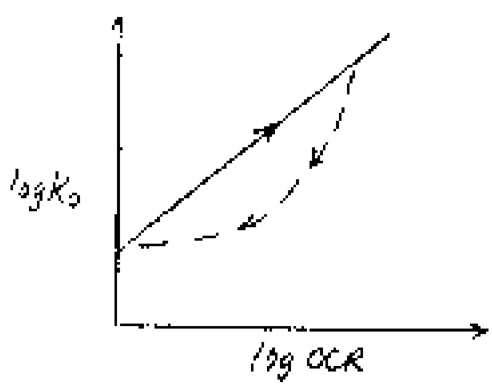
3) Limiting Value of $K_o = \dfrac{1 + (2c'/\bar{\sigma}'_{vc})\cos\phi' + \sin\phi'}{1 - \sin\phi'}$ $\left\{ \begin{array}{l} K_o = N_\phi + \dfrac{2c'\sqrt{N_\phi}}{\sigma'_{vc}} \\ \sqrt{N_\phi} = \dfrac{\cos\phi'}{1-\sin\phi'} \end{array} \right.$





4) Reloading after Unloading → Hysteresis

- Tokyo SOA (p4) Fig. 15 Sand Fig. 31 Clay



Effect of Side Friction

Unloading → K_o too high

Reloading → K_o too low

- Mayne & Kulhawy (1982) : Reloading from max. OCR

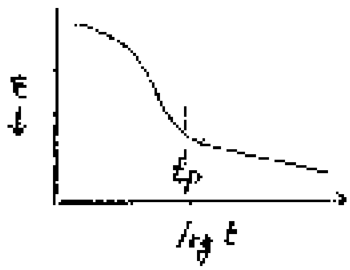
$$K_o = K_{oNC}\left[\frac{OCR}{(OCR_{max})^{1-n}} + 0.75\left(1 - \frac{OCR}{OCR_{max}}\right)\right]$$

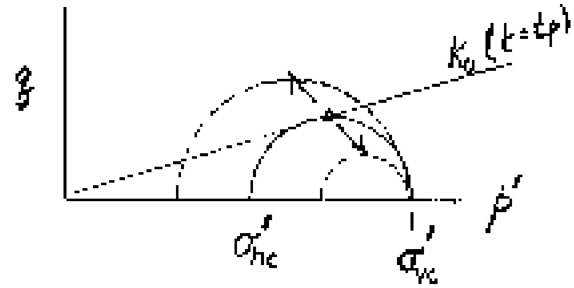
- Mesri & Hayat (1993) Inside recompression from σ'_{vo} $\quad K_o = \dot{K}_r + \dfrac{\sigma'_p}{\sigma'_v}(K_{oNC} - \dot{K}_r)$ (see p 28)

2.6 Effect of Secondary Compression on K_0

1) Schmertmann (1983) JGE, ASCE 109(1):

What happens to K_0 of NC clay during $t > t_p$?





ie: σ'_{hc} incr → K_0 incr
" const → " const
" decr → " decr

See p9a

2) References & Test Results : TRIAXIAL CELL DATA

(a) Kavazanjian & Mitchell (1984) JGE, ASCE 110(4)

(b) Discussion to above + closure (1986) 111(10)

(c) Mesri & Castro (1987) JGE, ASCE 113(3)

(d) Mesri & Hayat (1993) CGJ, 30(4)

(a) & (b) NC SFBM See p8 Figs 6 & 7 $\Delta K_0 / \Delta \log t = +0.02$

Hypothesize $K_0 \to 1$ with geologic time

(c) 4 NC Clays see p8 Table 1, Fig. 8 K_0 increases w/ log t

Replaced by p9a
Hypothesize K_0 incr. $\propto (OCR)^{n=\sin\phi}$

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

3) References & Test Results : OEDOMETER CELL DATA

(e) Jamiolkowski, et al. (1985)

(f) Holtz et al, (1986) JGE, ASCE 112(8)

(e) Undisturbed Clay OCR=1 & 10 — Sq. Oedometer - Transducer
p.9 Fig. 25 — $\Delta K_0 \approx 0$

(f) Undisturbed Clay OCR=1
p9 Fig. 2 & 4 $t/t_p \approx 10^4$

(e) 2 Undist./Remolded Clays OCR=1 — MIT LSO
p9 Fig 25 $t/t_p \approx 10^2$ — $\Delta K_0/\Delta \log t = 0.017 \pm 0.002$

4) Comparison

TX data → "large" increase with time
OED " → "no" " " "

5) Discussion Possible Experimental Errors

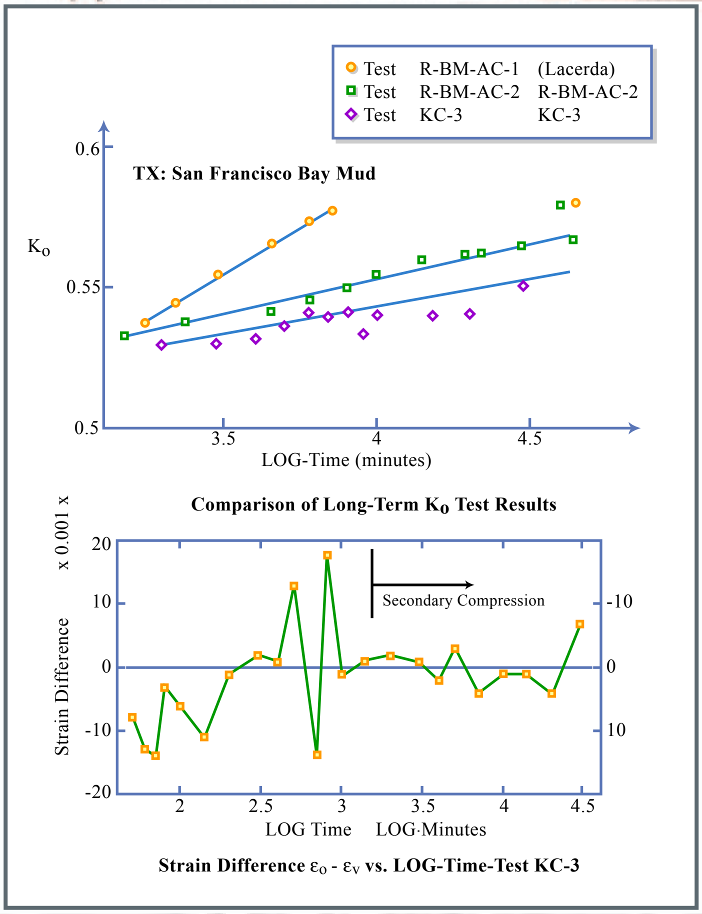
TX : Internal leakage (e.g. membrane) → K_0
External " →
"Weekend" Effect (perturbations) →

MIT LSO : Cell leakage → K_0 too low

Sq. Oed :

UofI Oed (p26) : M&H(93) say that secondary compr. → increase in side friction → reduced σ_v' → don't measure increase in K_0

6) Conclusion



Comparison of Long-Term K_0 Test Results

Strain Difference $\varepsilon_0 - \varepsilon_v$ vs. LOG-Time-Test KC-3

Figure by MIT OCW. Adapted from:

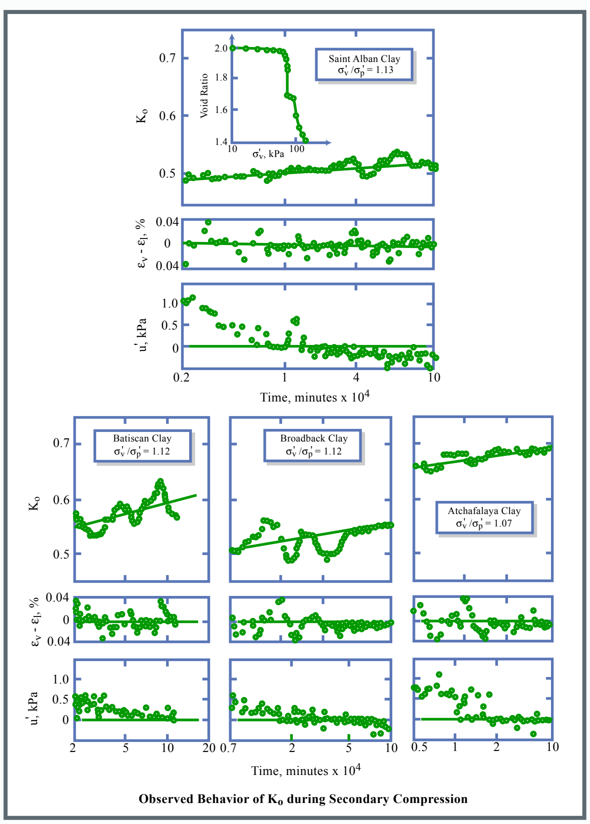
Kavazanjian & Mitchell (1985)

K_0 During Secondary:

TRIAXIAL CELL DATA

TABLE 1—Soft Clays Used in Investigation

Soft clay (1)	[illegible] (%) (2)	[illegible] (%) (3)	[illegible] (%) (4)	[illegible] (5)
Saint Alban	[illegible]	31–43	18–22	[illegible]
Broadback	[illegible]	28–36	19–23	[illegible]
Atchafalaya	57–78	62	[illegible]	[illegible]
Batiscan	[illegible]	49	22	[illegible]



Observed Behavior of K_0 during Secondary Compression

Figure by MIT OCW. Adapted from:

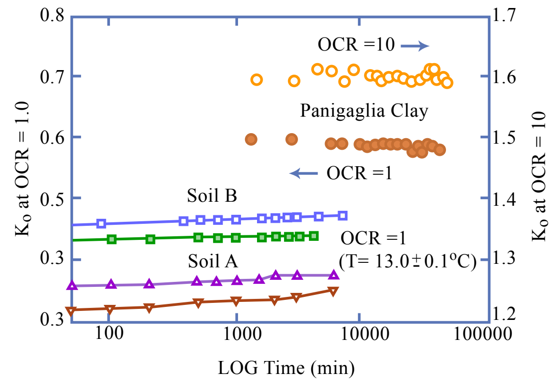
Mesri & Castro (1987)

K_0 During Secondary:

OEDOMETER CELL DATA

TUT : Square w/ transducer à la R.S. Ladd (65)

MIT : Circular w/ H_2O cell à la R.T. Martin

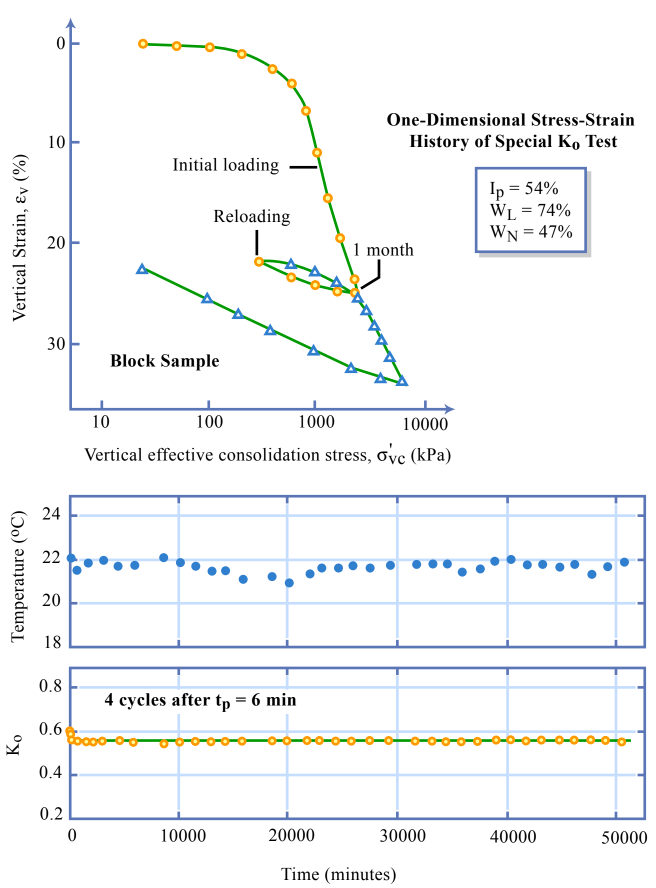


Panigaglia Clay		Soil A	Soil B
W_L = 65%, I_p = 40%, C_{T1}/CR = 0.08 ± 0.01		△ Undisturbed W_L = 138%, I_p = 78% σ'_{vc} = 50 kPa	□ Undisturbed W_L = 56%, I_p = 32% σ'_{vc} = 390 kPa
● At OCR = 1 σ'_{vc} = 1000 kPa T = 19.8 ± 0.5°C	○ At OCR = 10 σ'_{vc} = 475 kPa T = 21.5 ± 0.5°C	▽ Remolded W_L = 84%, I_p = 30% σ'_{vc} = 245 kPa	□ Remolded W_L = 56%, I_p = 17% σ'_{vc} = 390 kPa
TUT		MIT	

Coefficient of Earth Pressure at Rest vs. Time for Undisturbed and Remoulded Clay

Figure by MIT OCW.

Adapted from: Jamiolkowski, et al. (1985)



K_o and Temperature vs. Time for Undisturbed Montalto di Castro Clay

Figure by MIT OCW.

Adapted from: Holtz et al. (1986)

Mesri & Hayat (1993) CGJ, 30(4), 647-666 $\boxed{\text{Effect of Secondary Compression}}$

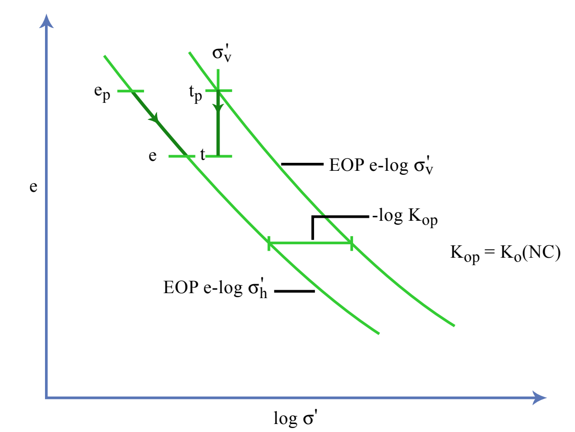
TABLE 1. Index properties of soft clays

	Clay	w_o (%)	w_l (%)	w_p (%)	CF (~2 μm%)	σ'_{vo} (kPa)	σ'_p/σ'_{vo}	ϕ' (deg.)	C_α/C_c
1	St. Hilaire	61–68	55	23	77	83.4	1.40–1.57	26	0.031
2	St. Esprit	73–92	75	27	76	36.5	3.00–3.30	28	0.026
3a	St. Alban 1	58–64	43	21	40	32.7	2.10–3.37	30	0.025
3b	St. Alban 2	48–74	31–42	18–22	56	33.1	2.13–3.04	31	0.024
4a	La Grande 15b	55–59	62	26	53	42.0	2.80–2.95	28	0.057
4b	La Grande 23a	55–58	64	26	52	82.7	1.75–2.00	28	0.052
5	Boston Blue	27–30	32–36	17	36–44	154.9	3.29	30	0.026
6	Vasby	94–103	121	40	67	28.3	1.20–1.34	23	0.055
7	Atchafalaya	52–78	82	33	61	99.9	1.14–1.22	22	0.022
8	Batiscan	82–88	49	22	80	53.1	1.62–1.72	30	0.030
9	Broadback	42–48	28–36	19–25	46	55.0	2.16–2.40	30	0.040

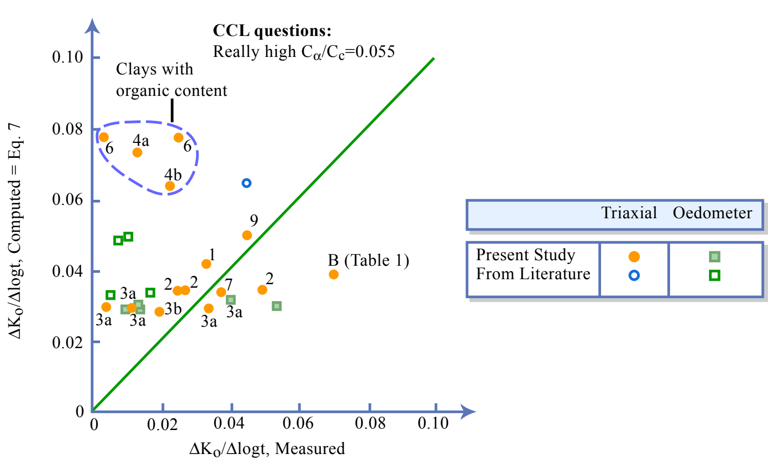
NOTE: w_o, initial water content; w_l, liquid limit; w_p, plastic limit; CF, clay fraction, less than 2 μm; σ'_{vo}, in situ effective vertical stress; σ'_p, preconsolidation pressure; C_α, secondary compression index; C_c, compression index.

Fig. 10 → $K_o = K_{op}\left(\frac{t}{t_p}\right)^{C_\alpha/C_c}$ = Eq 7

Note: $C_\alpha = C_{\alpha e}$



Soil Behavior Assumed in Formulation of e_p, void ratio at the end of primary consolidation.



Measured and computed increase in K_o during secondary compression in the NC compression range. Numbers refer to clays listed in Table 1.

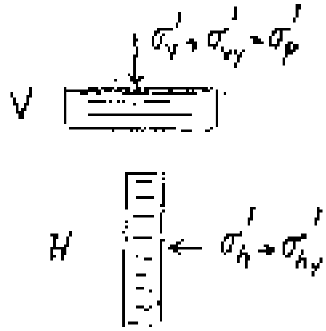
Figure by MIT OCW.

Authors comments on Fig. 11:

- Significant scatter related to experimental problems
- Ring friction in Oed. tests during secondary compression → unloading effect which may reduce or completely eliminate the increase in K_o

3. ESTIMATION OF INSITU K_0 FROM LAB TESTING

3.1 Oedometer Tests on Vertical & Horizontal Specimens



1) Assumption that $K_0 = \sigma'_{hy}/\sigma'_{vy}$ *
   - At best, would only work for OCR = 1

2) Becker et al (1987) CGJ 24(4)
   - See II p12 for details & example
   - CCL tried → doesn't work (J. Becriaques)

3) Conclusion: Doesn't work.

3.2 Estimation from $K_0 = f(OCR)$

1) Discussion of how get $K_0 = f(OCR)$
   - Empirical correlations
   - Lab testing via CK_0-TX
   - " " via Lateral Stress Oed

2) Discussion of problem due to unloading vs reloading to insitu OCR
   - Why unloading should → upper estimate

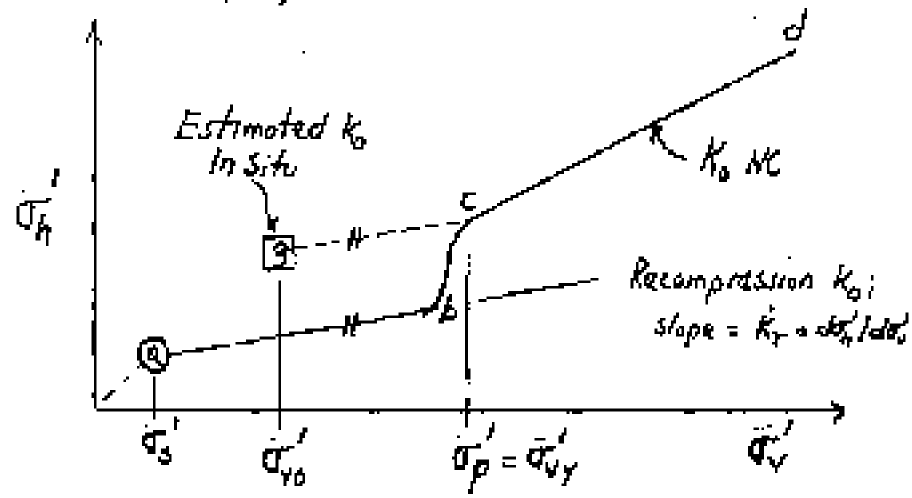
3.4 Conclusions

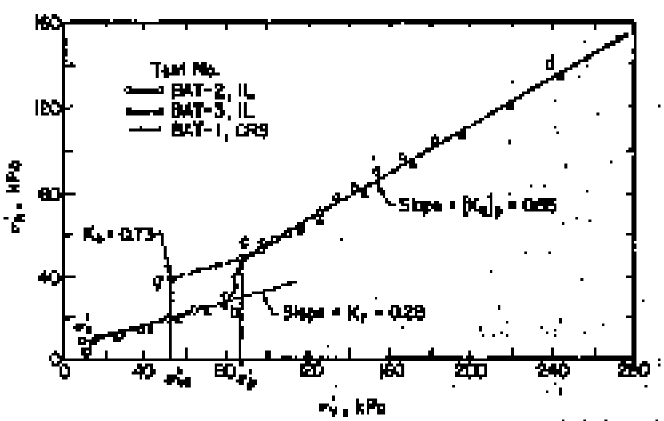
1) Always apply 3.2;
2) Also use 3.3 if have the data, e.g. from SHANSEP CK_0-TX testing

* Zeevaert (1953) 3rd ICSMFE V3, p113
Tavenas et al (1975) ASCE "In Situ" Conf., V1, p450-472

3.5 <u>Measure K_o During Recompression</u>

1) Mesri & Castro (1987) JGE, ASCE 113(3) + Lefebvre (1979)
+Mesri & Hayat (1993) CGJ





TX CELL Data

Also see p26 for m18 data & p11a, b/c... for CNT data in BBC

FIG. 8.—σ_h' versus σ_v' Path in One-Dimensional Compression and Construction for Estimating In-Situ K_o, Batiscan Clay

TABLE 3.—Estimates of In-Situ K_o, and Measured Values of K_r and $[K_o]_p$

Soft clay (1)	K_o (Eq. 14b) (t/t_p = 10,000) (2)	$\frac{[\sigma_p']_b}{[\sigma_p']_a}$ (3)	$\frac{\sigma_{ho}'}{\sigma_{hc}'}$ (4)	K_r (5)	$[K_o]_p$ (6)
Saint Alban	0.55	0.72	0.79	0.26	0.49
Broadback	0.62	0.66	0.78	0.31	0.53
Atchafalaya	0.72	0.87	0.72	0.50	0.66
Batiscan	0.64	0.80	0.73	0.28	0.55

Data 1/or

VSH Ord.

Above approach

NOTE: CCL doesn't understand reasoning of this approach, but agrees that <u>measured K_o at $\bar{\sigma}_{vo}'$ will be much too low</u>

CCL 5/26/92 CCL 2/25/93 1.322 "Mesri" Techique → K_o

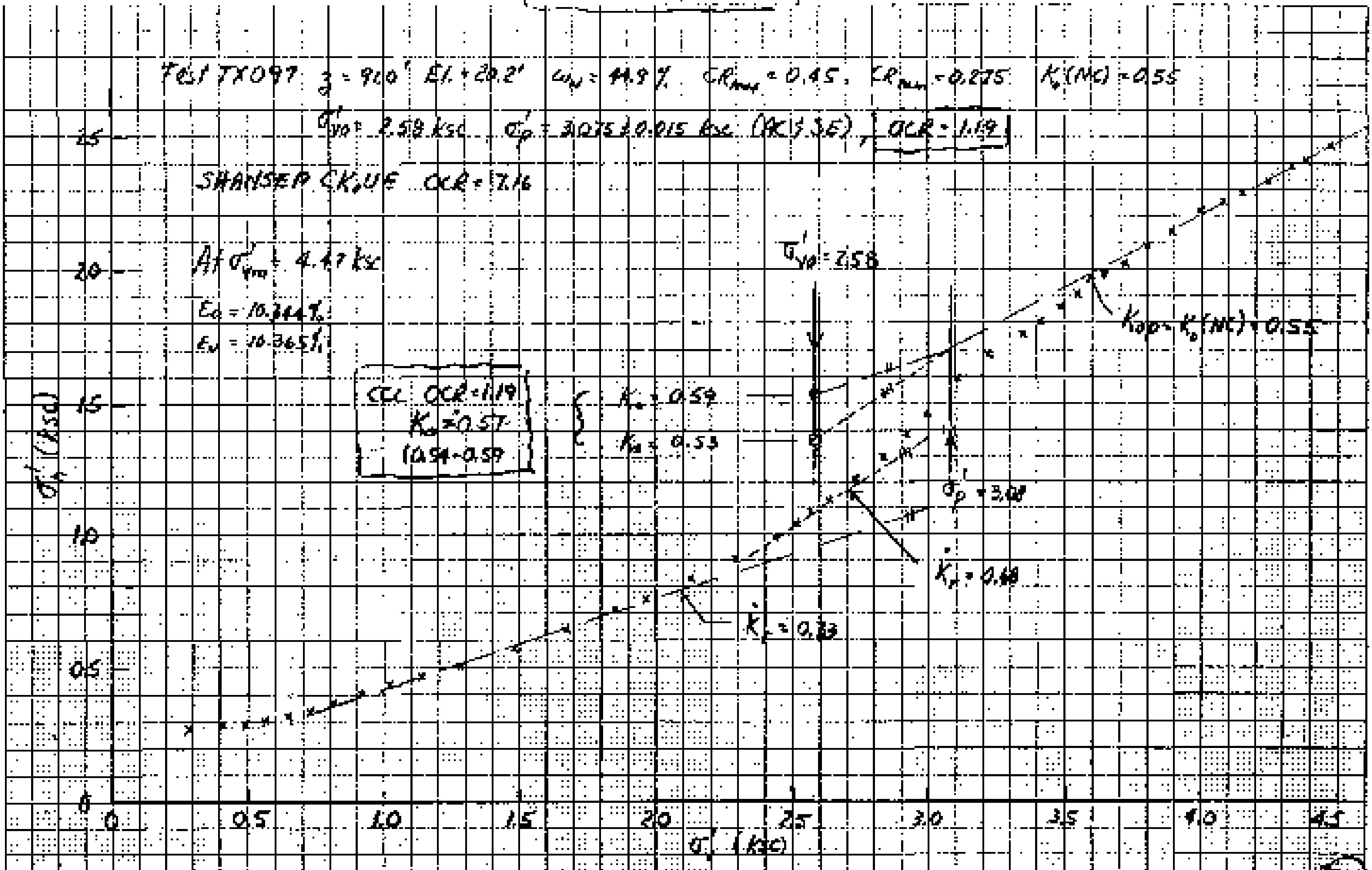


Fig. CK_o-TX97: σ'_h vs σ'_v 1-D Compression: BBC Lower "Structured" Clay

46 1517

CCL 5/25/92 CCL 2/25/93 1.322 "Mean" → K_o

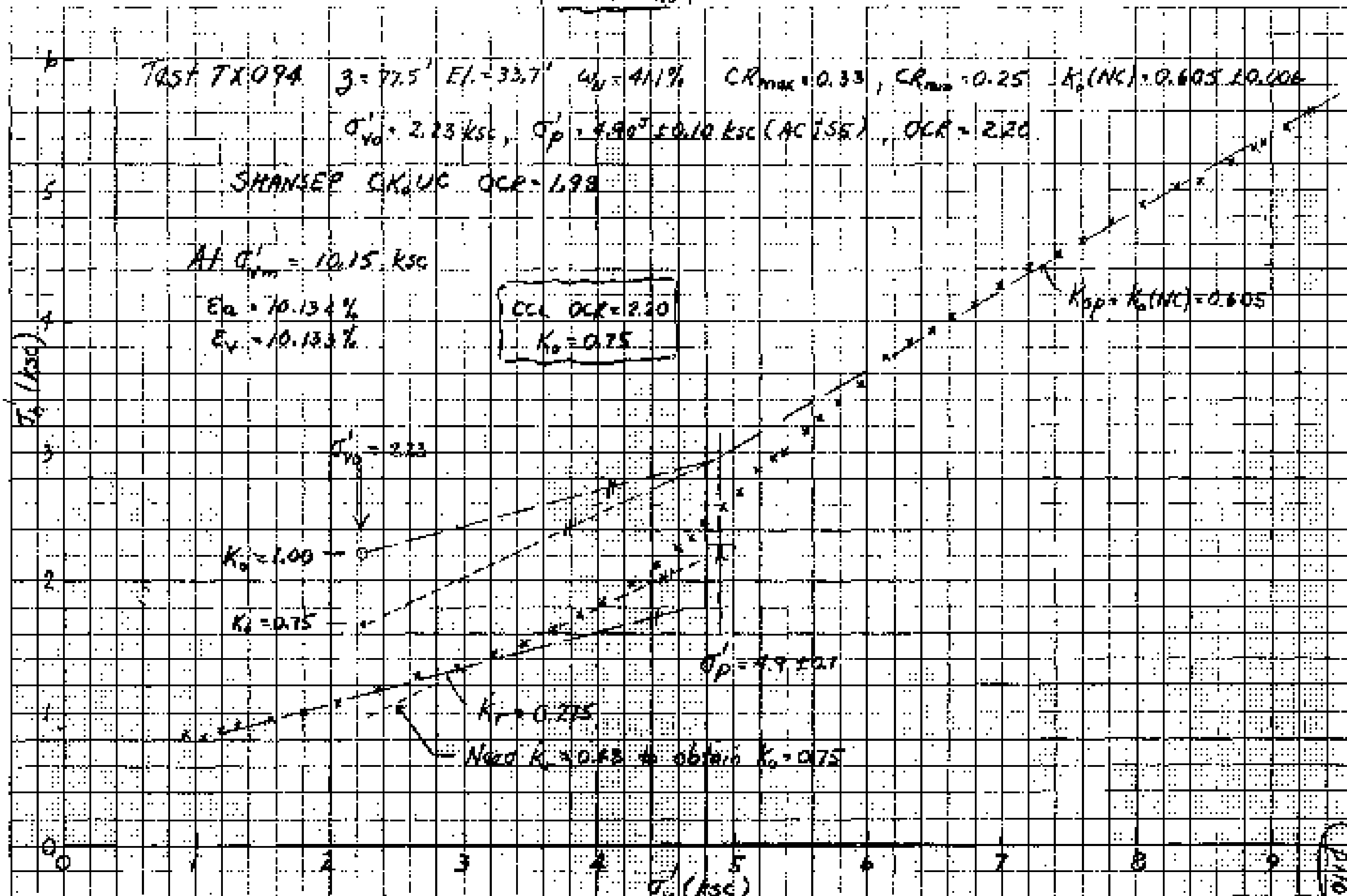


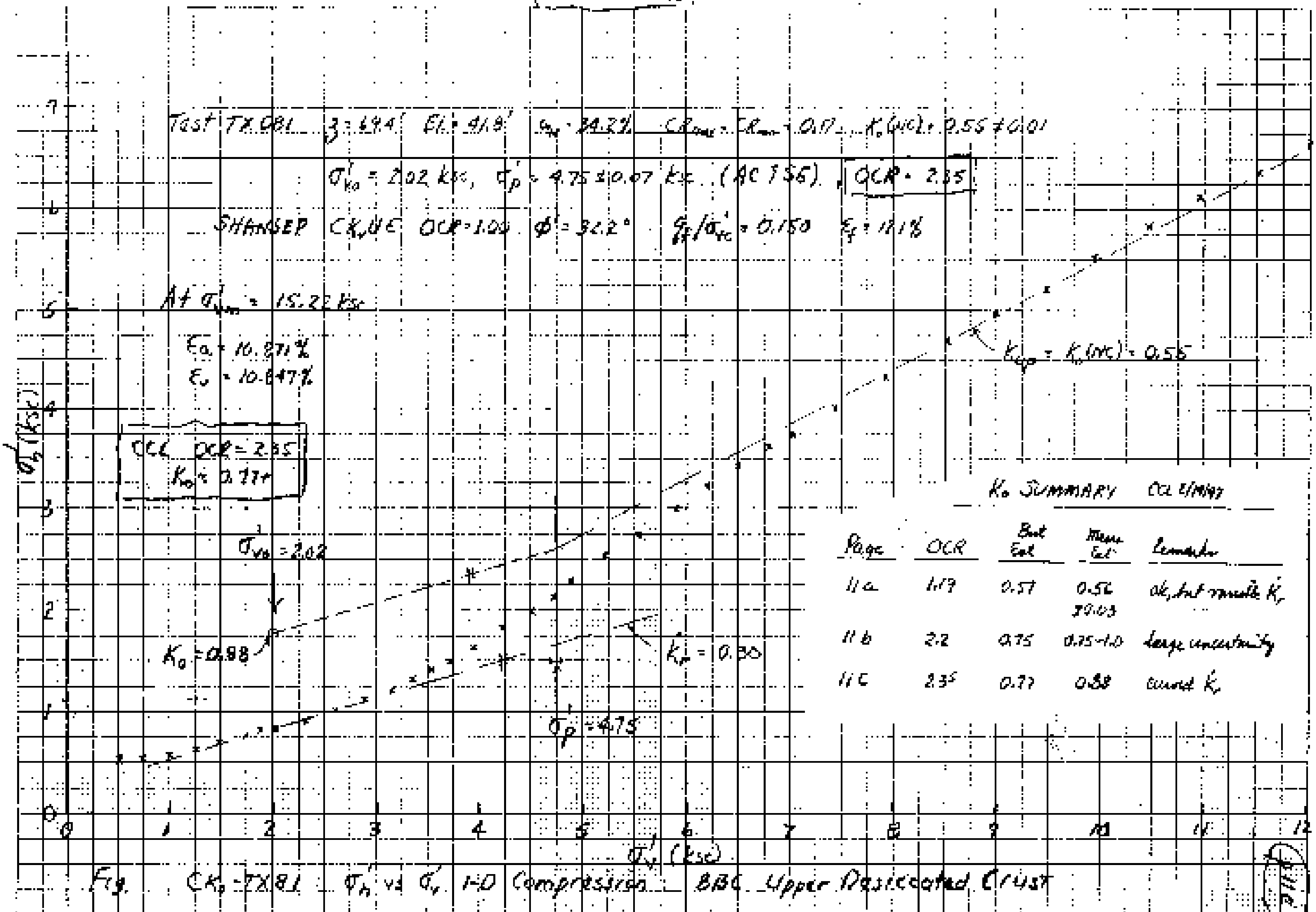
Fig. CK_o-TX94: σ_h vs σ'_v 1-D Compression: BBC Desiccated Crust with Some "Structure"

D110

CCL 3/24/97 CCL 2/25/93 1.322

46 1517

"Mean" → K_0



K_0 SUMMARY CCL 3/14/97

Page	OCR	Best Est	Mean Est	Remarks
11a	1.19	0.57	0.56 ±0.03	ok, but variable K_r
11b	2.2	0.75	0.75-1.0	large uncertainty
11c	2.35	0.77	0.88	curved K_r

Fig. CK_0-TX81 σ'_h vs σ'_v 1-D Compression BBC Upper Desiccated Crust

11c

4 ESTIMATION OF K_o FROM IN SITU TESTING

4.1 Tests Considered & Selected References

NOTE: ASCE Conference INSITU'86 "Use of In Situ Tests in Geotechnical Engineering", 1284p → many new papers

1) Total Stress Cell = Earth Pressure Cell (EPC)
   - Massarsch et al. (1975) ASCE IN SITU '75 Conf.
   - Jamiolkowski et al. (1985)

2) Hydraulic Fracturing Test (HFT)
   - See above

3) Self Boring Pressuremeter Test (SBPT)
   - Baguelin, et al. (1978) The Pressuremeter and Foundation Engineering, Trans Tech. Publ., Germany, 617p
   - Jamiolkowski, et al. (1985)

4) Marchetti Dilatometer Test (DMT)
   - Marchetti (1980), JGED, ASCE, 106(3)
   - Proc 1st Inter. Sym. on Penetration Testing, ISOPT-1, Orlando, March 1988, 2 Vol. Balkema
   - Jamiolkowski, et al. (1985)

NOTE 1), 2) & 3): "Measure" $\sigma_{ho} = \sigma'_{ho} + u$

∴ therefore need independent estimates of σ_{vo} & u to obtain $K_o = \sigma'_{ho}/\sigma'_{vo}$

4.2 Total Stress = Earth Pressure Cell (EPC)

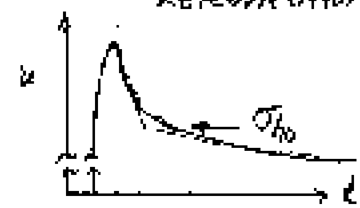
1) See Sheet E/H1 Fig.1

2) Inherent error → too high K_o with increasing OCR

3) Probably best method for low OCR clays

4.3 Hydraulic Fracturing Test (HFT)

Lefebvre (1990)



1) See Sheet E/H1 Fig.2

- Procedure

2) Bjerrum et al. (1972) - Limited $K_o < 1$ to get vertical cracks ($\perp$ to σ_3)
3) Lefebvre (1990-MIT) - Can still get vertical cracking for $K_o > 1$:
treat as total stress cavity expansion (rather than increase in pore pressure → crack $\perp$ to σ_3). 4) Use if have hydraulic piezometers

4.4 Self-Boring Pressuremeter Test (SBPT)

1) Sheets S1-S4

2) Historical development: 1972

- English → Camkometer French → PAFSOR

- 3 independent papers → "derived" stress vs. strain

$$\tau = \varepsilon_o\, dP/d\varepsilon_o = 0.434\, dP/d\log(\Delta V/V)$$

- Resultant values of c_u usually much too HIGH
  - End effects Yes (PAFSOR) L/D = 2-4 No (Camkometer) L/D = 8
  - Disturbance Yes if P_o too low
  - Variable $\dot{\varepsilon}$ Yes
  - Partial drainage ?
  - Anisotropy. No (opposite)

2/97
4.4 Cont.

3) Use to estimate σ_{ho} Techniques

PAFSOR (S1)

CAMKOMETER (S2,3)

Which preferred?

4) Some results (S4)

CAIT STP data → too scattered to be of any use

5) CCL conclusion: expensive waste of $

4.5 Marchetti Dilatometer Test (DMT)

1) Sheets D1-D5

2) Testing technique (D1)

JHS (3/88) Civil Engr. Mag.
Total Cost/test = $25±10

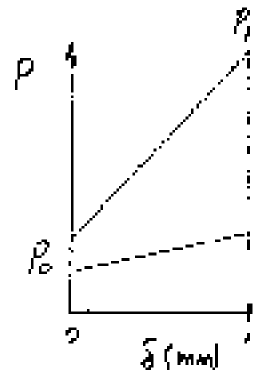
3) Overview of DMT "predictions" : ALL EMPIRICAL

2/97 2/01
4.5 Cont.

4) Material Index, $I_D = \dfrac{\Delta P}{P_o - u_o}$

Marchetti (1980)

- Reflects predominant "grain size"
- $\Delta P = f$(soil stiffness - rapid loading)
- $P_o - u_o = f$(soil strength - " ")



I_D	Soil Classification
< 0.1	Peat & Sensitive Clay
0.1	CLAY
0.35	Silty CLAY
0.6	Clayey SILT
0.9	SILT
1.2	Sandy SILT
1.8	Silty SAND
3.3	SAND

5) Horizontal Stress Index, $K_D = \dfrac{P_o - u_o}{\sigma'_{vo}}$

- K_o "uncemented & not aged" (D2 Fig. 11)

$$K_o = \left(\frac{K_D}{1.5}\right)^{0.47} - 0.60$$

(D2 Fig 4 - Other results)

- $OCR = \sigma'_p / \sigma'_{vo}$ "uncemented" $I_D < 1.2$ (D2 Fig 11)

$$OCR = (0.5\,K_D)^{1.56}$$

- Undrained shear strength: uses SHANSEP Eqn

$$c_u / \sigma'_{vo} = 0.22\,(OCR)^{0.8}$$ (but you can select S & m)

6) Modulus: $E_D + I_D + K_D + R_M \rightarrow M = 1/m_v$!

4.5 Cont.

7) Output from actual test site (D3-5)

8) JHS (3/88) promotes for φ estimates (also sells equipment)

5. CONCLUDING REMARKS

1) Practical uses of K_0

a) Required for CK_0UD Recompression technique (since K_c is much too low for 1-D reconsolidation to σ'_{vo} à la pages 2a & 2b)

b) Required starting point for FE analyses

c) To estimate σ_{ho} on underground structures (e.g., tunnels, retaining walls, etc)

2) Variation in σ_{ho}/σ_{vo}

- For simplicity, assume WT at GS and $\gamma_b = \gamma_w \rightarrow \gamma_t = 2\gamma_w$

$$\therefore \frac{\sigma_{ho}}{\sigma_{vo}} = \frac{K_0 z\gamma_w + z\gamma_w}{2z\gamma_w} = \frac{K_0}{2} + 0.5$$

& $K_0 = \pm 0.1 \rightarrow \Delta\sigma_{ho}/\sigma_{vo} = \pm 0.05$

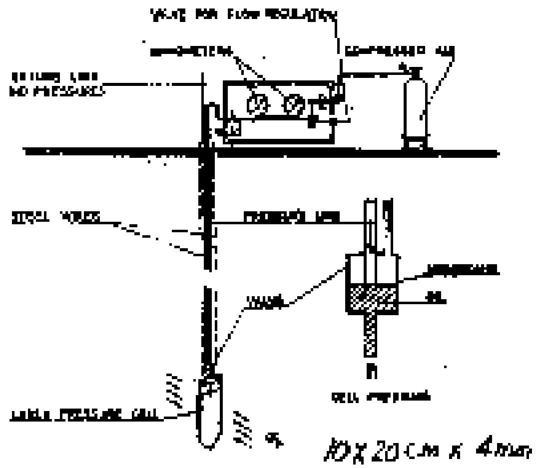
K_0	σ_{ho}/σ_{vo}
0.5	0.75
1.0	1.00
1.5	1.25

CCL 2/20/99 2/25/01

1.322 Class Schedule, Reading Assignments, Etc. on CONSOLIDATION (Part C)

Topics: From Handout Notes	Approx. No. Classes	Reading (Backup) Tokyo ('91)	SF ('85)	Other	Remarks
I Introduction • Background • K_0: trends & measurement • In situ testing	2	4.2.7 (2.2.4, 4.2.4)	(4.5 to 3.2)	–	Covers several in situ devices for estimating K_0 (Some also for OCR & strength)
II Amount of 1-D Consolidation (ρ_{cT}) • Consolid. tests & ρ_{cT} eqn. • σ'_p mechanisms & measurement • Effects of disturbance, creep, etc. • In situ tests for SH profiling	4-4½	–	2.2	–	"Main" problem: develop field & lab testing programs to determine best in situ test for stress history profiling
III Rate of Consolidation (ρ_t) • Terzaghi theory & meas. of c_v • Effects of SH, disturbance, etc. • Permeability • Non-linear consolidation	2	–	(3.4)	–	–
IV Secondary Compression (ρ_s) • C_α/C_c concept • Hypothesis A vs B • Surcharging	1½	(2.2.4)	2.5	–	"Major" Home Problem covering Parts I - IV
V 2 & 3-D Loading & Vertical Drains • Initial settlement (ρ_i) and ρ_{cT} • Rate of settlement • Consolidation with vertical drains	2-2½	(2.2.5)	(3.3)	Foott & Ladd (1981) 1.361 HW 16/12	Self-graded home problem
VI Problem Soils • High S_t • Peats • Collapsing & expansive • Residual • Varved clay	1½-2	–	–	–	Emphasis on peats and collapsing/expansive soils

From Massarsch et al (1975) ASCE ASMI Conf.



10X20 cm x 4mm

FIG. 1.—SCHEMATIC DIAGRAM OF THE EARTH PRESSURE CELL (GLÖTZL) METHOD

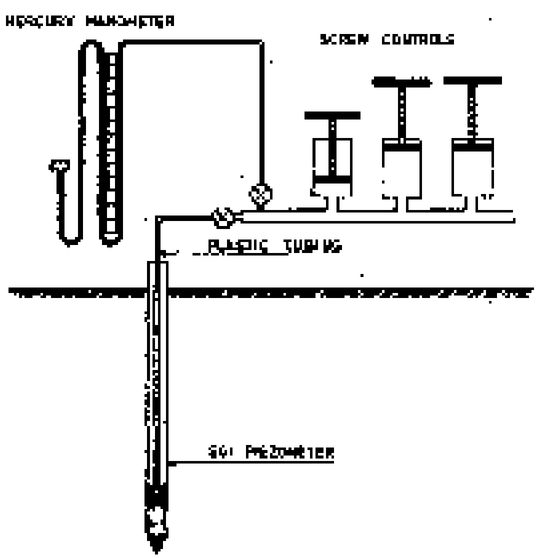


FIG. 2.—SCHEMATIC DIAGRAM OF THE HYDRAULIC FRACTURE METHOD

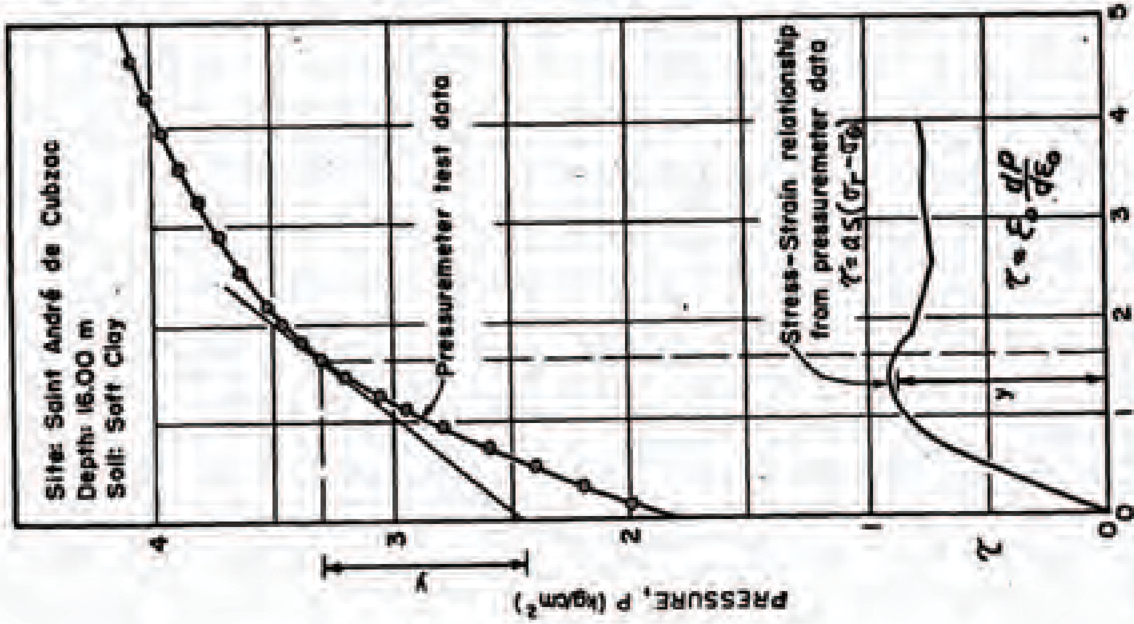
Earth Pressure Cell (EPC)

- Penetrate with protective casing until 30 cm above depth
- Push in Cell & wait few days for equilibrium
- Measure σ_h via "pressure balance" principle (deflection ≈ 5 μm)

Hydraulic Fracturing Test (HFT)

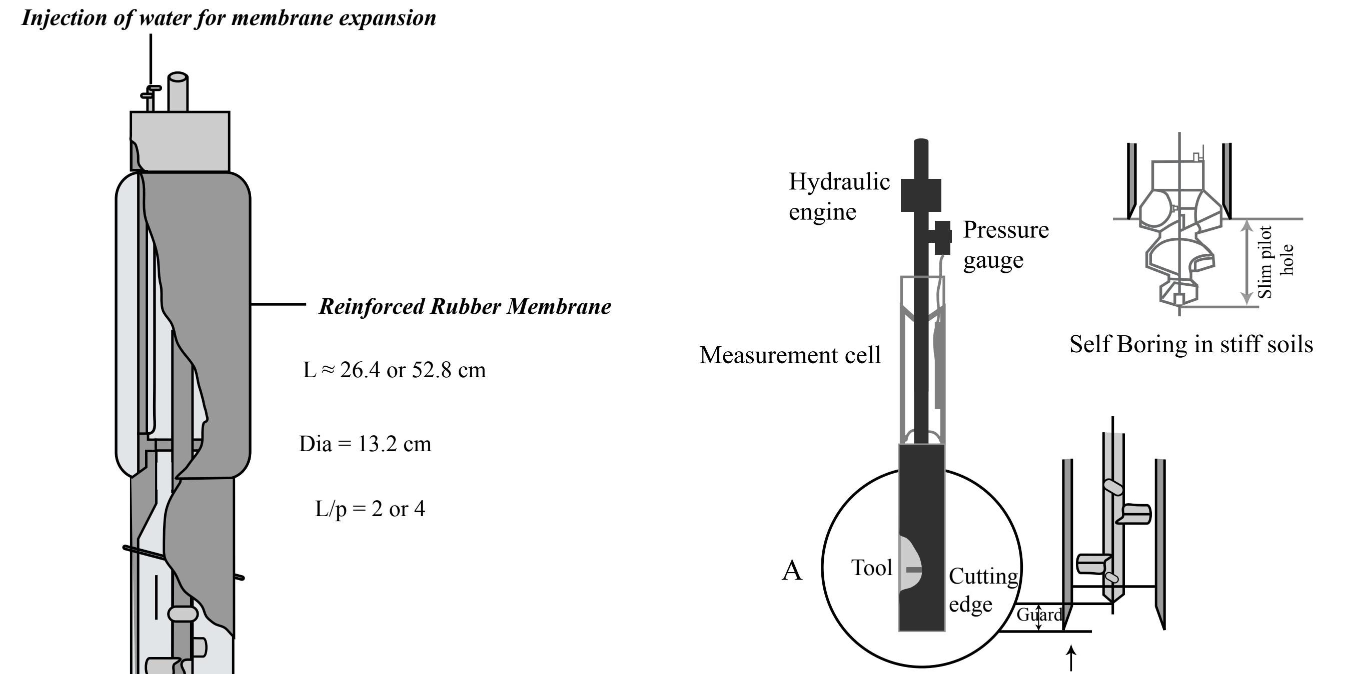
- Install push-in or Casagrande type piezometer
- Increase u in increments while monitoring dV/dt (flow rate). Large increase in dV/dt indicates formation of crack (hopefully radially-vertical)
- Reduce u with measurements of dV/dt →

  $\sigma_{ho} = u$ when dV/dt = precracking value



Tokyo (77)

Fig. 55 Data from an undrained Autoforeur pressuremeter test on clay (supplied by F. Schlosser).



Self boring in soft soils

Drawing of the autoforeure presuremeter (supplied by F. Schlosser).

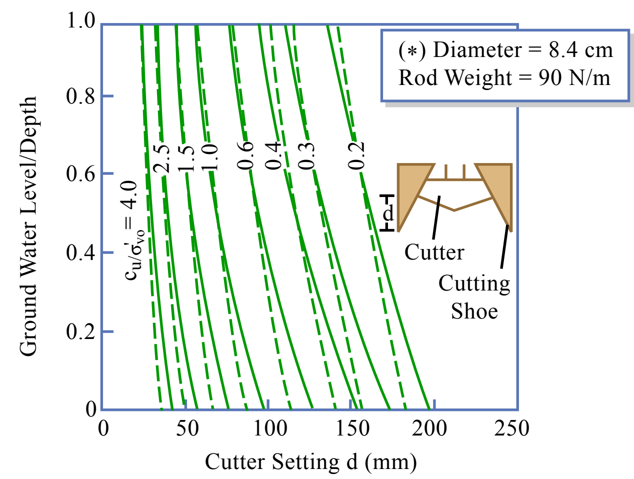
SBP Type Pafsor

Figure by MIT OCW.

Adapted from: Tokyo (77)

"French" PAFSOR

- Cutting drive mechanism above measurement (expansion) cell
- Inflated membrance during insertion (NOT RIGID)
- Expansion via water → AVERAGE P vs ΔV
- $\varepsilon_0 = \frac{\Delta r}{r_0} = \frac{1}{\sqrt{1-\Delta V/V}} - 1.00$



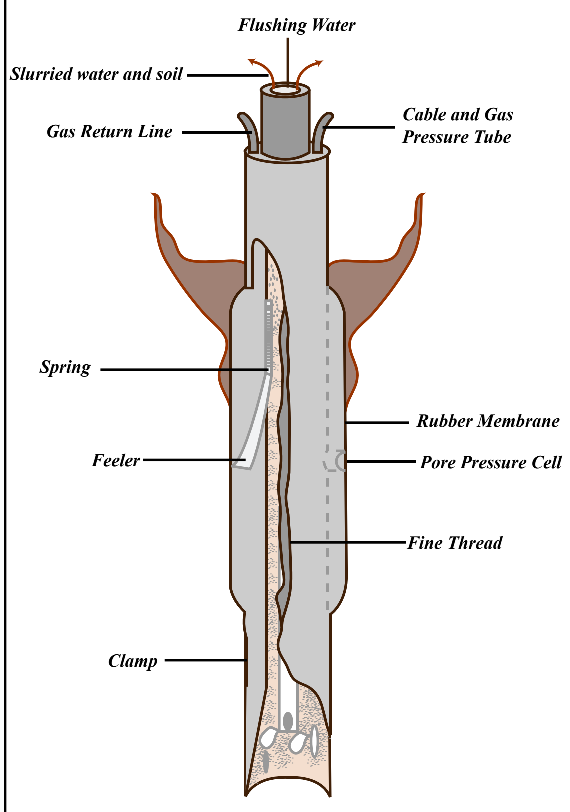
Cutter Setting for Clays, Applicable to Camkometer Type MK III*

"English" CAMKOMETER

- Cutting drive mechanism at ground surface (vibrations)

Figure by MIT OCW.

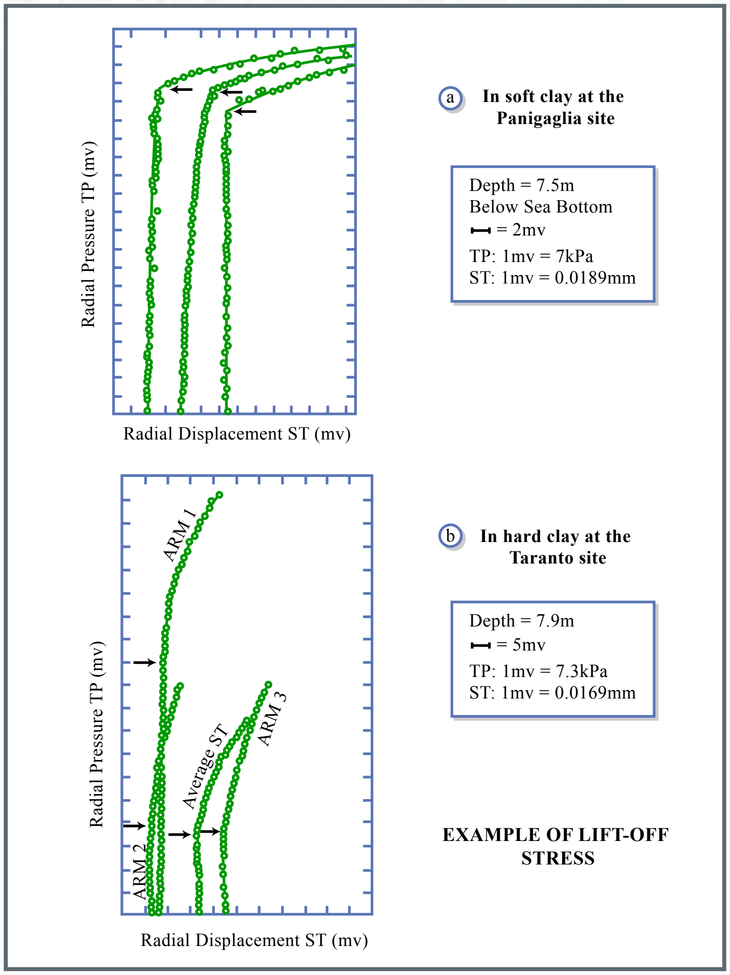
Adapted from Clarke (1981).



Camkometer

Figure by MIT OCW.

- Membrane against RIGID hollow cylinder during insertion
- Expansion via gas pressure with measurement of Δr by 3 "feelers" (electric sensors)
  → 3 separate P vs $\varepsilon_0 = \Delta r / r_0$ (or use average ε_0)



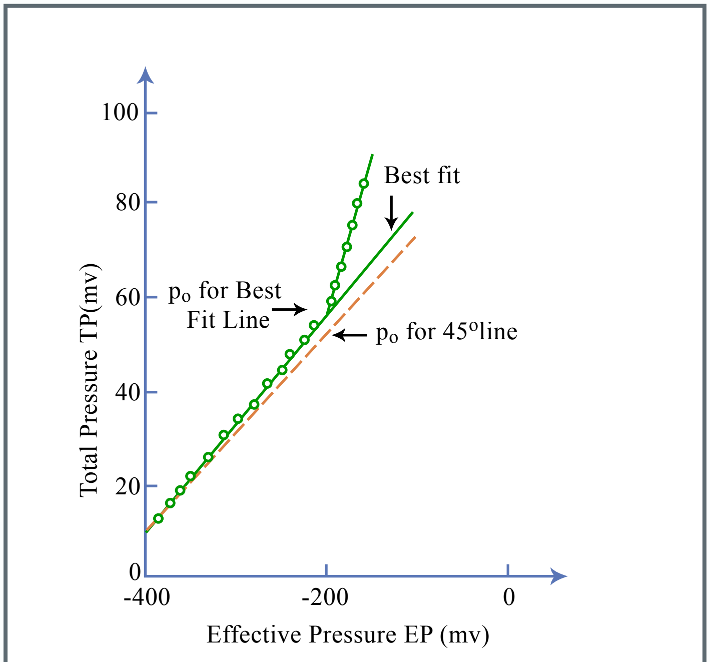
EXAMPLE OF LIFT-OFF STRESS

Figure by MIT OCW.

Use "lift off" $P = \sigma_{ho}$

(a) Soft Clay: 3 feelers → ≈ same σ_{ho}

(b) Stiff Clay: 3 feelers → different σ_{ho}



p_o Determined from Total vs. Effective Pressure Plot

Figure by MIT OCW.

Assume $P = \sigma_{ho}$ when $\Delta P \to +\Delta u$

Adapted from: Wroth (1982), Lacasse and Lunne (1982).

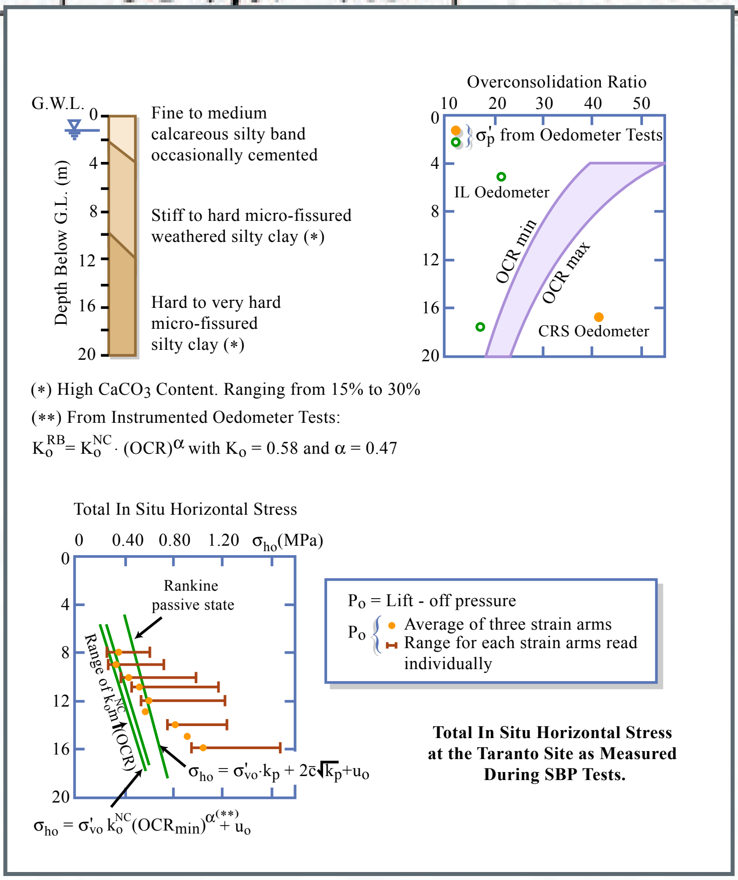


Figure by MIT OCW.

Adapted from: Jamiolkowski, et al. (1985) = SF (85)

Results CAMKOMETER - Stiff Clay Site

• A lot of σ_{ho} data exceed Rankine passive σ_{hp}!

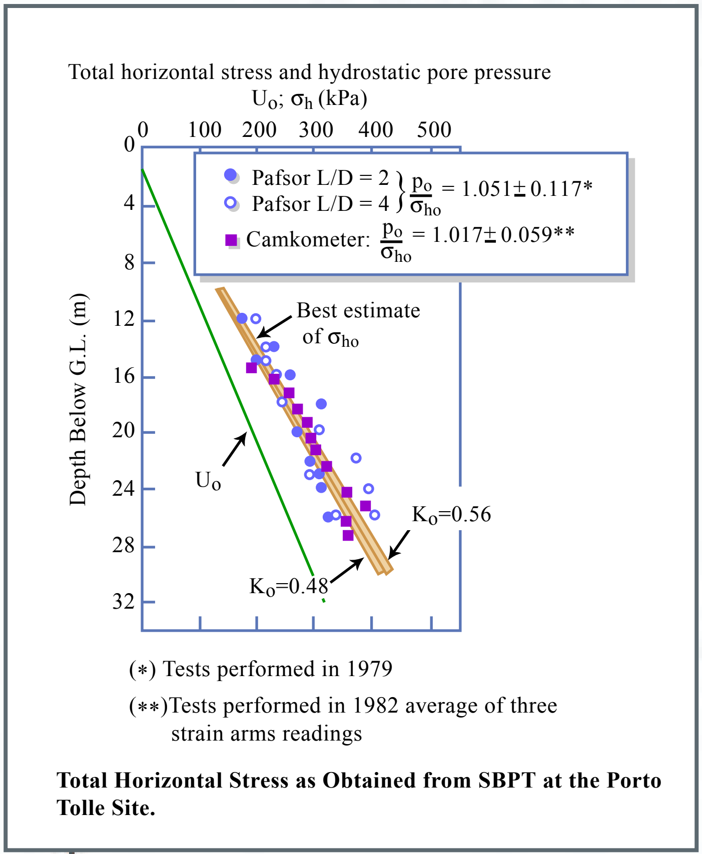


Figure by MIT OCW.

Adapted from: Ghionna et al. (1981, 1983).

Results PAFSOR + CAMKOMETER - Soft Clay Site

Mean of scattered data → reasonable K_o

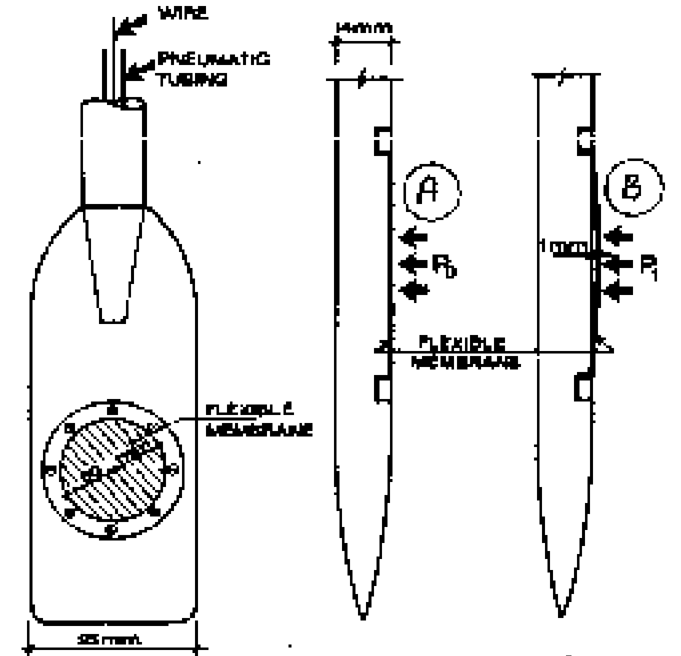


Fig.38: Marchetti Dilatometer. SF(85)

Marchetti Dilatometer Test (DMT)

Testing Procedure

1) Push (penetrate) at ≈ 2cm/s
2) Test at 20cm intervals without delay time ($t < 15s$)
   (Have beeping sound with membrane in contact)
3) Increase P via gas pressure + gage readings (±0.1 bar)
   - Beeping stops at liftoff = A reading → P_0
   - Beeping starts again with $\delta \approx 1mm$ = B reading → P_1
   - Do this within 15-30s
4) Decrease P_1, beeping stops & then starts when membrane again in contact = C reading → equilibrium u in granular soils

Calibration Information, P_0 & P_1

Z_m = gage reading at zero pressure*

ΔA & ΔB = membrane stiffness corrections*

$P_0 = 1.05(A - Z_m + \Delta A) - 0.05(B - Z_m - \Delta B)$

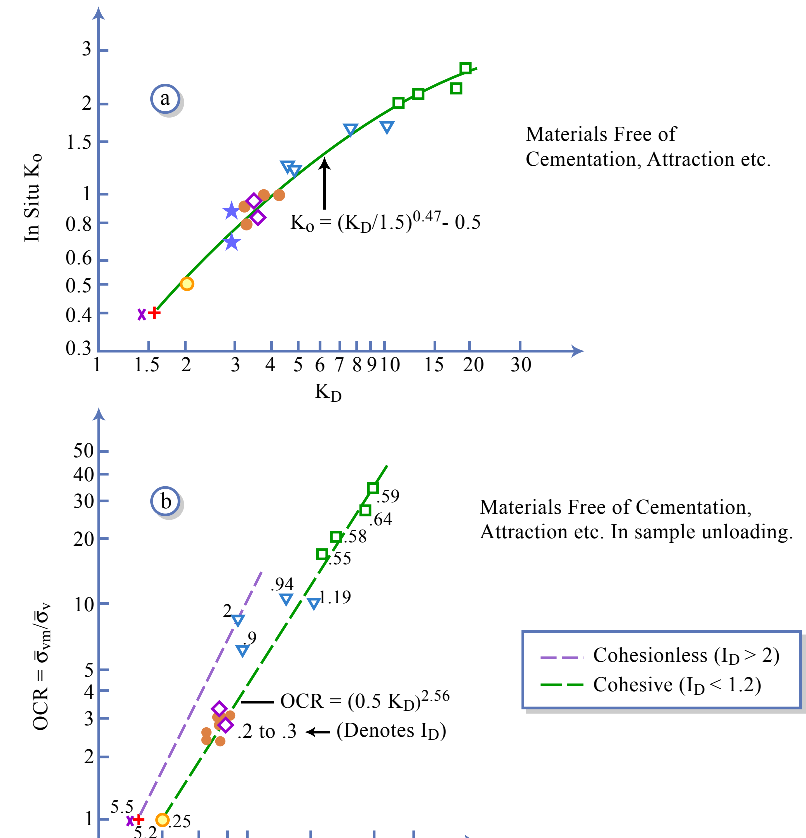
$P_1 = B - Z_m - \Delta B$

$\Delta P = P_1 - P_0$

* Very important soft cohesive soils

DMT Parameters (u_0 = equilibrium u)

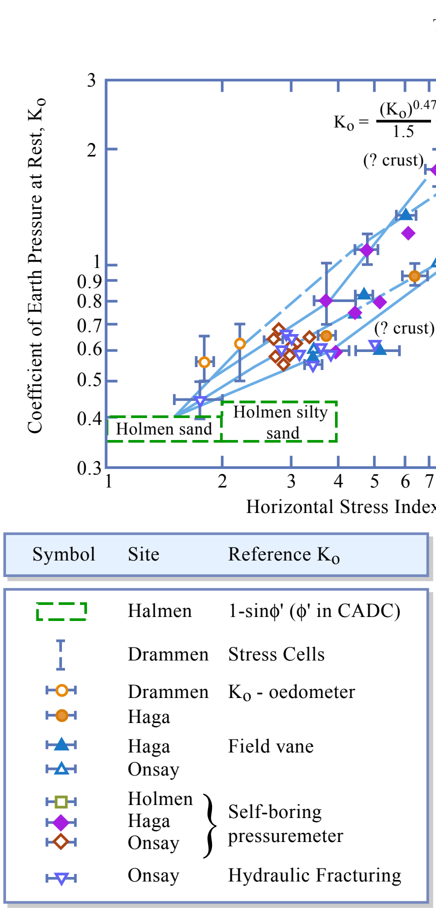
1) Material Index, $I_D = \dfrac{\Delta P}{P_0 - u_0}$
2) Horizontal Stress Index, $K_D = \dfrac{P_0 - u_0}{\sigma'_{v0}}$
3) Dilatometer Modulus, $E_D = \dfrac{E}{1-\mu^2} = 38.2\,\Delta P$



Correlation between (a) K_o and K_D; (b) OCR and K_D

Figure by MIT OCW.

Adapted from: Marchetti (1980)



Relationship between horizontal stress index, K_D, and coefficient of earth pressure at rest at NGI sites.

Figure by MIT OCW.

Adapted from: Lacasse & Lunne (1988) ISOPT-1

1.322 UMT

CCL 3/1/87 (03)

2/88-3/89

LOCATION: D-9 - BLADE ORIENTATED EAST-WEST MEMBRANE FACING NORTH
PERFORMED - DATE: 24 FEBRUARY 1987
BY: DR. CHARLES LADD, MARVIN OOSTERBAAN, ALFRED NYERS, JEFF GOODWIN, GARY CLIFFORD

CALIBRATION INFORMATION:
DELTA A = .20 BARS (ΔA) DELTA B = .45 BARS (ΔB) GAGE 0 = .20 BARS (Z_m) ENT DEPTH= 1.00 M

1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	σ'_{vo} SV (BAR)	σ'_p PC (BAR)	OCR	KO	c_u CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
8.31		5.70	14.00	279.	1.75	9.20	.717	1.800	.504	11.33	22.47	1.47		30.6	474.7	SANDY SILT
8.92		4.20	8.60	137.	1.22	5.90	.777	1.800	.548	3.02	5.52	1.13		27.7	270.7	SANDY SILT
9.53		5.10	7.80	75.	.52	6.98	.837	1.800	.596	4.19	7.03	1.46	.626		159.2	SILTY CLAY
10.14		5.00	10.80	188.	1.41	5.97	.897	1.800	.644	4.50	6.99	1.14		28.3	375.1	SANDY SILT
10.75		3.70	7.00	97.	1.07	3.79	.957	1.700	.689	1.87	2.71	.95			148.2	SILT
11.36		4.80	11.80	231.	1.92	4.71	1.017	1.900	.736	6.07	8.24	.96		29.3	414.7	SILTY SAND
11.97		6.80	15.80	304.	1.65	6.70	1.077	1.950	.792	9.04	11.41	1.20		29.4	644.1	SANDY SILT
12.58		8.60	20.00	392.	1.63	8.16	1.134	1.950	.849	13.29	15.66	1.36		29.9	902.9	SANDY SILT
13.19		5.30	7.80	60.	.41	4.68	1.196	1.800	.901	3.40	3.77	1.11	.574		105.6	SILTY CLAY
13.80		5.40	7.20	42.	.30	4.32	1.256	1.700	.946	3.14	3.32	1.04	.545		68.7	CLAY
14.41		5.80	7.80	49.	.32	4.47	1.316	1.700	.988	3.46	3.51	1.07	.594		82.4	CLAY
15.02		5.80	7.50	38.	.25	4.25	1.376	1.700	1.030	3.33	3.24	1.03	.580		62.0	CLAY
15.63		5.80	7.50	38.	.26	4.02	1.436	1.700	1.072	3.19	2.98	.99	.565		59.9	CLAY
16.24		6.00	7.60	35.	.22	4.00	1.496	1.700	1.114	3.29	2.95	.99	.585		54.0	CLAY

TEST NO. D-9 (CONTINUED)

$$P_0 = 1.05\,(A - Z_m + \Delta A) - 0.05\,(B - Z_m - \Delta B)$$

$$P_1 = (B - Z_m - \Delta B)$$

$$I_D = (P_1 - P_0)/(P_0 - u_0) \rightarrow \text{Soil Type } \& \ \gamma_t$$

$$K_D = (P_0 - u_0)/\sigma'_{vo} \rightarrow K_0 = \left(\tfrac{2}{3} K_D\right)^{0.47} - 0.60$$

$$\rightarrow OCR = \left(\tfrac{1}{2} K_D\right)^{1.56}$$

$$\rightarrow c_u = \sigma'_{vo}\ 0.22\,(OCR)^{0.8}$$

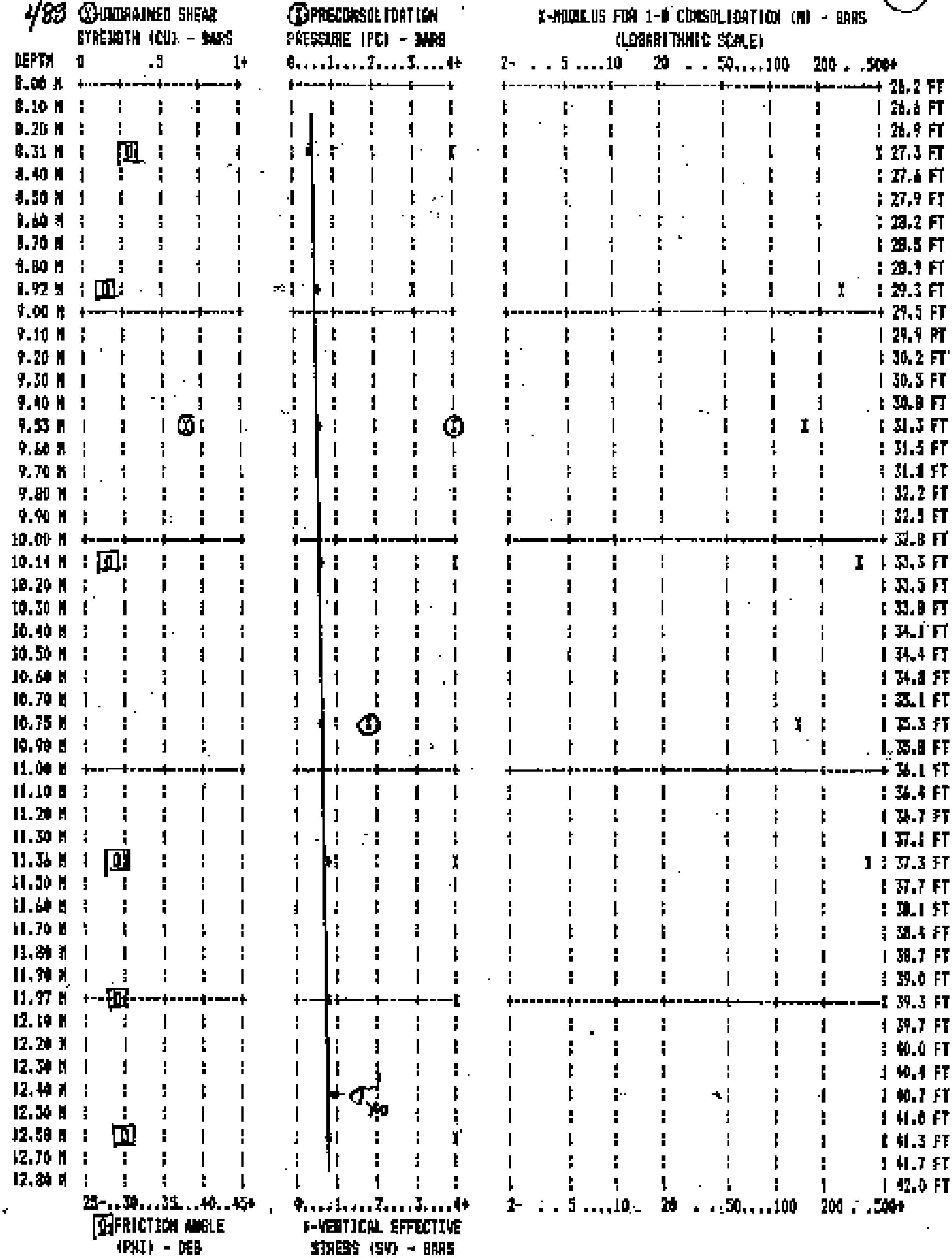
Cohesive Soils

1.322

CCL 3/87 Cohesive

3/89 4/88

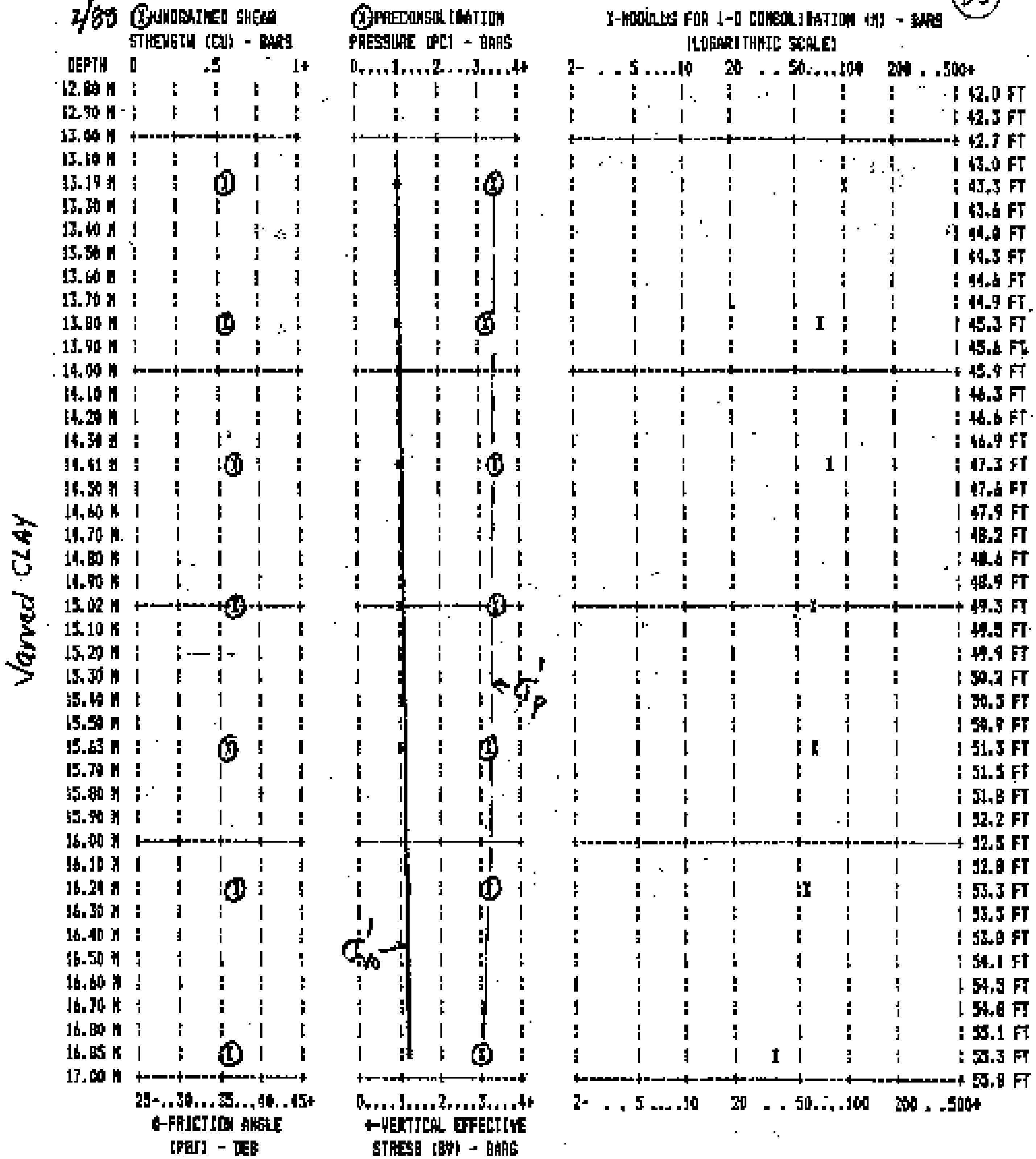
(04)



Granular

1-322
CCL 3/87 3/89

(D5)



END OF SOUNDING

Note: After inputting u_o & σ'_{vo} for 1st test, data plus correlations with γ_t → computed u_o & σ'_{vo} vs depth

K_0 INFORMATION from SOUTH BOSTON CAIT STP

A. General Soil Profile (Not to scale)

Project El. = NGVD +100.0' = BCB +94.35'

El.(ft) | Depth(ft)
111.2 ——— 0
102.0 ▽ FILL — 9.2 γ_t = 110 pcf
91.4 ——— 19.8
Marine SAND γ_t = 120 pcf
74.1 ——— 37.1
γ_t = 116.4
20 — Marine
CLAY = 110.6
−5 — (BBC)
= 116.5
−28.9 ——— 140.1
Glacio-Marine or SAND

See Fig. 2 for w_N & AL

B. Stress History

- Fig. SH-1 El. vs. Linear Regression σ'_p
  Note: Crust σ'_p thought to be due to desiccation
- Fig. SH-2 El. vs. u, σ'_{vo} & σ_v

C. K_0 Data from Lab Testing

- Fig. K_0-1 El. vs K_0(NC) from CK_0-TX
- " K_0-2 K_0 vs $(1-\sin\phi')$
- $K_0 = K_{0NC}(OCR)^n$ LSO & CK_0-TX suggest $n = 1.11 - 1.30^{?}$ K_0 NC

D. K_0 Data from Field Testing

- Fig STP-1 Sketch of SBPT using jetting to mount
- " -2 El. vs u_0, σ'_{vo}, σ_{vo}, est σ_{ho} & σ_{ho} data from SBPT
- " -3 El vs K_0 from EPC, SBPT & DMT

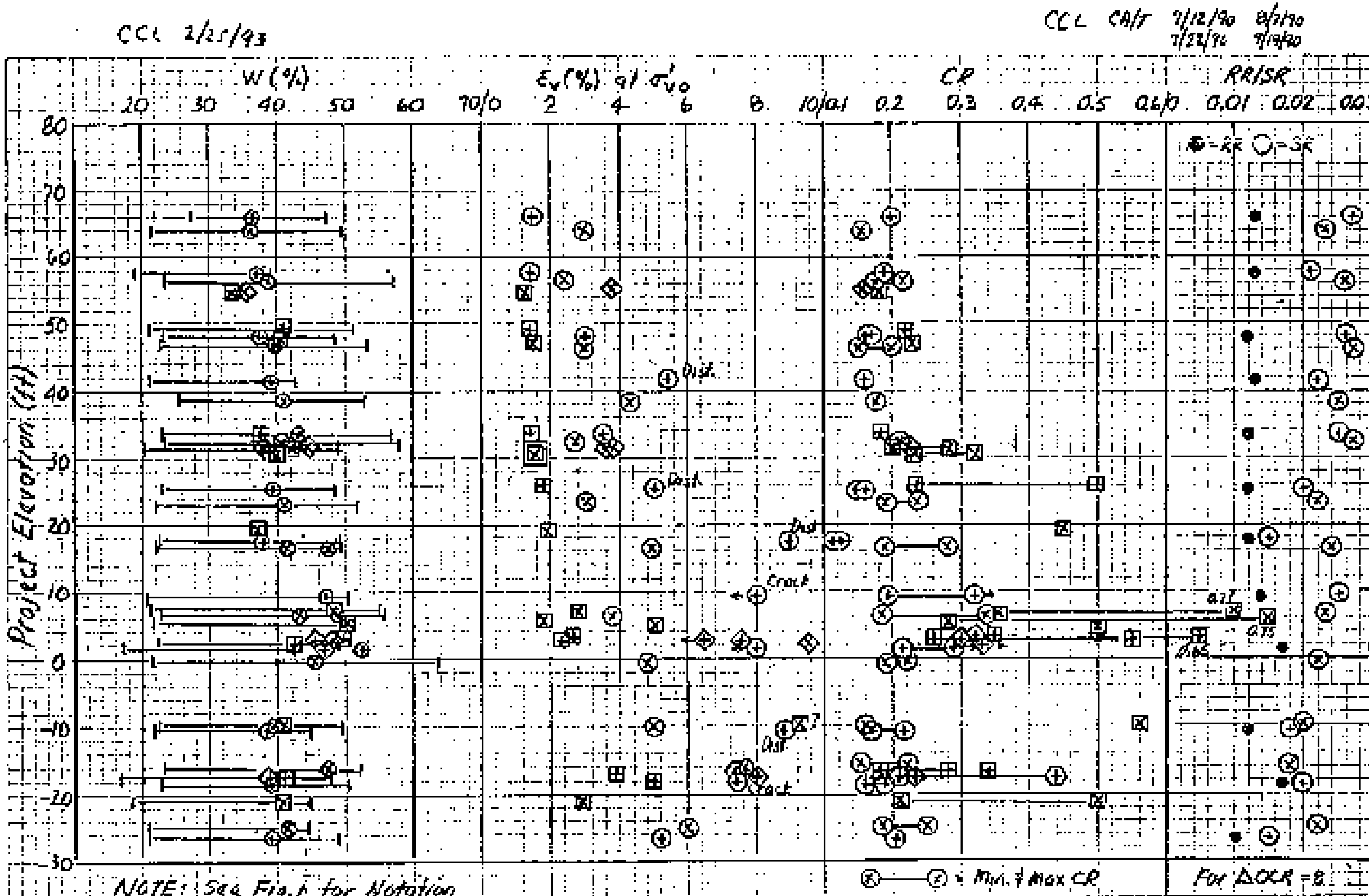
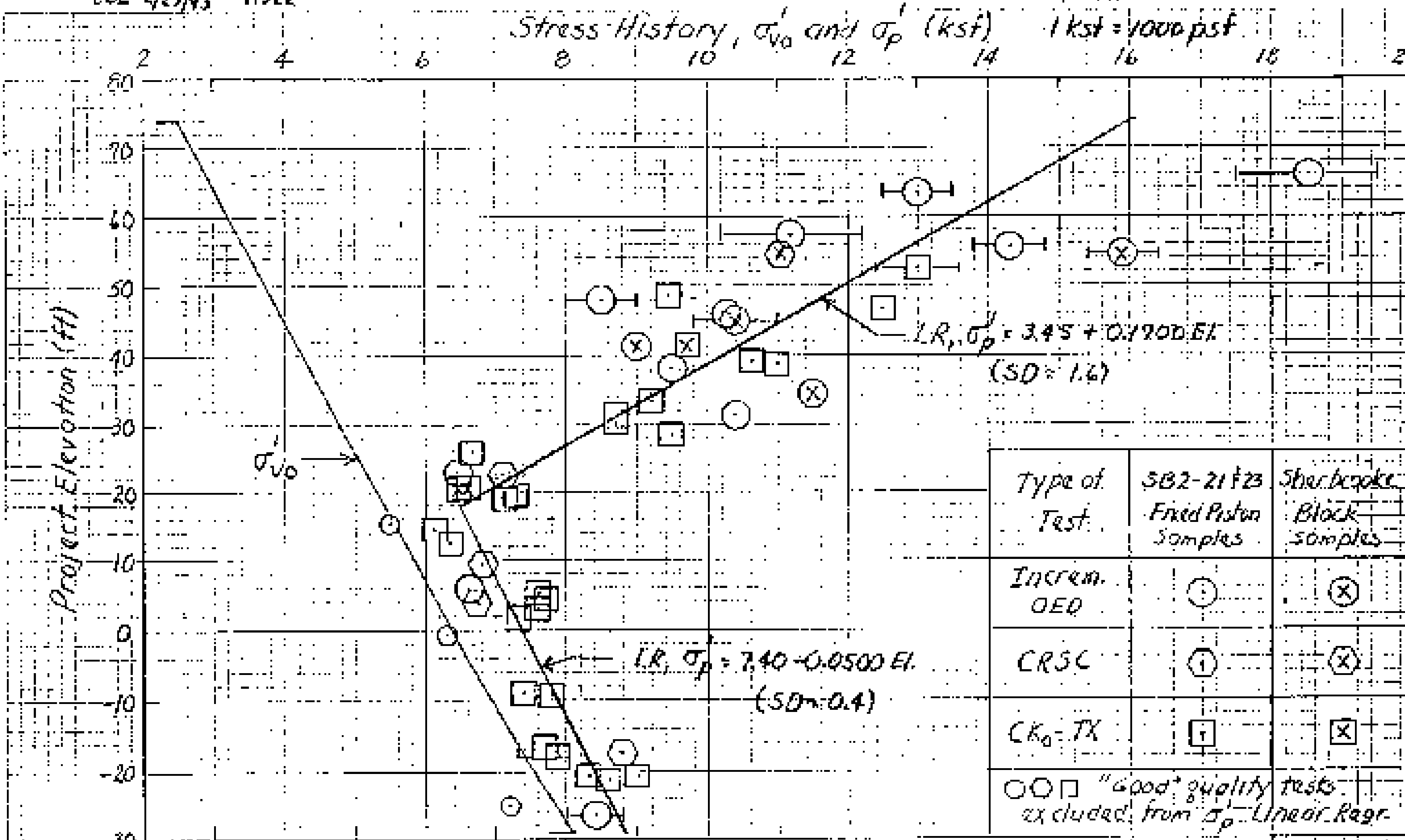


Fig. 2 CA/T South Boston STP: Index Properties and Compressibility from Lab. Consolidation Testing

CCL CA/T 12/30/90
3/28/91

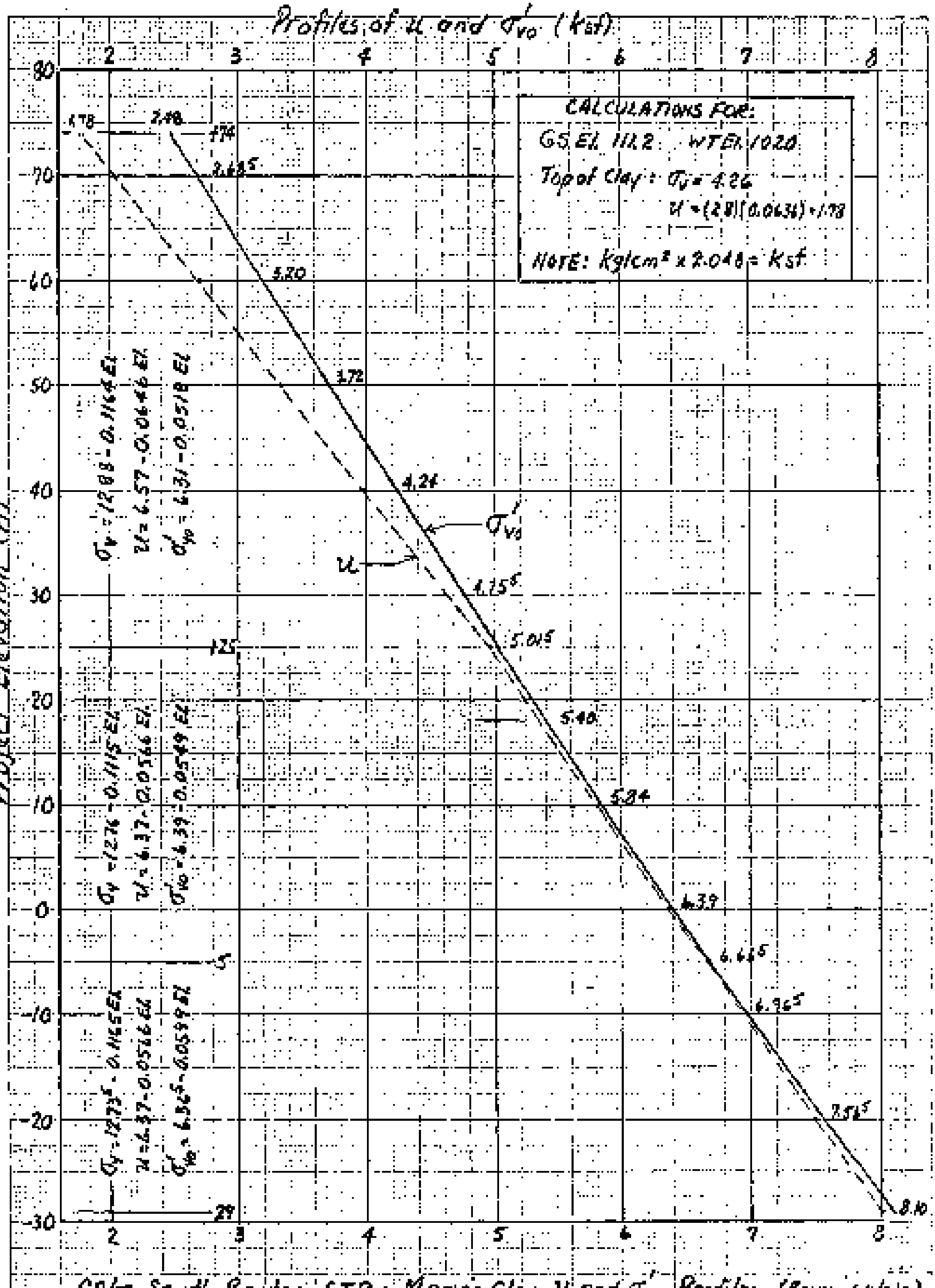
CCL 2/23/93 1.322



NOTES: GS El. = 111.2 ; WT El. = 102.0 with γ_w = 63.6 pcf , u in Marine Clay from piezometers at El. 50, 25, -1 and -32

Fig SH-1 CA/T South Boston STP: Stress History from Laboratory Consolidation Tests

CA/T CCL 11/19/90



CA/T South Boston STP: Marine Clay u and σ'vo Profiles (Revised 11/2/90)

Fig. SH-2

CCL 2/25/93 1.322

CCL 2/25/93 1,322

CCL 9/14/90 9/24/90 10/10/90 10/20/90

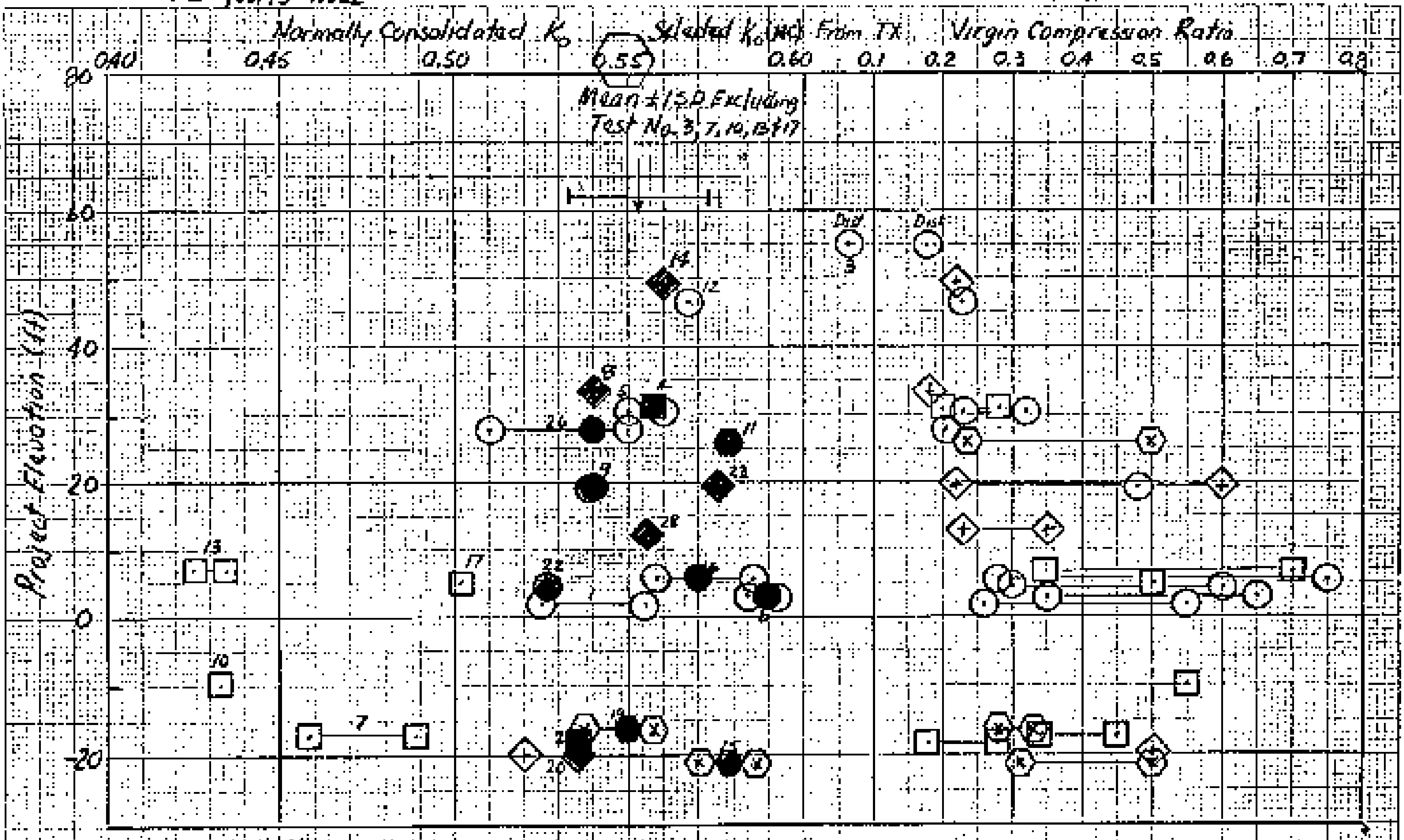


Fig. K-1 CA/T SB STP: Triaxial K_0 and CR vs. Elevation for Normally Consolidated Clay

CA/T CCL 9/24/90
10/20/90

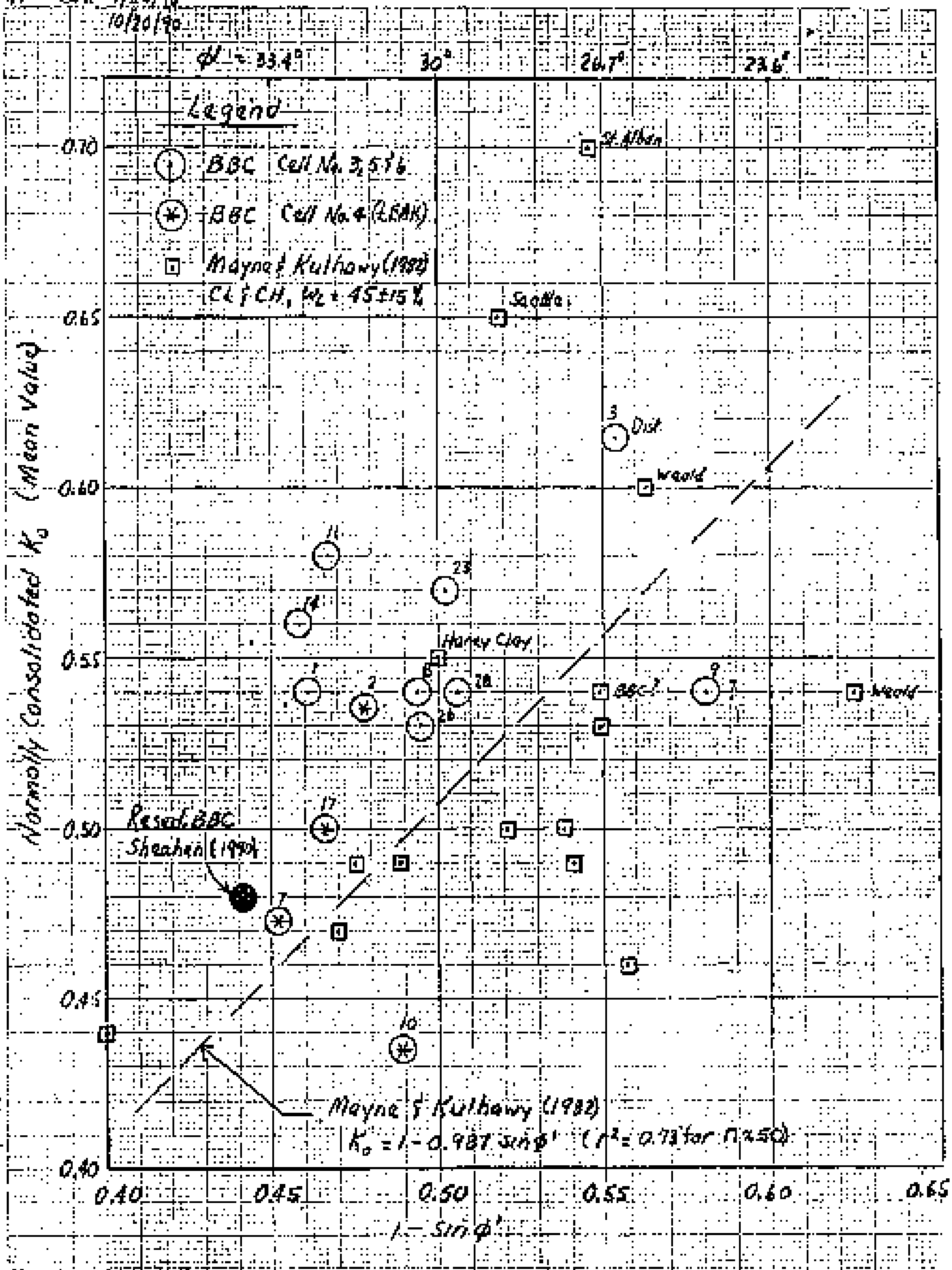


Fig. K_o-2 CA/T SB STP: K_o vs $(1-\sin\phi')$ for OCR=1

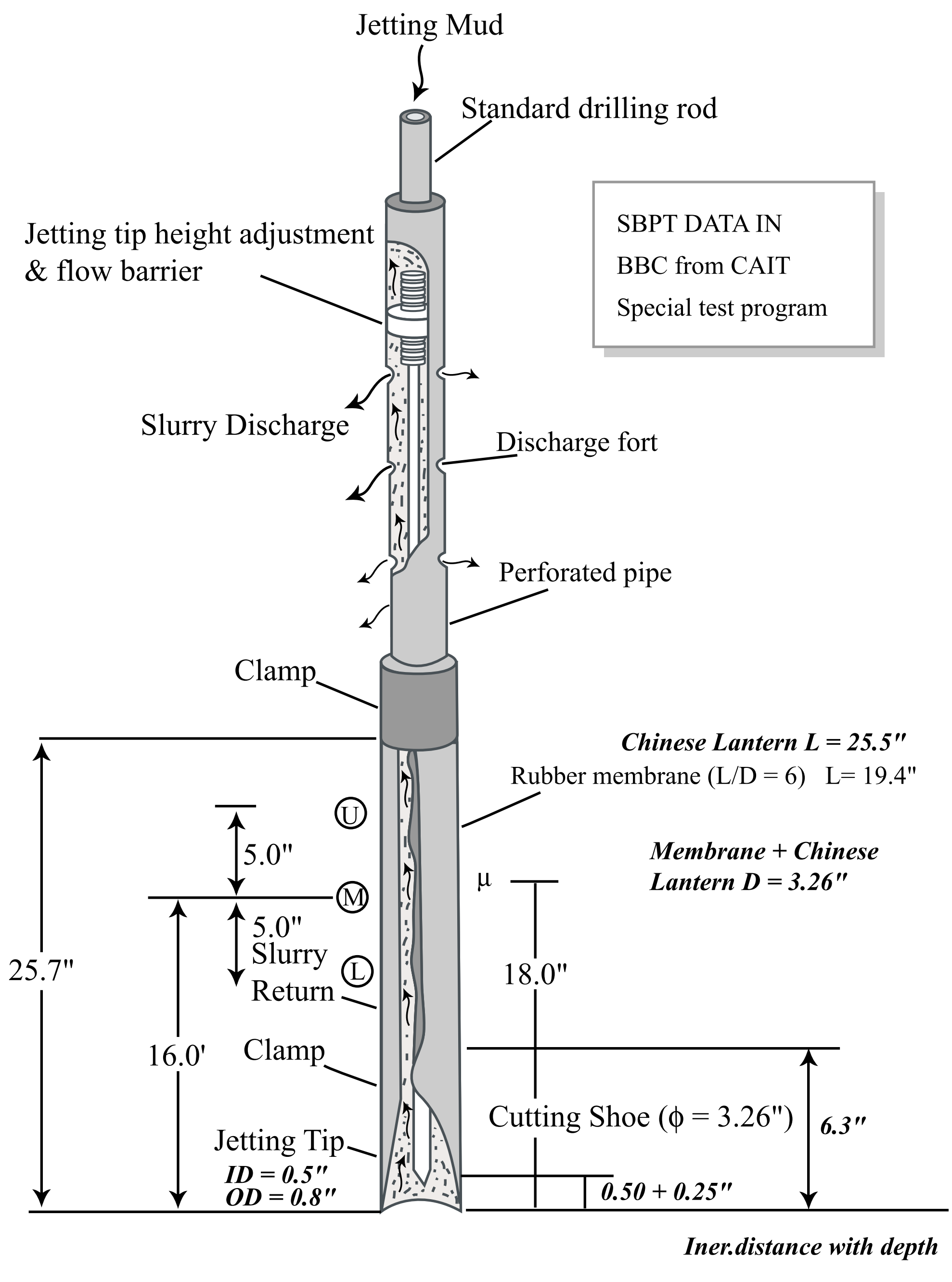
CCL 2/20/93 1.32

1.322 Consolidation Part I

Supplement for Section 4

2/23/93
CCL 3/2/92 1.322

CCL 8/9/91



Sketch of the Jetting pressure meter (Adapted from Hughes, Jefferies and Morris, 1984)

Figure by MIT OCW.

(Adapted from Hughes, Jefferies and Morris, 1984)

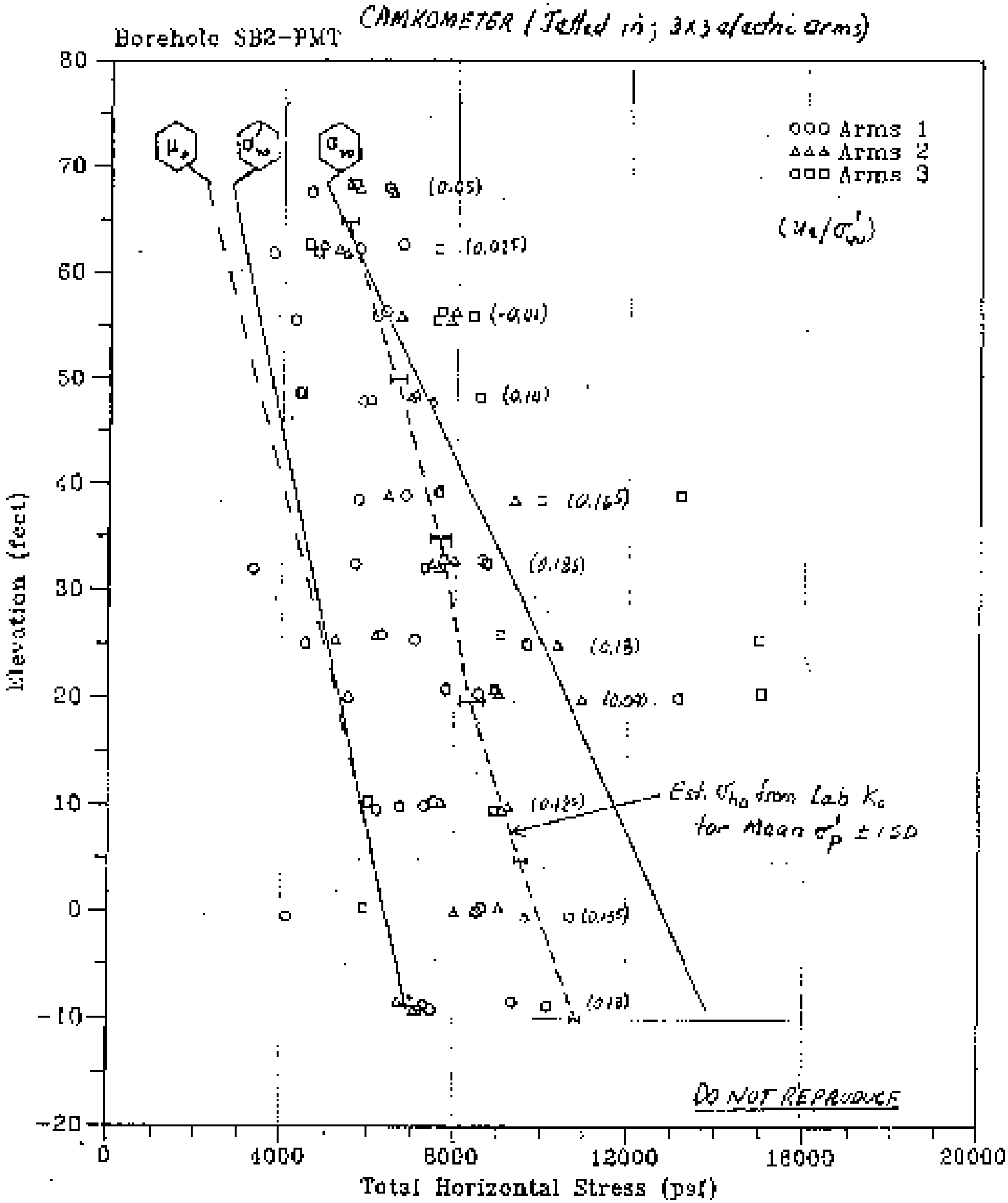
(UNH Final Report to HSA, 6/91)

Fig STP-1

CCL 8/1991

2/22/93 1.322 CA/T STP Boston Blue Clay

CENTRAL ARTERY (I-93)/THIRD HARBOR TUNNEL (I-90)
SELF-BORING PRESSUREMETER TESTING
TOTAL HORIZONTAL STRESS



"Corrected"

Figure 6: Total Horizontal Stresses from Self-Boring Pressuremeter Tests

(From UNH Final Report to HTA, 6/91)

Fig STP-2

CCL 8/91 3/3/92 1.322 CA/T STP Boston Blue Clay
1/28/93

CENTRAL ARTERY (I-93)/THIRD HARBOR TUNNEL (I-90)
SELF-BORING PRESSUREMETER TESTING
COEFFICIENT OF EARTH PRESSURE AT-REST

Borehole SB2-PMT CAMKOMETER (jetted in; 3x3 electric arms)

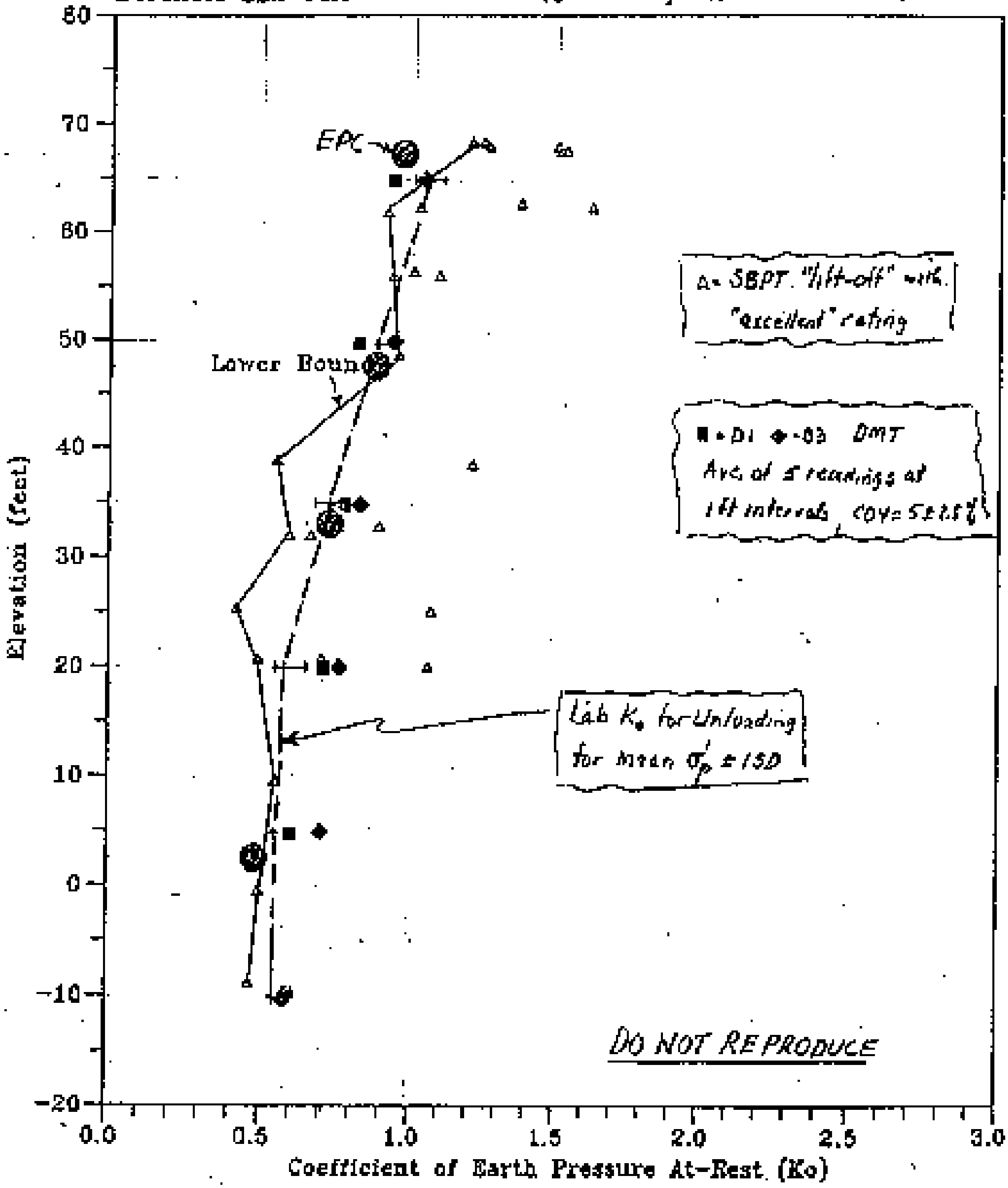


Figure 7: Coefficients of Earth Pressure At-Rest

EPC
SBPT
DMT

Fig STP-3

CCL 2/20/99

1.322 Class Schedule, Reading Assignments, Etc. on CONSOLIDATION (Part C)

Topics: from Handout Notes	Approx. No. Classes	Reading (Backup): Tokyo ('77)	Reading (Backup): SF ('95)	Reading (Backup): Other	Remarks
I Introduction • Background • K_o: trends & measurement • In situ testing	2	4.2.7 (2.2.4 – 4.2.6)	(1.5 – 3.2)	–	Covers several in situ devices for estimating K_o (Some also for OCR & strength)
II Amount of 1-D Consolidation (ρ_{cf}) • Complete tests & ρ_{cf} eqn. • σ'_p mechanisms & measurement • Effects of disturbance, creep, etc. • In situ tests for SH profiling	4 - 4½	–	2.2	–	"Mini" problem: develop field & lab testing programs to determine best in situ test for stress history profiling
III Rate of Consolidation (ρ_c) • Terzaghi theory & meas. of c_v • Effects of SH, disturbance, etc. • Permeability • Non-linear consolidation	2	–	(3.4)	–	–
IV Secondary Compression (ρ_s) • C_α/C_c concept • Hypothesis A vs B • Surcharging	1½	(2.2.6)	2.5	–	Major Home Problem covering Parts I - IV
V 2 & 3-D Loading & Vertical Drains • Initial settlement (ρ_i) and ρ_{cf} • Rate of settlement • Consolidation with vertical drains	2 - 2½	(2.2.5)	(3.3)	Foott & Ladd (1981) L 361 HP 16/2	Self-graded home problem
VI Problem Soils • High s_t • Peats • Collapsing & expansive • Residual • Varved clay	1½ - 2	–	–	–	Emphasis on peats and collapsing/expansive soils
	13 - 14.5				

2/98
3/4/99
3/1/01

1-D CONSOLIDATION: MAGNITUDE OF FINAL SETTLEMENT

(Note: Replace $\bar{\sigma}$ with σ')

1-D CONSOLIDATION: MAGNITUDE OF FINAL SETTLEMENT

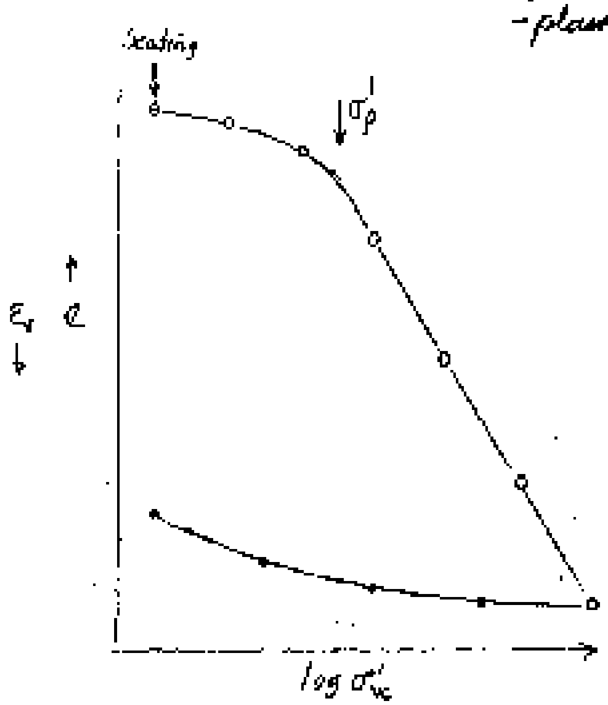
1. ROLE OF OEDOMETER (1-D Consolidation Test)

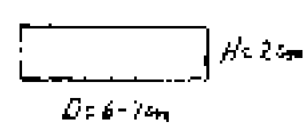
1.1 Objectives of Test - 5 items

1) σ'_p = yield stress
2) CR, RR, SR = compressibility
3) c_v → Rate of primary consolidation
4) C_α = Rate of secondary compression
5) $K_0 = \sigma'_{hc}/\sigma'_{vc}$ for $\varepsilon_h = 0$ (special projects, e.g., using FE w/ GSM)

1.2 Std. Procedure - Incremental (ASTM D2435-90)

1) Seating $\sigma \approx 0.1$ atm. : when add water?
2) LIR = 1 (Standard)
   When reduce? < a) b)
3) t_c = 24 hr. (Std) : How get EOP?
4) Misc -
   - Max. stress to define VCL & σ'_p →
   - S_i - always check
   - Filter material - paper (Con sol) - plastic



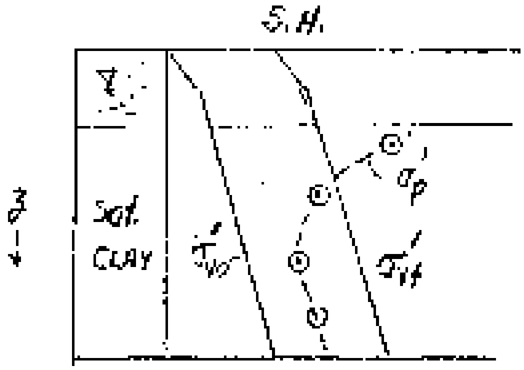


Why D/H ≥ 3?

3/6/90 2/99 3/1/01

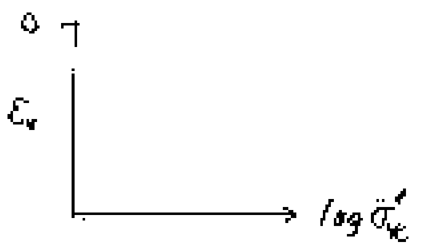
2. SETTLEMENT COMPUTATIONS

2.1 Problem



$RR = \frac{C_r}{1+e_0}$; $CR = \frac{C_c}{1+e_0}$

Have raw data from 4 oed. tests



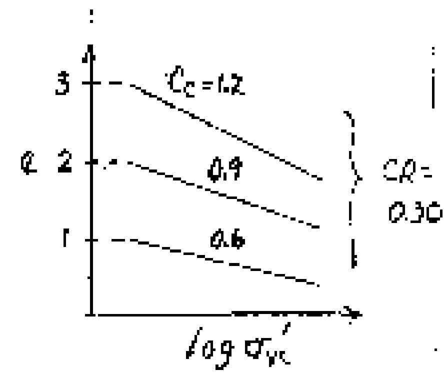
2.2 Questions (Ignoring effects of disturbance & creep)

1) Eqn for $\rho_{cf} = \Sigma H_i (RR \log \bar{\sigma}_p/\bar{\sigma}_{vo} + CR \log \bar{\sigma}_{vf}/\bar{\sigma}_p)$

2) How evaluate parameters?

3) Most important variable = σ'_p

2.3 Discussion



1) Plotting e (research) or ε (practice) vs $\log \sigma'_{vc}$

2) Typical values CR
(soft → med. stiff, low-mod. S_t)

CL → 0.25 ± 0.1
CH → 0.35 ± 0.1 } for non-structured → const. CR

3) Typical values of RR/CR ≤ 0.1-0.2 (unless significant "structure" - S-curve)

a) Collective evaluation of RR & CR

b) Supplemental information → σ'_p profile

(1) Geology → help explain/predict trends

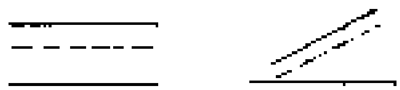
(2) In situ testing → spatial variability (Mini-problem)

3. MECHANISMS OF VOLUME CHANGE (Part A, V)

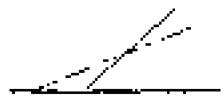
Rebound

1) Elastic particle deformation
   especially "bending" platy particles

2) Change in "closest" spacing (≈ constant orientation)



3) Change particle orientation & sliding at contacts



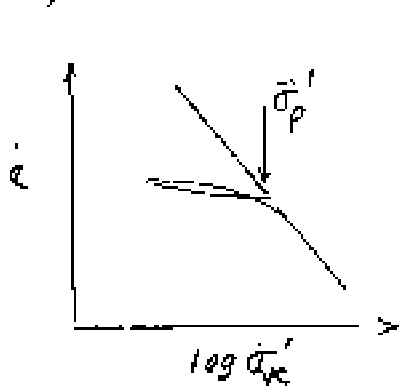
4) "Particle" crushing
   - Clay flocs & aggregates
   - Sand

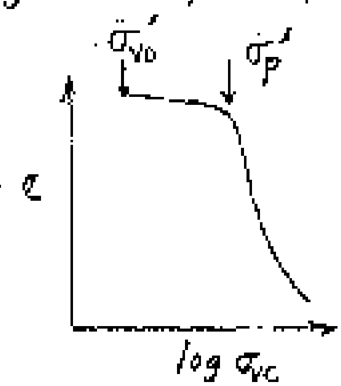
4. MECHANISMS CAUSING PRECONSOLIDATION PRESSURE

4.1 Physical Significance

$$\bar{\sigma}_p' \equiv \sigma_p' \equiv \bar{\sigma}_{vm} \equiv P_c'$$

Yield Stress for 1-D loading separating
"elastic" behavior - small strains & ≈ recoverable
vs "plastic" behavior - large strains, mostly non-recoverable





3/89 3/97

4.2 "Four" Principal Mechanisms

TABLE V SF'85 p66

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)
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)

4.3 "Mechanical" $\Delta\bar{\sigma}' = \Delta\sigma - \Delta u$

a) $\Delta\sigma$

	Constant $(\bar{\sigma}_p' - \bar{\sigma}_{vo}')$	$\bar{\sigma}_p' / \bar{\sigma}_{vo}'$	Erratic

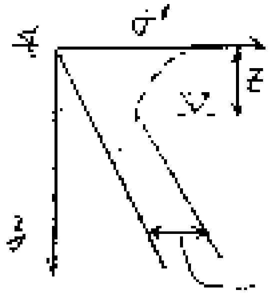
1) Overburden
2) Prior structures
3) Glaciation
4) Waves - $\Delta\tau$ (Madsen, 1978 Geot)

NOTE: Also review 1.361 Notes Part IV-4

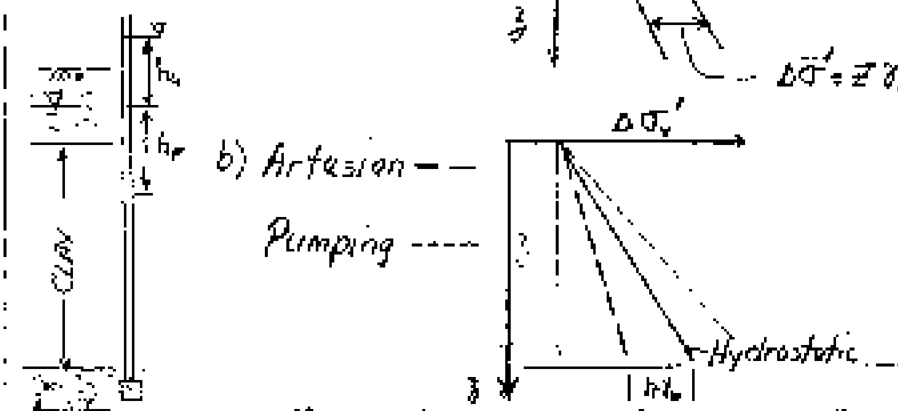
$(\bar\sigma'_p - \bar\sigma'_{vo})$ $\bar\sigma'_p/\bar\sigma'_{vo}$ Erratic

b) Δu at Boundaries

i) Δ Water Table



$\Delta\bar\sigma' = Z\gamma_w$ if $\bar U \to 100\%$
& $S \approx 100\%$ (within crust)



b) Artesian
Pumping

$\boxed{\Delta\bar\sigma_v' = z(\gamma_b \pm i \cdot \gamma_w)}$

• Mexico City, Houston, Tokyo, Taipei, Bangkok

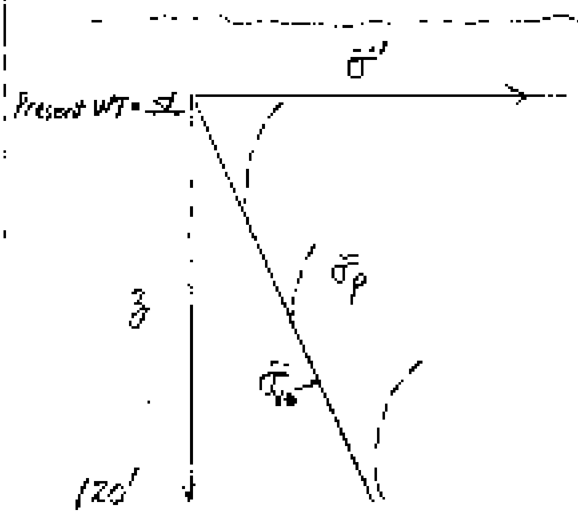
4.4 Desiccation (Drying crust)

1) Evaporation, vegetation, etc.
1. is very significant (esp. trees) | changes during seasons within "active" zone. Can reach 10 ± 5 m!

2) Frost

. Both can → v. high "soil suction"

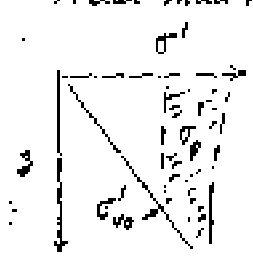
• K_o or not



• Observed deltaic flood plain, e.g. Mississippi River

How explain?

• Tidal mud flat deposits (Holocene)



How get OC?

See Kenney (1964) p 56?b

CCL 2/94 1.322 p. 5a

SEA-LEVEL MOVEMENTS AND THE GEOLOGIC HISTORIES OF THE POST-GLACIAL MARINE SOILS AT BOSTON, NICOLET, OTTAWA AND OSLO

Geotechnique (1964) 14(3) 203-230

by

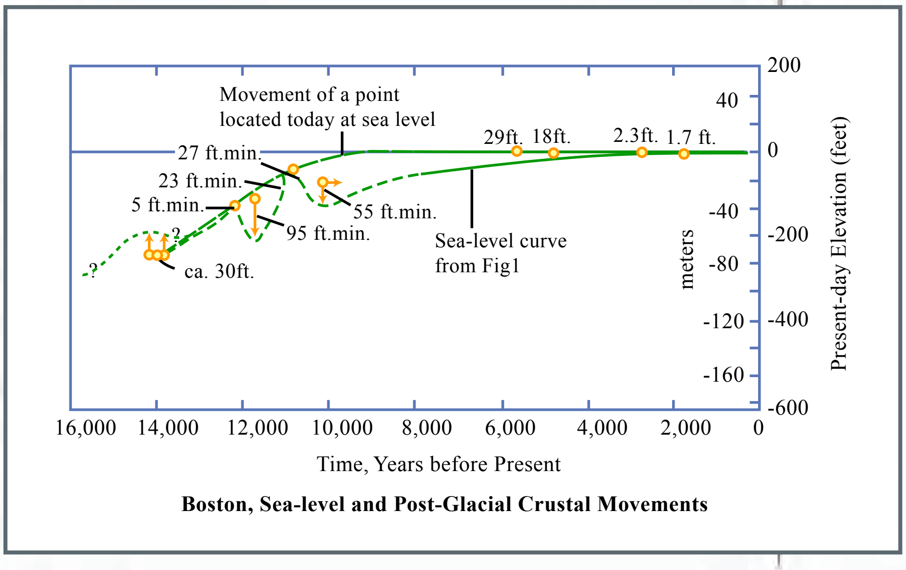
T. C. Kenney*

SYNOPSIS

The Paper is divided into two separate parts; the first part deals with eustatic sea-level movements which have occurred during the past 20,000 years, and the second part concerns the geologic history of marine soil deposits at Boston, Nicolet, Ottawa, and Oslo.

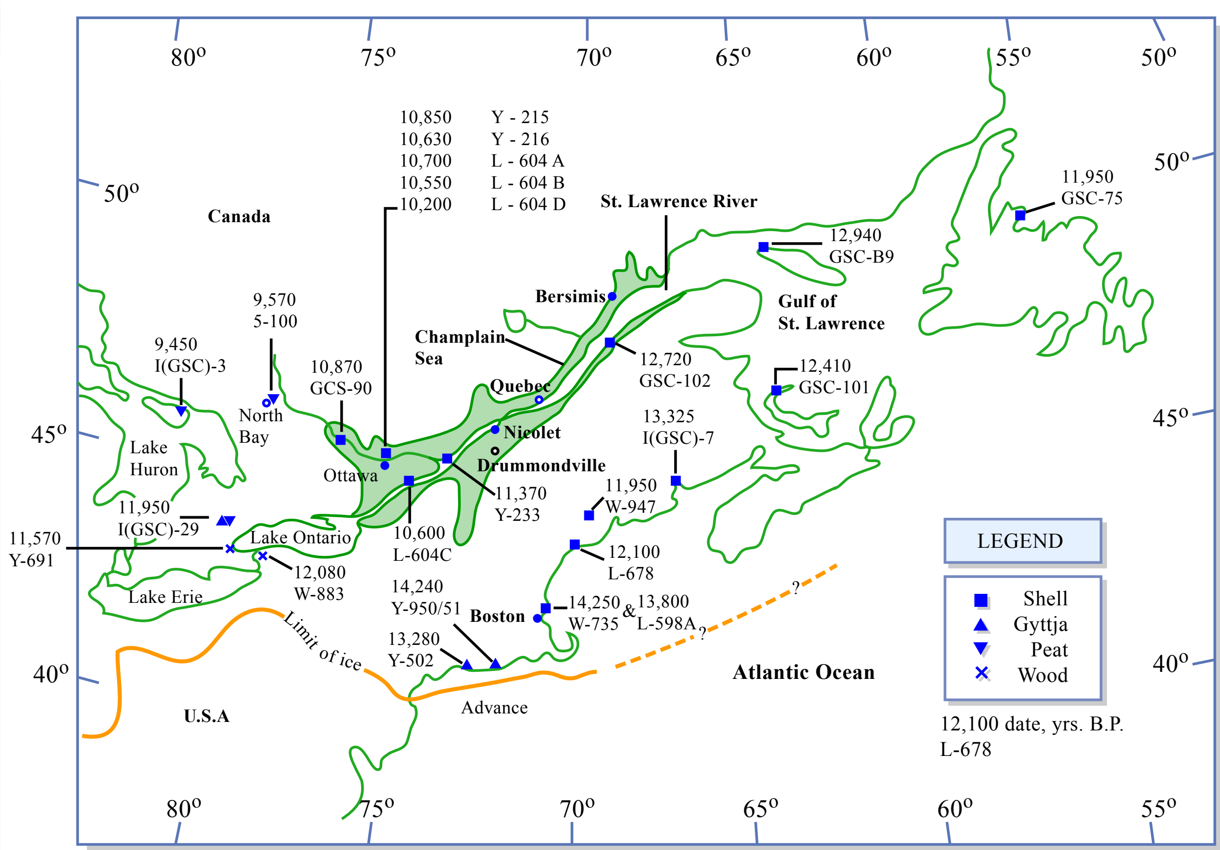
Eustatic sea-level movements are determined by synthesizing direct and indirect evidence concerning sea-level movements. Direct evidence consists of the ages and elevations of marine fossils and other materials, and elevations and ages of deposition and erosion surfaces which were controlled by sea-level movements. Indirect evidence consists of the dates of climate and temperature changes and the dates of major activity of the continental glaciers. From these data a provisional sea-level movement curve has been drawn for the period extending over the past 20,000 years.

Geologic histories of marine soil deposits are dependent on, among other things, sea-level and local crustal movements. For each of the above mentioned sites, time curves of sea-level and crustal movements are drawn, and from a study of these curves and other geological evidence, the general geologic history of the soils at each site is determined. Geotechnical data are presented in the form of boring profiles and results of laboratory tests, and these are discussed with respect to the previously determined geologic history. In certain cases there are apparent discrepancies between the geologic histories and the interpretations of geotechnical data, and these apparent discrepancies are commented upon.



Boston, Sea-level and Post-Glacial Crustal Movements

Figure by MIT OCW.



Extent of the "classical" Wisconsin glaciation of North America

Figure by MIT OCW.

CCL 2/94 1.322 56

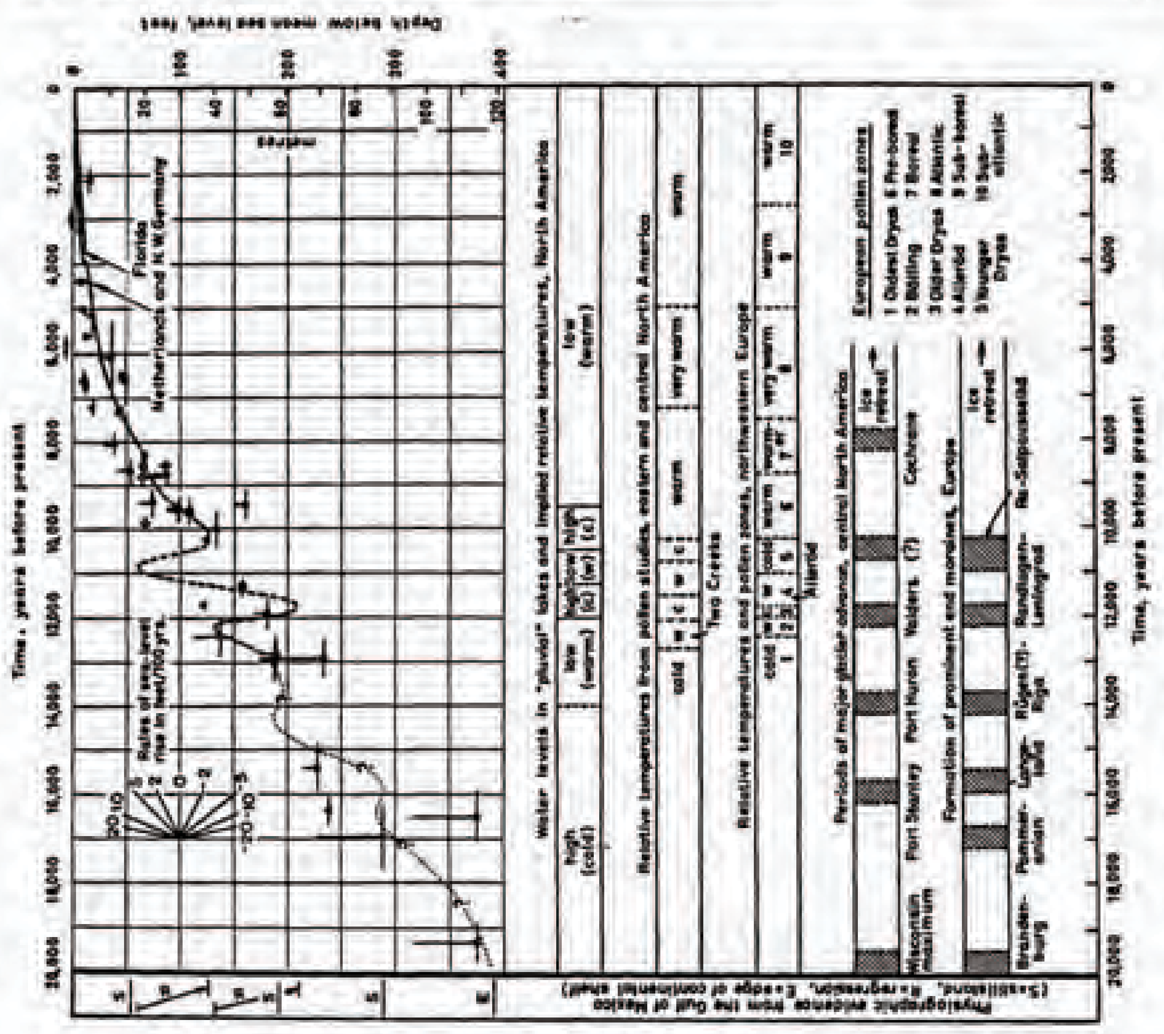
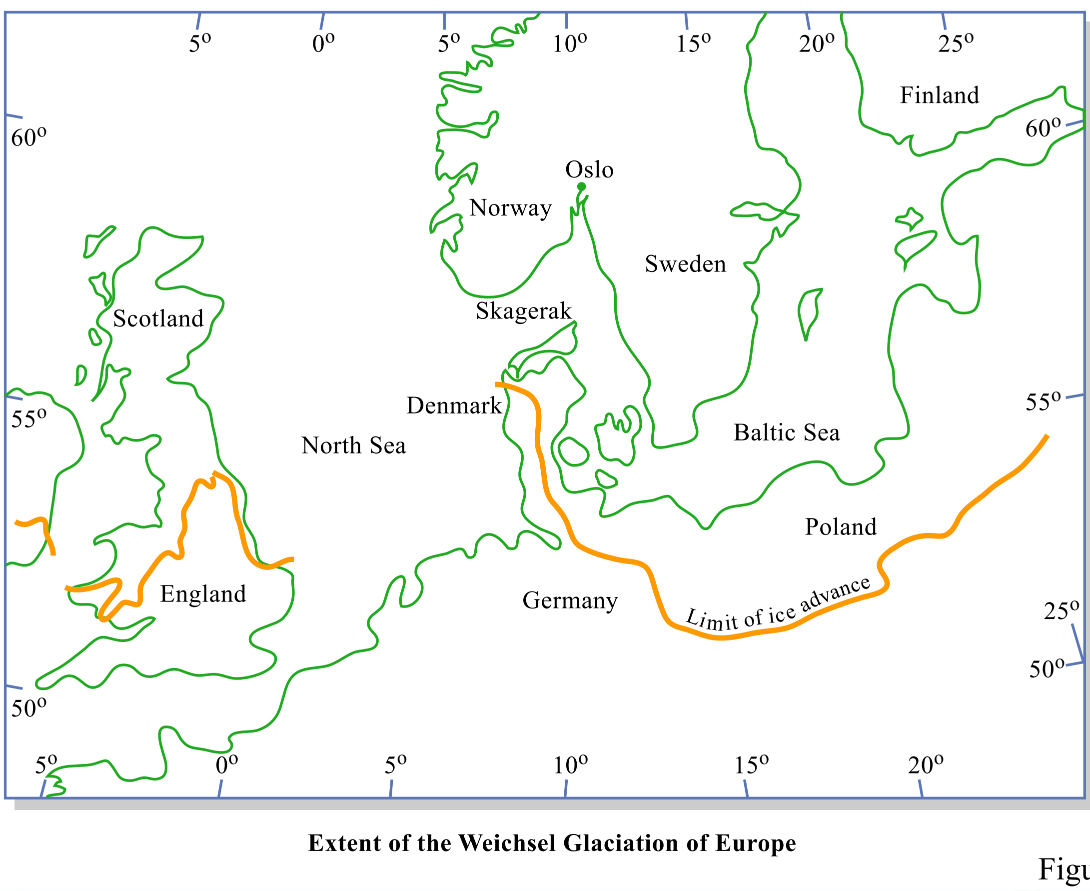


Fig. 1. Eustatic sea-level movement curve for the late Pleistocene period



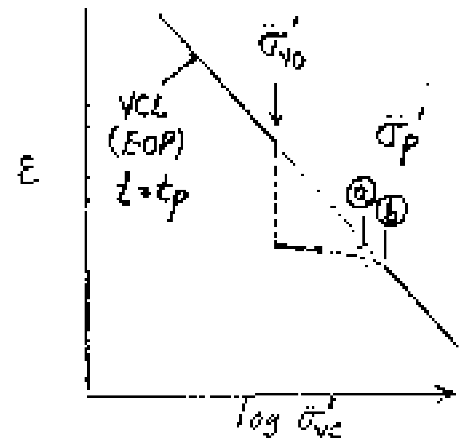
Extent of the Weichsel Glaciation of Europe

Figure by MIT OCW.

2/97 3/01

4.5 Drained Creep (Sec. Comp. = Aging*)

* Note: Some authors use term "aging" to include D = Physico-Chem.



ⓐ $C_\alpha \log(t/t_p) = CR \log(\bar{\sigma}'_p/\bar{\sigma}'_{v0})$

$\log OCR = \frac{C_\alpha}{CR} \log(t/t_p)$

$OCR = (t/t_p)^{C_\alpha/CR}$

$C_\alpha/CR = 0.045 \rightarrow$

No. cycles	$m=1$ OCR	$m=0.8$ OCR
1	1.11	1.14
2	1.23	1.30
3	1.365	1.475

≈ 10-15% /LC

ⓑ Mesri & Castro (JGE 3/87)

$C_\alpha \log(t/t_p) + RR \log(\bar{\sigma}'_p/\bar{\sigma}'_{vc}) = CR \log(\bar{\sigma}'_p/\bar{\sigma}'_{vc})$

$OCR = (t/t_p)^{\frac{C_\alpha/CR}{m}}$ where $m = 1 - C_s/C_c = 1 - RR/CR$

NOTE: No difference in predicted ε_{vf} if $\sigma'_{vf} > \sigma'_p$

Discussion

1) Does $\bar{\sigma}'_p$ lie on EOP? CCL believes it should if no physico-chemical cementation.

2) Tokyo Fig 39 à la Bjerrum (1972) → Incr. OCR with incr I_p

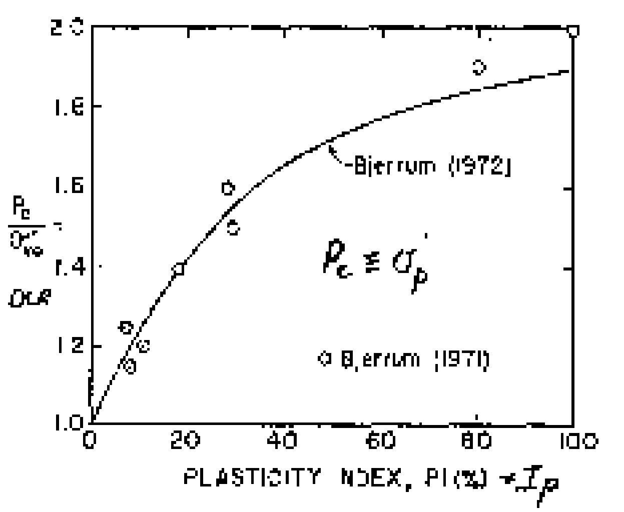


Fig. 39 Precompression of late glacial and post glacial clays attributed to aging.

Why is this plot suspect (CCL regrets including)?

Ans: If C_α/CR = constant & if deposits of same age, then OCR due to aging should not vary w/ I_p

3) Confusing terminology:

"Young" NC = little or no aging

"Old" NC = significant aging

↑ Should use "normally loaded"

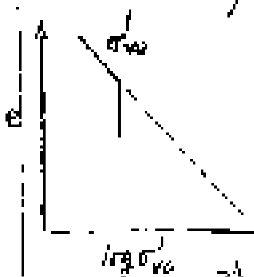
4.6 Physico-Chemical (see Table V, p4)

1) Discussion (Cementation & other causes of "bonding")

- Certain deposits do contain potential cementing agents like carbonates, Al-Fe oxides, silica, organic matter, etc.
- Other causes even less well documented
- CCL believes can be very significant in some deposits, but hard to prove

NOTE: If combination of high I_L + high $\bar{\sigma}_p'$ + brittle clay behavior } then quite likely e.g. Champlain Clays

2) Example - James Bay B-6 (p8)

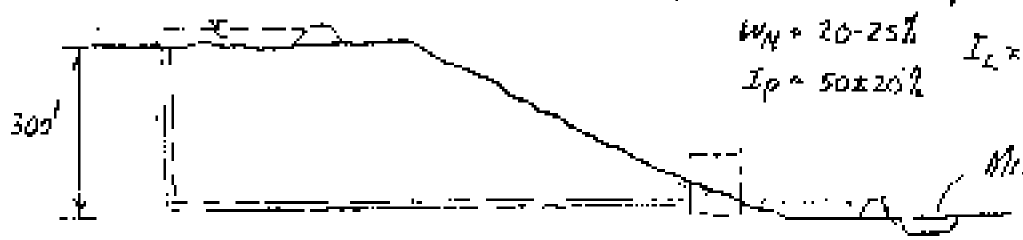


Why conclude Marine Clay had significant cementation?

- $\bar{\sigma}_p'$ inconsistent of mechanical, desiccation and aging
- Variable $\bar{\sigma}_p'$ in block samples
- 1-D → very compressible at $\bar{\sigma}_v' > \bar{\sigma}_p'$
- CK₀UC → very brittle with v. high yield surface

3) Example - Nebraska Pumped Storage Project (p9)

Pierre Shale : up to 50% $CaCO_3$
$w_N \approx 20\text{-}25\%$ $I_L \approx 0$
$I_p \approx 50 \pm 20\%$



- Most oedometer data → $\bar{\sigma}_p' \approx 160 \pm 20$ atm vs Geology predicted only 80 atm vs ave. $\bar{\sigma}_{vo} \approx 10$ atm
- If mechanical $\bar{\sigma}_p'$ → v. high K_o → significant impact on slope stability ("spalling") and tunnel lining design
- McGowan SM - is high $\bar{\sigma}_p'$ due to cementation?
  - leach with HCl → lower $\bar{\sigma}_p'$?
  - correlation with % $CaCO_3$

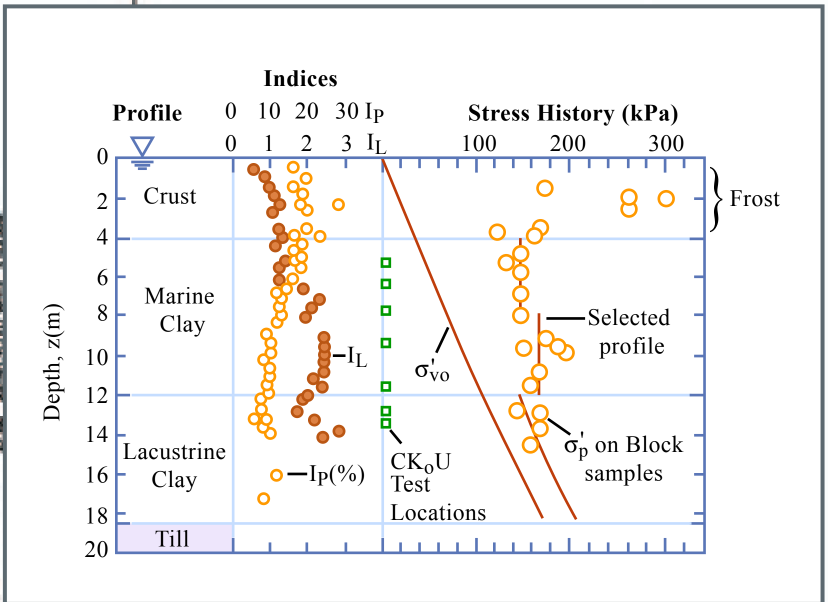
4) Examples - MIT Biology Bldg (p9a) ; EB CA/T (p11a)

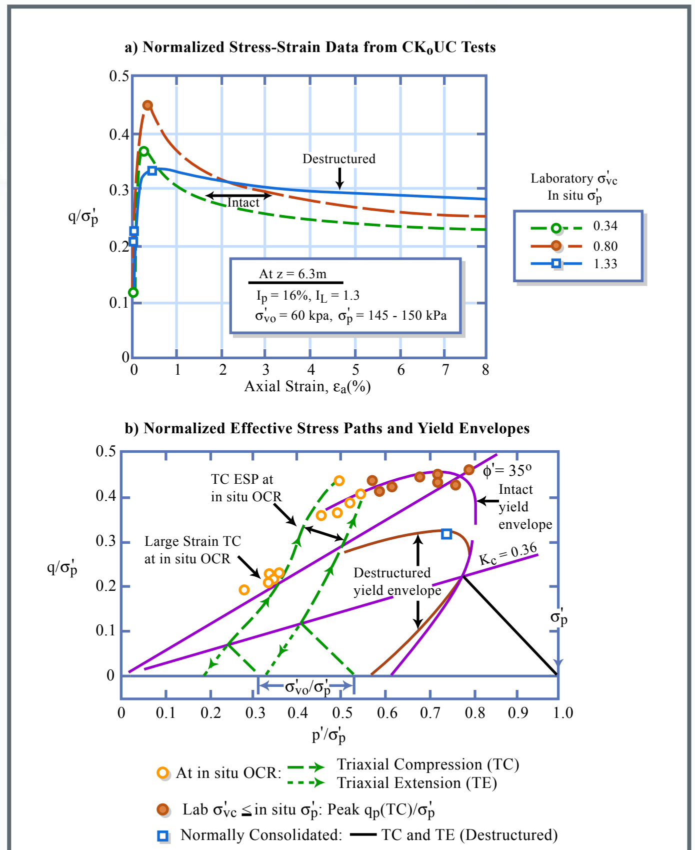
Soil profile, index properties & stress history at James Bay B-6

Marine Clay, z = 4-12m

$I_L = 2 \pm ½$

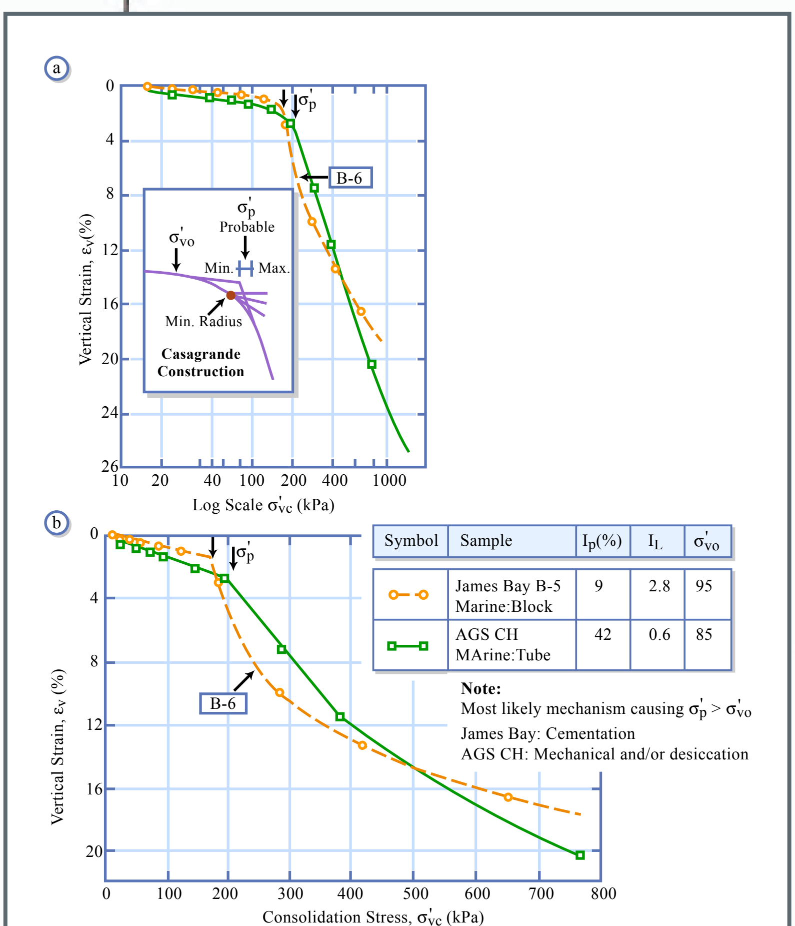
$\sigma'_p = 1.6 \pm 0.3$ bar





(a) Normalized Stress-Strain, (b) Effective Stress Paths and Approximate Yield Envelopes from CK_oU Tests on James Bay B-6 Marine Clay.

Data from Lefebvre et al. (1983).



Compression curves for clays having different preconsolidation pressure mechanisms
(a) Semi-Log scale; (b) Natural scale.

1-D & CK_oUC data (from SF '85)

Figures by MIT OCW.

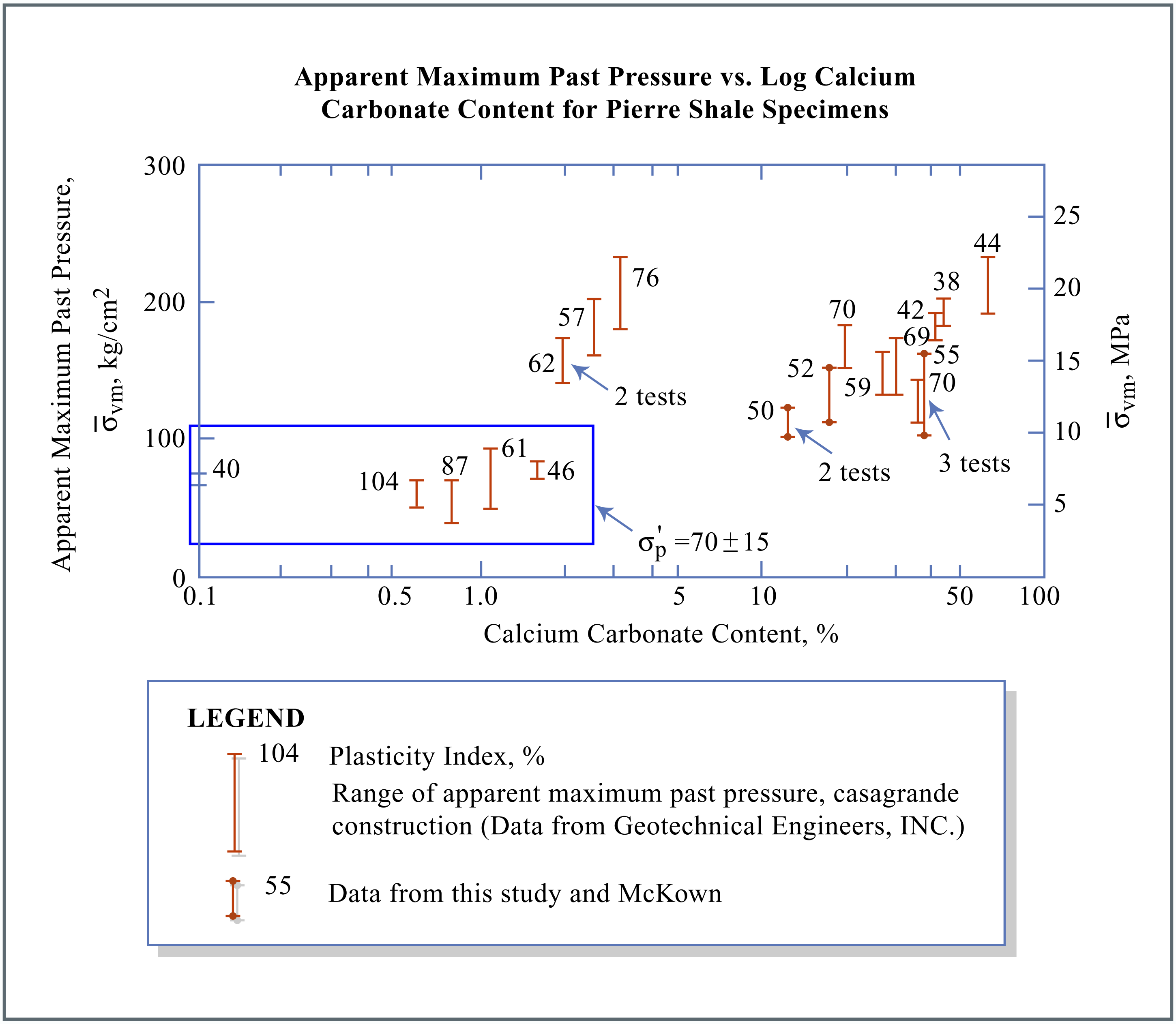


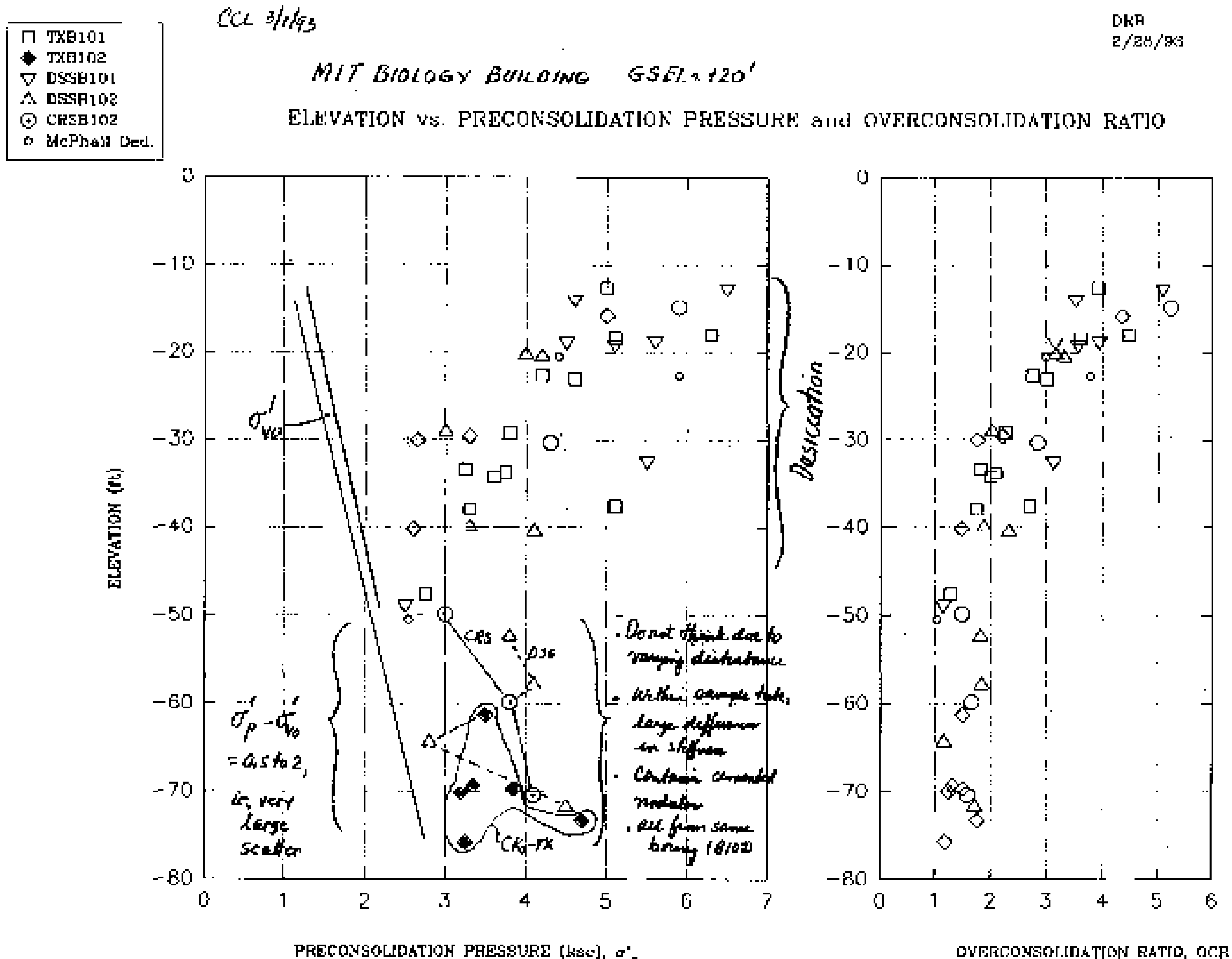
Figure by MIT OCW.

Adpated from: McGowen & Ladd (1982) ASTM STP 777

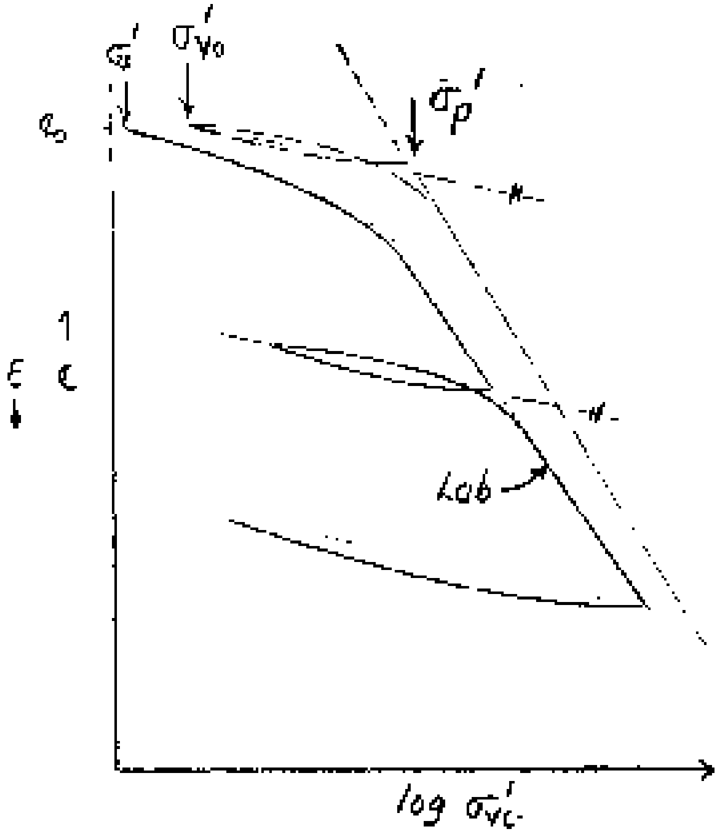
CCL 3/3/93 1.322
3/27/97

Consolidation II

p9a



5. SAMPLE DISTURBANCE (see Fig 2·6, p10a)



5.1 Schematic

· Moderate quality

· Add severe disturbance

← Validity of parallel assumption is Mechanisms → σ_p' ?
(Not for cementation)

5.2 Effects of Disturbance

1) Lower curve

2) Obscure & usually lower est. $\bar{\sigma}_p'$

3) Signifance incr. recompression compressibility

∴ Should include?

4) May lower virgin compressibility
(p Notes)

-Table 2-2 -low-moderate S_t

(using JHS) Corr/meas. CR ≈ 1.15 ± 0.05 + but can be much larger for S-shaped curves

e.g., CA/T BBC 0.25 → 0.7 (p116)
Orinoco Clay 0.25 → 0.35 (p134,4)

CCL 3/1/99

ccL 4/28/67

Reconstruction of In situ Compression Curve using Schmertmann's Method

JHS(1955) "The undisturbed consolidation of clay", 1955 Trans ASCE, 120, 1201-1230

NOTE: CCL recommends using linear (not curved) recompression and virgin compression lines to obtain Δe vs $\log \sigma'_{vc}$

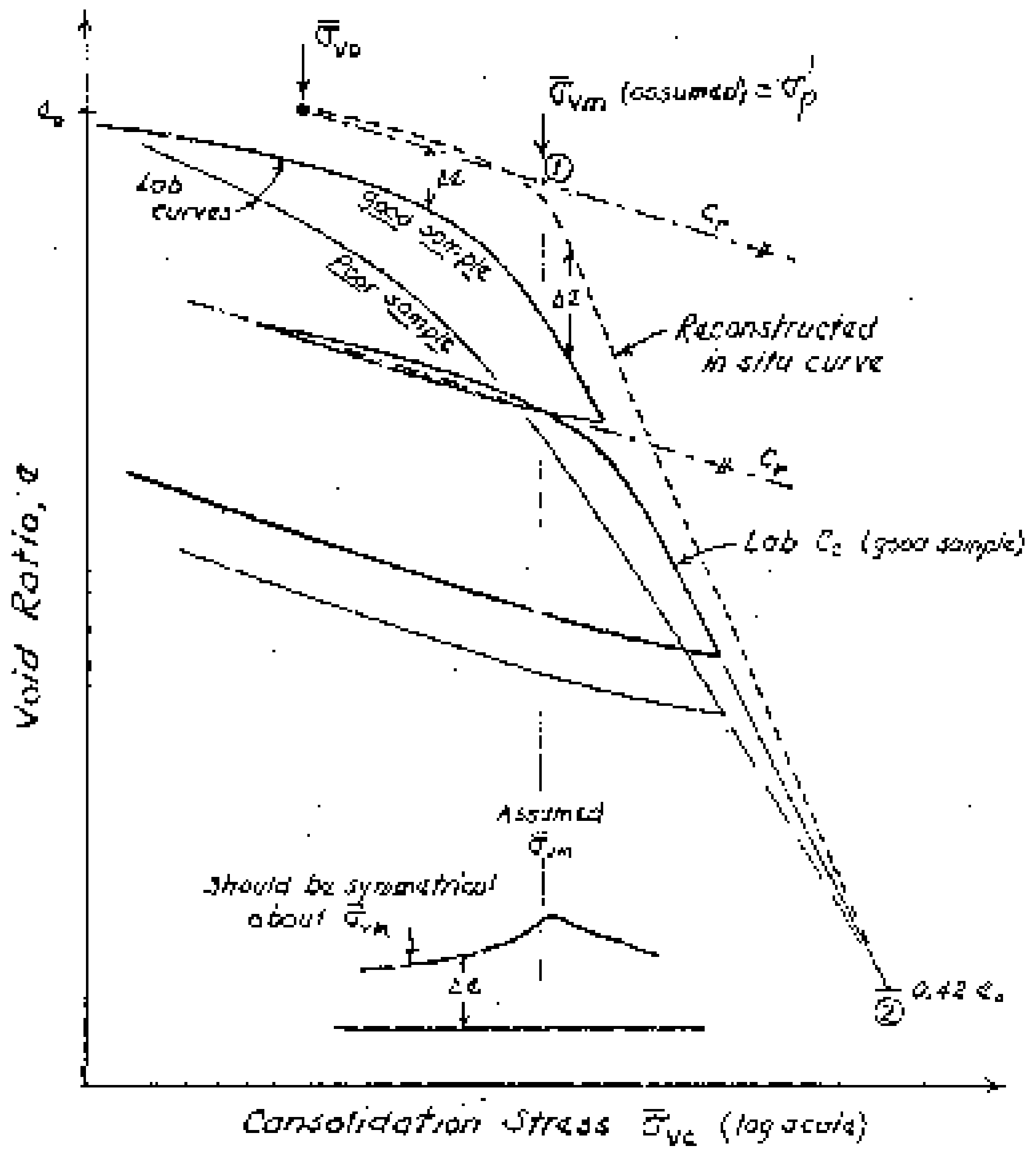
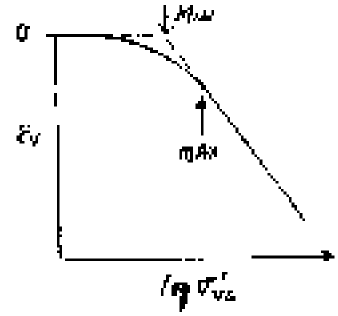


Figure 2-6

6 GRAPHICAL METHODS TO ESTIMATE σ_p'

6.2 Casagrande (AC)

- Most common
- Add min-max.
- Use standard size scale: CCL prefers 3 cycle $8\frac{1}{2} \times 11$ with $\Delta\varepsilon_v/\Delta LC = 10 \pm 2\%$



6.3 Schmertmann (Fig. 2-6 of Notes, p10a)

- No published update since 1955
- Advantage → "in situ" curve
- NOT APPLICABLE TO LOW OCR
- CCL prefers using linear (not curved) recompression - virgin compression lines to obtain $\Delta\varepsilon$ vs $\log \sigma_{vc}'$

6.1 Testing Soils with S-Shaped VCL

- p11a SB/EB CAIT Test Series: Stress history
- p11b " " " " : Typical compression curves & values of CR

6.4 Butterfield (1979 Geot. No. 4)

- Not much backup
- CCL - MIT experience → not valid (e.g. HP No. 4)

(NOTE: MIT-SI uses $\log e$ vs $\log \sigma_{vc}'$ to cover very large range in σ_{vc}')

6.5 Strain Energy = Work/Unit Volume (Covered 1.34)

- See p12 & p12a (Note: delete UR data; must use max. CR to VCL)
- Use of linear scale → more precise σ_p' than via AC
- Need data at $\sigma_{vc}' < \sigma_{v0}'$ to define initial slope

6.6 Recommendations

1) Always use AC since std practice & simple to apply
2) But SE preferred since more accurate, less judgment (esp. w/ rounded curves) and can automate, plus linear σ_{vc}' scale
3) See p12b for example of comparing SE vs AC

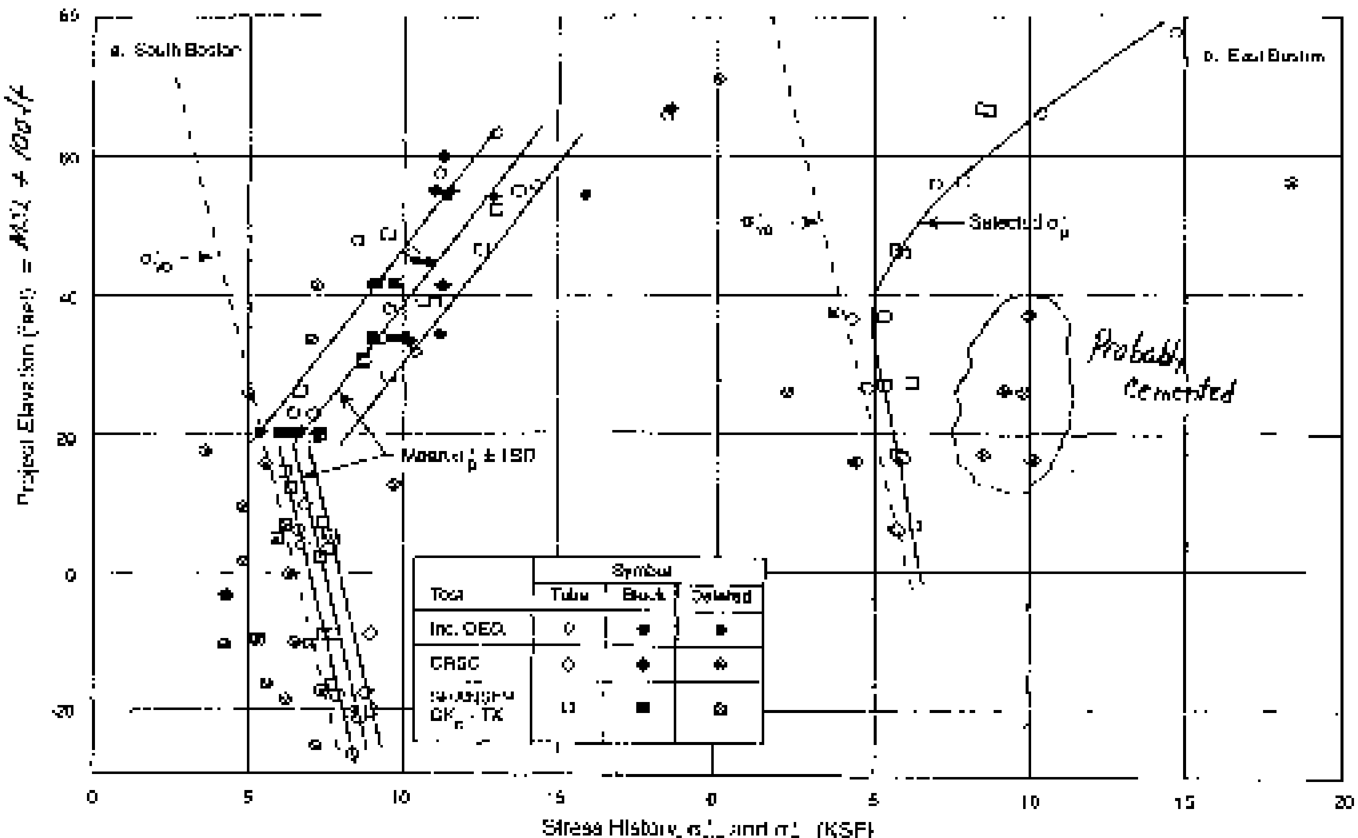
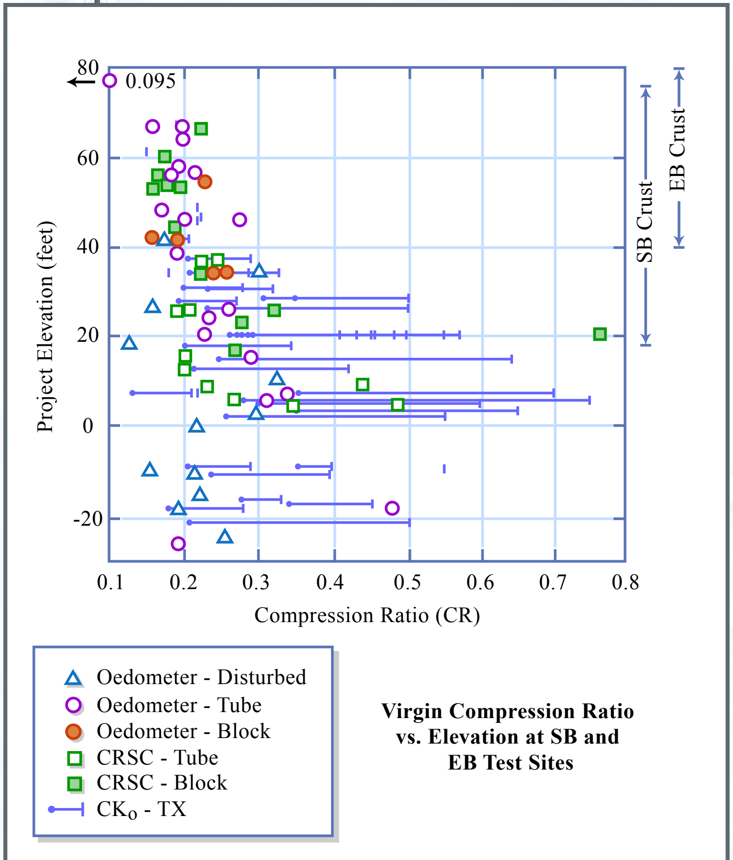


FIG. 6. Stress History from 1-D Consolidation Tests at SB and EB Test Sites (CA/T)

Notes: 1) Most of "good" σ'_p values from continuous loading tests (CRSC & SHANSEP CK_o-TX)

2) Deleted values due to excessive disturbance (ε_v @ σ'_{vo} too large for sample) or σ'_p very high (probably localized cementation)

3) Used heavy weight mud → $\sigma_v = \frac{?}{3}\gamma_m$ ≃ est. σ_{ho} & fixed piston sampler (3" φ) and Sherbrooke 30cm φ block sampler (at SB to El. 20)

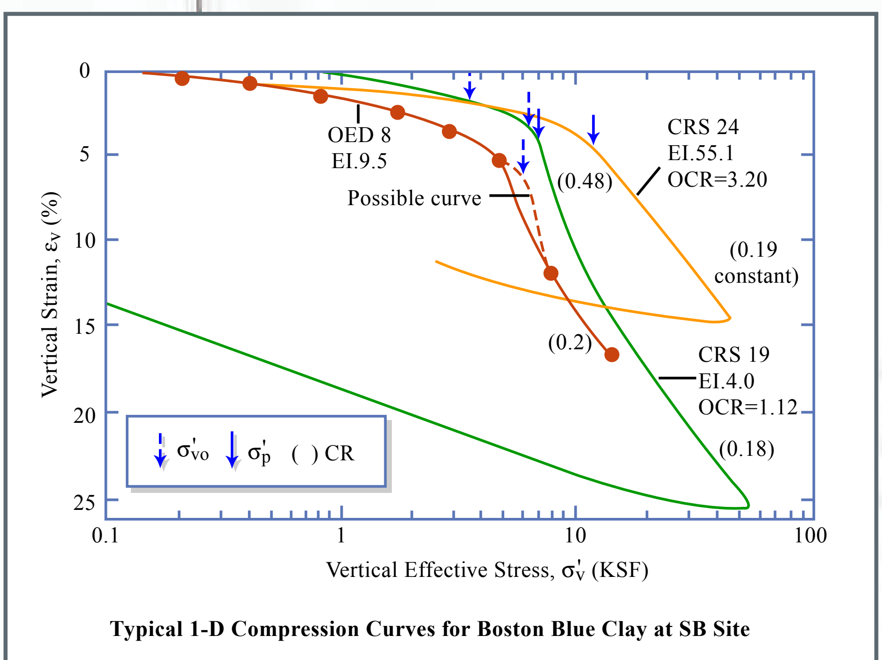


Virgin Compression Ratio vs. Elevation at SB and EB Test Sites

Figure by MIT OCW.

1) In upper crust, CRS, CK_o-TX & OED → same values of max. CR ≈ 0.2 (since similar compression curve).

2) At greater depths, need continuous loading tests to define max. CR ≈ 0.4 – 0.7. OED → much lower values of max CR, partially due to more disturbance (tests run before used special extrusion technique)



Typical 1-D Compression Curves for Boston Blue Clay at SB Site

Figure by MIT OCW.

Adapted from: Ladd et al. (1998) Geo-Congress 98

1) CRS 24 in crust → linear VCL w/ CR = 0.2.

• Oed ≈ CRS • SE better than AC

2) CRS19 in low OCR clay → S-shaped VCL w/ CR decreasing at increasing σ'_{vc}/σ'_p

3) OED8 in low OCR clay → ill-defined VCL. Can't define σ'_p and max. CR. Plus this specimen was moderately disturbed

2/97
3/99

STRAIN ENERGY ≡ WORK PER UNIT VOLUME
SE W

References

(1) Crooks & Graham (1976), Geot. 26(2)

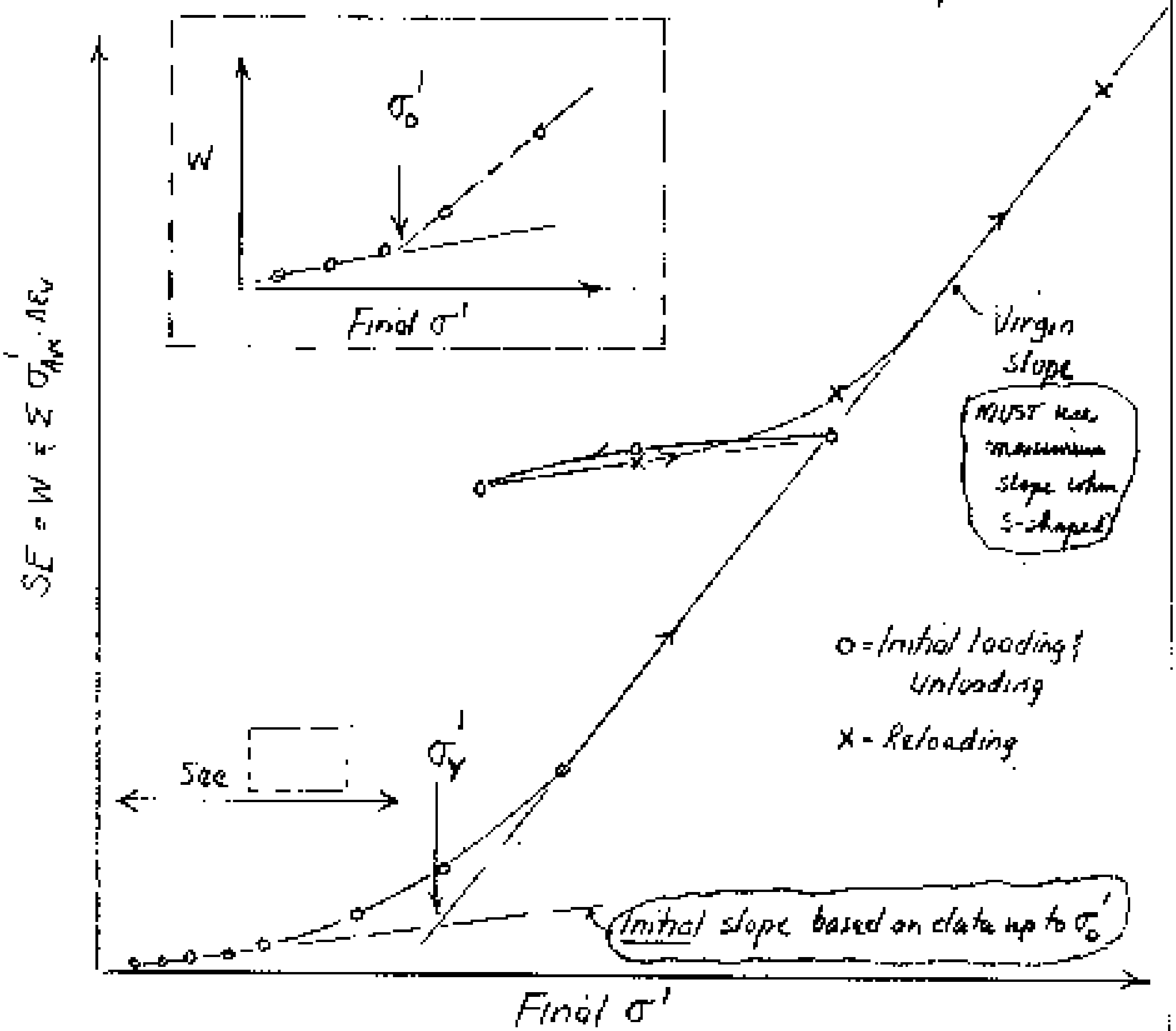
(2) Tavenas, et al (1979), Geot. 29(3)

(3) Becker, Crooks, Been & Jefferies (1987), Can Geot. J., 24(4)

Definition

- Strain Energy = Work/Unit Volume $= \int (\sigma_1' d\varepsilon_1 + \sigma_2' d\varepsilon_2 + \sigma_3' d\varepsilon_3)$
- Oedometer (Ref. 3) $W = \int \sigma_v' d\varepsilon_v \approx \Sigma(\sigma_{vc_{Ave}}' \times \Delta\varepsilon_v)$ for each increment
  MUST be natural strain $= \Delta H/H = \Delta e/(1+e)$

Technique à la Ref(3): Obtain both YIELD (σ_Y') & INSITU (σ_0') Stresses, V & H!
(see p 12a)



CCL
3/89
3/99

12/a
1.322

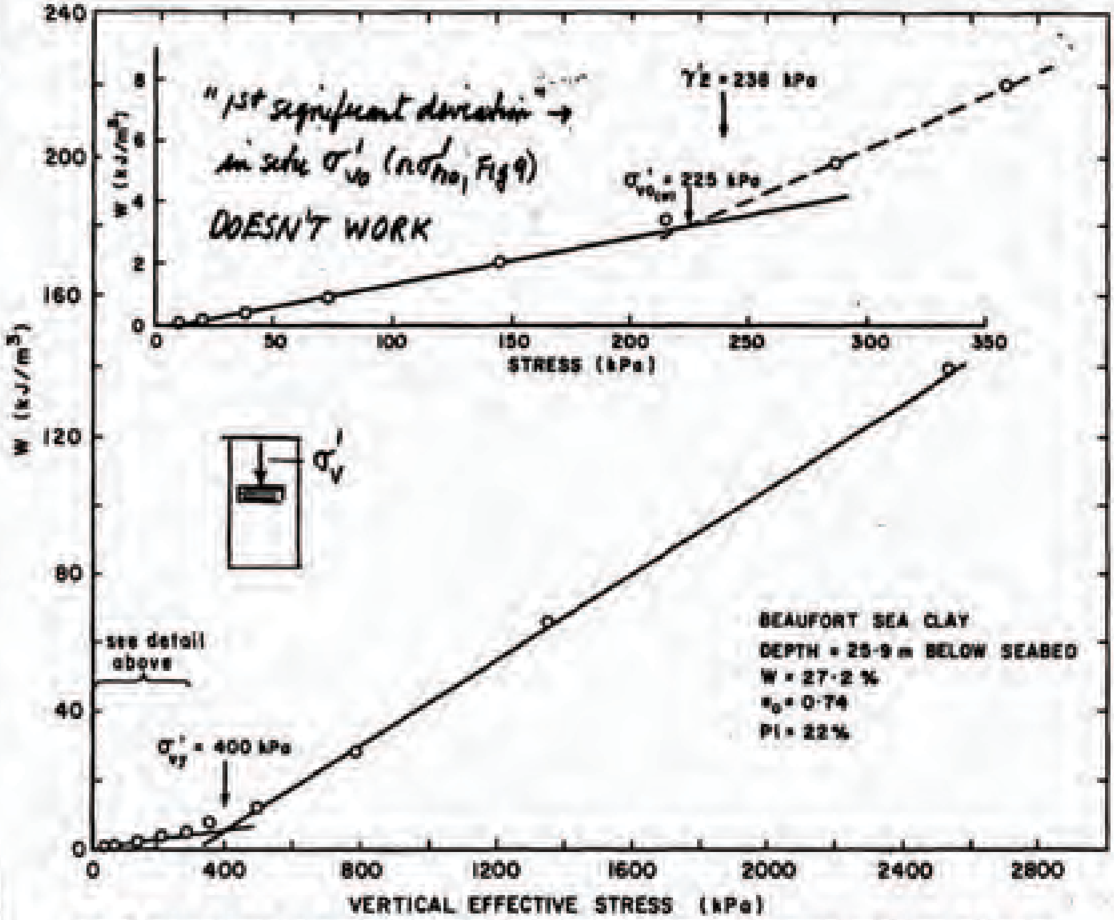
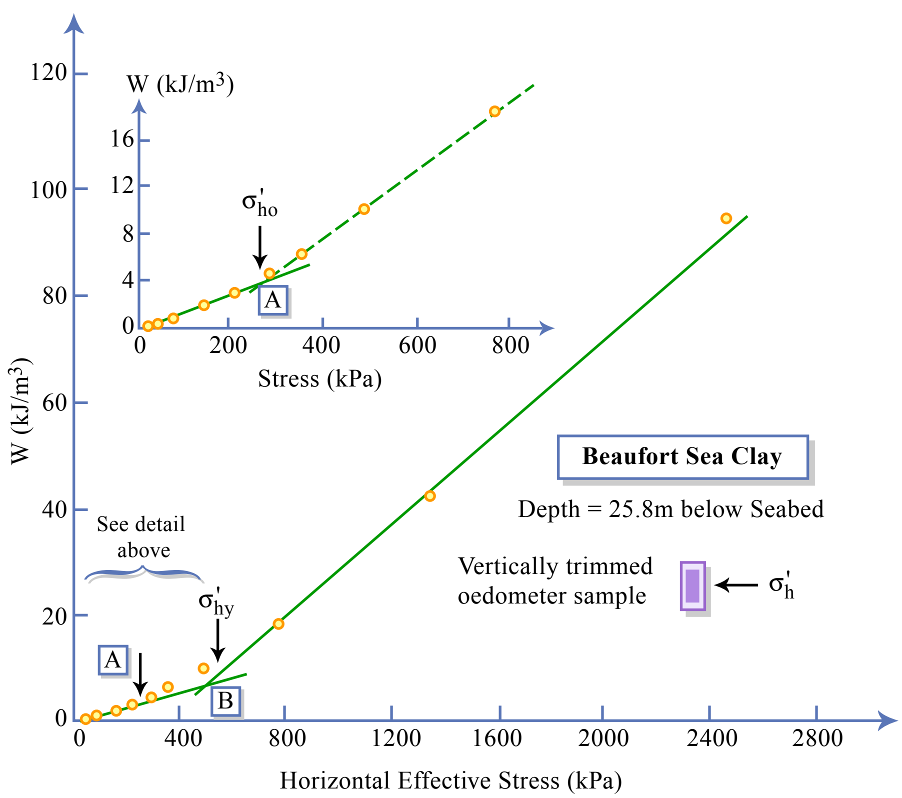


FIG. 6. Work per unit volume interpretation of oedometer test data for overconsolidated Beaufort Sea clay.



Work per unit volume interpretation of VTO test on overconsolidated Beaufort Sea clay.

Figure by MIT OCW.

Experimental data from Becker, et al (1987) showing estimates of σ'_{vo}, σ'_{ho}, $\sigma'_{vy} = \sigma'_p$ & σ'_{hy} obtained from Work Per Unit Volume = Strain Energy Technique

MMD
CCL 11/21/94

CCL 3/9/95
2/97

1.322 II

p126

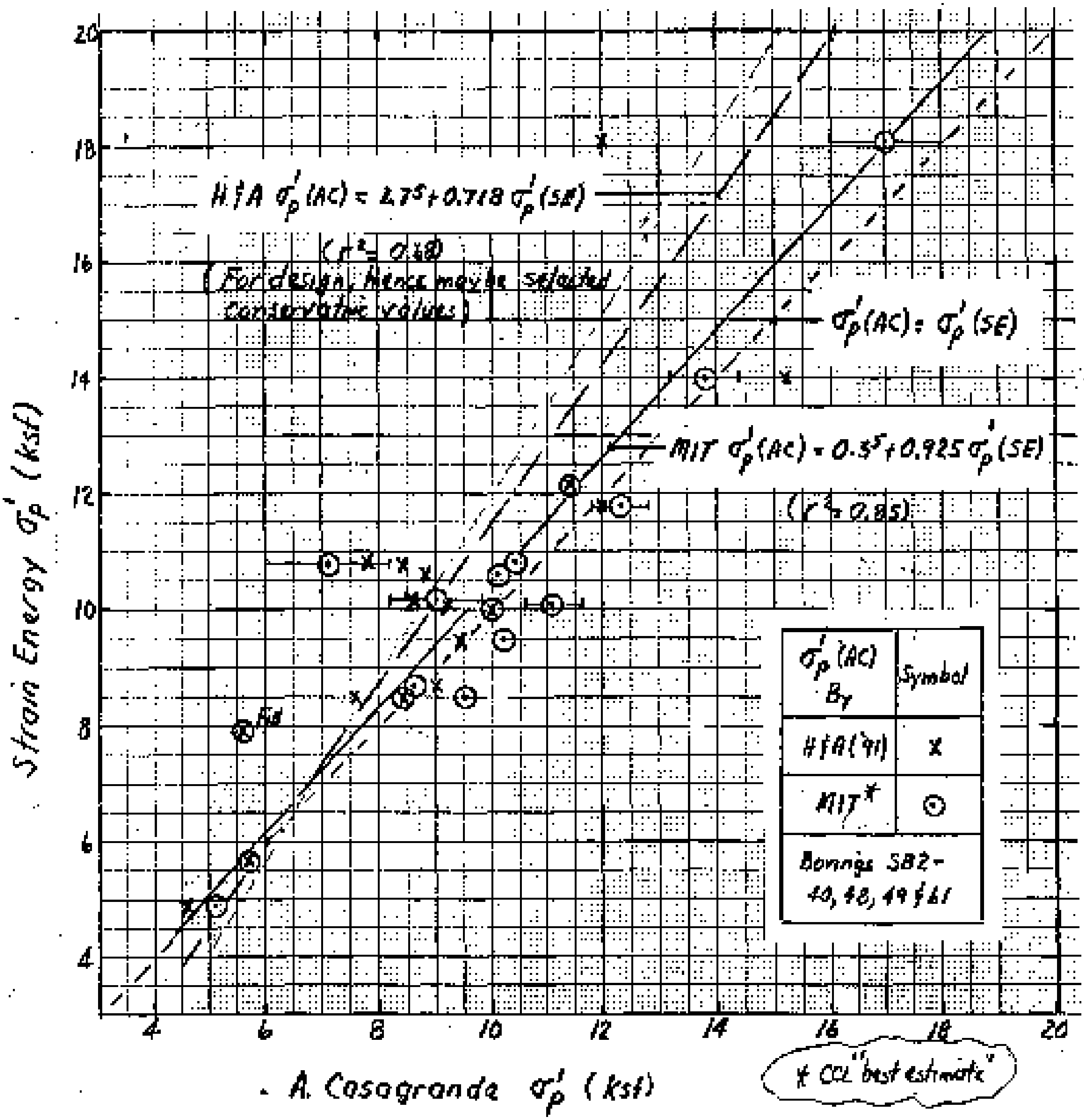


Fig. Comparison of Preconsolidation Pressures Estimated From Casagrande and Strain Energy Techniques for 17 Oedometer Tests on BBC (Fixed piston samples from SB D004A)

7. ASSESSMENT OF EFFECTS OF SAMPLE DISTURBANCE ON σ'_p

7.1 General Guidance

1) Used mudded hole ($\gamma_b V_m = \sigma'_{h0}$), FP samples & debond

2) Always use radiography whenever possible
   - Select best quality soil for testing, i.e., avoid more highly disturbed soil
   - Even best quality may show evidence of disturbance, e.g., rounded near edges

3) Always run s_u index above/below consolidation specimens to see if soil is weaker/stronger than typical (e.g., p 13a, b)

4) Measurements of σ'_s in companion UU tests also helpful

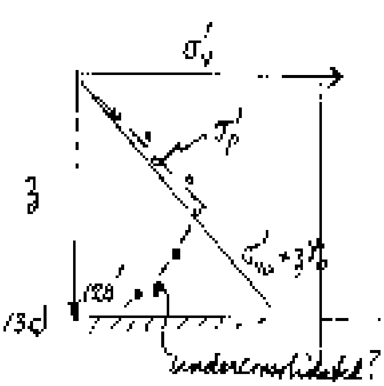
5) Always compare σ'_p profile with in situ testing data from FVT, CPTU, etc. (Mini-prof..., Section 10)

7.2 Evidence of "Excessive" Disturbance

1) Increased ε_v at σ'_{v0} compared to typical data (goes with lower σ'_s/σ'_{v0})

2) Lower CR than typical and/or less S-shaped than typical and more rounded than typical near σ'_p

3) However, as of now, no definitive methodology to obtain corrected values of σ'_p



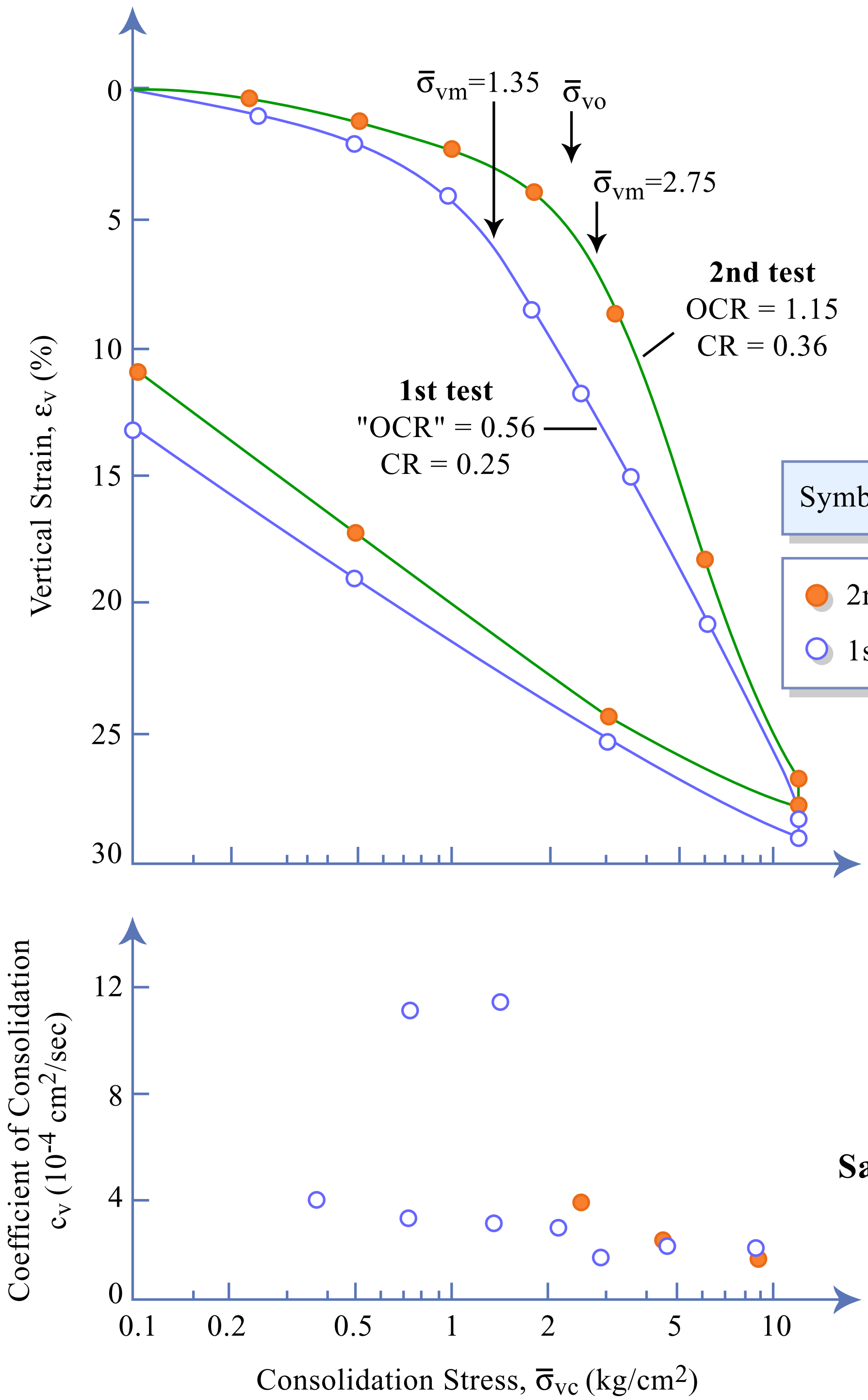
7.3 Examples

1) Offshore Venezuela, Orinoco Clay (p13a/b + Fig 10, p13d)
   - 1st test → "OCR" = 0.56 vs 2nd test → OCR ≈ 1.15
   - Note very large increase in s_u with depth below top of tube (gross disturbance)
   - See Fig. 10 for correlation with ε_v at σ'_{v0}

2) Floating foundation on very thick varved clay (Fig. 4, p13c)

3) ε_v at σ'_{v0} data on BBC from CAIT SB STP (p13d)
   - New sample extrusion technique à la Dr. Germaine (developed for Arch. site) uses piano wire around perimeter after pre-cut top/bottom based on X-ray

4) TPM (96) Sample Quality Designation (A → E) for OCR < 4±) (plotted p13d)
   - What SQD needed for reliable σ'_p? • Incr. σ'_{v0} → incr. ε_v even with very high quality samples (p13d)

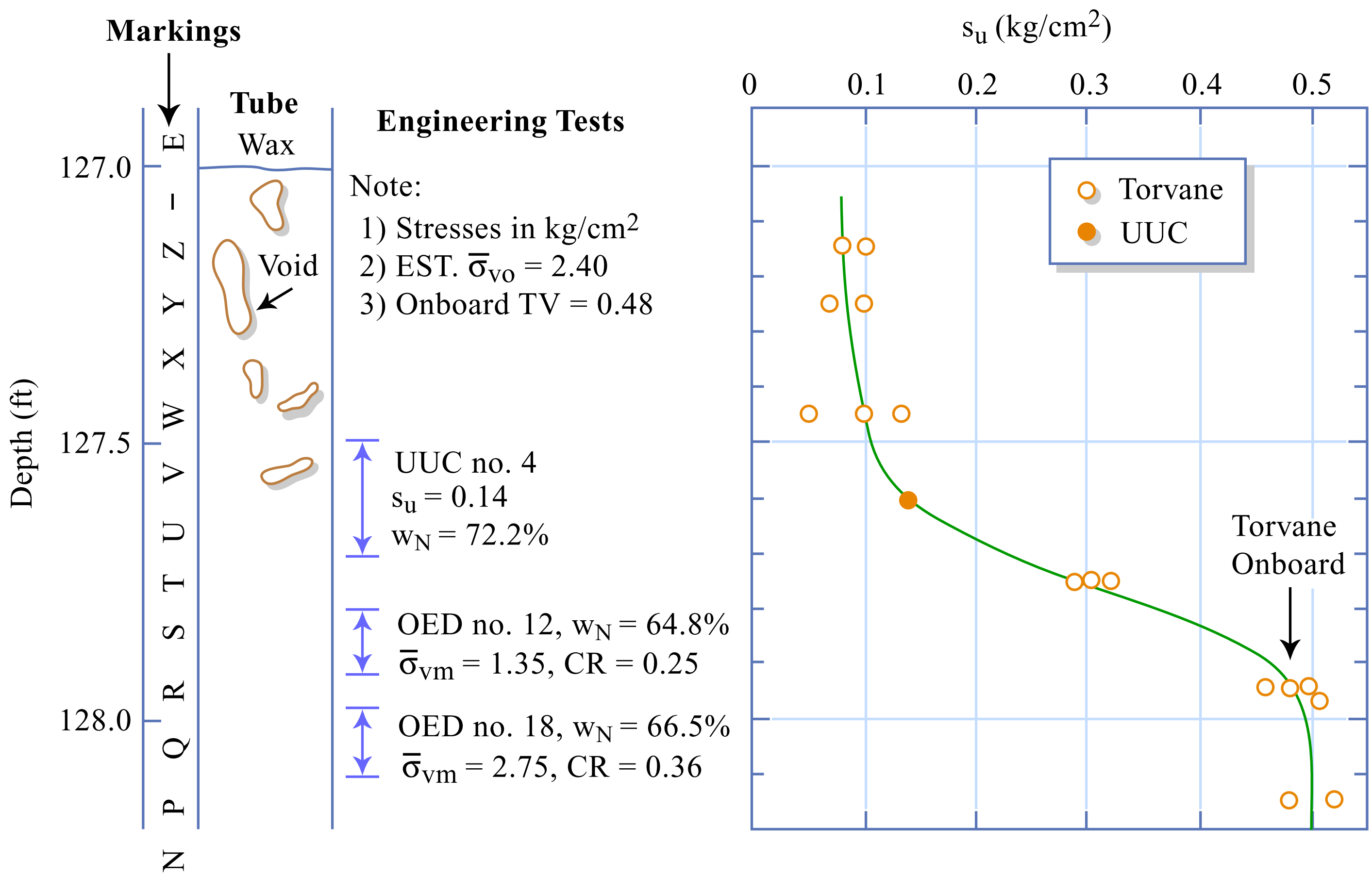


Symbol	Test No.	w_N (%)	TV (TSF)
● 2nd	18	66.5	0.51
○ 1st	12	64.8	0.30

Effect of Disturbance on Oedometer Test Data: Sample F1S57 (Depth = 128 FT)

Figure by MIT OCW.

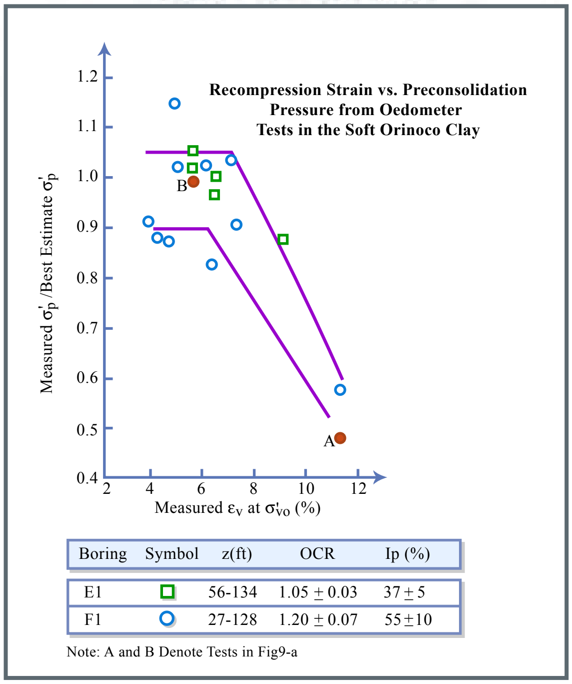
Adapted from: Ladd et al. (1980)



Comparison of Oedometer and Strength Data with Radiograph for Push Sample of Orinoco Clay

Figure by MIT OCW. Adapted from: Ladd et al. 1980.

From SF'BS SOA



Adapted [from Ladd et al. (1980)].

Figure by MIT OCW.

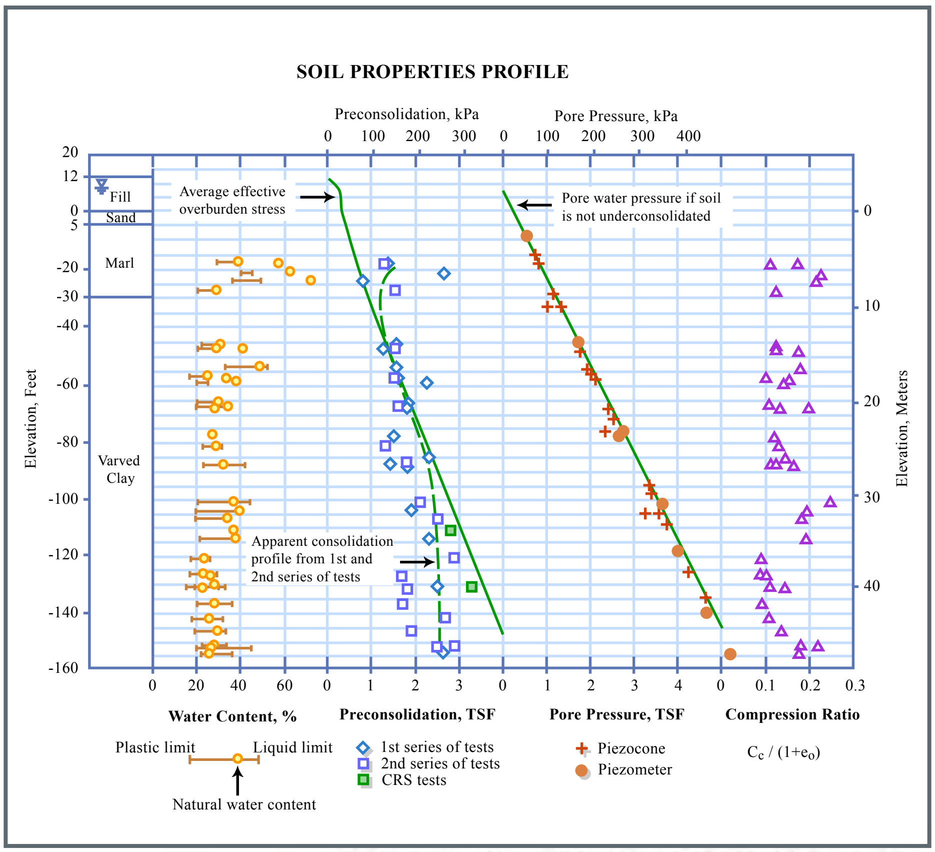


Figure by MIT OCW. Adapted from Steward, Lacy & Ladd : ASCE ρ'94 Conf.

Large shopping mall with floating foundation on thick deposit of varved clay in upper state NY

1) Initial oedometer → "underconsolidated" from bottom 70'
(altho. CPTU dissipation & piezometer → hydrostatic u)

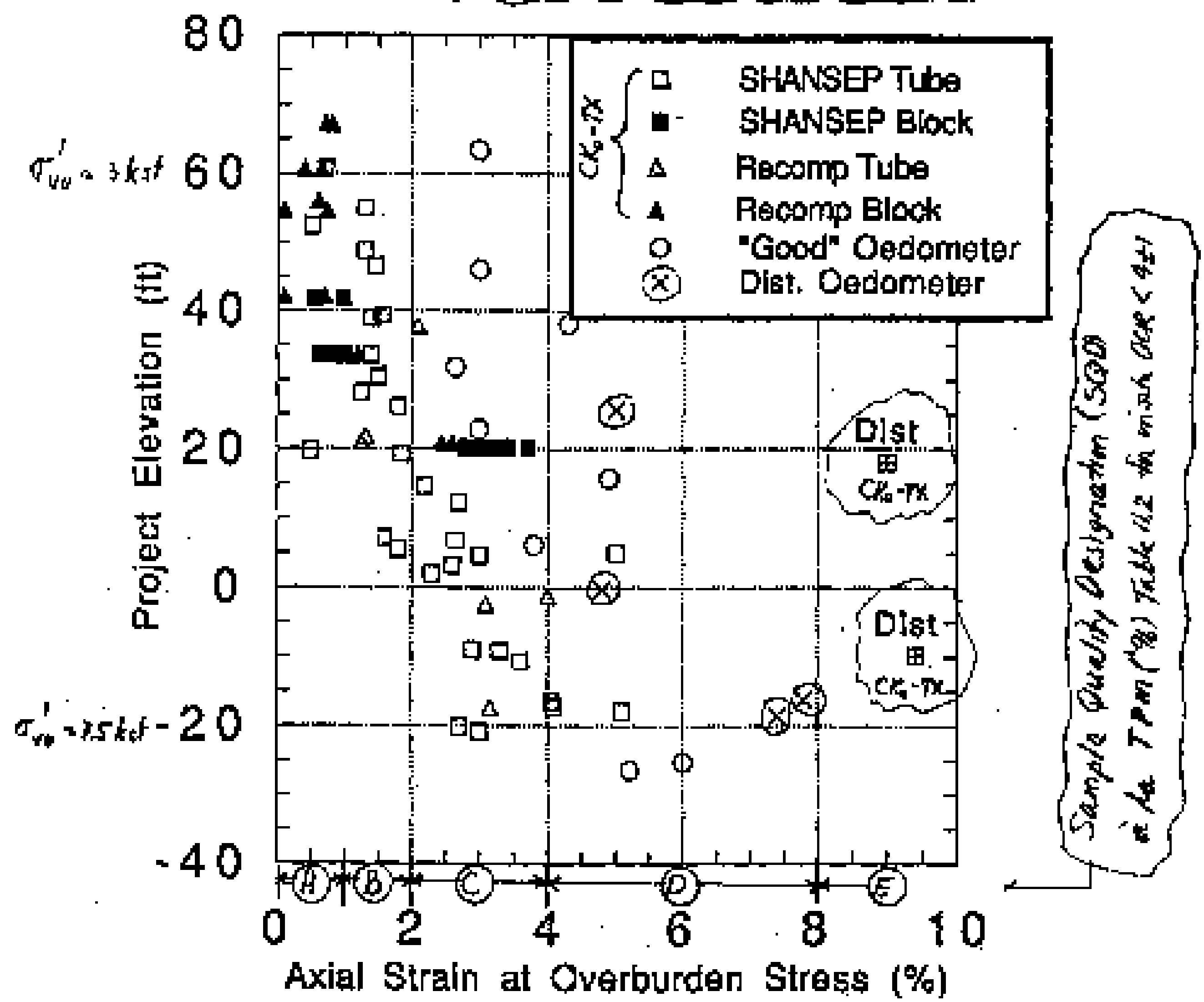
2) Subsequent CRSC at MIT on best quality clay from radiographs → 2 values of σ'_p slightly below hydrostatic σ'_{vo}

3/2/97
3/1/01

13d

CCL 3/5/92 1.322 I:Consol.:Part II

2/97
1/97 Note that CK_0-TX → smaller E_v than oedometer tests
Also block → smaller E_v than FP tube, except El. 20

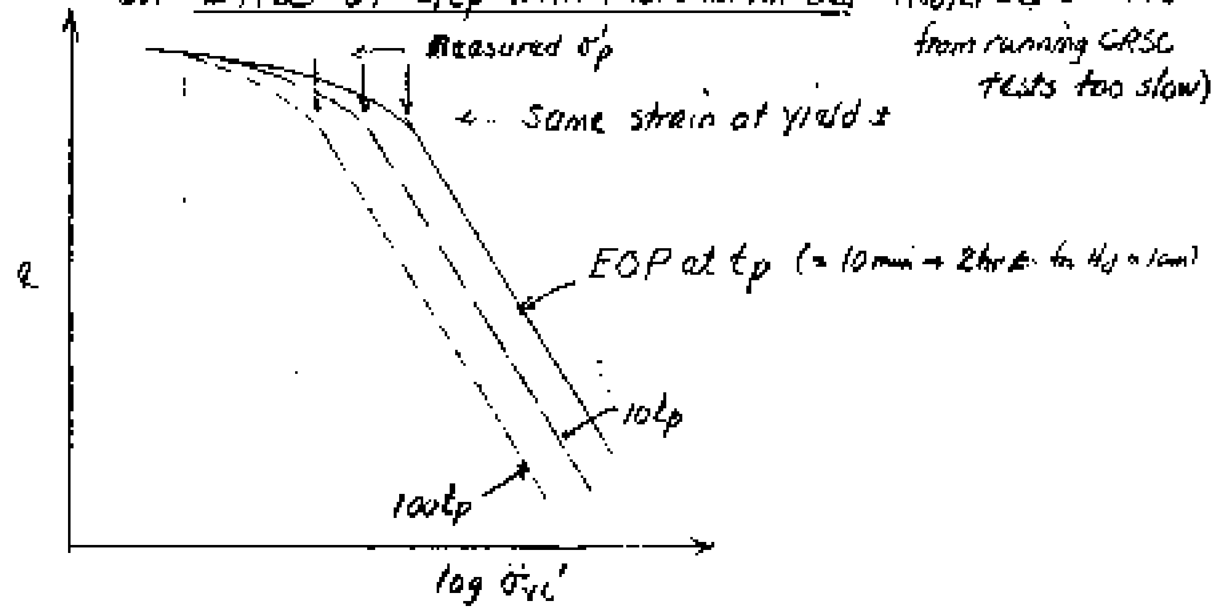


⊗ Dist. oedometer: mainly occurred during extrusion → new technique where used piano wire to cut bond between BBC & Shelby tube

Figure 5-14: South Boston Elevation vs. Axial Strain at Overburden Stress from CK_0 and Typical Oedometer Tests

8. EFFECT OF TIME AND EOP

8.1 Effect of t/t_p with Incremental Oed (Note: Get same results from running CRSC tests too slow)



- Obtain above via different tests with varying t or same test with data plotted varying t, or CRSC with varying $\dot{\varepsilon}_v$
- Not controversial concept for $t \geq t_p$

8.2 How to Obtain EOP From Incremental Tests

1) See Fig. 2-11 & Notes (p14a)

2) Typical differences σ'_p EOP vs 1 day ≈ 15±5%

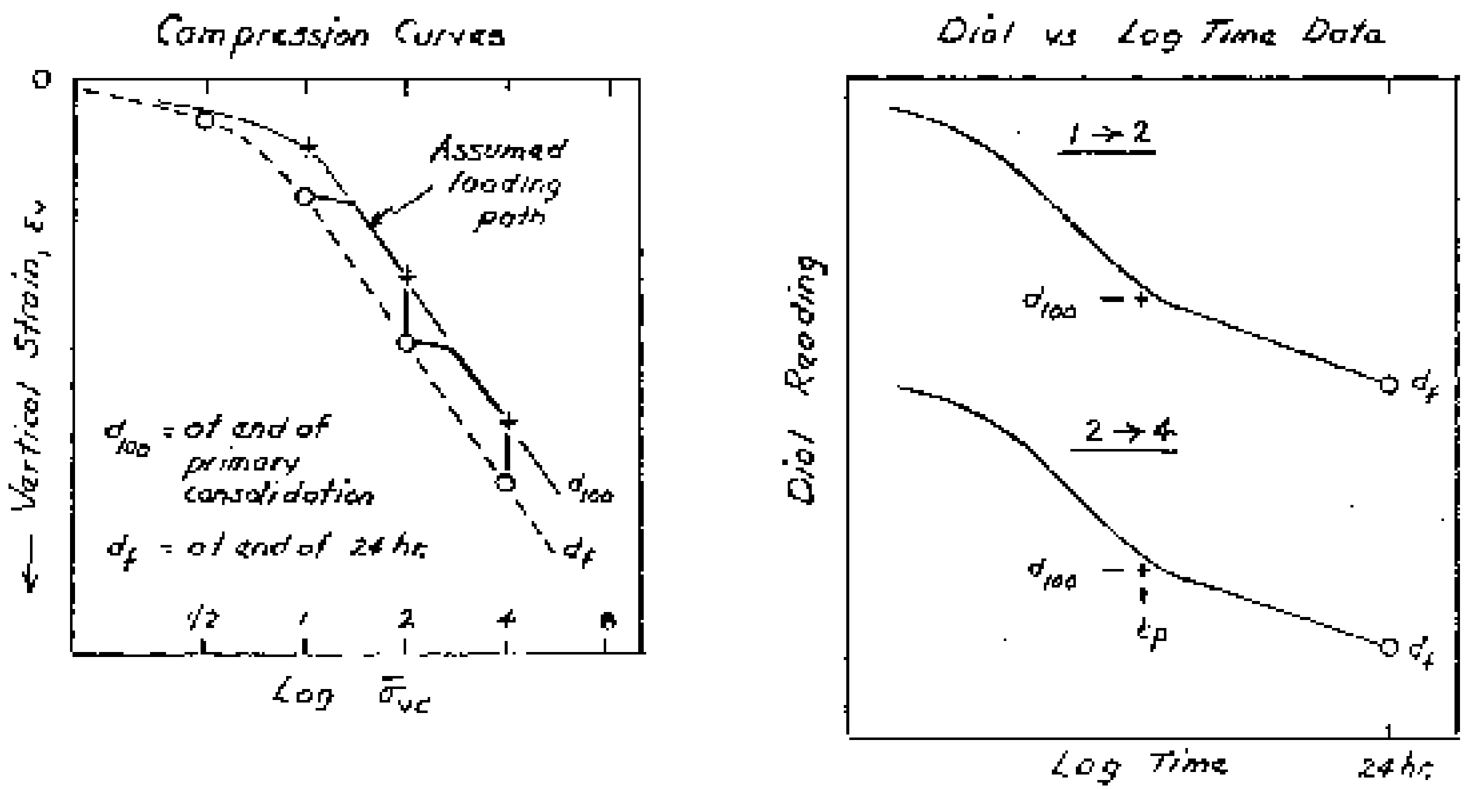
3) Std. practice (ASTM D2435-90 allows either t_p or constant t_c up to 24hr). Most use t_c = 24hr (except U of I / MIT)

4) Problems in practical application
   - t_p varies OC→NC
   - Hard to define low-LIR & near σ'_p (true with both $\sqrt{t}$ & log t methods to estimate d_{100} at t_p)

CCL recommendation:
- get typical NC t_p & use throughout, e.g. t ≈ 10 min. low c_v, t ≈ 2 hr high c_v

Very Important

CCL 3/10/79



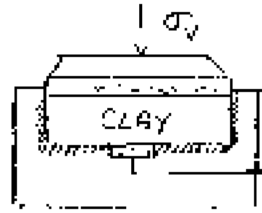
CORRECTION OF OEDOMETER TEST DATA TO OBTAIN END OF PRIMARY CONSOLIDATION COMPRESSION CURVE

3/89 3/90 4/92 4/97 3/01

ASTM D 4186-89

8.3 CRSC (Wahls et al + Wissa et al) ASCE, JSMFD, 97(10)

Fig 2-7 & Notes



PLUS BACKPRESSURE

$u_e \approx u_b < 5\text{-}10\%\ \sigma_v$

1) Principle & how operate

Linear Theory

$$\bar{\sigma}_v' = \sigma_v - \frac{2}{3} u_b$$

$$k = \frac{\dot{\varepsilon} H_d^2 \gamma_w}{2 u_b}$$

$$c_v = \frac{H_d^2}{2 u_b}\left(\frac{\Delta\sigma_v}{\Delta t}\right) = \frac{\dot{\varepsilon} H_d^2}{2 u_b m_v} = \frac{k}{m_v \gamma_w}$$

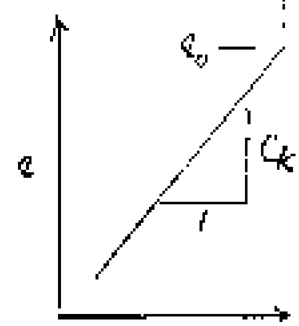
2) Effects of $\dot{\varepsilon}$ on measured σ_p' - see p16

Cover in Part III

5) $k = k_0 (10)^{\left[\frac{e - e_0}{C_k}\right]}$

4) Advantages & limitations

- Cont. data
- 1/10th time
- Too fast $\dot{\varepsilon} \rightarrow \sigma_p'$ too high
- No C_α
- Capital investment



$C_k = de/d\log k$

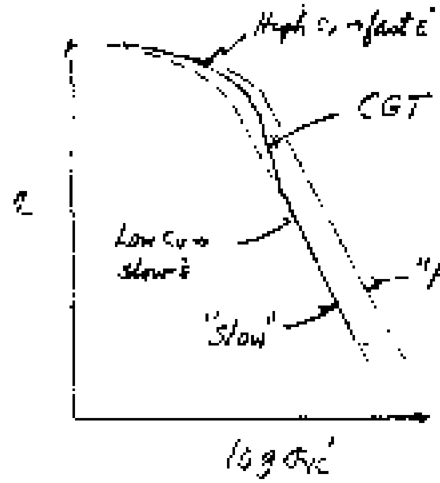
3) Mesri & Castro (3/87 JGE)

$$\dot{\varepsilon} = \frac{k_0}{2^{C_c/C_k}} \times \frac{\bar{\sigma}_p'}{H_d^2 \gamma_w} \times \frac{C_{\alpha e}}{C_c}$$

(NOTE: $\dot{\varepsilon}$ to obtain EOP with $u_b \times 1$ kPa ∴ too slow for CRSC tests to measure k_v & c_v)

8.4 CGT (Maintains constant $i = u_b/H_d$)

Why variable $\dot{\varepsilon}$ can → erroneous compression curve



8.3 Continued: 6) Discussion p16

- Most of data on high I_L - high S_t clays that are probably cemented
- Will data conclude that Champlain clays have cementation bonds → Yield envelope = $f(\dot{\varepsilon})$
- However, even more ordinary clays probably have a "structural viscosity" → $\sigma_p' = f(\dot{\varepsilon})$ at v. high rates

Cover in Part IV

Leroueil et al (1983) CGJ No4

TABLE 1. Geotechnical properties of clays Champlain Clays

Site	Number and symbol	Depth (m)	w (%)	w_L	I_P	I_L	S_t fall cone	C_u field vane (kPa)	σ'_{vo} (kPa)	$\sigma'_{p\,oed}$ (kPa)	Reference
Berthierville	1 ■	3.7	72	59	34	1.4	15	14	21	47	Samson et al. 1981 Morin et al. 1983
St-Césaire	2 +	4.2	89	68	42	1.5	22	25	55	80	Samson et al. 1981
St-Césaire	2 +	6.8	85	70	43	1.3	19	37	68	90	
Gloucester	3 ×	3.7	88	52	28	2.3	70	20	35	65	Samson et al. 1981 Leroueil et al. 1983
Gloucester	3 ×	4.1	76	53	29	1.8	35	20	38	67	
Gloucester	3 ×	7.5	93	53	29	2.4	88	25	58	87	
Varennes	4 ✳	5.9	62	65	39	0.9	28	60	64	216	Samson et al. 1981
Joliette	5 ★	6.7	65	41	19	2.3	108	29	40	110	Samson et al. 1981
Ste-Catherine	6 ●	3.8	86	60	35	1.8	30	18	20	60	Samson et al. 1981 Morin et al. 1983
Mascouche	7 ▽	3.8	65	55	30	1.3	52	70	34	270	Leahy 1980 Marchand 1982
St-Alban	8 △	3.9	60	40	18	2.1		13	25	55	Leroueil et al. 1978a Leahy 1980
Port Lennox	9 □	6.1	60	45	22	1.7	30	30	54	105	Leahy 1980 Paquin 1983
Louiseville	10 ○	9.2	75	70	27	1.1	22	45	58	160	Leahy 1980 Leblond 1981
Batiscan	11 ◇	7.3	60	43	21	2.7	65	25	60	88	Bouchard 1982
Other sites	●										Authors' files

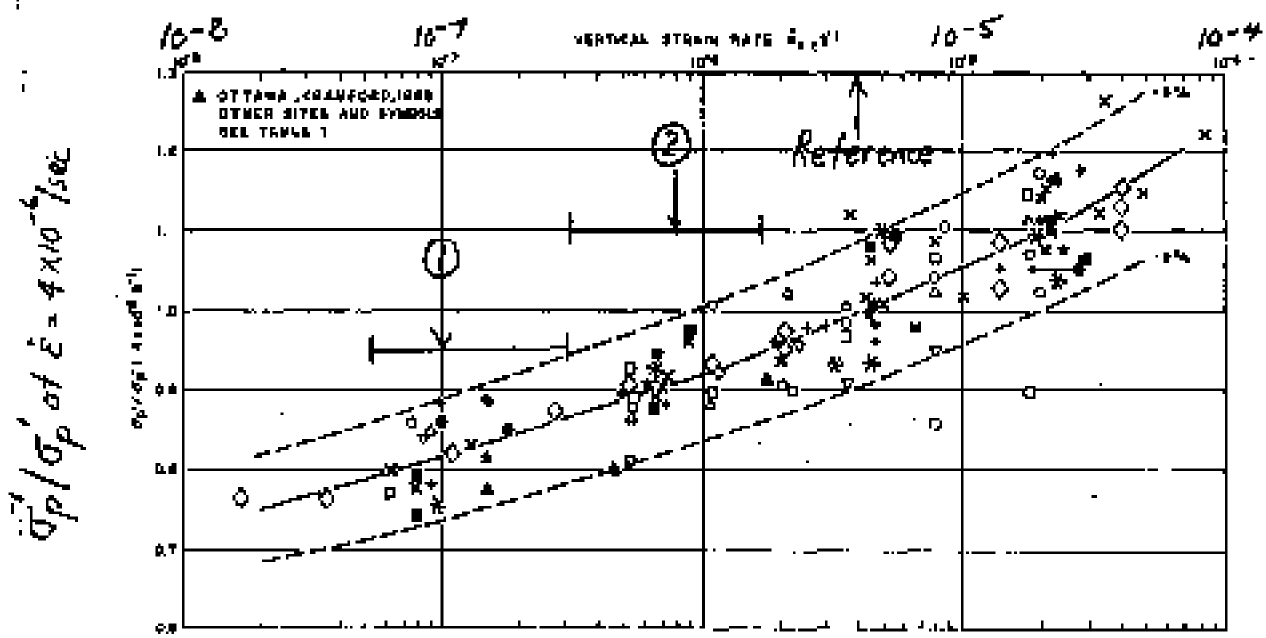


FIG. 7. Normalized preconsolidation pressure – strain rate relationship.

① Mean + range for $\dot{\varepsilon}$ typical oed. at t = 1 day

② " " " " " " " t = t_p or u_b = 0 in CRSC

3/89 2/97 3/2/99 slides

9. MISCELLANEOUS

9.1 Temperature [Also Baldi, et al. (1988), CGJ 25(4), 807-825]

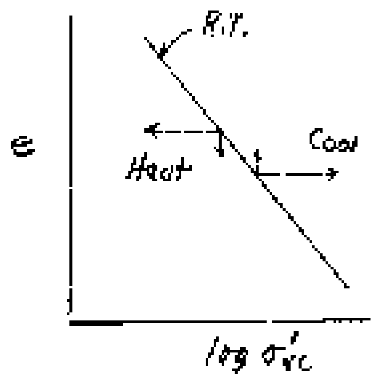
1) Results for Arctic Silt (see p17a)

2) Other results (see p17b & c)

Typical in situ T ≈ 10°C NC
≈ −2°C Arctic
≈ 5°C at great depth G. Mexico

$\frac{[\sigma'_p(T) - \sigma'_p(20°)]}{\sigma'_p(20°)} \longrightarrow \frac{\%\ \text{Decr.}\ \sigma'_p}{\Delta T = +10°C} \approx 5\text{-}10\%$

3) ΔT during test

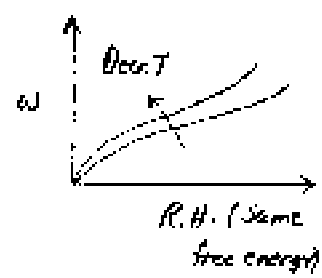


$\Delta\sigma'_v >> \Delta e$ – why?

Not Δ(R-A)

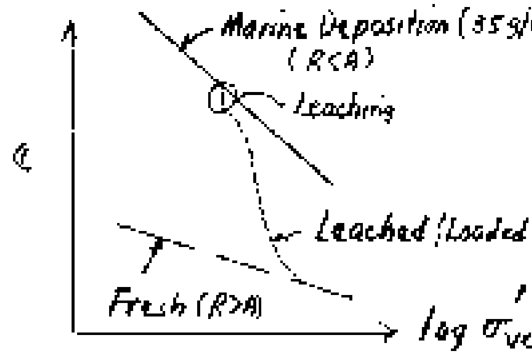
· Contact stress

Incr. water adsorption with decr. T → incr. $\bar{\bar{\sigma}}_c$?



9.2 Pore Fluid (Covered Part A, II)

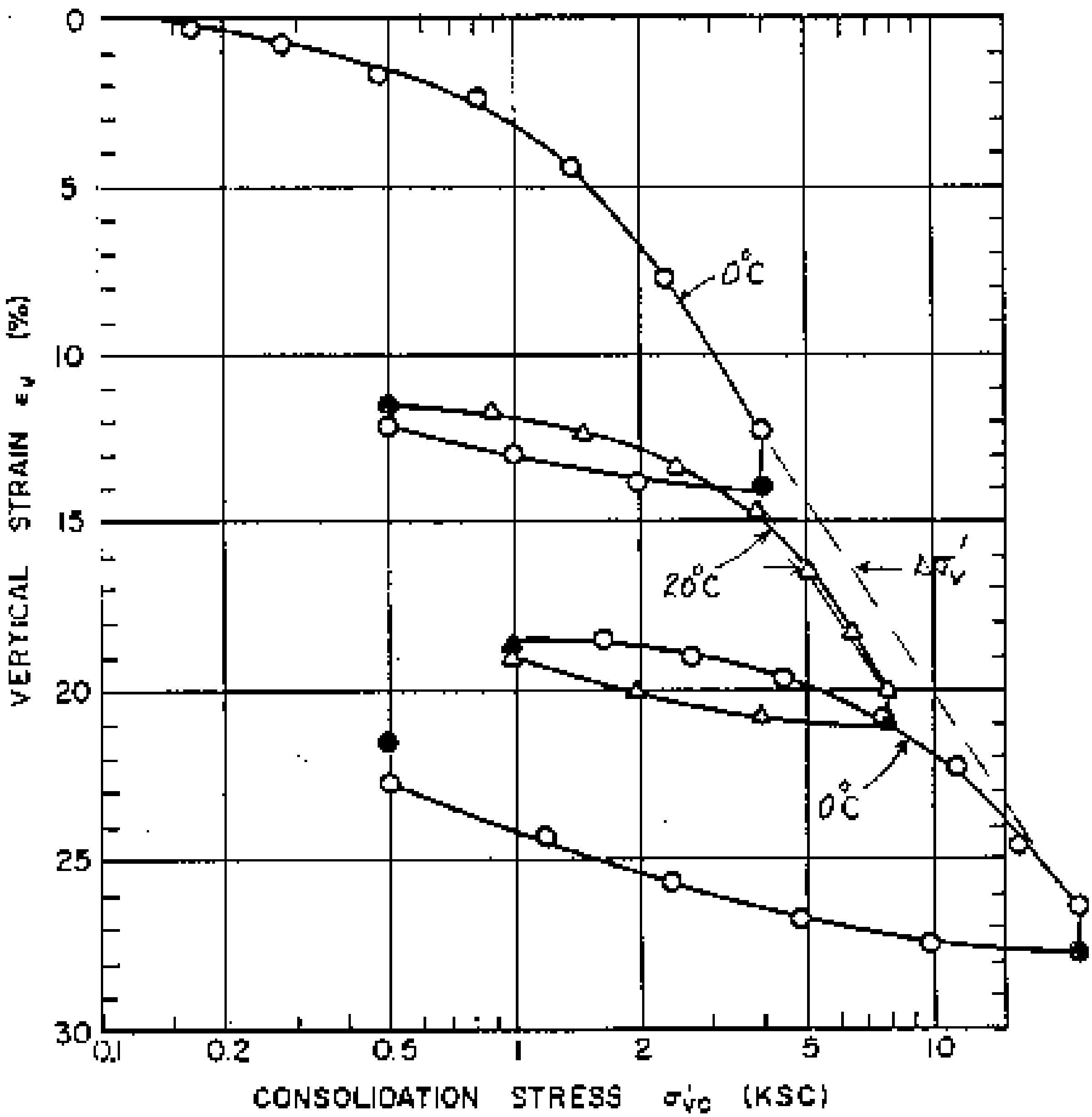
1) Schematic – marine illitic clays



2) Other potential effects

$\bar{\bar{\sigma}}_c$ · Kaolinite: anything that changes + edge charge

R-A · Smectite: amt. swelling = f(salt conc., cation valence & dielectric cond.)



Sample No.	MP - B2 - S7	W_N (%) 52.1	Estimated		
Depth	16.3'	W_L (%) 66.3	σ'_{vo} 0.402	σ'_p 1.7 KSC	
Soil type	Gray Clayey Silt (MH)	W_P (%) 31.3	CR 0.168	RR 0.027	
		I_P (%) 35.0	G_s 2.78	e_o 1.494	
				S (%) 97.0	

○ At t_p

● At t_f

Remarks — Corrected for apparatus compressibility.

COMPRESSION CURVE TEST NO. OED - B2 S7 TC - 1

Figure 4-14 Compression Curve for Temperature Controlled Oedometer Test (OED-B2S7TC)

MIT Center Scientific Excellence in Offshore Engr. SM Thesis . Yin (1985)

ARCTIC SILT Harrison Bay, Alaska

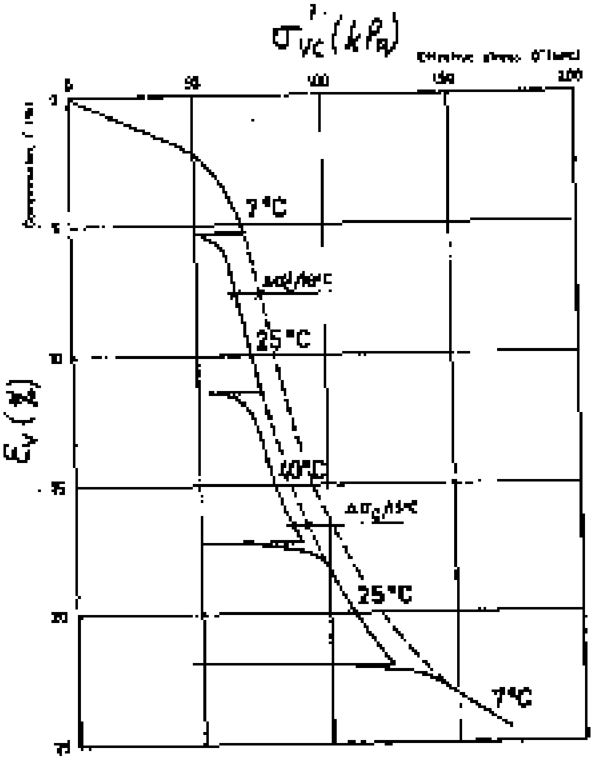


FIG. 5—CRS test with varying temperature. Clay from Bäckebol.

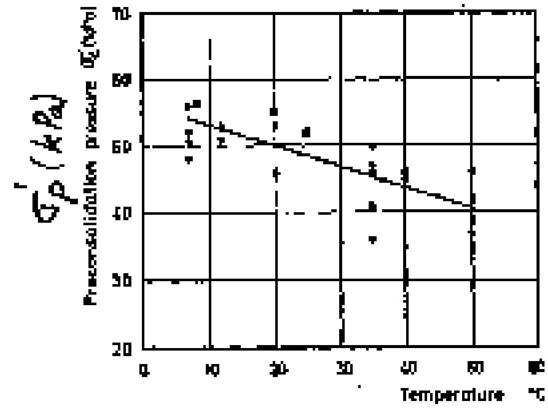
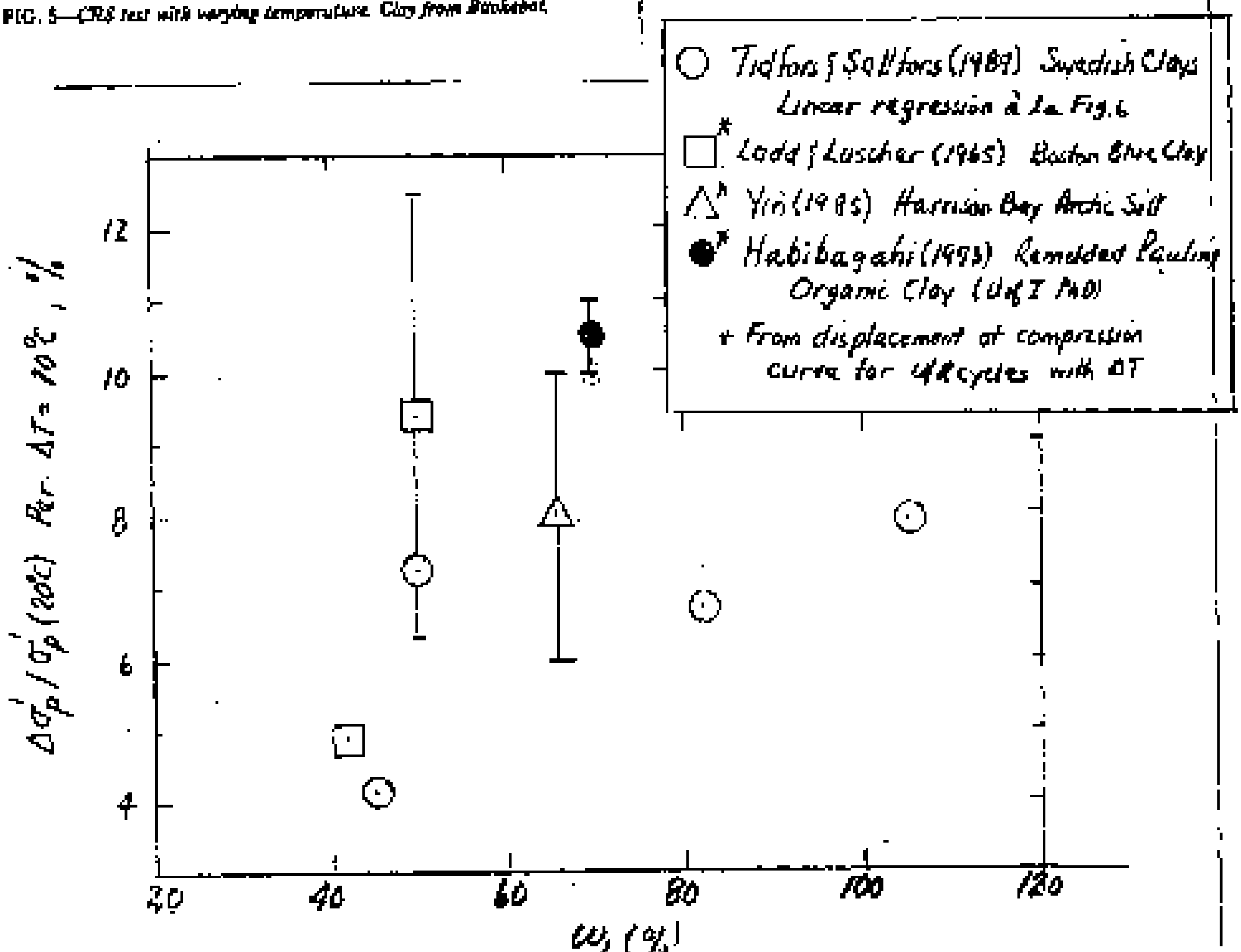


FIG. 6—Preconsolidation pressure as a function of test temperature for specimens taken at 7 m depth at Bäckebol. Full line shows results of linear regression analysis.

Tidfors & Sollfors (1989)
GTJ, ASTM 12(1)

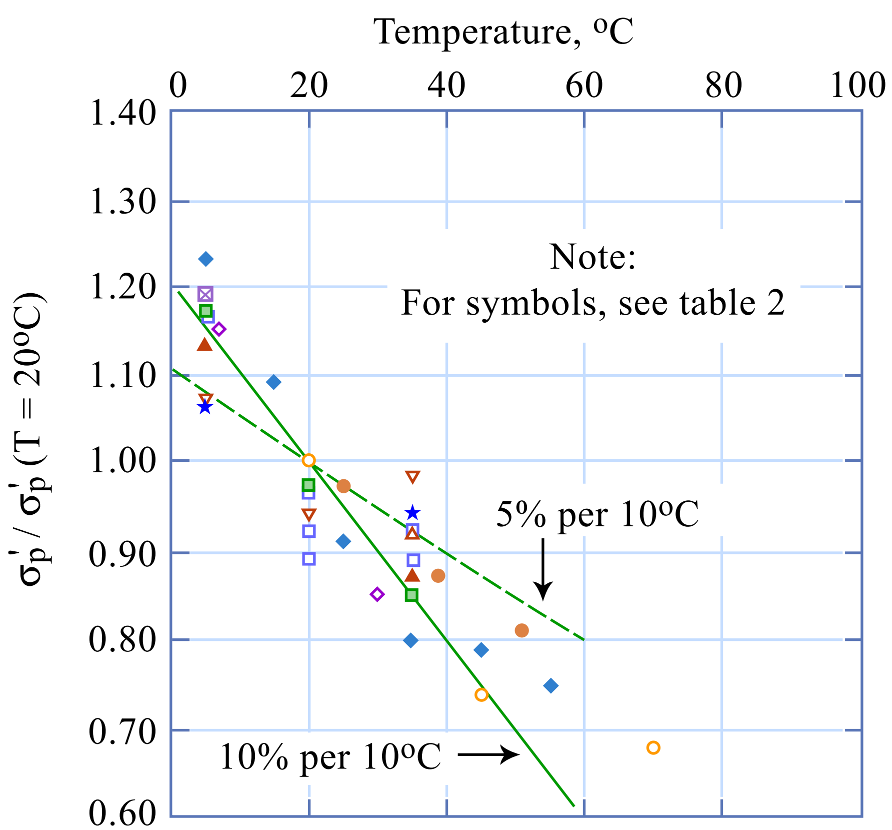
T Data for 4 Swedish clays



Boudali, Leroueil & Murthy (1994) "Viscous behaviour of natural clays"
13th ICSMFE, New Delhi, Vol. 1, 411-416

Site	Depth (m)	I_p (%)	Type of oedometer test	σ'_p (20°C) (kPa)	Range of temperature, °C	Reference	Symbol used in Fig. 8
Berthierville	3.15-3.50	25	CRS, $\dot{\varepsilon}_v$=1.0 x $10^{-6}s^{-1}$	68.5	5-35	Present study	□
"	"	"	CRS, $\dot{\varepsilon}_v$=1.5 x $10^{-6}s^{-1}$	58	"	"	■
"	"	"	CRS, $\dot{\varepsilon}_v$=1.6 x $10^{-7}s^{-1}$	62.5	"	"	■
Louiseville	8.70-8.76	39	CRS, $\dot{\varepsilon}_v$=1.5 x $10^{-6}s^{-1}$	175	"	"	▲
"	8.76-8.82	39	"	190	"	"	▲
St-Jean-Vianney	4.44-4.54	14	"	980	"	"	★
"	5.60-5.72	14	"	1060	"	"	★
Argile noire	--	32	Conventional	--	20-95	Despax, 1975	○
Illite	--	--	Isotropic consolidation	--	25-51	Campanella and Mitchell, 1968	●
Bäckebol	3.0-7.0	60	CRS, 0.0024mm/min	54	7-30	Tidfors and Sällfors, 1989	◇
Luleå	4.0	80	Conventional	50	5-55	Eriksson, 1989	◆

Table 2. Clays considered in Fig. 8



Variation of the Normalized Preconsolidation Pressure ($\sigma'_p/\sigma'_p(20°C)$) with Temperature

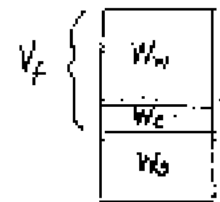
Figure by MIT OCW.

3/88 3/90 4/97 3/00 4/…

3) Check effect of Δ salt conc. on AL, esp w_L

4) Effect of salt on measured w (C = salt)

· Measured $w = \dfrac{W_w}{W_s + W_c}$



· Salt G_c = specific gravity (≈ 2.3)

C = conc. g/cc of solution

· Fluid $\gamma_f = \gamma_w + C\left(\dfrac{G_c - 1}{G_c}\right)$

· Corrected $w' = \dfrac{V_f \cdot \gamma_w}{W_s} = \dfrac{w}{1 - C\left(\frac{1}{\gamma_c} + w\right)}$ = weight of voids if all pure H_2O

→ consistent $e = V_v/V_s$ & $G_s w' = Se$

{ GSL C = 0.150 g/cc $w = 51.5\% \rightarrow w' = 60\%$ }

· Potentially large effect on AL - Plasticity Chart - empirical correlations, but constant I_L (see p18a)

8.3 Side Friction Covered 1.37

10. PRACTICAL PROBLEM (Assuming Pcf - Pood)

· See p19 for problem - Class Discussion on _ _ __

· For in situ consider : FVT
CPTU
DMT

· 2* sheets w/ answers + rationale

2/97
2/22/98 Supplement to Effect of Salt Conc. on Water Content (for $\gamma_w = 1.00 g/cc$)

Definitions

$w = \dfrac{W_w}{W_s + W_c}$

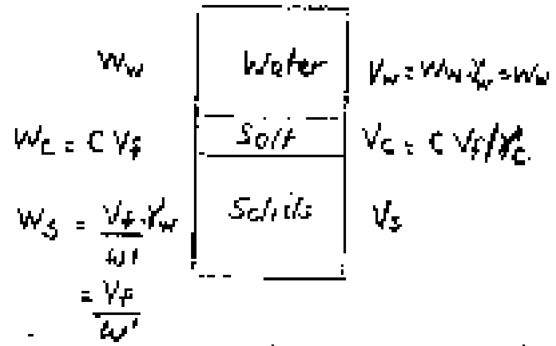
$w' = \dfrac{V_f \cdot \gamma_w}{W_s}$

$C = \dfrac{W_c}{V_f}$

$G_c = \dfrac{\gamma_c}{\gamma_w}$

$\gamma_c = \dfrac{W_c}{V_c}$ $\quad \dfrac{W_c}{W_s} = \dfrac{C V_f}{V_f} w' = C w'$

Phase Relations



$V_f = W_w + \dfrac{C V_f}{\gamma_c} = \dfrac{W_w}{1 - C/\gamma_c}$

$\gamma_f = \gamma_w + C\left(\dfrac{G_c - 1}{G_c}\right)$

Derivation

$$w' = \frac{V_f \gamma_w}{W_s} = \frac{W_w}{(1 - C/\gamma_c) W_s} = \frac{w(W_s + W_c)}{(1 - C/\gamma_c) W_s} = \frac{w(1 + W_c/W_s)}{(1 - C/\gamma_c)} = \frac{w(1 + C w')}{(1 - C/\gamma_c)}$$

$$\rightarrow w' = w \Big/ \left[1 - C\left(\frac{1}{\gamma_c} + w\right)\right]$$

Application when Add/Substract water at Constant W_c

Change from w_0 to w_1, use $C_1 = C_0(w_0'/w_1') \approx C_0(w_0/w_1)$

Example ($G_c = 2.33 = \gamma_c$ for $\gamma_w = 1.00$)

1) Initial condition: $C_0 = 0.150 g/cc$ $\quad w_0 = 51.5\% \rightarrow w_0' = \dfrac{51.5}{1 - 0.150(0.429 + 0.515)} = 60.0\%$
(at w_N)

2) Increase to $w_1 = 100\%$: $C_1 \approx 0.150\left(\dfrac{51.5}{100}\right) = 0.0773 \rightarrow w_1' = \dfrac{100}{1 - 0.0773(0.429 + 1.00)} = 112.4\%$ vs 112.5%
(at w_L)

3) Decrease to $w_2 = 40\%$: $C_2 \approx 0.150\left(\dfrac{51.5}{40}\right) = 0.193 \rightarrow w_2' = \dfrac{40}{1 - 0.193(0.429 + 0.40)} = 47.6\%$ vs 47.5%
(at w_P)

4) Plasticity Chart

Measured $w \rightarrow w_L = 100\%$, $I_p = 100 - 40 = 60\%$ → on A-line

Corrected $w' \rightarrow w_L = 112.4\%$, $I_p = 112.4 - 47.6 = 64.8\%$ → below A-line

5) Liquidity Index

Measured $w \rightarrow I_L = (51.5 - 40.0)/60 = 0.192$
Corrected $w' \rightarrow I_L = (60.0 - 47.6)/64.8 = 0.192$
} same

3/16/97 3/01

PROBLEM SOILS

Sheets

1. Highly Structured and Sensitive — S1-S3

- S1 Summary of material covered in Consolid. V
- S2 Laval field data
- S3 DM7.1 I_L vs $\sigma_p' = f(S_t)$

2. Peats and Highly Organic Soils — P1-P5

- P1, 2 "text"
- P3, 4, 4a Correlations
- P5 Fundamentals of decrease in σ_{vc}' due to settlement

3. Collapsing and Expansive Soils — C/E1-C/E4

- C/E1 Overview
- C/E1, 2 Collapsing soils
- C/E3, 4 Expansive clays

ES1-4 Effects of changes in climate on behavior of expansive soils

ES5 Fdn. design on " "

Note: Limitations of double oedometer testing and influence of stress level on collapse/swell behavior will be covered under Part G (Compacted Clays)

4. Tropical Residual Soils — R1-R6

- R1 "text"
- R2-4 Backup information
- R5, 6 Sowers (1994)

Note: CCL has very little experience with residual soils

5. Varved Clays — V1-V12

- V1, 2 "text"
- V3-12 Backup information

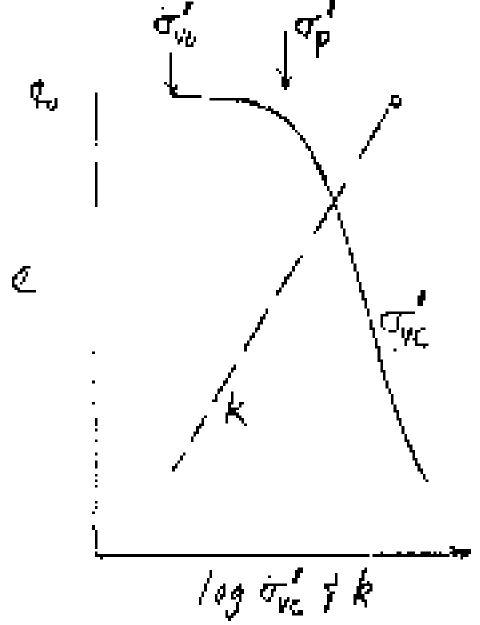
6. References (Outdated) — Ref1

3/96 3/99 3/01

Highly Structured & Sensitive Clays

V. high I_L vs σ_p'
Often cemented & leached
See 53

Mesri - Hypothesis A



ILLICON

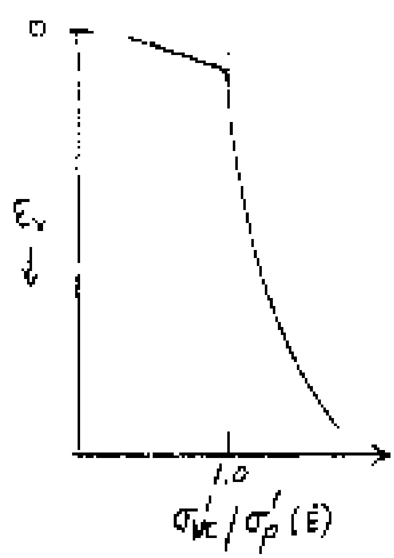
- Assumes unique EOP → insitu = lab
- Input e vs log σ_{vc}' & log k

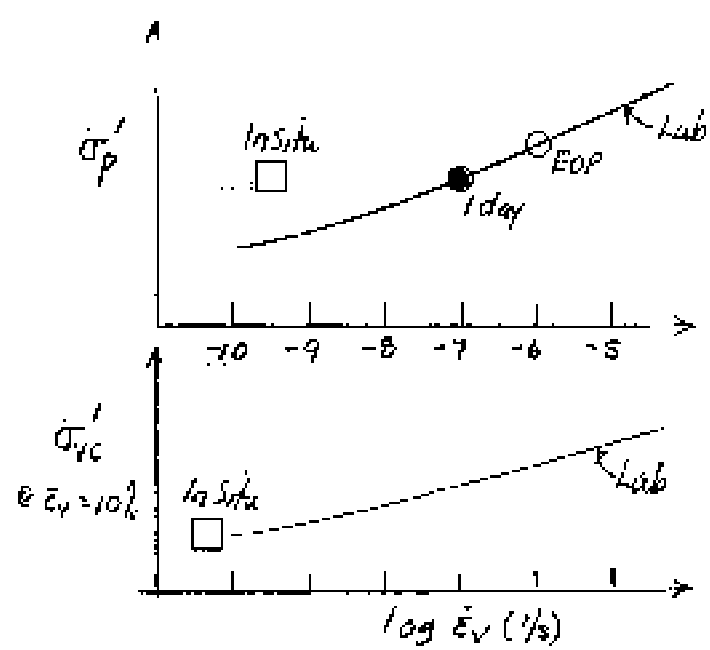
NOTE: Either case, often get CR → 1 ± 0.5

Laval - Hypothesis B

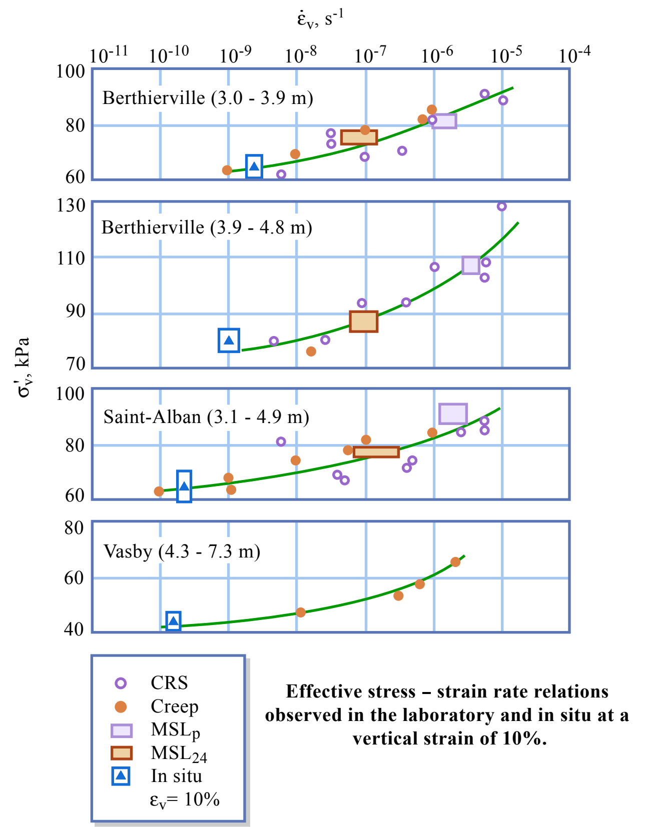
Leroueil (1988) CGJ 25(1)

- Lab → unique σ_{vc}' - ε_v - $\dot{\varepsilon}_v$
- Insitu → ε_v > lab EOP See 52

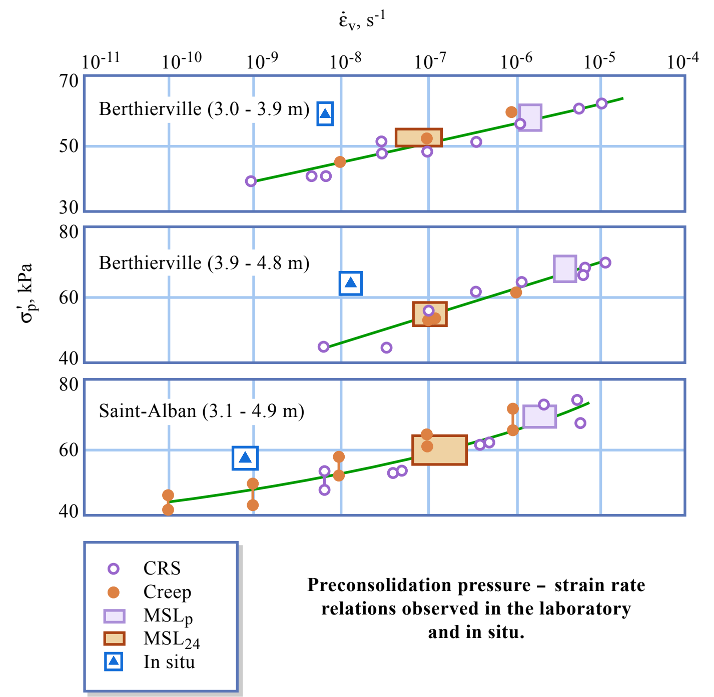




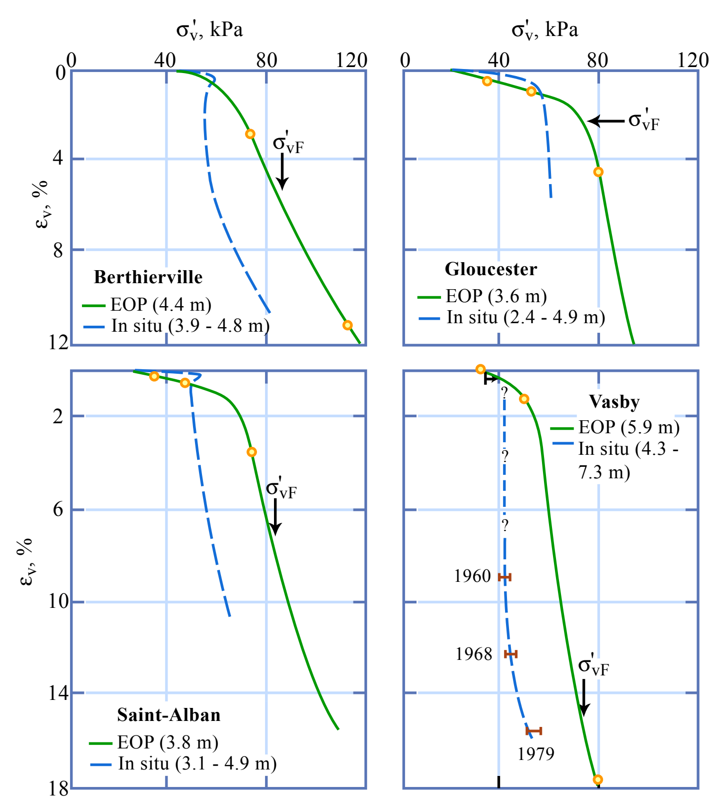
CCL would use Hyp. B for Champlain clays



Effective stress – strain rate relations observed in the laboratory and in situ at a vertical strain of 10%.

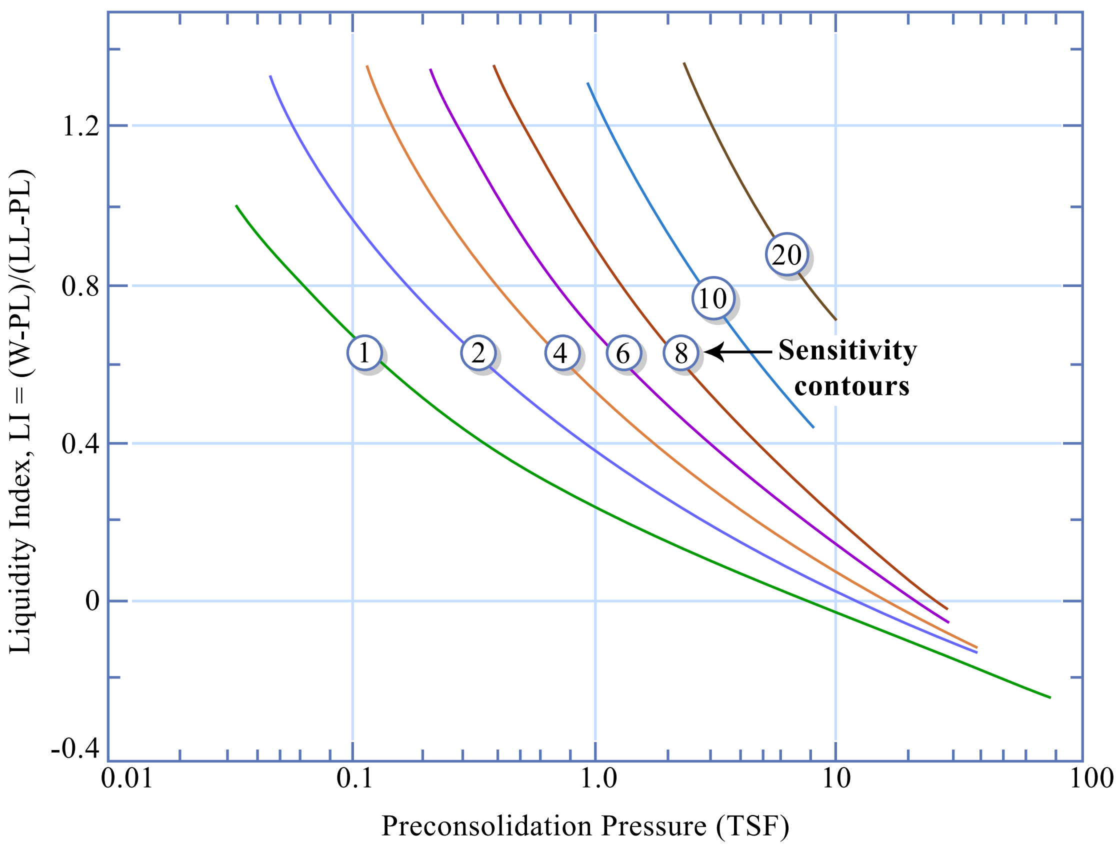


Preconsolidation pressure – strain rate relations observed in the laboratory and in situ.



Comparison of in situ stress-strain curves & laboratory end-of-primary compression curves

(after Leroueil *et al.* 1988b).



Preconsolidation Pressure vs. Liquidity Index

Figure by MIT OCW.

Adapted from: NAVFAC DM 7.1 Soil Mechanics May 1982

1) CCL has no idea of the reliability of this correlation

2) Most Champlain clays plot above this correlation, e.g. Lefebvre et al (1987) JGE, 113(5), 476-489 for 5 clays

Clay	I_L	σ_p' (bar)	S_t
GB #12	2.85	1.1	>300
" #39	2.85	1.9	500
NBR B1	1.8	1.45	100
" B6	2.5	1.75	450
St. J.V.	1.4	9.4	100

CCL 4/1/89 1.322 | PEATS (PL)

3/93 3/96 3/99 3/01

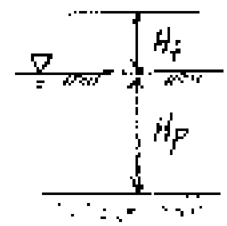
1. Introduction

- Characterized by: Low σ_p' & s_u
  (Often) Low E_u & highly creep susceptible
  High compressibility, CR
  $C_\alpha / CR \approx 0.07 \pm 0.02$ (P4a, Table 1 & Fig 10)

- Example - Typical NC deposit

$\rho > H_f$ for $H_f \leq \frac{1}{2} H_p$

e.g. 20' peat, 5' fill → ρ below WT



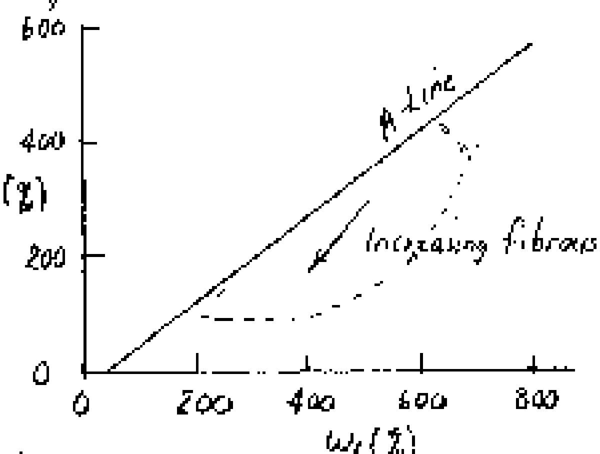
2. Classification & Index Properties

1) Types (Muskeg Engr. Handbook, 1969)

Fine Fibrous (grassy)	↑ w_N	400 - 1500%
Coarse Fibrous (woody)		
Amorphous - granular		200 - 600%

2) Plasticity Chart

Difficult to obtain w_p



3) Correlations with w_N (P3)

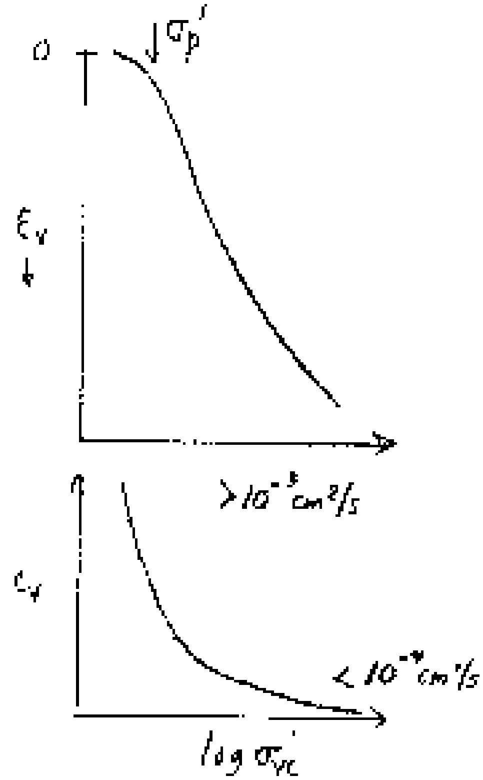
- Increasing w_N corresponds with increasing OM - best via chemical analysis
  - burn at 440 or 750°C (ASTM D 2974) → higher values

- Increasing OM → decreasing G_s

- Typical unit weights

w_N(%)	γ_t(TCM)	σ_{v0}' at $z = 5$ m
300	1.15	0.75 TSM = 155 psf = 7.5 kPa
1000	1.03	0.15 " = 30 psf = 1.5 kPa

3. Consolidation (1-D): General Behavior



Compressibility

- Often low σ_p'
- See P4 for w_N vs CR → 0.5 ± 0.1 (top) (Also P4a, Fig. 1) typical w_N = 200-1000%
- Usually S-shaped

Coefficient of Consolidation (DM-7 NG)

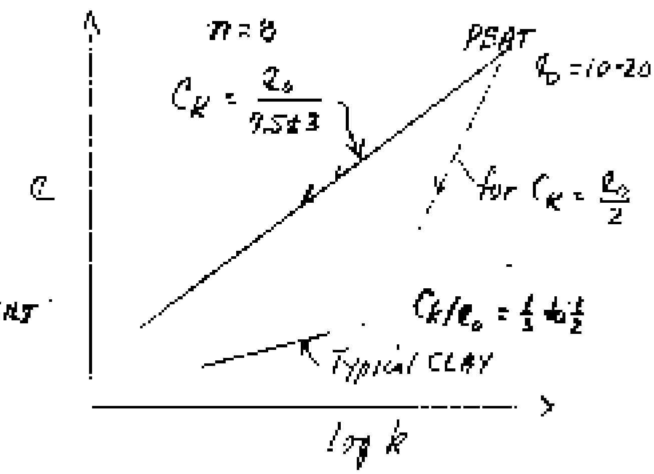
- c_v decreases substantially (often 1-2 orders of magnitude) (see P4 bottom)

Coefficient of Permeability

Lefebvre et al. (1984)

* k decreases more rapidly with decreasing e than for inorganic clay

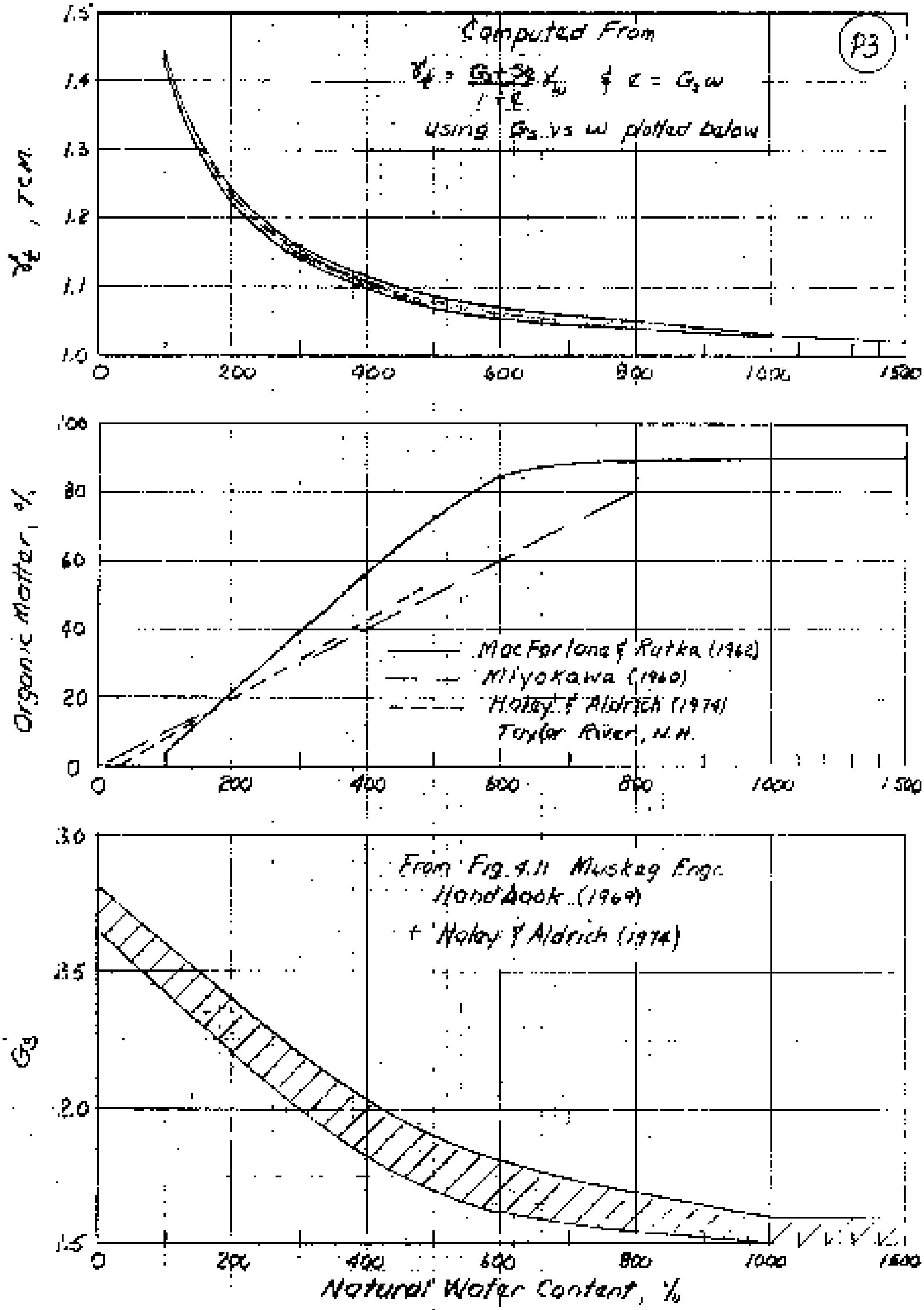
e ≈ 15 k = 10^{-5} cm/s non-plastic SILT
≈ 3 " 10^{-8} " CH CLAY



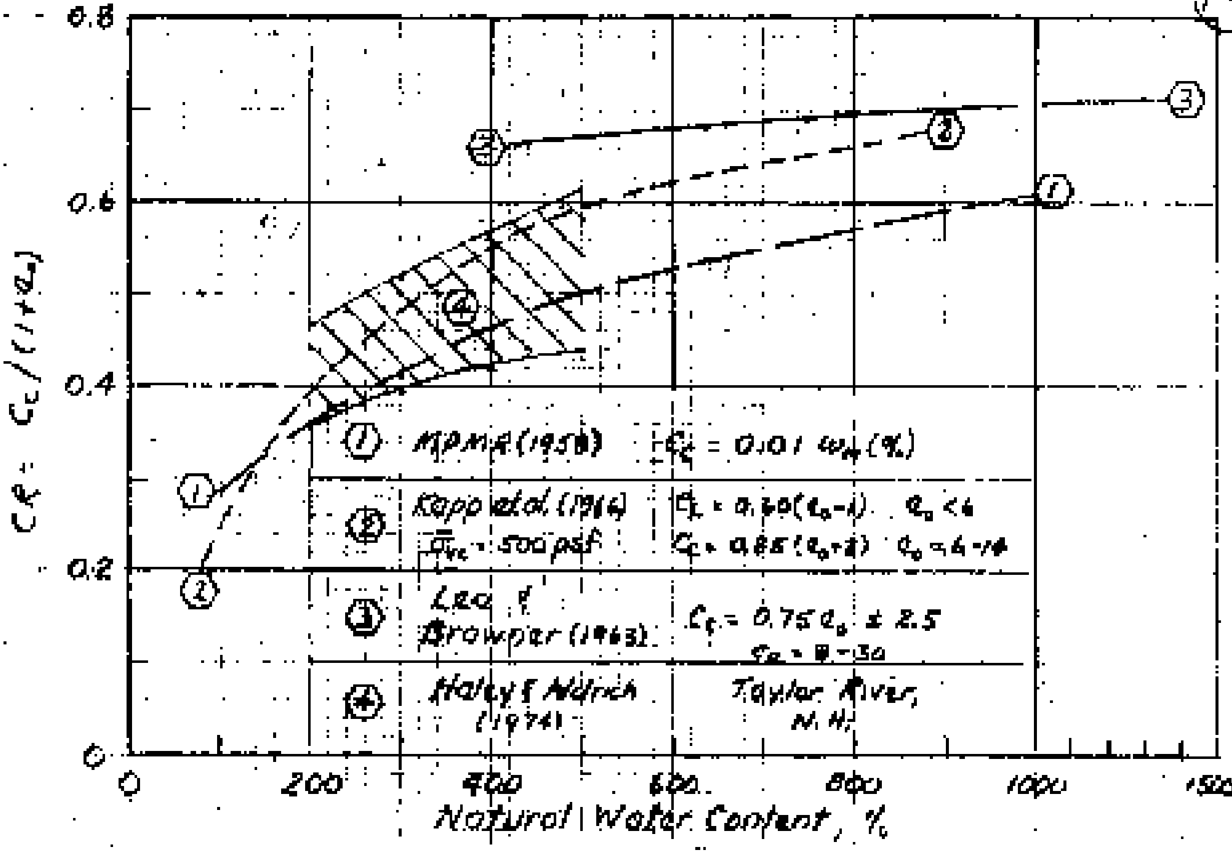
4. Estimating Field Settlements See P4a for data on e_0 vs w_N, C_k vs C_c & C_α/C_c

1) V. rapid consolidation at very low σ_{vc}' ∴ ρ_s can be very important
2) Major jobs warrant non-linear, finite strain (due to large ΔH_d) (input $e - \log \sigma_{vc}' - \log k$)
3) Simplified approach $t_f = t_\ell \left(\frac{H_f}{H_\ell}\right)^{n \approx 1.3 \pm 0.2}$ (vs. Terzaghi n = 2)
4) See P5 for reduction in σ_{vc}' due to 1-D settlement
5) WATCH OUT FOR LARGE LATERAL DEFORMATIONS (Taylor River)

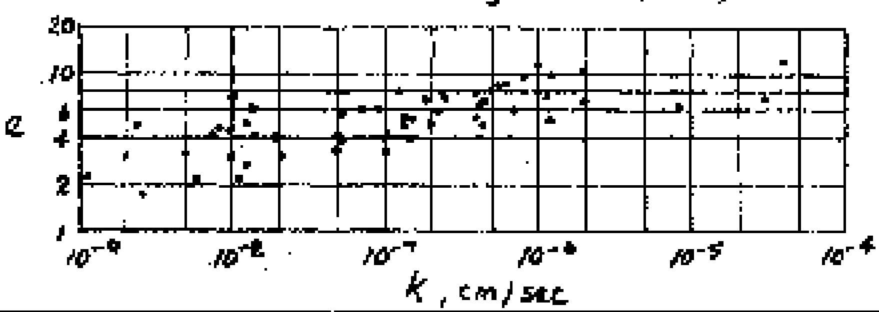
CCL 3/74

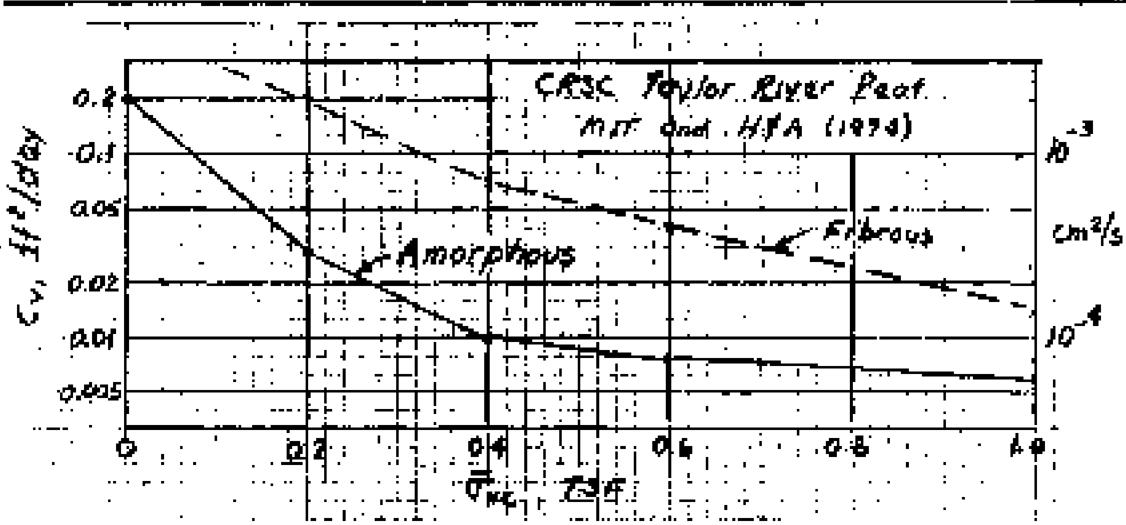


INDEX PROPERTIES OF PEAT M.I.T. 1.322
3/74



From L. Casagrande (1966)

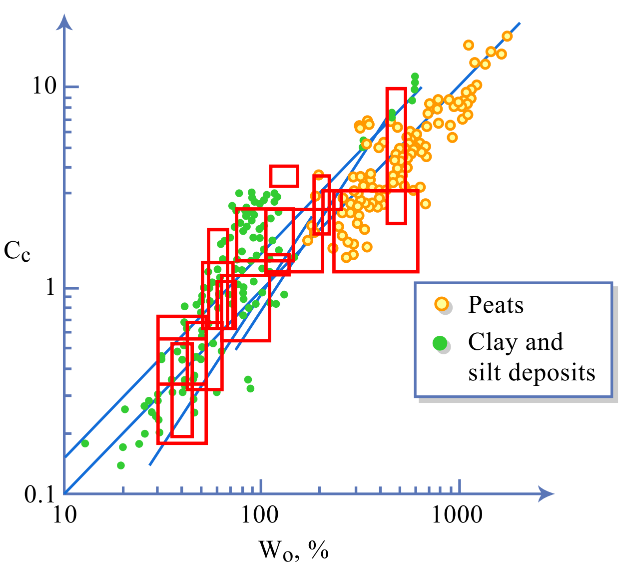




ENGINEERING PROPERTIES OF PEAT

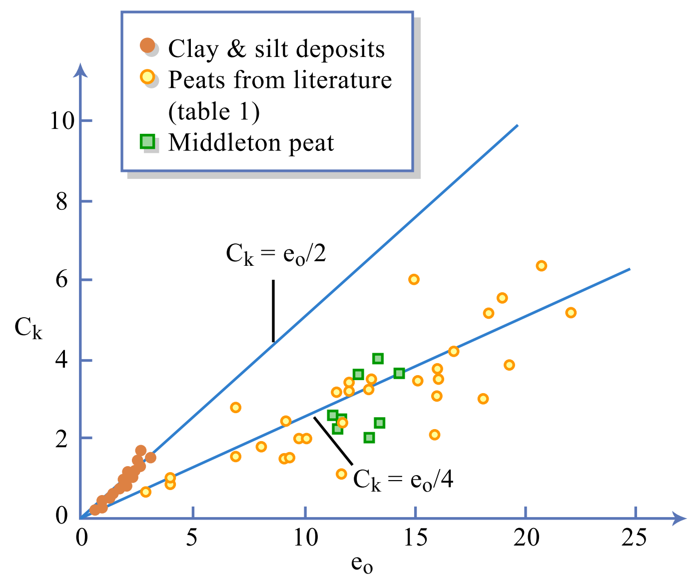
TABLE 1. Values of Natural Water Content, w_o, Initial Vertical Coefficient of Permeability, k_{vo}, and C_α/C_c for Peat Deposits

Peat (1)	w_o % (2)	k_{vo} m/s (3)	C_α/C_c (4)	Reference (5)
Fibrous peat	850	4×10^{-6}	0.06–0.10	Hanrahan (1954)
Peat	520	—	0.061–0.078	Lewis (1956)
Amorphous and fibrous peat	500–1,500	$10^{-7}–10^{-6}$	0.035–0.083	Lea and Brawner (1963)
Canadian muskeg	200–600	10^{-5}	0.09–0.10	Adams (1965)
Amorphous to fibrous peat	705	—	0.073–0.091	Keene and Zawodniak (1968)
Peat	400–750	10^{-6}	0.075–0.085	Weber (1969)
Fibrous peat	605–1,290	10^{-6}	0.052–0.072	Samson and LaRochell (1972)
Fibrous peat	613–886	$10^{-6}–10^{-5}$	0.06–0.085	Berry and Vickers (1975)
Amorphous to fibrous peat	600	10^{-6}	0.042–0.083	Dhowian and Edil (1981)
Fibrous peat	660–1,590	$5 \times 10^{-7}–5 \times 10^{-6}$	0.06	Lefebvre et al. (1984)
Dutch peat	370	—	0.06	Den Haan (1994)
Fibrous peat	610–850	$6 \times 10^{-8}–10^{-7}$	0.052	Present study (1997)



Values of natural water content and compression index for peats as compared to those of soft clay and silt deposits.

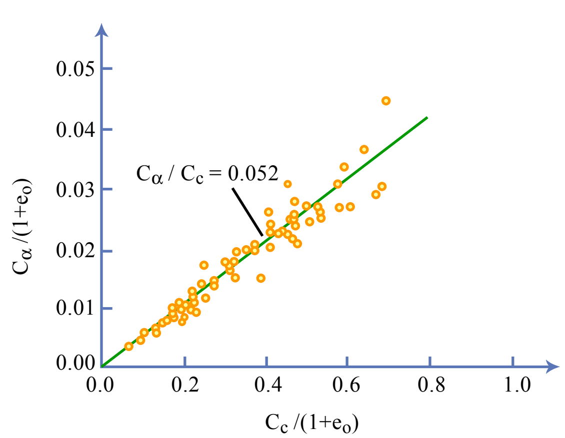
Figure by MIT OCW.



Values of C_k for peats as compared to those of soft clay and silt deposits.

Figure by MIT OCW.

Adapted from: **Mesri and Rokhsar (1974); Mesri et al. (1994); Watanabe (1977)**



Relationship between secondary compression index and compression index for Middleton peat.

Most comprehensive study of secondary compression behavior of peat (NC / OC), plus other "goodies"

Figure by MIT OCW. Adapted from: Mesri et al (1997) JGGE 123(5)

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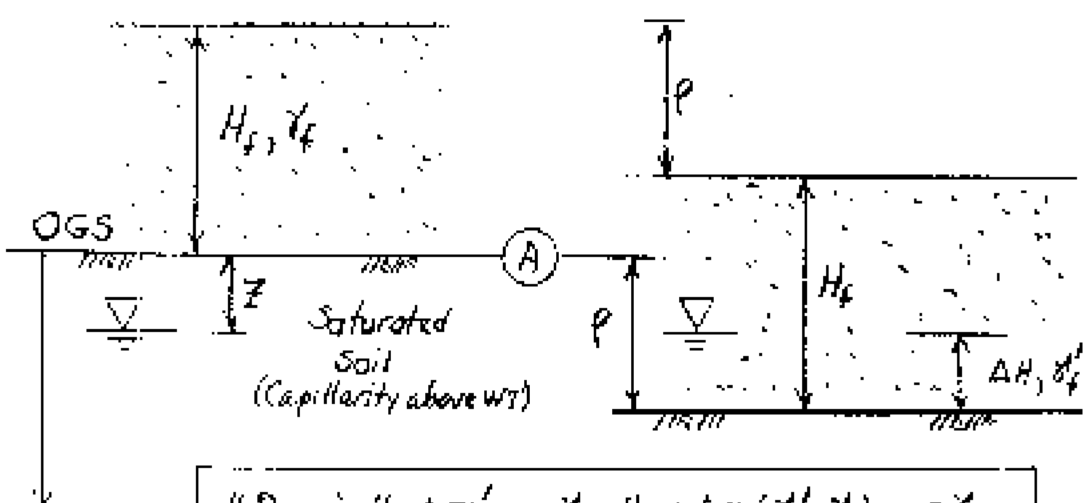
(P5)

Influence of Large Settlement on Consolidation Stress

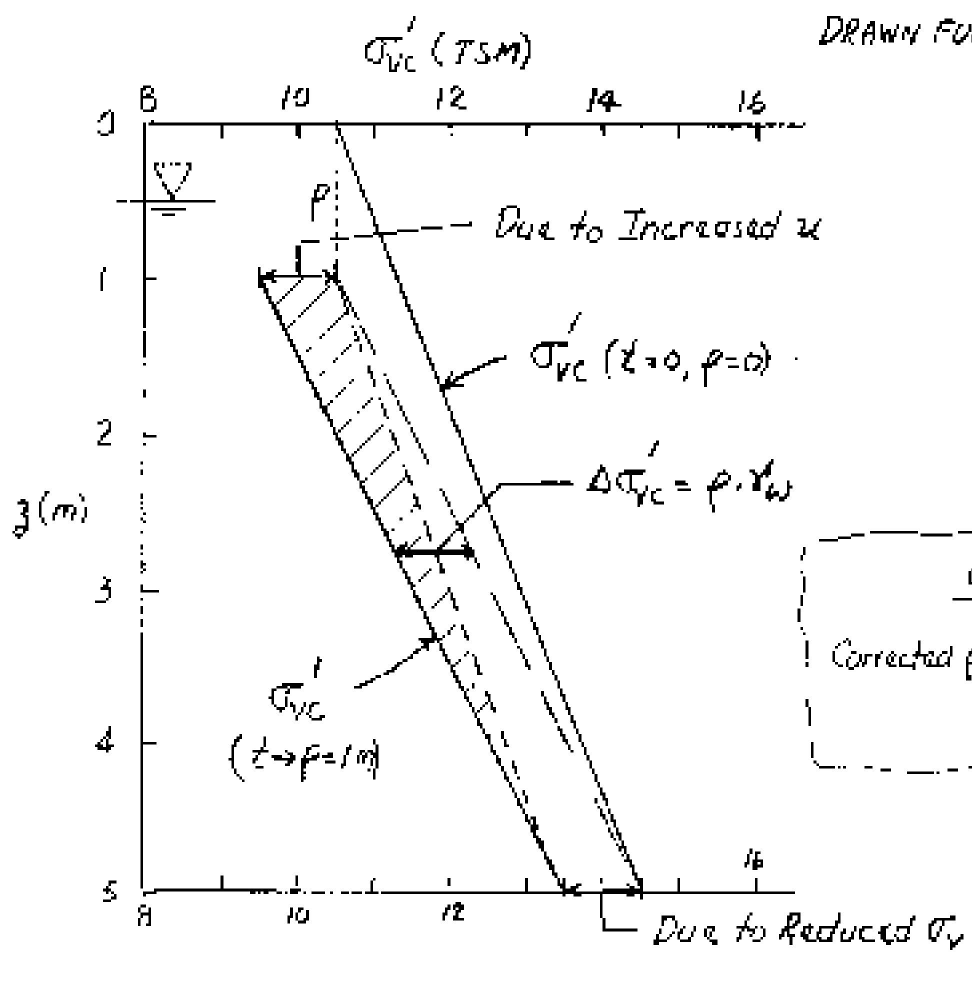
INITIAL CONDITION ($t=0$) WITH SETTLEMENT ρ



$$\text{"Driving" } \Delta\sigma_{vc}' = \underbrace{\gamma_f \cdot H_f + \Delta H(\gamma_f' - \gamma_f)}_{\Delta\sigma_v \text{ at } (A) = \text{ bottom of fill}} - \rho\gamma_w$$

$$\Delta\sigma_v - \Delta u$$

DRAWN FOR: $H_f = 5m$
$\gamma_f = \gamma_f' = 2TCM$
$z = 0.5m$
$\rho_c = 1.0m$
$\gamma_b = 0.8TCM$

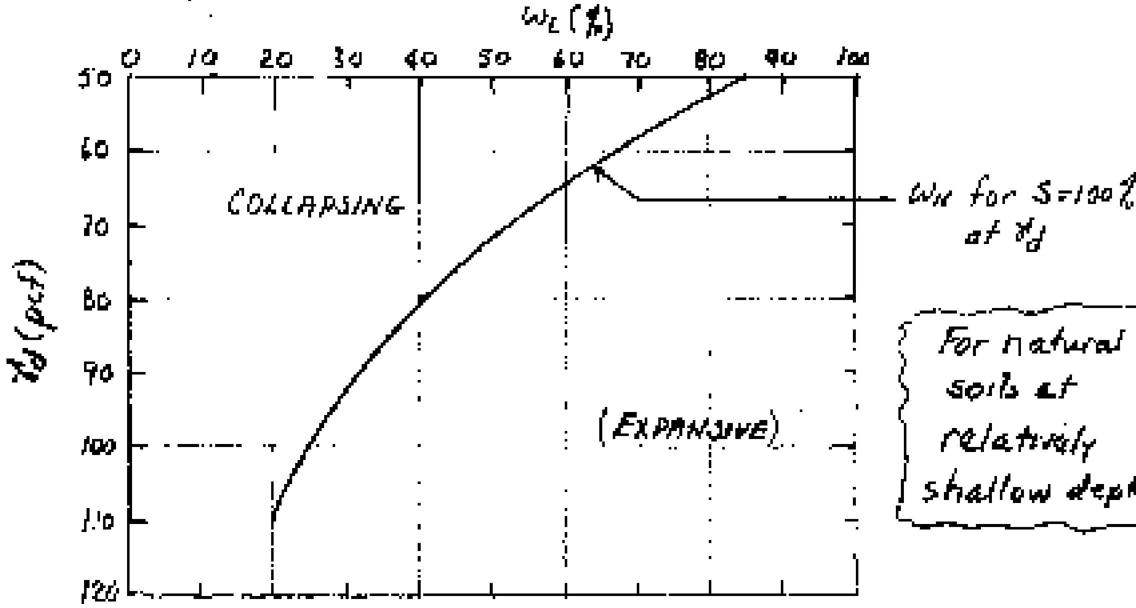


Constant m_v

$$\text{Corrected } \rho_{cf} = \frac{\rho_{c0}}{1 + \rho_{c0}\frac{\gamma_w}{\Delta u}}$$

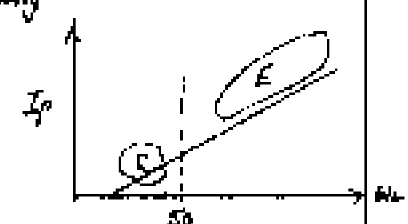
1. Introduction

- Occurs when some partially saturated soils have "free" access to water ((Say S < 50-75%)
- Holtz & Hilf (1961)



For natural soils at relatively shallow depth.

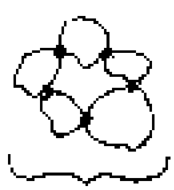
- Low γ_d & low I_p : w (S=100%) > w_L → collapsing
- High γ_d & high I_p : → expansive



2. Collapsing

1) Index properties → ML, CL-ML & CL (all low I_p)
LOESS (wind blown silty soil) + sometimes mudflow, alluvial, residual

2) Structure or "bonding"



Natural "dry" state

- Carbonates
- dry clay
- capillarity

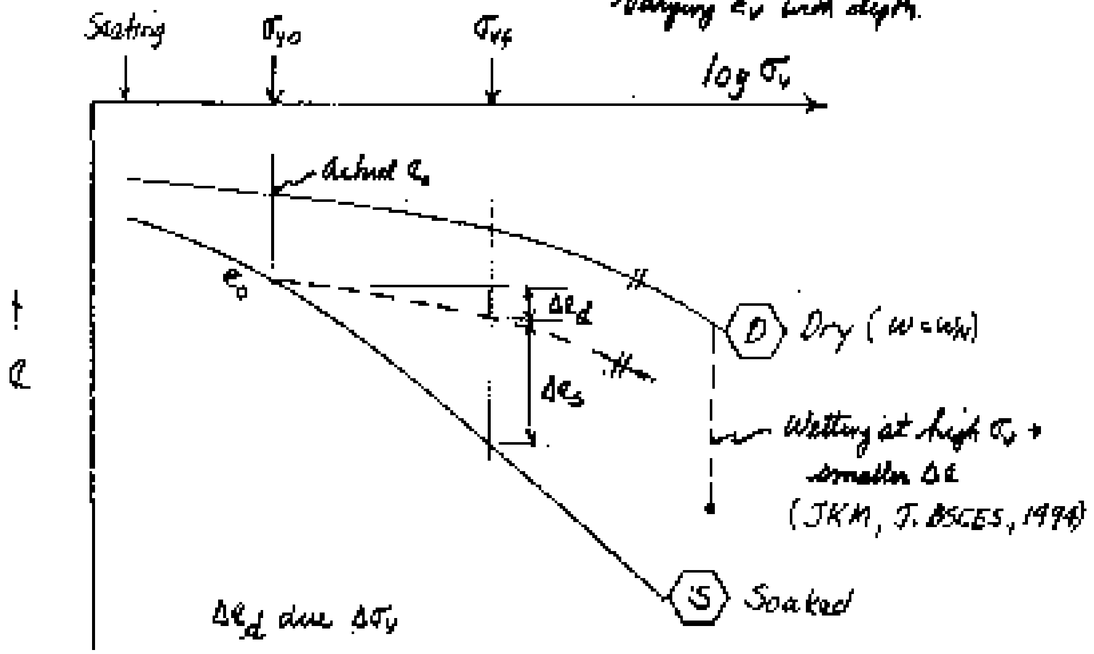
} adding H_2O weakens/destroys bonding

3) For cuts in Loess - use vertical, because sloping will erode

3) Example . San Joaquin Valley, CA : $\rho \rightarrow 15'$! (Dudley, 1970)

4) Estimating Settlements

- Boring with undisturbed sampling : MUST USE ___ HOLE
- "Double Oedometer" (Clemence & Finbarr 1981)
  - ? For homogeneous deposit, one pair to tests can be used to predict varying ε_v with depth.



Δe_d due $\Delta\sigma_v$

Δe_s due saturation = "Collapse"

$$\rho = \Sigma\left[H_i\left(\frac{\Delta e_d + \Delta e_s}{1+e_0}\right)\right]$$

(Paper used soaked e_0 to be conservative; also use of D & S curves tends to overpredict ρ)

"Environmental Stress Path Testing"

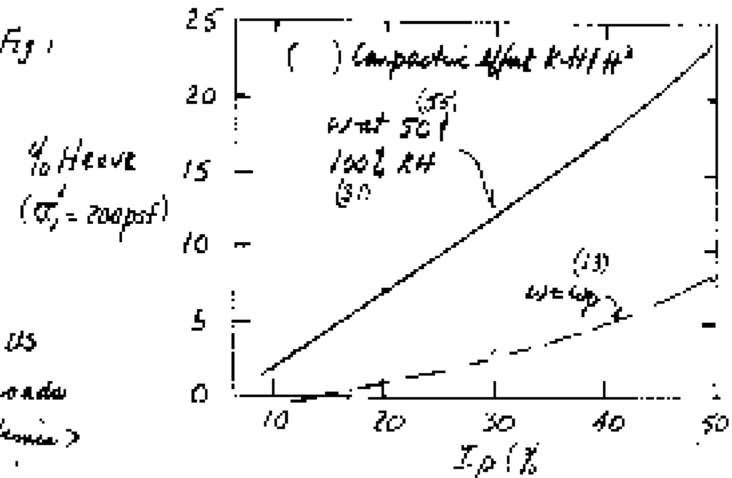
Better procedure is to load dry & then add H_2O at each depth

3. Expansive Clays

1) General Conditions & Scope of Problem

- CH Clays (usually significant smectite content) in climate where rate of evaporation greatly exceeds rate of rainfall, i.e. clay starts out in desiccated condition with low degree of saturation

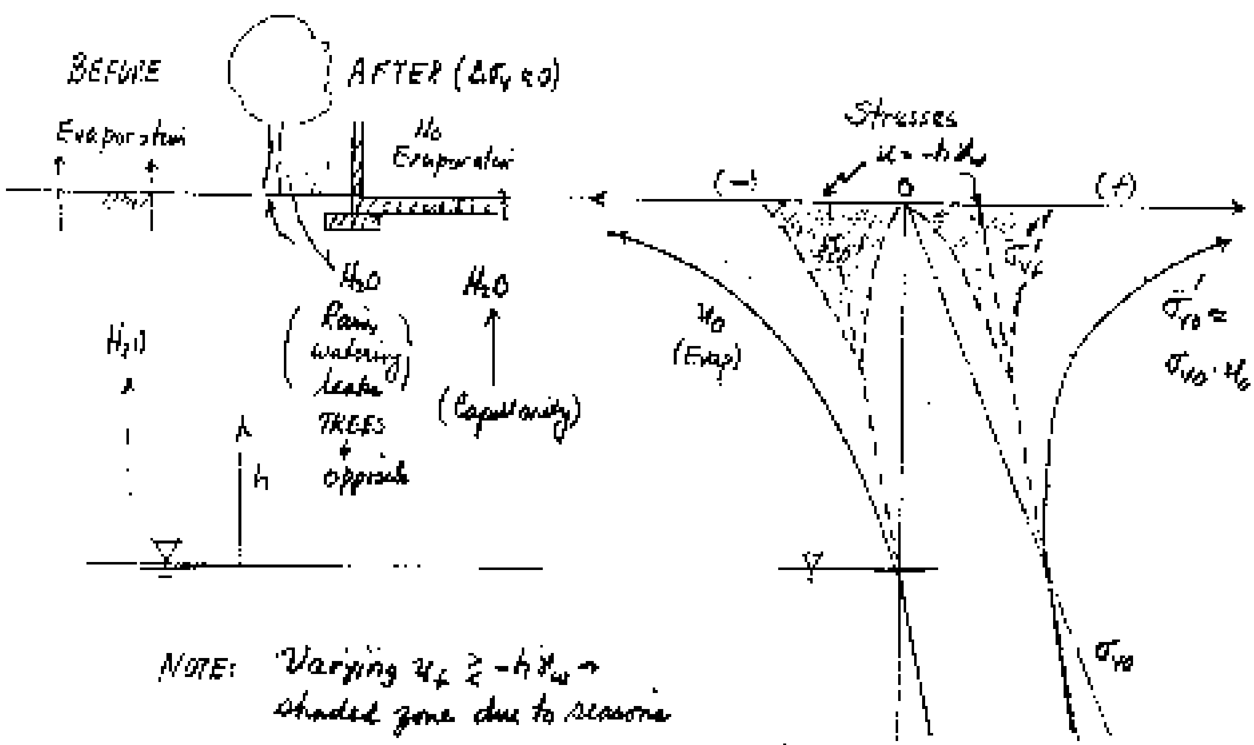
- Ladd & Lambe (1961): Fig:



NRC (1984) Comm. Ground Failure Hazards

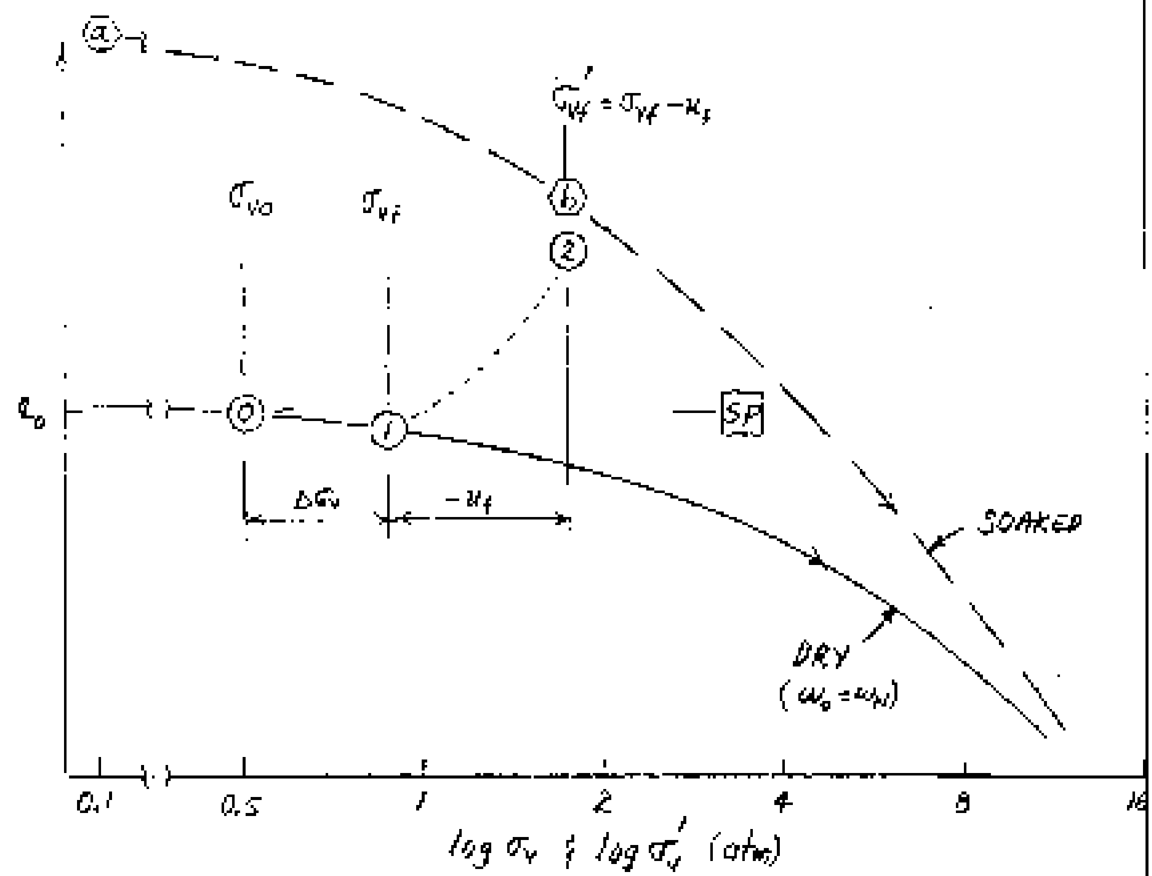
- \$5-10B/yr damage in US to homes, buildings, roads & utilities (several times > than Σ floods + hurricanes + tornadoes + earthquakes!)
- In 20% of US, 10% of new homes suffer severe damage due to poor or no geot. engr.

2) Illustration of $\Delta\sigma'$ due to Construction



NOTE: Varying $u_f \gtrless -h\gamma_w \rightarrow$ shaded zone due to seasons

3) "Environmental Stress Path Testing" (Sampling in DRY hole)



Steps (At each depth)

- Start with DRY sample ($w = w_N$)
- Apply σ_{vo} & σ_{vf} : $e_1 - e_0 = -\Delta e$ due to applied load
- Add water plus increase stress to σ'_{vf} : $e_2 - e_1 = +\Delta e$ due to swelling
- (∴ SP = Swell Pressure to prevent swelling at e_0) Heave $\varepsilon_v = \frac{e_2 - e_1}{1 + e_1}$

4) Double Oedometer (Jennings & Knight, 1957, 4th ICSMFE)

- Duplicate samples: use results at varying depth
- One loaded DRY to get $-\Delta e$ due to $+\Delta\sigma_v$
- Other SOAKED at seating stress → (a) (≈ 0.1 atm)
- Predicted final e at (b)

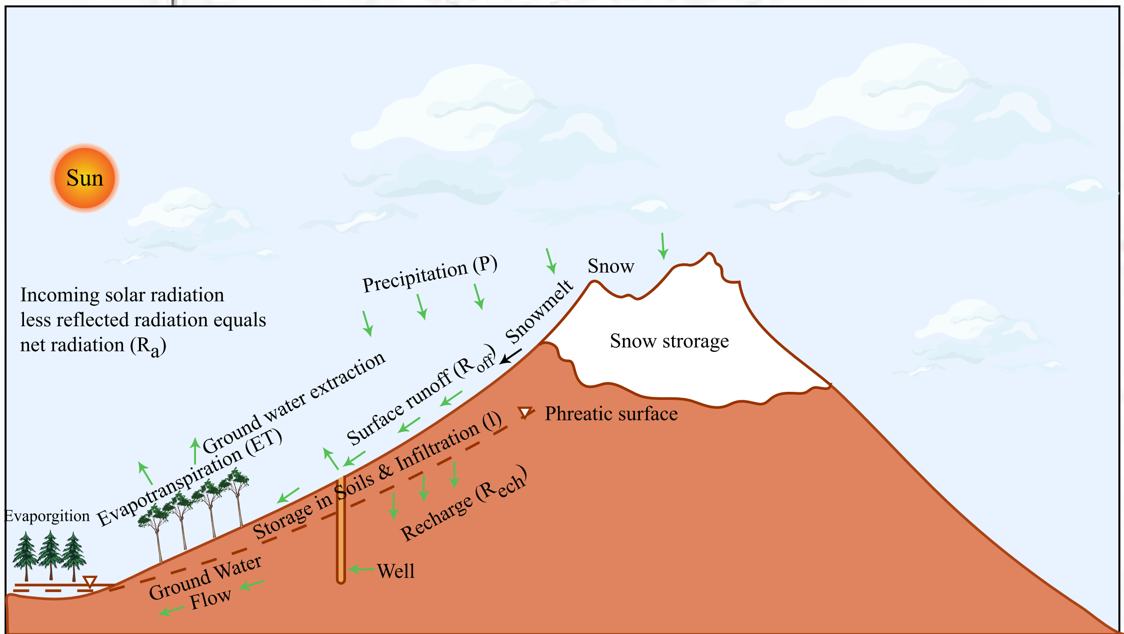
NOTE: Estimation of u_f a problem with both methods.
Assuming $u_f = 0$ → most unfavorable condition → maximum heave, but neglects seasonal Δu_f that can cause increased damage

A EFFECTS OF CHANGES IN CLIMATE (Seasonal Wetting & Drying)

1. Climatic Changes

Blight (1997) Rankine Lecture
Geot. 47(4)

a) Components of water balance



Components of the soil water balance.

• Complex

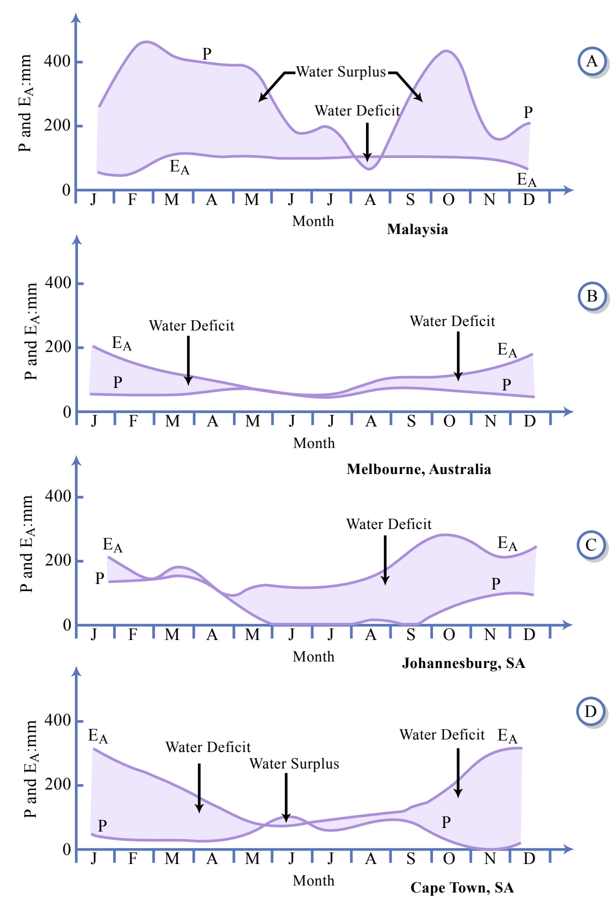
* Precipitation − (Intercepted + Runoff) } Input
(P)

=

* Evapotranspiration (E_A)
+
Recharge (Flow to W.T.)
+
Change Soil Water Storage } Output

b) Examples: Rainfall vs Evaporation

Worse case = wet season with alot of rain and very dry season (hot & dry)



Illustrations of different types of atmospheric water balance: (A) Kuala Tahan, Malaysia, 1984; (B) Melbourne, Australia, 30-year average; (C) Johannesburg, South Africa, 1987; (D) Cape Town, South Africa, 1992. E_A, A-pan evaporation; P, precipitation.

Figures by MIT OCW.

Adapted from: (Blight 1977)

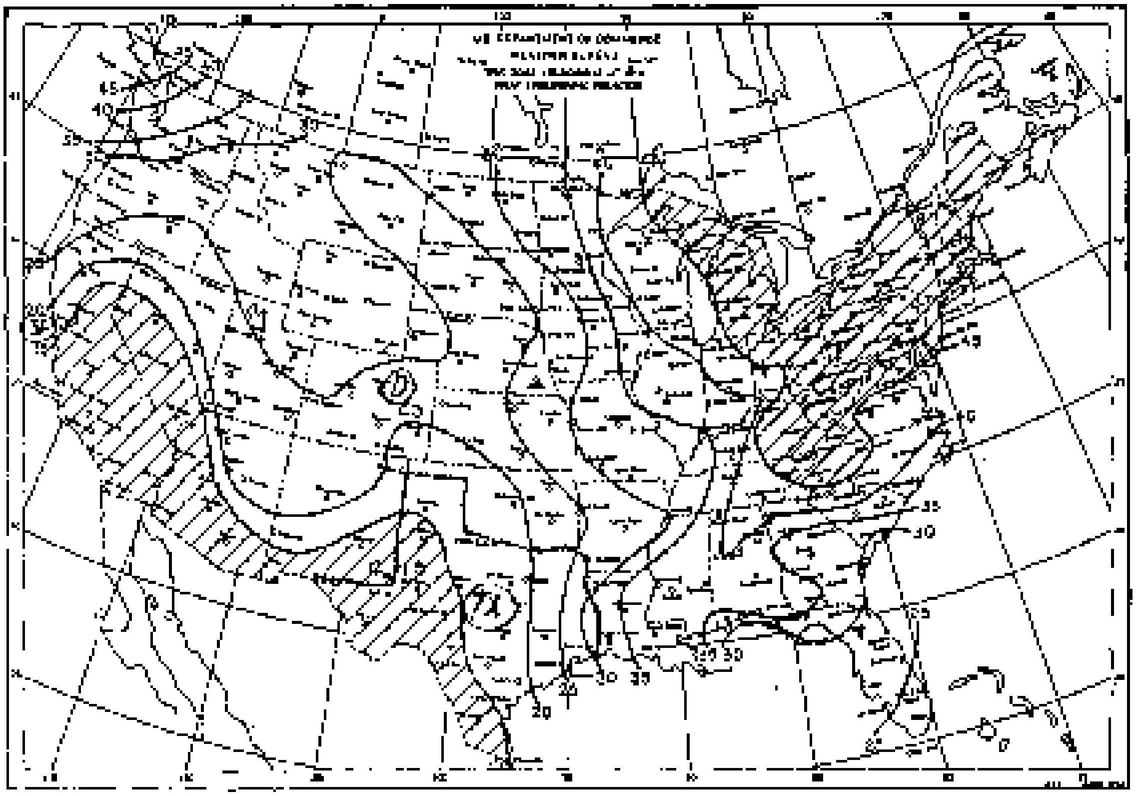
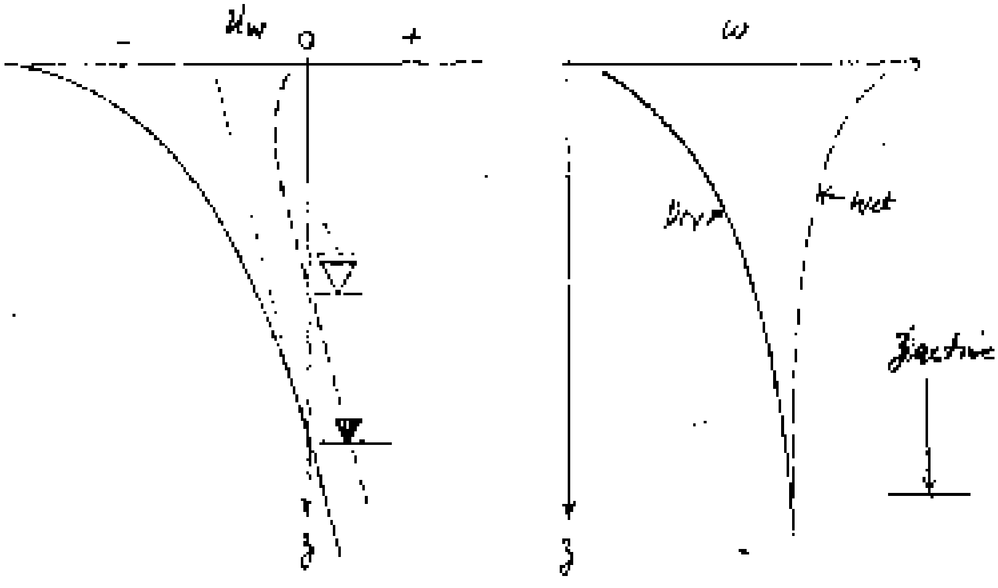


FIG. 1 Climatic Ratings (C_w) for Continental United States (NRC Report 1968)

Severe problems: San Francisco area, Denver, Texas, etc.
Increasing C_w → high P/E_p ; $C_w = 20 \pm 5$ → seasonal damage

2. Active Zone = Depth of "seasonal" change in water content → cycles of swelling / shrinkage

▼—— Dry
▽- - - Wet

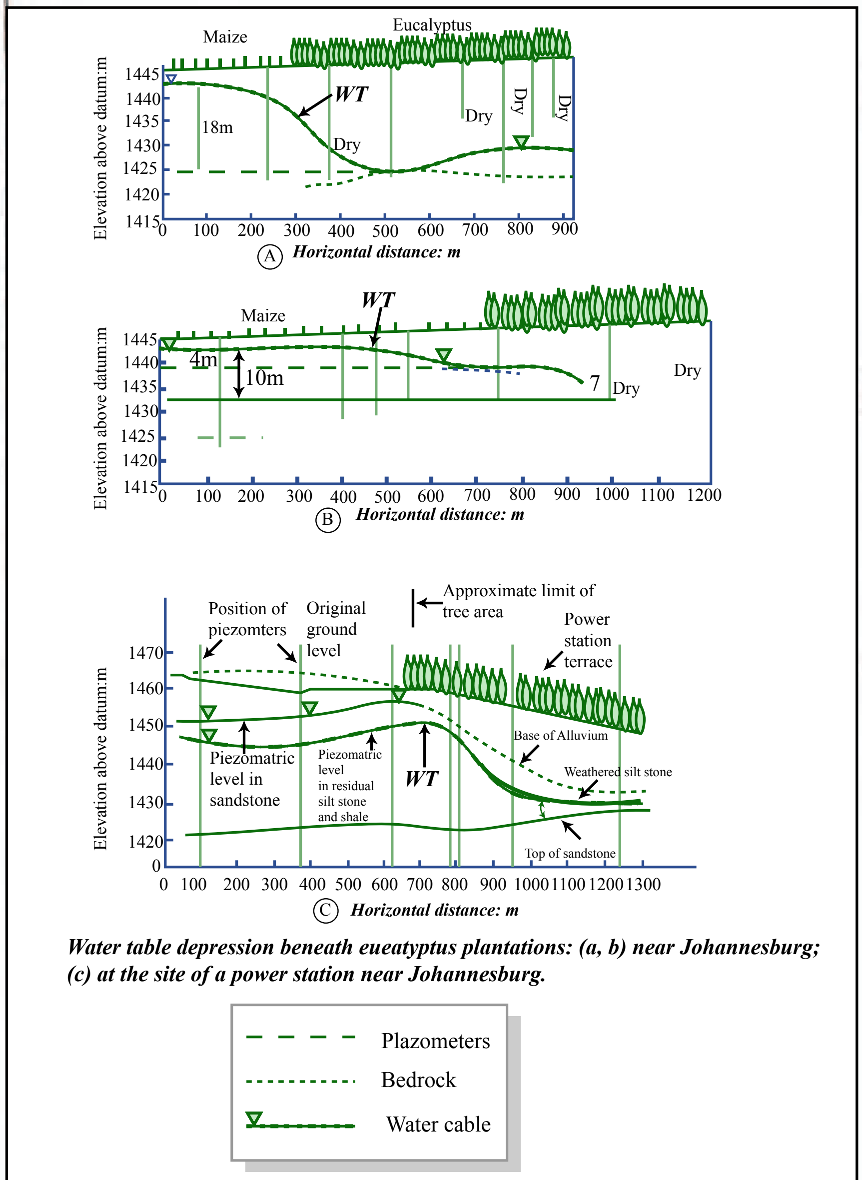


3/18/01 3. Some Examples (Blight 1997)

a) Trees → depressed water table

+ also increase amt of drying during dry season

- Effect extends to 3/4 ± 1/4 × tree height horizontally
- Depth >> depth of roots
- Why trees do this?



Water table depression beneath eueatyptus plantations: (a, b) near Johannesburg; (c) at the site of a power station near Johannesburg.

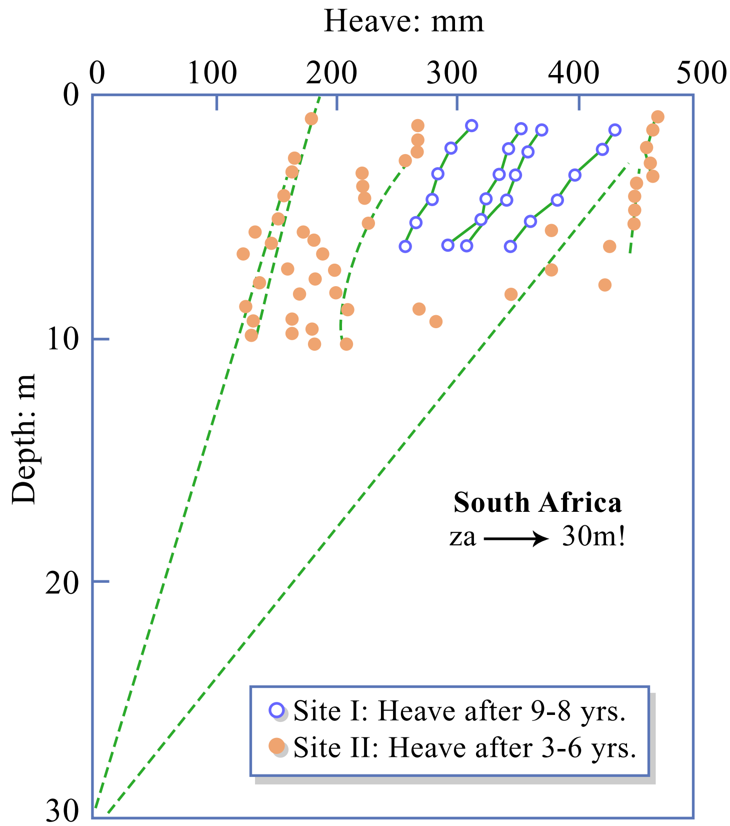
>20m

≅10m

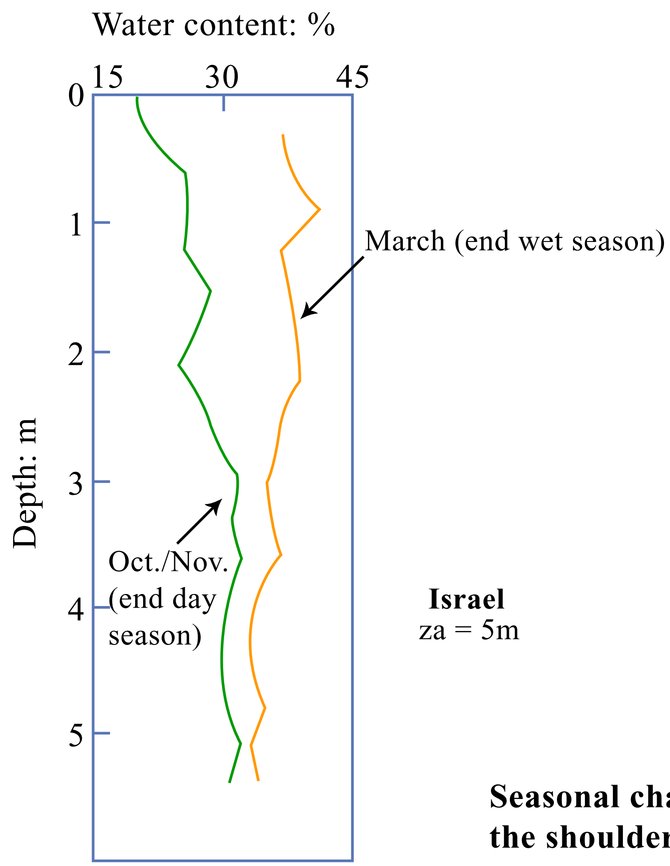
≈15m

Figure by MIT OCW.
Adapted from: (Blight 1977)

b) Depth of active zone (z_a)



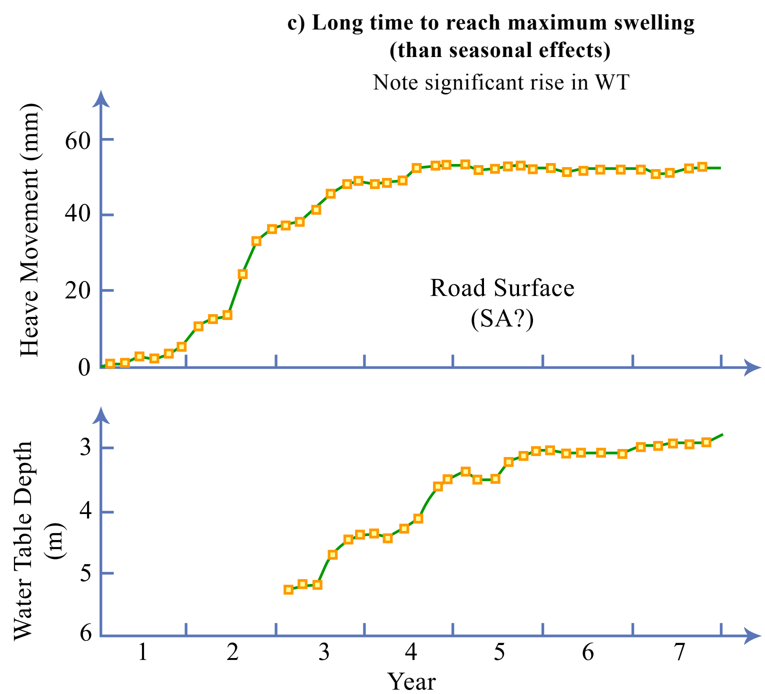
Measurements of heave with depth, indicating a depth of active zone of about 30m.



Seasonal changes in water content observed under the shoulders of an airport pavement in Israel.

Figure by MIT OCW.

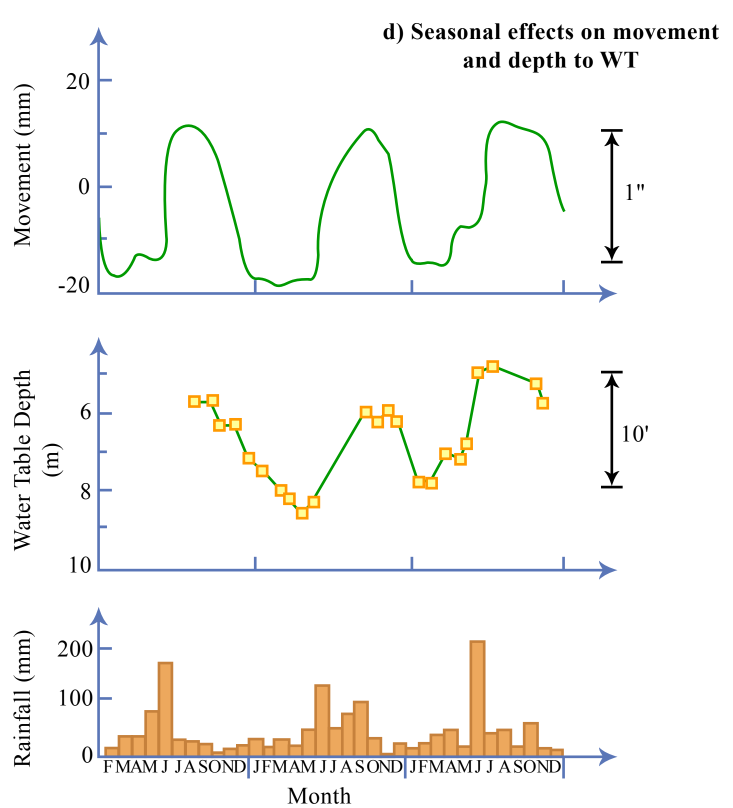
CCL 3/17/98 1,322 Expansive Soils ES-4
3/99



Surface heave movements and a rise of the water table that occurred as a new water balance was established in a recently urbanized area

c) Long time to reach max. swelling (than seasonal effects)

· Note significant rise in WT



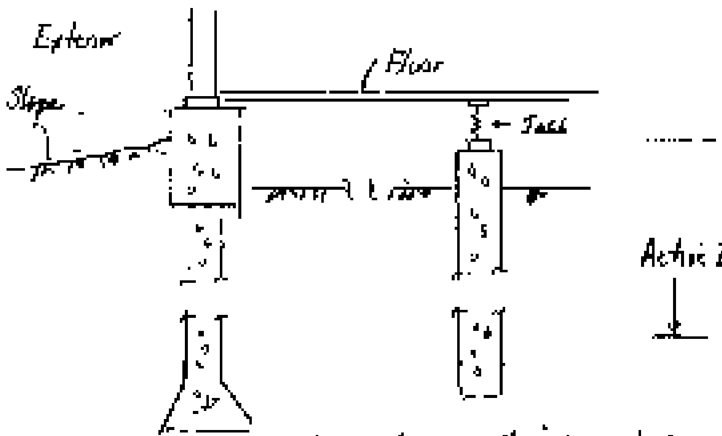
Seasonal variations in surface movement and water table depth for an old building in Cap Town

d) Seasonal effects on movement and depth to WT

Figures by MIT OCW.

B. FOUNDATION DESIGN ON EXPANSIVE SOILS

1. Best Solution = Isolate Bldg from Active Zone → Piers + Grade Beams



NOTE: Do NOT attach structures to bldg such as fences, patios, etc.

Must have sufficient embedment to resist uplift forces due to swelling soil around piers

2. Partial Solutions

a) Prewet f(dn) soils – but will not help with seasonal wetting/drying → poor practice

b) Prestressed slab – at minimum, replace upper soils with granular fill
 – add cut-off wall around perimeter to reduce seasonal effects

Reference: Book by Nelson & Miller (1992) Expansive Soils: .... J. Wiley

1. Background

1) Residual Soil = in situ weathering of rock → soil (see R5,6)

2) Composition in WARM-WET climates (see R2)

- Crystalline rock + poor drainage → smectite
- " " + good " → red laterites (iron oxides)
  - Usually cemented
  - Considered "good" ML-MH soil
- Volcanic ash & rock → andisols
  - Generally high w_N-I_p → "poor" MH soil

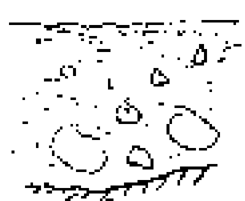
Saprolite: retains structure of parent rock

2. Comments on Engineering Practice

1) Index Properties (see R2,3)

- Can't use empirical correlations based on sedimentary clays
- Results = f (preparation method)

2) Structure & Resultant Problems (R3, 5&6)



- Extremely heterogeneous: fine grained at top, mostly rocks at bottom
- Can't sample for meaningful lab testing
- Usually high in situ k

3) Estimates of Settlement

- Local experience
- In situ testing via SPT or Menard Pressuremeter (R4) or PLT (costly) or DMT (if can insert)

4) Slope Stability (R3)

- Increased %S during rainfall → failures — Hong Kong, California Bay area
- Definition of $\bar{\sigma} = f_1(\sigma - u_a) + f_2(u_a - u_w)$
  (For compacted clays, often use $f_1 = 1$, $f_2 = \chi$)

(R2)

TROPICAL RESIDUAL SOILS*

C.C. Ladd 1.322 3/82

1.0 DEFINITIONS AND SPECIAL COMPOSITION

1.1 Tropical = ±22° N-S

. Residual soil = in situ weathering of rock to produce soil.

1.2 Composition of Tropical Residual Soils in Warm-Wet Climates

(1) Crystalline rock and poor drainage → smectite

(2) Crystalline rock and good drainage → Red Laterites (also called Oxisols)

- Kaolinite plus Fe/Al. oxides (reddish color)
- Low "activity" with a lot of cementation
- Considered "good" MH soil

(3) Weathering of volcanic ash/rock → Andisols

- Halloysite (tubes + spheres) plus amorphous alumina & silica (very high SSA but low surface charge) and maybe smectite (usually dark color)
- Generally high w_N and P.I.
- Considered "poor" MH soil

2.0 CHARACTERISTICS OF RED RESIDUAL SOILS (LATERITES) WHICH OFTEN REQUIRE DIFFERENT ENGINEERING PRACTICE (Compared to saturated sedimentary clays).

2.1 Index Testing and Correlations with Atterberg Limits (See Mitchell & Sitar, 1982, for examples).

(1) Halloysite

- Tubular structure → very low dry density
- Dehydration when dried

(2) Fe & Al. oxides plus silica gel act as strong cementing agents.

- Decreases effective SSA
- Highly variable in situ

---

* Panel discussions and Proceedings ASCE GED Spec. Conf. on Engr. and Construction in Tropical and Residual Soils, Honolulu, Hawaii, Jan. 1982 (Available from ASCE).

R3

(3) Drying soil generally increases amount of cementation and reduces plasticity.

(4) Amount of mechanical remolding can greatly affect measured Atterberg Limits (more remolding → increased plasticity).

(5) Conclusions

- Can't use empirical correlations developed for temperate clays
- Any correlation with index properties likely to be very scattered

2.2 Heterogeneity

(1) Profile characterized by differential weathering and cementation. See Brand (1982) for classification system for Hong Kong.

(2) Because of above, properties highly variable and

- Undisturbed sampling difficult to perform
- Conventional size samples don't reflect mass properties

(3) Conclusions

- Base design on local experience and/or large in situ testing

2.3 Saturation - Rainfall

(1) Strata of main interest usually occur above the water table and are characterized by:

- Partial saturation
- Generally high in situ permeability

(2) How to define $\bar{\sigma}$ in partially saturated soils?

- If $S \geq 80\%$, $\bar{\sigma} = \sigma - u_w$ probably reasonable (discontinuous air voids)
- Otherwise, must consider two components, i.e. $\bar{\sigma} = f_1(\sigma - u_a) - f_2(u_a - u_w)$

(3) Variation in u_w greatly affect slope stability

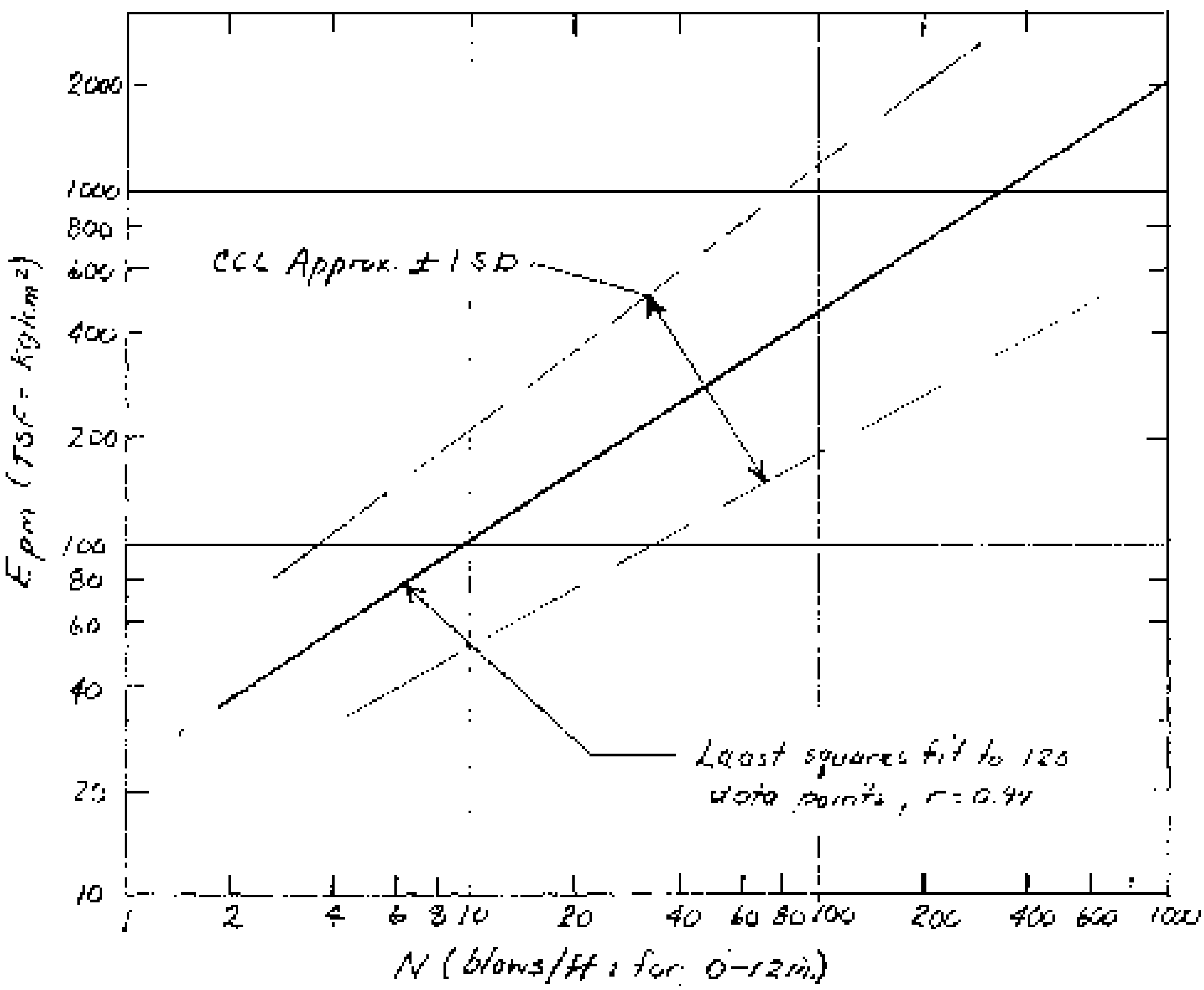
- Seasonable variations
- Effect of heavy rainfalls
- Influence of modifying drainage pattern

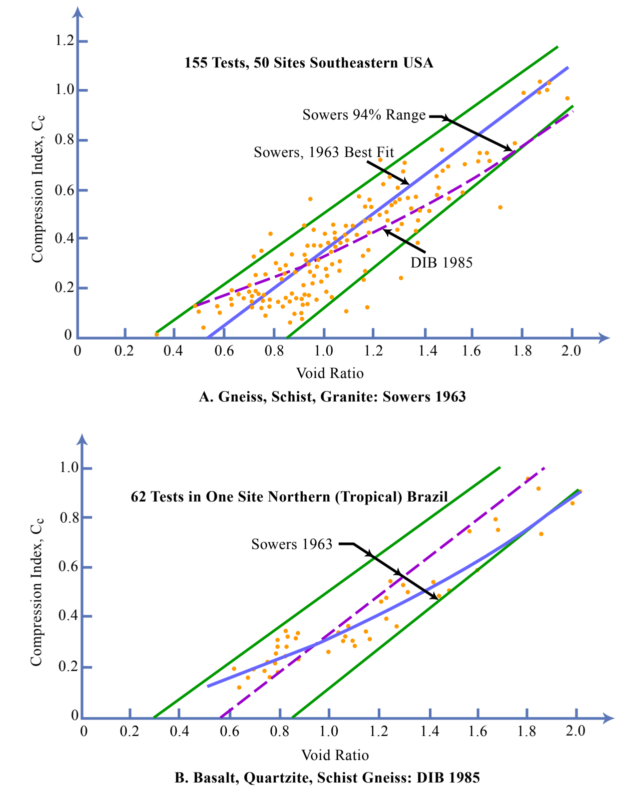
Martin, R.E. (1977) "Est. Fdn. Settlements in Residual Soils" ASCE, JGED V103, GT3, pp. 197-212

- For residual soil developed from igneous & metamorphic rock, mostly SM to ML nonplastic materials with 30-70% fines.
- Recommends using Schmertmann method

$$\rho = C_1 C_2 \Delta q \, \Sigma\left(\frac{I_3}{E_s}\right)\Delta z \quad \text{with } E_s = E_{pm} \text{ (Menard Pressuremeter)}$$

- If E_{pm} not available, can use approx. N vs. E_{pm} correlation below
- Method supported by 5 case histories (but $\rho < 0.5''$ most cases) in greater Wash. D.C. area only





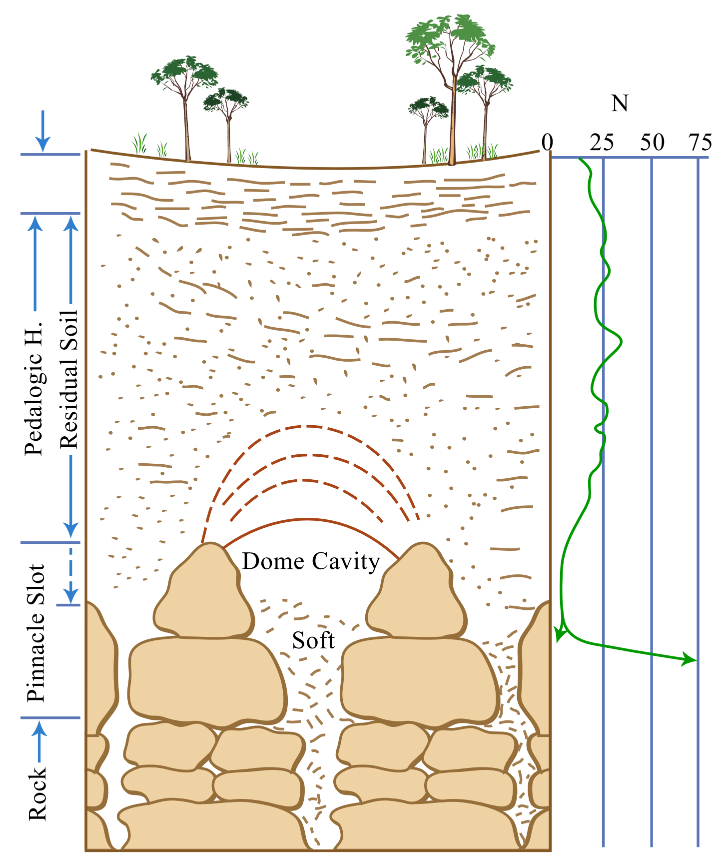
A. Gneiss, Schist, Granite: Sowers 1963

B. Basalt, Quartzite, Schist Gneiss: DIB 1985

Relationship Between the Void Ratio and Compression Index in Residual Soils Derived from Crystalline (Igneous and Metamorphic) Rocks.

Figure by MIT OCW.

← MUST check for underground voids



Residual Soil Profile in Limestones

Residual Soil Settlement Related to the Weathering Profile

Figure by MIT OCW.
Adapted from:

MUST study when encounter residual soils

ASCE Geol. Spec. Publ. No. 40
"Settlement '94" V2, p 1689-1702

→ RESIDUAL SOIL SETTLEMENT RELATED TO THE WEATHERING PROFILE

George F. Sowers,[1] Honorary Member, ASCE

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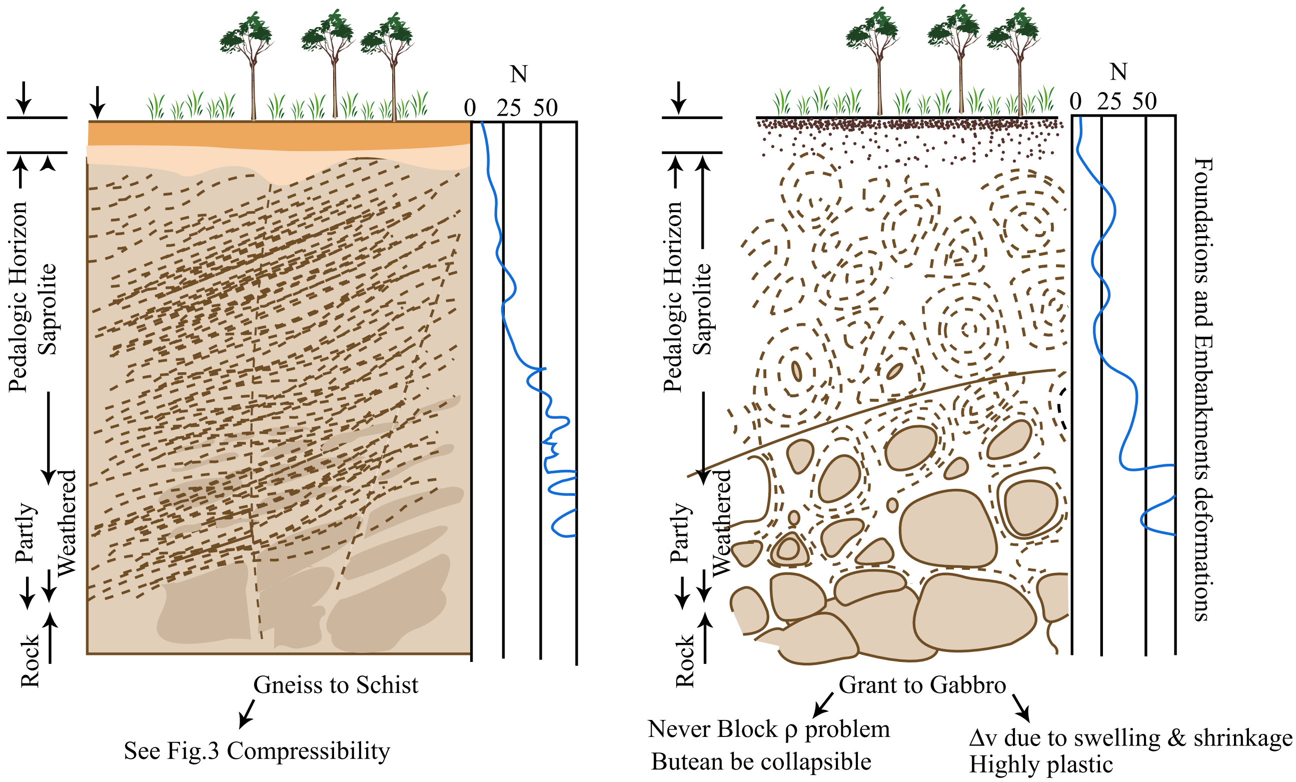
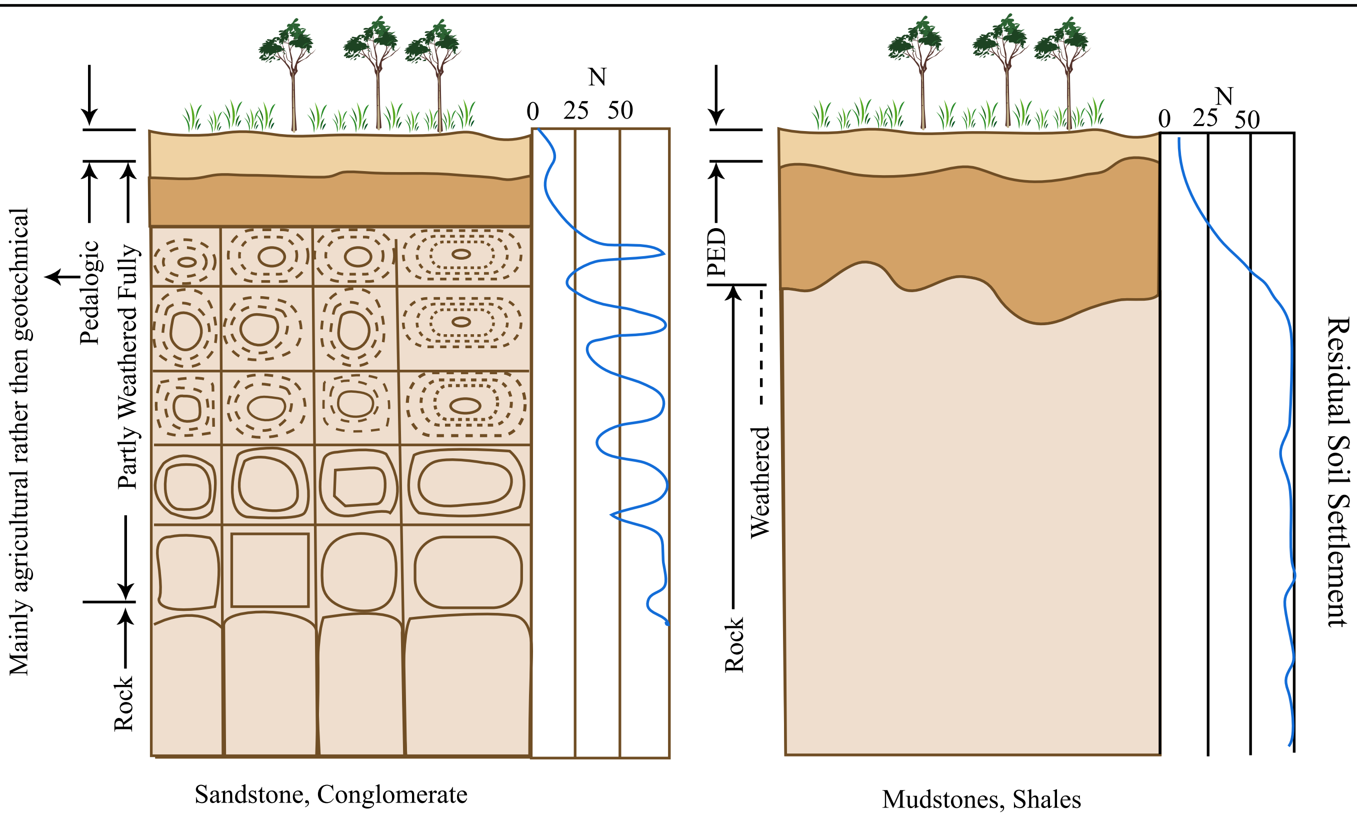
4/17/95

R6



Weathering profiles in crystalline Rock: Gneiss to Schist, Granite to Gabbro



Weathering Profiles in Clastic Sedimentary Rock: Sandstones and Shales

Figures by MIT OCW.

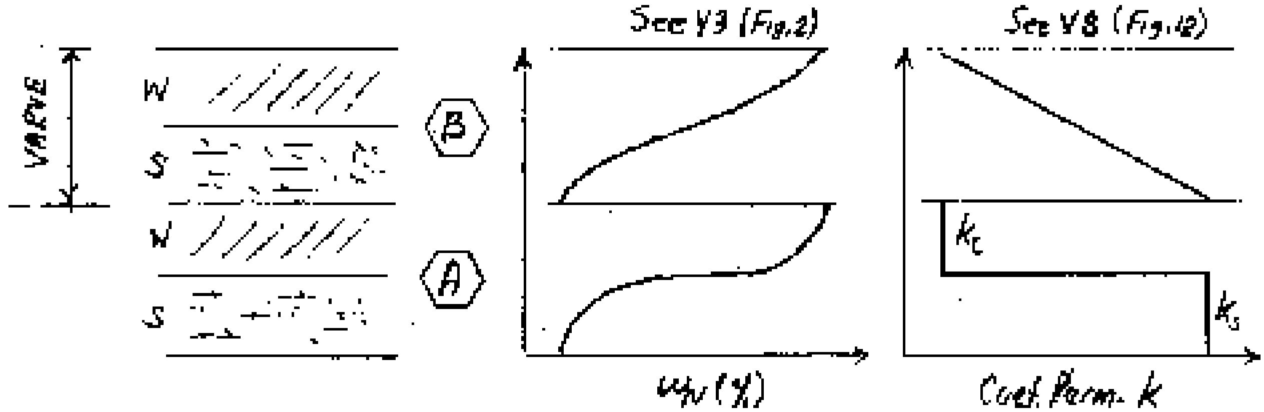
Adapted from: Sowers (1994)

1. Deposition & Composition

1) Deposition = rhythmically banded glacial lake deposit (fresh, cold H_2O)

Summer: meltwater with heavy load → coarser grained fraction settling out

Winter: little flow & frozen over → finer fraction settling out



2) Composition (North America, esp. NE US) e.g. Conn. Valley, NJ Hackensack Valley and upper state NY

- Typical varve thickness = 2±1 cm
- Summer → "Silt" layer ML → CL
- Winter → "Clay" layer CH

} Plasticity Chart V4 (Fig.3)

- Can get thick (several ft) layer of rock flour (cohesionless silt size ground quartz, etc)
- Radiography BEST; drying alternative
- Transition within varve: A = abrupt, B = gradual
- Often find decreasing varve thickness at higher elevations (retreating glaciers)

2. 1-D Consolidation

- Compression curves (V5) A = stiff → C & D = soft
- CR vs w_N (V6) w_N = 30 - 60 → CR = 0.05 - 0.4 ± 0.1 (NJ)
- c_v(NC) vs w_N (V7) w_N = 30 - 70 → c_v ≈ 1 to < 0.1 ft²/day (Mass & NJ)

NOTE: Oedometer specimen should include at least one varve

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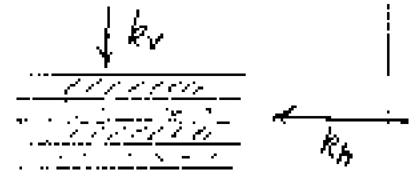
3/18/91

3. Anisotropic Flow



$r_k = k_h/k_v$

i) Importance

- Combined vertical & horizontal drainage
  See V9 (Fig. 13)

- Radial drainage to Vertical Drains

  See V10 (Fig. 14) Want $c_h = \frac{k_h}{m_v \gamma_w} = \frac{k_h}{k_v} c_v = r_k c_v$

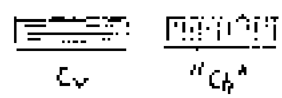
($H_d = 50'$, $c_v = 0.1\ ft^2/day \rightarrow t_{90} = 50\ yr$
Sand drain $s = 20'$, $c_h = 10 \rightarrow t_{90} = 1\ yr$)

2) Theoretical Relationship for $r_k = k_h/k_v$ (Kenney, 1963)

See V8 (Fig. 12) For $k_s/k_c = 100$ A (Abrupt) $\rightarrow r_k = 25$
B (Gradual) $\rightarrow r_k = 2.5$

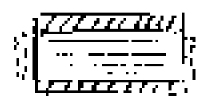
3) Methods to Measure c_h & r_k · V11 (Table C-1)

LAB a) V & H oedometer



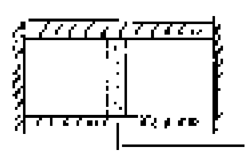
} incorrect m_v

b) Oedometer w/ radial drainage



Side friction! Smear

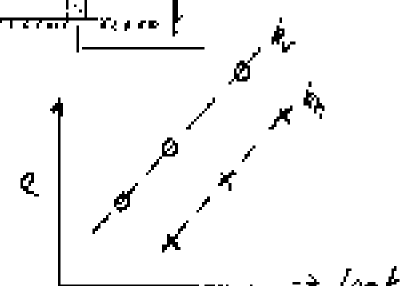
c) "Rowe" Cell with central drain
Need large diameter specimen



d) Separate measurements of k_v & k_h : TX

Cylinders k_v k_h Cube k_v k_h

MIT 4" cube



FIELD a) Piezometers (Casagrande) – Falling/rising head

SF SCA b) Self-boring

c) Pumping from sand drain

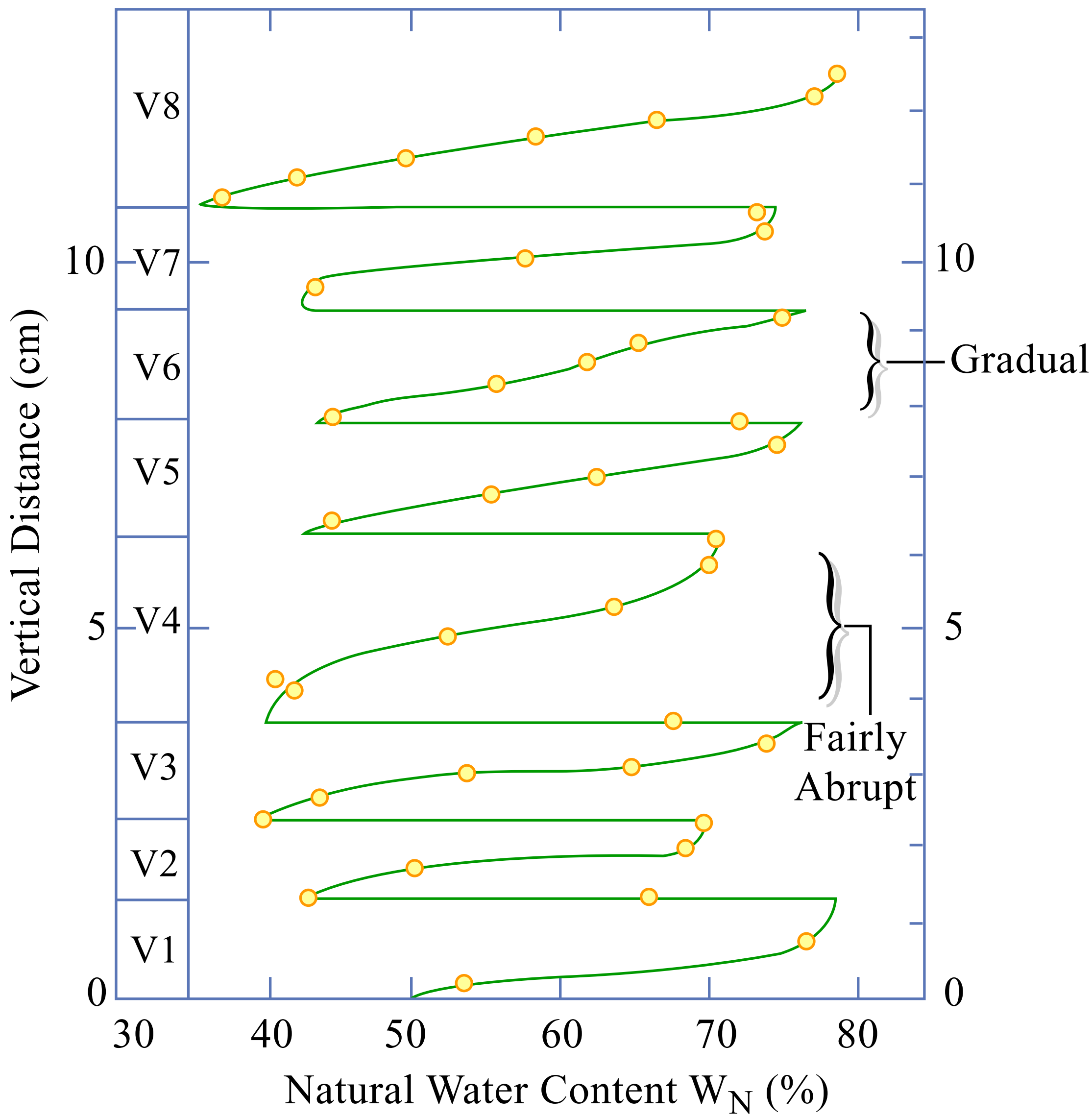
4) Results V12 (Table C-2) All $k_v \approx 14 \pm 5 \times 10^{-8}$ cm/s

- New Liskeard Lab & field $\rightarrow r_k = 3 \pm 1$ 500±100 40±10
- Conn. Valley $r_k \sim 10 \pm 5$ · NJ Field >> Lab
  ↳ rootflow layer?

1.322
4/89
3/01

V3

Northeastern US Varved Clays

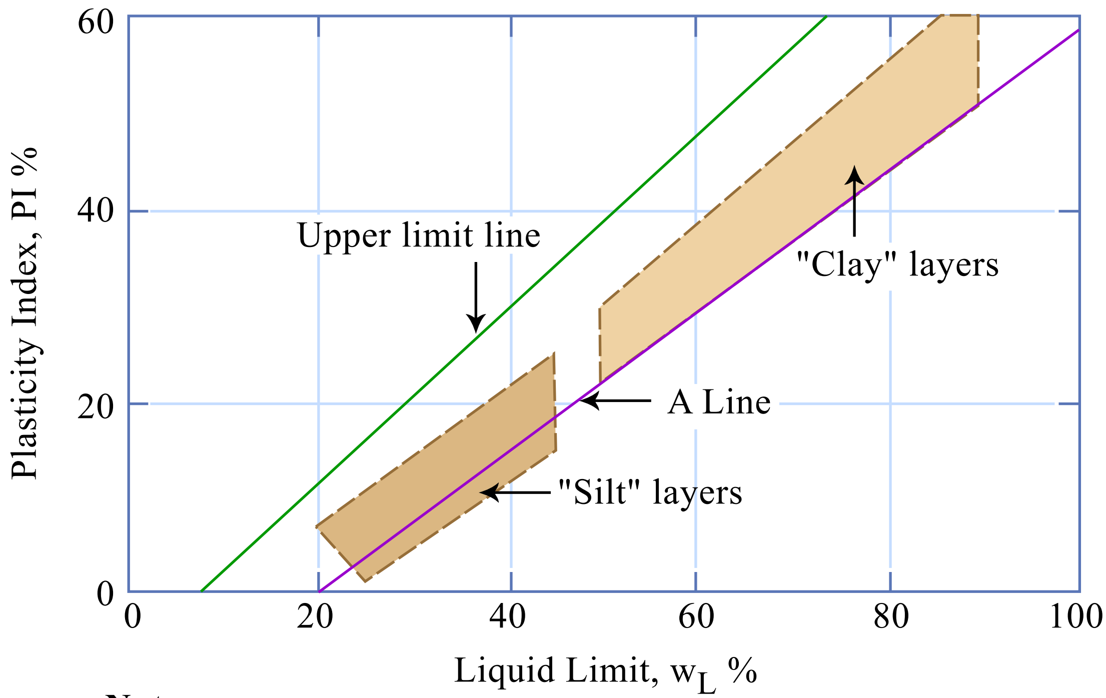


Notes:
1) V1, V2, etc. refer to separate varves
2) Sample from Northampton, Ma., W_N = 56.7%
3) Data from Ladd and Wissa (1970)

Water Content Variation within a Varved Clay from the Connecticut Valley

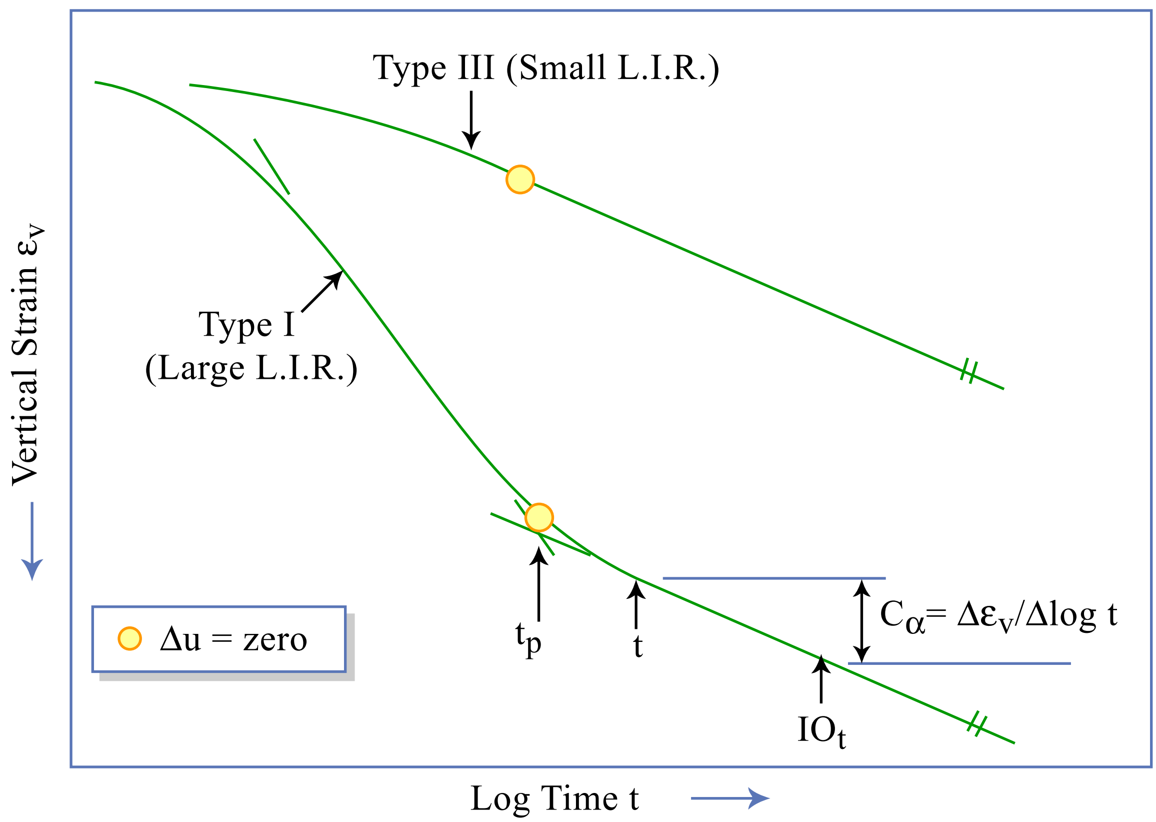
Figure by MIT OCW.

Adapted from: Ladd (1987) Lecture Notes N.Y. ASCE Met. Section



Note:
Range shown for varved clays having w_L >30 % for bulk material

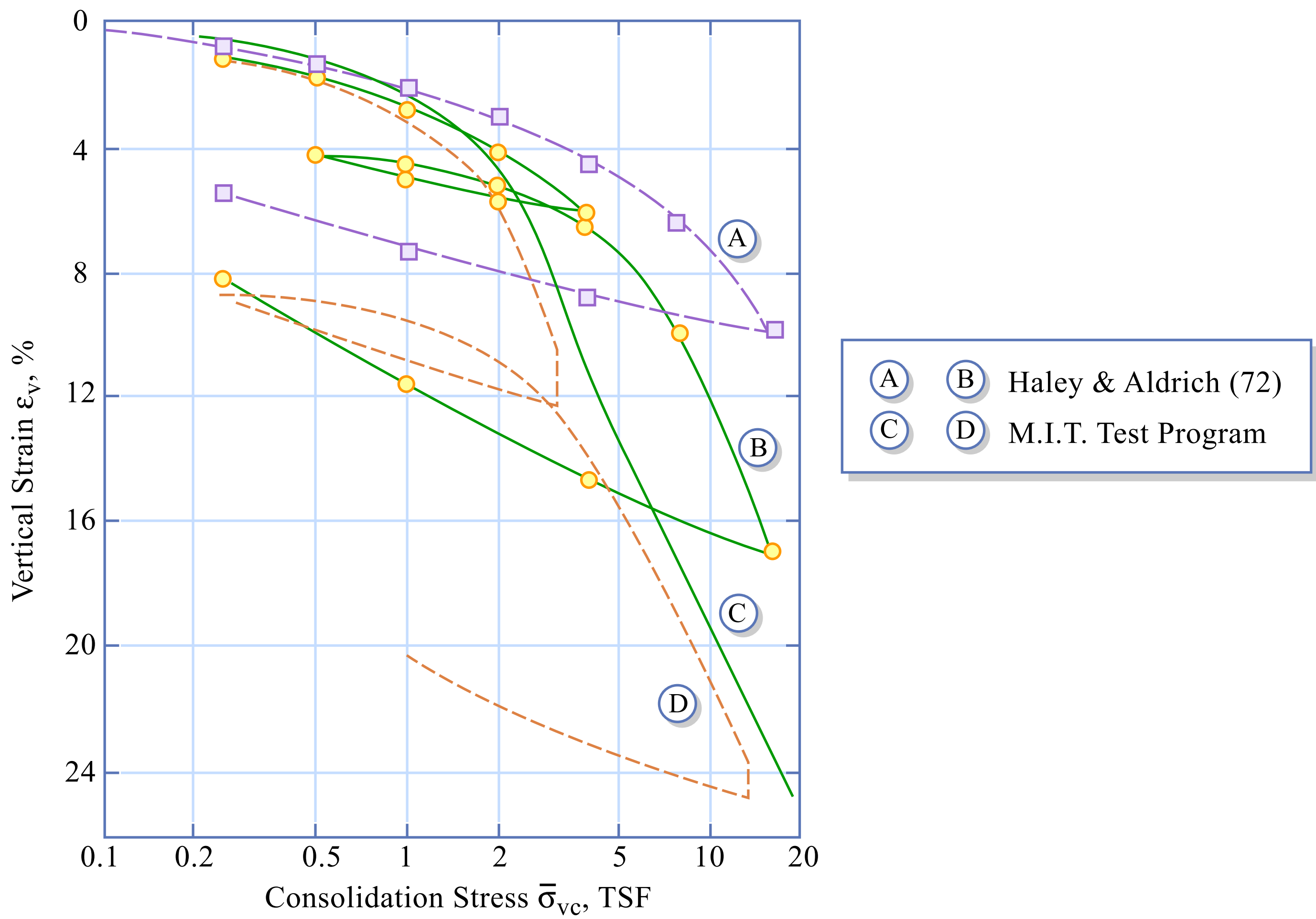
Plasticity Chart for Typical varved clays from Northeastern United States.



Strain vs. Log Time from Incremental Consolidometer Test Showing Effect of Load Increment Ratio (N.C. Clay)

Figure by MIT OCW.

V5



1 TSF = 95.8 kN/m^2

Curve	Type of Test	W_N (%)	$\bar{\sigma}_{vm}$ TSF	CR Max.	CR Min.	Location
A	24 hr. Incremental	36.2	8 *	0.11		Chicopee-
B		50.2	6	0.23		Holyoke
C	CRSC	61.9	2.5	0.30	0.20	Northampton
D		65.4	2	0.25	0.19	Amherst

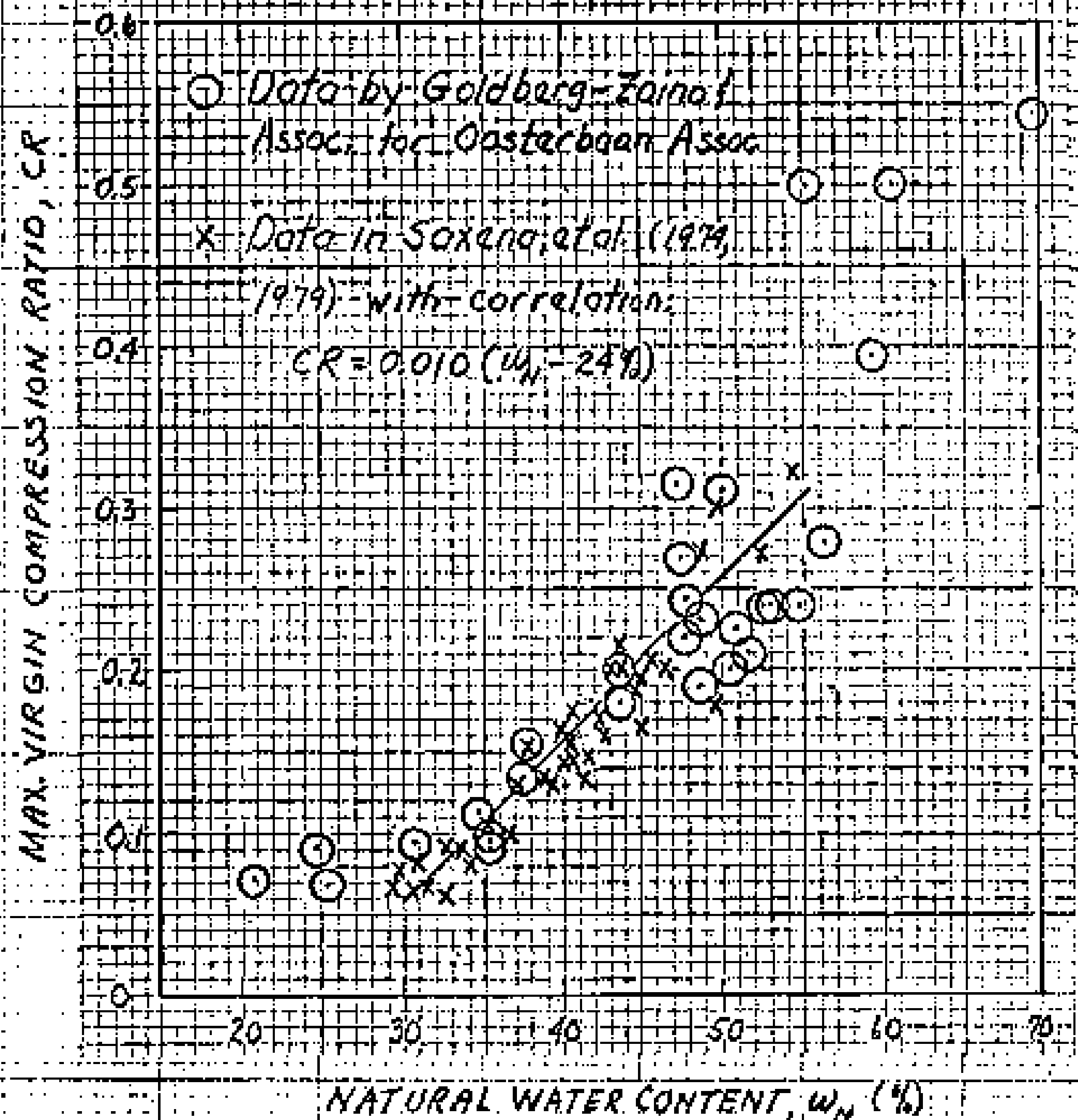
Typical Compression Curves for Connecticut Valley Varved Clays

Figure by MIT OCW.

Adapted from: Ladd & Foott (1977)

CCL
10/87

(V6)

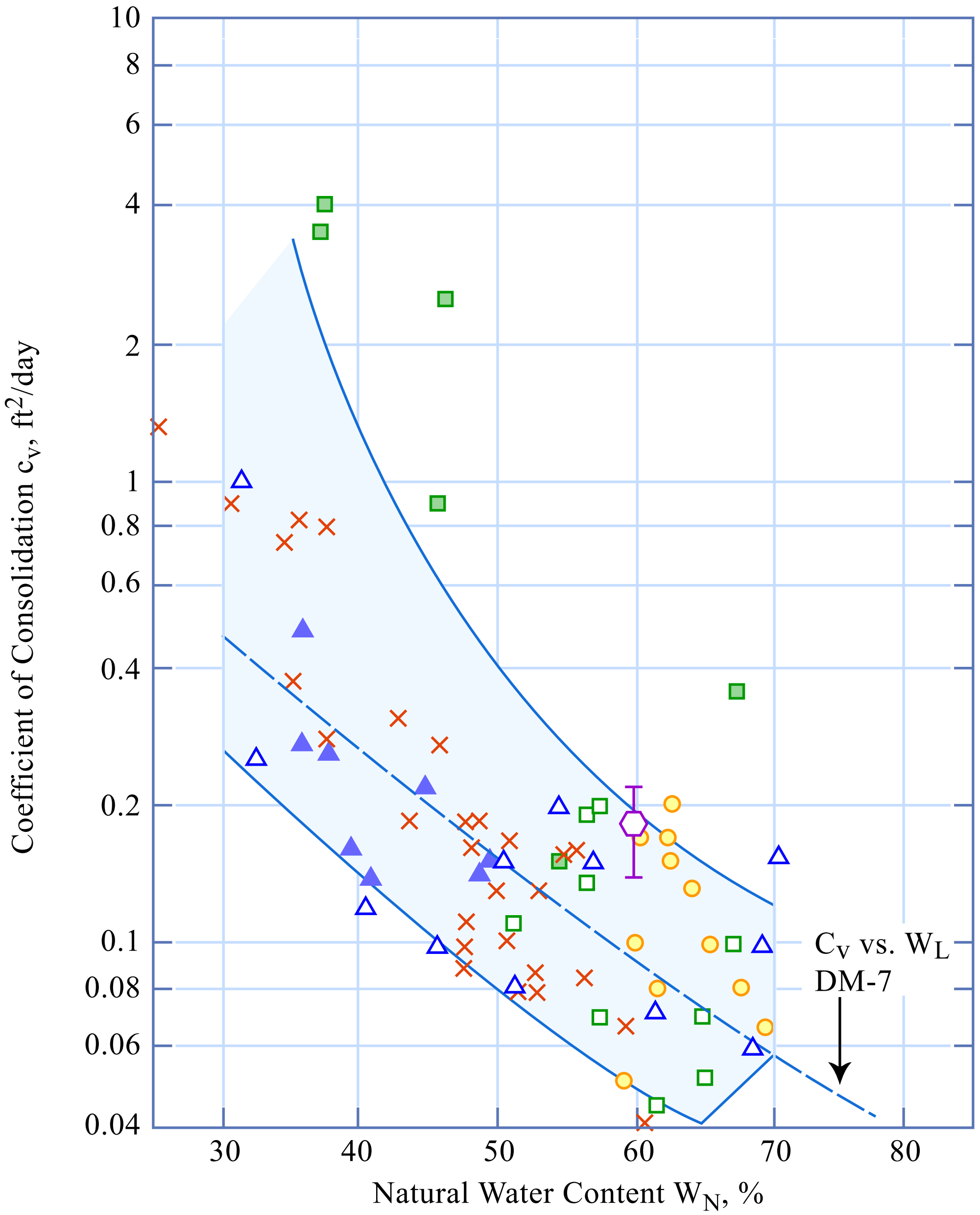


NOTE: For sites near Newark Airport

FIGURE 6 VIRGIN COMPRESSION RATIO VS. NATURAL WATER CONTENT FOR HACKENSACK VALLEY VARVED CLAYS

CCL 10/87

V7



Location*	Type of Test	Symbol
Amherst, MA.	CRSC & Increm.	○
Northampton, MA.	CRSC & Increm.	□ ■ **
E. Windsor, CT.	Increm.	⬡
Jersey City, N.J.	Increm.	△
Secaucus, N.J.	CRSC	▲
New Jersey	Increm. by GZA	× **

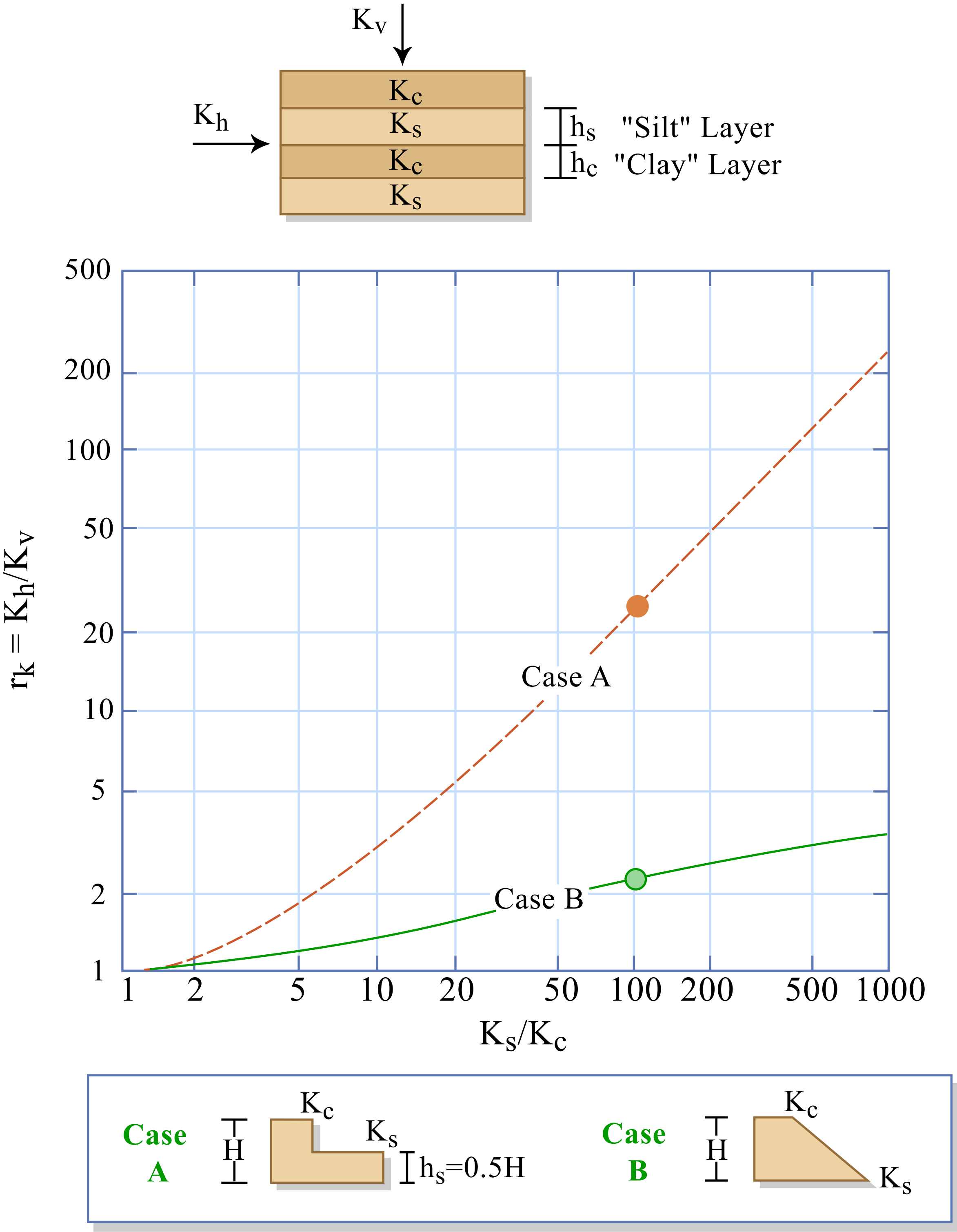
Coefficient of Consolidation (Vertical Drainage) vs. Natural Water Content for Normally Consolidated Varved Clays

* See Ladd (1975) for data sources

** From square root time method

1 ft^2/day = 0.093 m^2/day

Figure by MIT OCW.

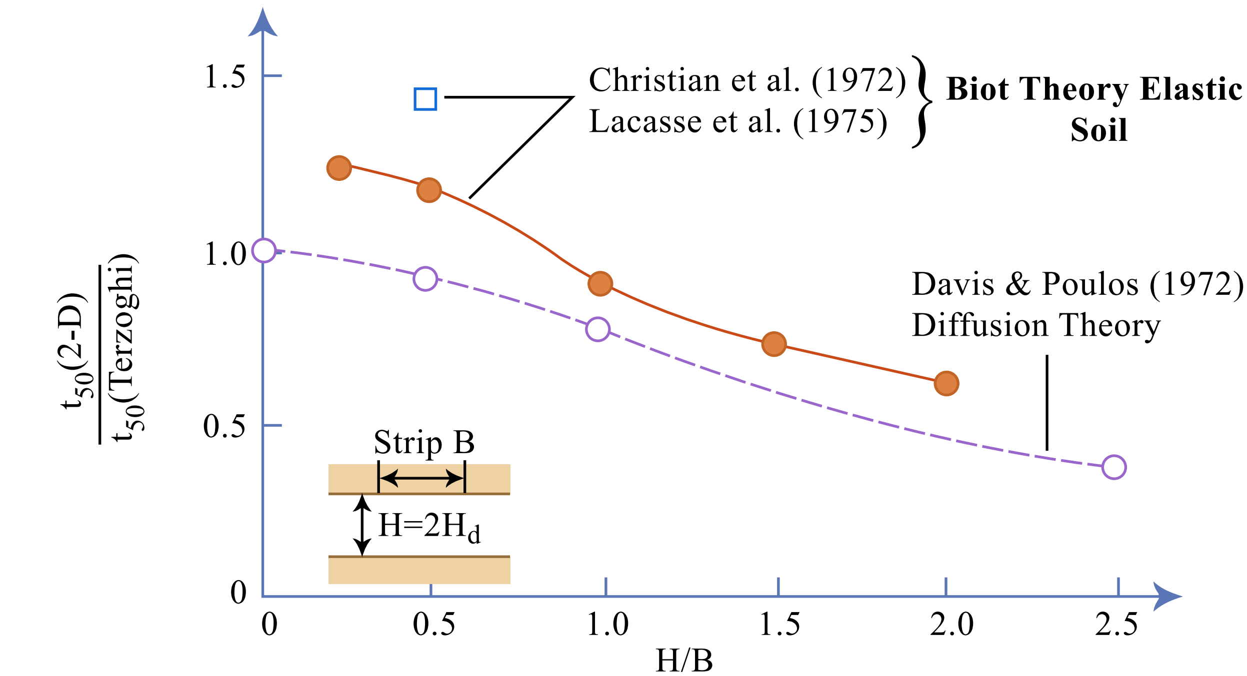


Relationship Between K_h/K_v Ratio and Permeability of the Silt and Clay Layers

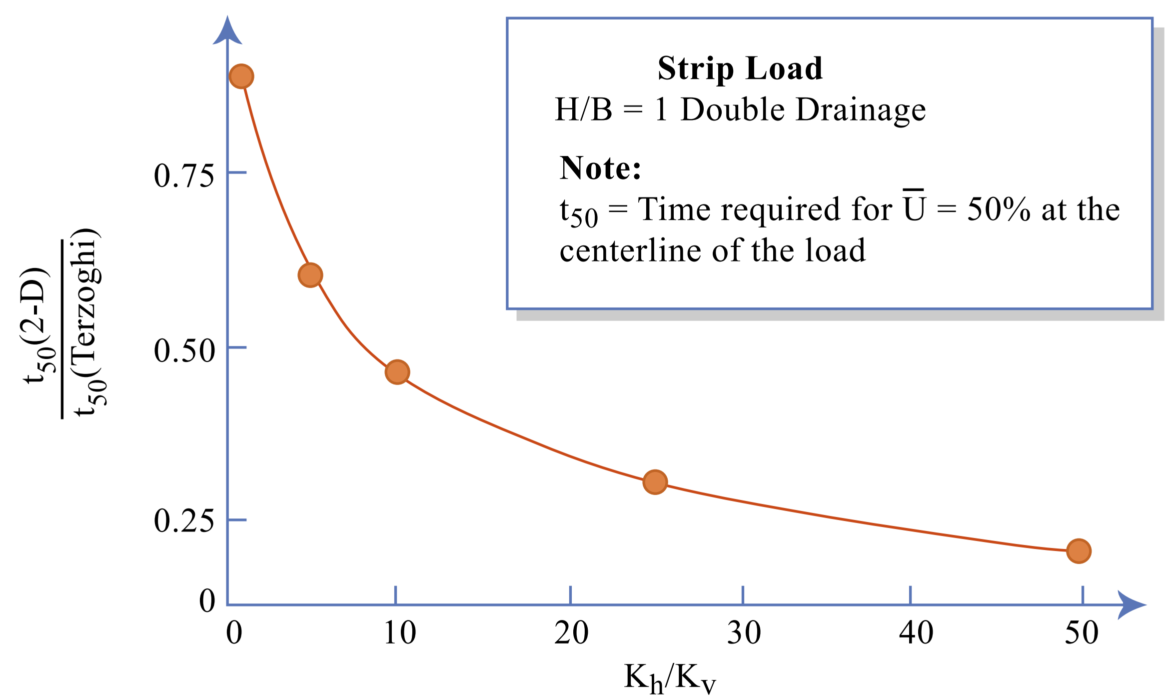
Figure by MIT OCW.

(After Kenney, 1963)

V9

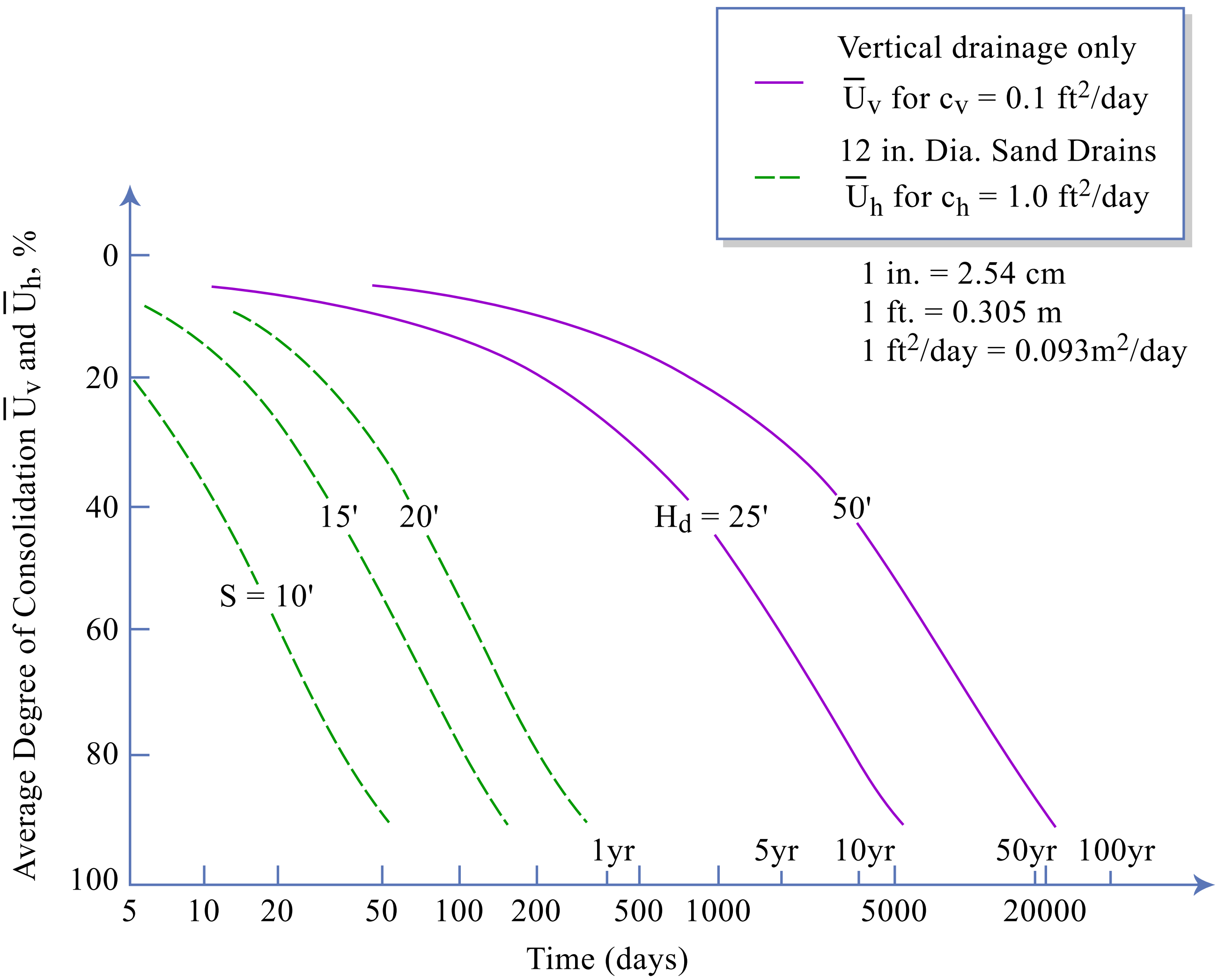


(a) Effect of Lateral Drainage on Rate of Consolidation from Different Theories with Isotropic Permeabilities



(b) Effect of Anisotropic Permeability Ratio on Rate of Consolidation for H/B =1 with Double Drainage

Figure by MIT OCW.



Effect of Sand Drains on Rate of Consolidation of Normally Consolidated Varved Clay with c_h/c_v = 10

Figure by MIT OCW.

METHODS FOR MEASUREMENT OF c_h AND k_h/k_v

NO.	METHOD AND PARAMETER	REMARKS	REFERENCE
1	Laboratory consolidometer test on horizontal ($\theta=90^o$) sample (c_h)	(1) Wrong m_v (2) Sample size influences results	Rowe (1959)
2	Laboratory consolidometer test with radial drainage to sides (c_h)	May have problems with side friction and scale effects	McKinley (1961)
3	Laboratory consolidometer test with radial drainage to vertical sand drain (c_h)	Large sample recommended to minimize scale effects	Rowe & Barden (1966) Shields & Rowe (1965)
4	Laboratory permeability tests on vertical and horizontal samples (c_h)	Problem with variability when using different samples	
5	Laboratory permeability tests on cubic sample (k_h/k_v)	Better than No. 4; large (10 cm) samples recommended	Chan & Kenney (1973) Ladd & Wissa (1970) Saxena et al. (1974)
6	Field constant head flow tests with hydraulic piezometer (c_h, k_h)	(1) Method of installation important (2) Need to consider length to diameter ratio	Mitchell & Gardner (1975)
7	Field pumping test from vertical sand drain (k_h)	(1) Method of installation important (2) Pervious layers can have important effect	Casagrande & Poulos (1969)

Table C-1

Ladd & Foott (1977)

VII

SUMMARY OF ANISOTROPIC PERMEABILITY RATIOS FOR VARVED CLAYS (Ladd, 1975)

LOCATION	LABORATORY k_v (10^{-8} cm/sec)	ANISOTROPIC PERMEABILITY RATIO: Method	k_h/k_v	Remarks	REFERENCE
1 New Liskeard Canada (Thick varve Layer)	7 (5.2 - 9.5)	Lab: 2.5" Cube	3.3 ± 0.4	6 Samples, Avg. Varve 1.6" Thick	Chan & Kenney (1973)
		Field: Flow Pattern in Natural Slope	< 5	Upper Limit	Kenney & Chan (1973)
			3	Best Estimate	
2 Amherst, Mass.	9	Lab: 4" Cube	4.5, 8, 11	Increase with depth d = 28,43,67'	M.I.T.
3 Northampton Mass.	6 ± 2	Lab: 2" Cube	5 (3.3-6.3)	4 Tests; El.79'	Ladd & Wissa (1970)
		Lab: 4" Cube	14, 10	2 Tests; El.59', 51'	
		Field: Inferred from Settlement Data	60 ± 30	May be Questionable	Connell et. al. (1973)
4 East Windsor, Conn.	20	Lab: k on Separate Samples	6 - 7	Quoted	Healy et. al.(1970)
			4 - 6	Est. by Writer	
		Lab: c_h/c_v from Consolidometer Tests	≤10	Est. by Writer	
5 Secaucus, N.J.	5 - 10	Lab: 4" Cube	5 ± 2.5	2 Samples Varves Inclined	Saxena et. al.(1974)
6 Section 7A, New Jersey Turnpike	17 ± 7	Lab: 2" Samples	20 ± 10	d = 15 - 40'	Casagrande and Poulos (1969)
	5 - 6		10 ± 5	d = 50 - 75'	
		Field: Pumping Test from Instrumented Jetted Sand Drain	500 ± 300	d = 22 - 85' Values by Writer	

1 in. = 2.54 cm 1 ft = 0.305 m

Table C-2

V/2

CCL 3/30/86 3/87 3/88 4/89 3/98 3/99 1.322 VI Consolidation: PROBLEM SOILS (Pet1)

Some References on "Problem" Soils

Highly Structured & Sensitive

Hypothesis B using changes $\sigma_v' - \varepsilon_v - \dot{\varepsilon}_v$ relationship

1) Mesri & Choi (1985) ASCE, JGE #4 - Computer program
2) Leroueil et al (1983) CGJ #4 - Estimating σ_p'
3) Leroueil et al (1985) Geotechnique #2 - Lab compressibility
4) Kabbaj et al (1988) Geotechnique #1 } in situ compressibility; Several case histories
5) Leroueil et al (1984) Soils & Fdn #3
6) Leroueil (1988) CGJ 25(1) State of the Art
7) Liu & Carter (1999) "Virgin compression of structured soils" Geot. 49(1)

Peats

1) MacFarlane, I. C. (1969), Muskeg Eng. Handbook, U. Toronto Press
2) Skempton & Petley (1970), Geotechnique, #4 - Index properties
* 3) Lefebvre et al (1984), CGJ, #2 - Lab & field data NBR peats
4) ASTM (1983) STP 820 Testing of Peats & Organic Soils
5) Mesri et al (1997) "Secondary compression of peat w/ & w/o surcharging" JGGE, 123(5)

Collapsing - Loess

1) Holtz & Gibbs (1961), 5th ICSMFE, Vol. I, p 673 - Settlement
2) Clemence & Finbarr (1981), ASCE, JGED, #3 - Design & references
3) Houston et al (1988) ASCE, JGE, #1 - Case history

Expansive Clays

1) Ladd & Lambe (1961) 5th ICSMFE, Vol I, p 201 - Correlations
2) O'Neill et al. (1980) ASCE, JGED, #12 - Good references
3) Nelson & Miller (1992) Expansive Soils: Problems and Practice in Fdn & Pavement Eng., J. Wiley

Residual Soils

1) 1.322 handout w. Martin (1977), ASCE, JGED, #3 - SPT - E
2) Townsend, F.C. (1985), ASCE, JGE #1 Updated references

Varved Clays

1. Ladd (1987) Lecture Notes, NY ASCE Met. Section
2) Ladd & Foott (1977) "Fdn design emb. - ", FHWA TS-77-214, USDOT

4/97
3/99
4/01

Strength-Deformation Behavior of Saturated Clays and Drained/Undrained Stability (Parts D & E of Outline)

I STABILITY PROBLEMS AND DRAINED STRENGTH PARAMETERS

Handout Sheets

A. Classes of Stability Problems & Types of Stability Analyses — (I A1,2)

- Review of 1.361 Part V-3

B. Determination of Effective Stress Failure Envelopes for CD Case — (I B)

1. Use of CD & CU Tests — 1,2
   1) CD DS 2) CD TX 3) CU TX
2. Miscellaneous
   1) Variation in ESE: OCR=1 — 3
   2) " " " : High OCR — "
   3) Common triaxial testing problems — "
   4) Comparison of ESE and correlations — 4-6

C. Long Term (CD Case) Stability: Problem Soils — (I C)

C.1 Stiff Fissured and Stratified Clays & Clay Shales

1) Introduction — 1
2) Definition of 3 envelopes (peak, fully softened & residual) — 2
3) Measurement of residual envelope — 3
4) Overview of fully softened vs residual envelopes — 4
5,6,7) Recommendations for selecting c' & φ' as per ≈1995 and results from recent research — 4-7
8) Basic research on ϕ'_r — 8,9
9) Empirical correlation — 5,10,11

C.2 Highly Structured, Sensitive Clays (Quick Clays)

1) Background — 12
2) Norway — 12
3) Quebec — 11,13

C.3 Conn. Valley Varved Clays — 14

Mini-Problem No. 1 on Strength of Clays

Topic in HO Notes	Approx Date	Questions
I B Measurement of c' & ϕ'	4/4/01	1) What can cause major errors in the measurement of c' & ϕ' for CD analyses of homogeneous cohesive soils
I C Problem Soils	4/8/01	1) For cuts in stiff fissured & stratified soils a) When safe to use peak envelope? b) " " " " NC " ? c) When must use ϕ'_r? d) When get combination of above? Are these 3 envelopes curved? (σ'_{ff} = 50-400 kPa)? 2) What is effect of using undisturbed vs remolded clay on value of ϕ'_r 3) For cuts & natural slopes in quick clays, is CD ESA w/ equilibrium & peak ESE safe?
II A s_u for UU Case	4/9/01	1) FVT: a) Why does μ decrease with increasing PI? b) What soils → unsafe to use μ? 2) CPTU a) How is N_k determined? b) What is the major cause of problem in getting consistent q_t profiles in soft clays? 3) DMT: How reliable is $s_u/\sigma'_{v0} = 0.22\,(OCR)^{0.8}$ where $OCR = (0.5K_D)^{1.56}$? 4) When would you replace unconf compression test with a UUC test ($\sigma_c = \sigma'_{v0}$)? 5) Did Bishop & Bjerrum (1960) conclude that both UUC & s_u(FV) → reliable s_u for UU case?

Class Schedule & Reading Assignments: STABILITY & STRENGTH OF COHESIVE SOILS

◯ = STUDY

Topics	Handout Notes	'77 SOA Tokyo	'85 SOA S.F.	CCL '91 TL	Other	Comments (≈ No. Classes)
1) Stability Classes & Types of Analyses	IA			1, 2	1.361 IV3-1&2	(2+)
2) CD Case · Measurement c' & φ' · Problem Scale	IB IC					
3) UU Case: Std. Practice & In Situ Testing	IIA	4.2	3.2, 3.3	4, 6	1.361 IV4-1&2	(1-)
4) Sample Disturbance	IIB	2.2.7	2.3	4	Above	(2)
5) Stress System & Anisotropy (Combined old & new notes)	IIC	2.2.2	2.4	4	—	(4±) Discuss HP#4 Q&A MTE 4/18
6) Time Effects: Strain Rate & Creep	IID	2.2.6	–	–	-	(2+)
7) CU Case	IIE	–	–	3, 5 & 6	Koutsoftas & Ladd (1985)	(1)
8) Home Problem on Design: Discussion						(1)

NOTE: There will be a series of mini-problems, mostly in the form of questions for class discussion on Topics 1 → 7. Topic 7 will have a major design problem.

4/97
3/98
4/01

Strength-Deformation Behavior of Saturated Clays and Drained/Undrained Stability (Parts D&E of Outline)

I STABILITY PROBLEMS AND DRAINED STRENGTH PARAMETERS

Handout Sheets

A. Classes of Stability Problems & Types of Stability Analyses (IA1,2)

· Review of 1.361 Part V-3

B. Determination of Effective Stress Failure Envelopes for CD Case (IB)

1. Use of CD & CU Tests 1,2
   1) CD DS 2) CD TX 3) CU TX

2. Miscellaneous
   1) Variations in ESE: OCR=1 3
   2) " " " : High OCR "
   3) Common triaxial testing problems "
   4) Comparison of ESE and correlations 4-6

C. Long Term (CD Case) Stability: Problem Soils (IC)

4/2/01 CHARLES C. LADD

1
"
"
2

Wed 4/4/01 since 3,4

rewriting-updating not 3,5,6

yet finished 7,8

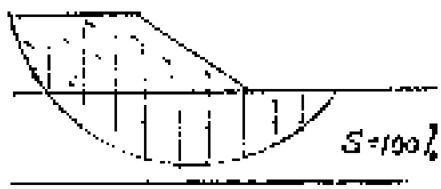
9

3/30/92 4/3/95 4/2/96

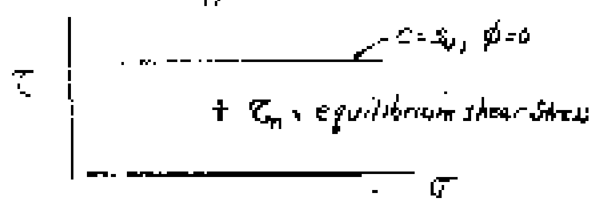
IA: CLASSES OF STABILITY PROBLEMS AND TYPES OF STABILITY ANALYSES (Sheet A2; Table 6)

CASE 1 = UU Case (Undrained)

Embankment on Soft Clay



Typical Slice



Conventional Practice to Est. s_u
In Situ FVT CPT etc
Lab UUC LV TV etc.

Also BC $q_{ult} = cN_c + \frac{1}{2}\gamma B N_\gamma + \gamma d N_q$

TSA = Total Stress Analysis   NOTE: Also can/should use USA

CASE 2 = CD Case (Fully Drained)

ESA = Effective Stress Analysis (Drained Strength Analysis)

- Slow const. or long time → $u_e = 0$
- Slow failure → $u_s = 0$

Sheet A2, Fig. 1

For same FS on c' & ϕ' →
$FS = \tan\phi' / \tan\phi'_m$

Next: Testing → c' & ϕ' (IB)

CASE 3 = CU Case (Partial drainage prior to Undrained Failure)

Sheet A2, Fig 3

- $u_e \geq 0$
- $u_s > 0$

USA = Undrained Strength Analysis
$c_u = f(\sigma'_{vc}\ \&\ \sigma'_p)$

CCL Methodology - Use CK_0U
QRS " " UU & CIU (Sheet A2, Fig. 2)

CRITICAL CONDITIONS (à la 1.361)

Loading (Construction → $+\Delta p$)

- Footings, tanks, emb., dams ...
- UU critical since $+u_e$ & drainage → incr. str. (esp. low OCR)

Unloading (Construction → $-\Delta p$)

- Excavations, etc.
- CD critical since $-u_e$ & drainage → decr. str. (esp. high OCR with $-u_s$)

TABLE 6. Stability Problems Classified According to Drainage Conditions and Definition of Factor of Safety

Case (1)	Common description (2)	Proposed description (3)	Proposed classification (4)	Definition of factor of safety[a] (5)
1	Undrained, short-term or end-of-construction	No consolidation of soil with respect to applied stresses and undrained failure	Unconsolidated-undrained = UU case	s_u/τ_m or c_u/τ_m (Eq. 5 or Eq. 8)
2	Drained or long-term	Full consolidation of soil with respect to applied stresses and drained failure ($s_d = 0$)	Consolidated-drained = CD case	$s_d/\tau_m = \tan\phi'/\tan\phi'_m$ (Eq. 7)
3	Partially drained or intermediate	Partial or full consolidation of soil with respect to applied stresses and undrained failure	Consolidated-undrained = CU case	c_u/τ_m (Eq. 8)

[a] τ_m = mobilized shear stress required for equilibrium; s_u = undrained shear strength obtained from conventional testing associated with typical $\phi = 0$ analyses; c_u = undrained shear strength obtained from techniques recommended in Section 5; and s_d = drained shear strength defined in Eq. 1.

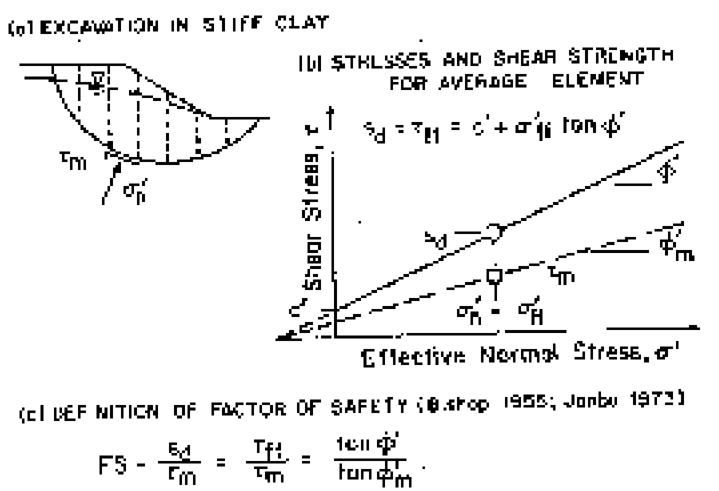


FIG. 1. Conventional Effective Stress Analysis Applied to Critical CD Case for Unloading Problem

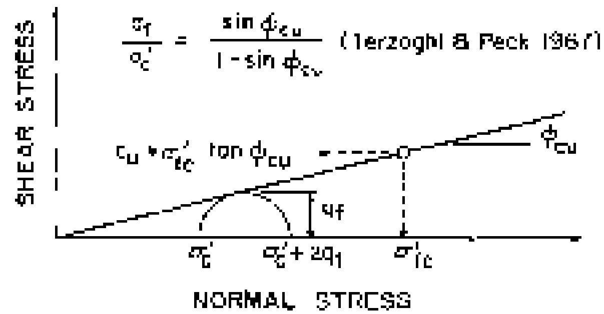


FIG. 2. Angle of Shearing Resistance ϕ_{cu} from Isotropically Consolidated-Undrained Triaxial Compression (CIUC) Tests as Defined by A. Casagrande

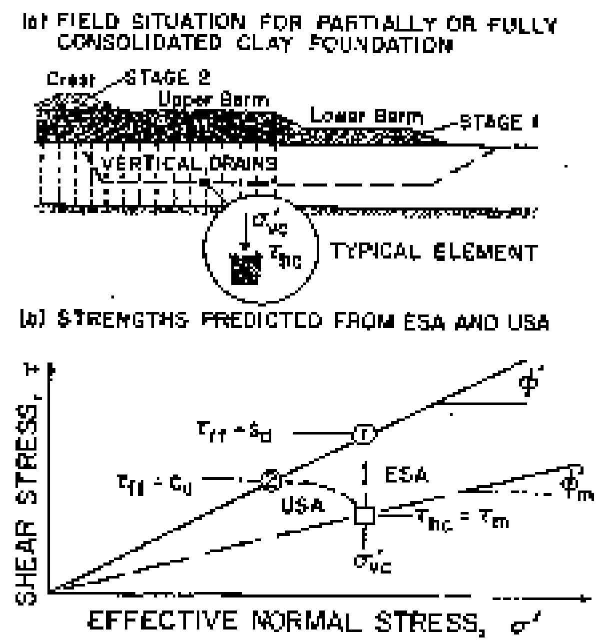


FIG. 3. Comparison of Effective Stress and Undrained Strength Analyses for Evaluating Stability during Staged Construction

IB DETERMINATION OF EFFECTIVE STRESS FAILURE ENVELOPE FOR CD CASE (Saturated Natural Cohesive Soils)

1. USE OF CD & CU TESTS

Note: Limited data on "ordinary" clays indicates that t_f has little effect on values of c' & ϕ' (i.e., using $t_f > t_{100}$ required obtain $u_s \approx 0$)

1.1 CD Direct (Box) Shear Tests

a) Advantages

- Simple equipment & easy to run
- Low cost
- Short $t \leq 1$ day

b) Disadvantages

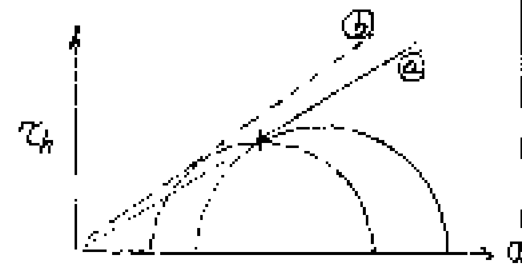
1) Non-uniform stress-strain conditions → no τ vs γ data

2) Unknown stress condition at failure

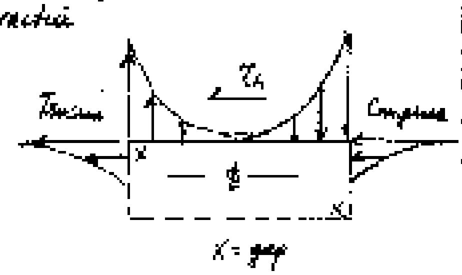
ⓐ $\tau_h = \tau_{ff}$; $\delta = 45 + \phi'/2$; $\tau_h/\sigma_v' = \tan\phi'$

ⓑ $\tau_h = |q_f|$; $\delta = 45°$; $\tau_h/\sigma_v' = \sin\phi'$

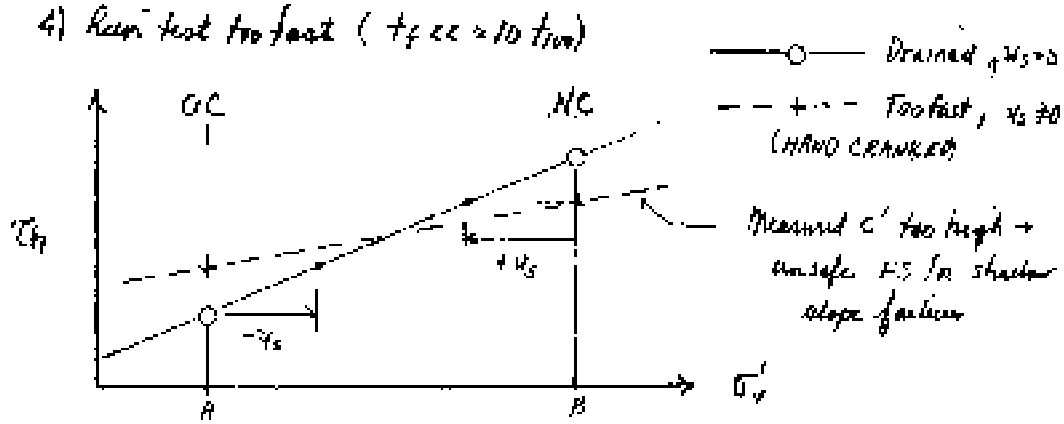
$\tau_h/\sigma_v' = 0.5 \rightarrow \phi' = 26.6°$ ⓐ ← Std. practice
$= 30.0°$ ⓑ



3) Tilting at high τ_h/σ_v'
- Tensile σ at leading portion
- Compressive σ at trailing portion



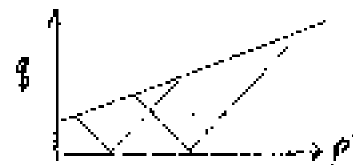
4) Run test too fast ($t_f << \approx 10\ t_{100}$)



1.2 CD Triaxial (Usually CIDC & U since K_c does not affect c' & ϕ')

a) Advantages

- Know stress conditions and meaningful stress-strain data
- Can vary ESP to define ESE at low stress
- Most reliable



b) Disadvantages

- More complex equipment & harder to run
- Much longer time (1-2 weeks) & more expensive

1.3 CU Triaxial (Usually CIUC)

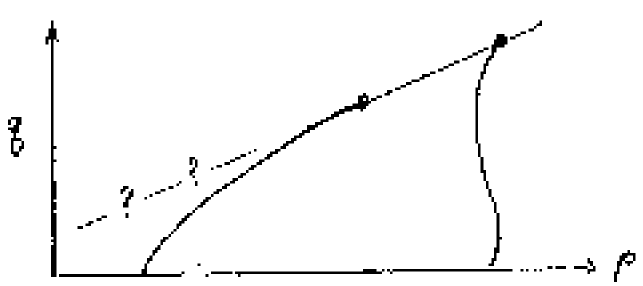
a) Advantages

- Obtain information on undrained behavior for CU case
- Less time than CIDC

b) Disadvantages

1) Procedures more complex to ensure reliable u & σ' data

2) Cannot define ESE at low σ' for high OC soils



(Also have curved ESE, see p IB6)

3) Varying ESE at q_f, max obl & tangency (see 2.1 & 2.2)

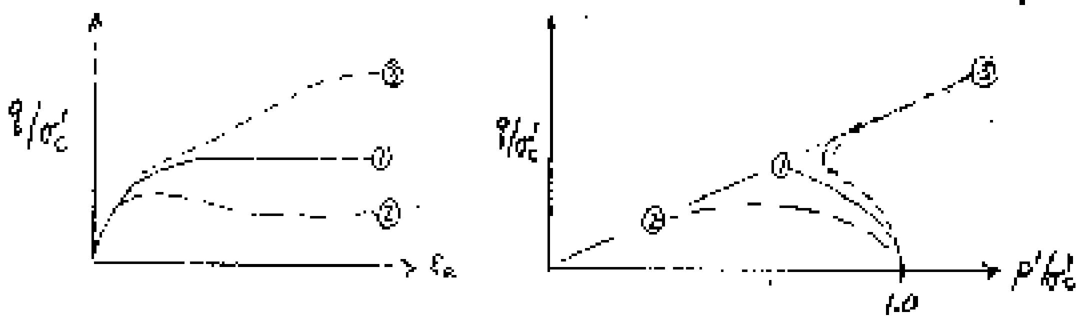
Note: generally use max obl. or tangency to estimate c' & ϕ' for CD case

4) Potential large unsafe error in c' if do not obtain pore pressure equalization (See 2.3)

2. MISCELLANEOUS

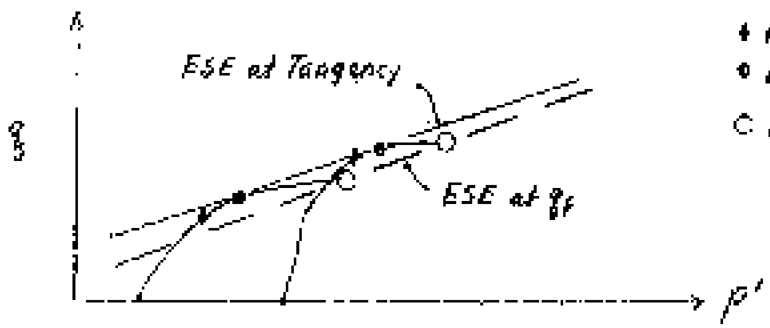
2.1 Variation in ESE : OCR=1 (CIUC Tests)

① Simple clay type behavior : $\phi'_{qf} = \phi'_{mo}$ ② Sensitive clay : $\phi'_{qf} < \phi'_{mo}$ ③ Arctic Silt : not really NC
Cohesionless (stress + φ)



- Value of $q = f$(mobilized ϕ') × (magnit. of p') ; High ε_a, $\phi'_{qf} < \phi'_{mo}$ by 5-10°
  Incr. w/ ε_a Decr. w/ ε_a
- CDC ϕ' (q_f : mo) ≈ CIUC ϕ'_{mo} − (0-3°)

2.2 Variation in ESE : High OCR (CIUC Tests)



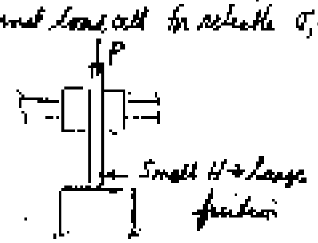
• At Max. Obl. } Similar
• At tangency }
○ At q_f → lower ESE

- Normally select ESE at Max. Obl. or tangency to estimate ESE for CD case
  But extrapolated envelope is TOO HIGH at low p'_f (See IB6)

2.3 Common Triaxial Testing Problems (Germaine / Ladd, 1988 ASTM STP 977 P.421)

a) Piston Friction (CU / CD)

- Need ball bearing - rolling diaphram or internal load cell for reliable $\sigma_1 - \sigma_3$
- Solid bushing liable → serious errors

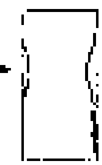


b) Filter Strips (CU/CD)

- Compression : 10cm² x 8cm Typical correction Δq = 100psf = 5kPa
- Extension : Need spiral + precut notches + maximum flexibility

c) Area Correction (CU/CD)

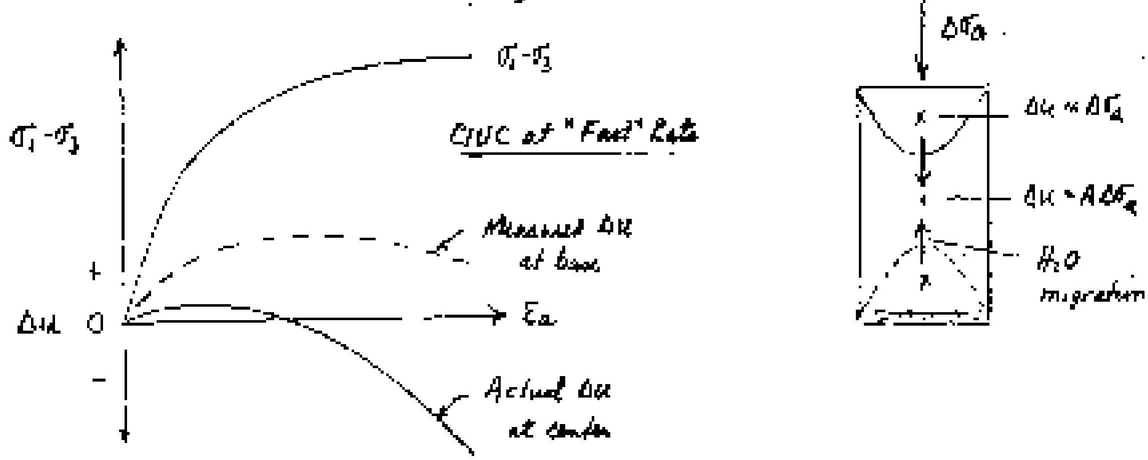
- Compression : See GSL (B8) for cylinder, parabolic & bulging
- Extension : Discount data when noticeable necking occurs →

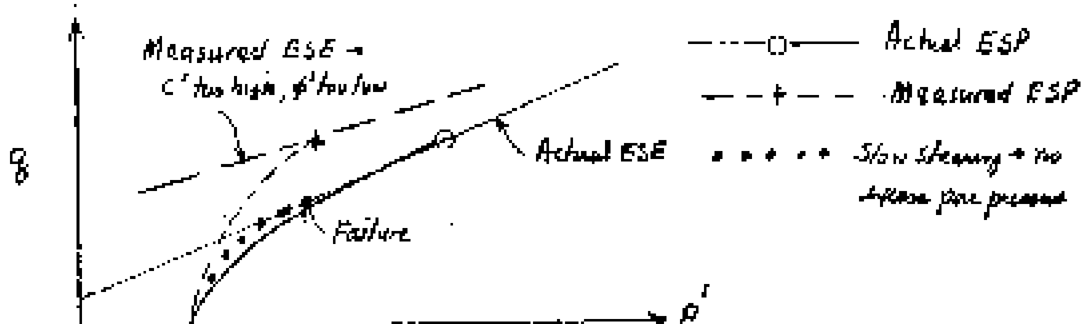


d) Saturation (CU)

- Need min. u_b = 2-3 atm - Always check that B ≥ ~ 95% few minutes

e) Frictional Ends - Pore Pressure Equalization at "High" OCR (CIUC)





Correct ESE requires : 1) Either u measured at ℄ of test (+ correct s_u)
(with frictional ends) 2) Or very slow if u measured at base (≈ too low)

CCL 4/1/94 1.322
4/2/01

IBS

2.4 Comparison of ESE and Correlations

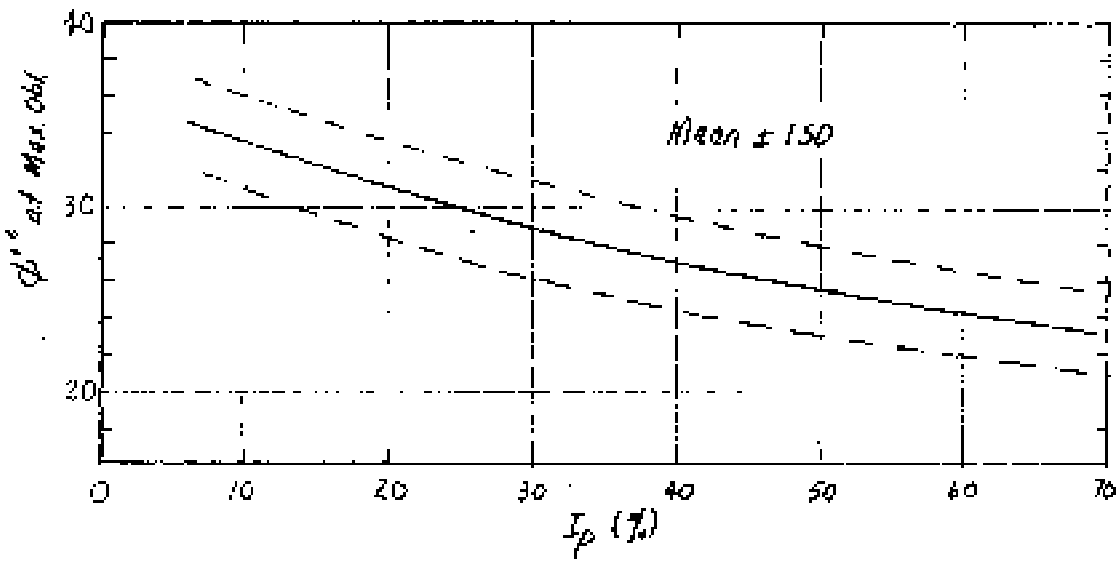
a) Natural BBC : CK_0UC/E ; OCR = 1.5–6 ; Max. Obl. (CA/T Project)

	SHANSEP c'/σ'_{vm}	SHANSEP ϕ'	Recompression c'/σ'_p	Recompression ϕ'
TC	0.017	28.5	0.044	29
TE	0.055	19	0.031	27
	Large difference TC vs TE		Similar values TC vs TE	

NOTE: Will discuss different between TC vs TE more fully under IIC

b) Friction Angle vs Plasticity Index : Normally Consolidated Soils

1) HAVDOCKS Day·7 (1961)



2) Mesri & Abdel-Ghaffar (1993) JGE, ASCE, 119(8), 1216–1249

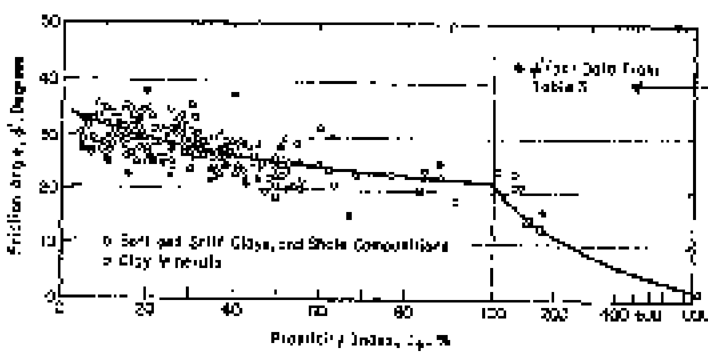


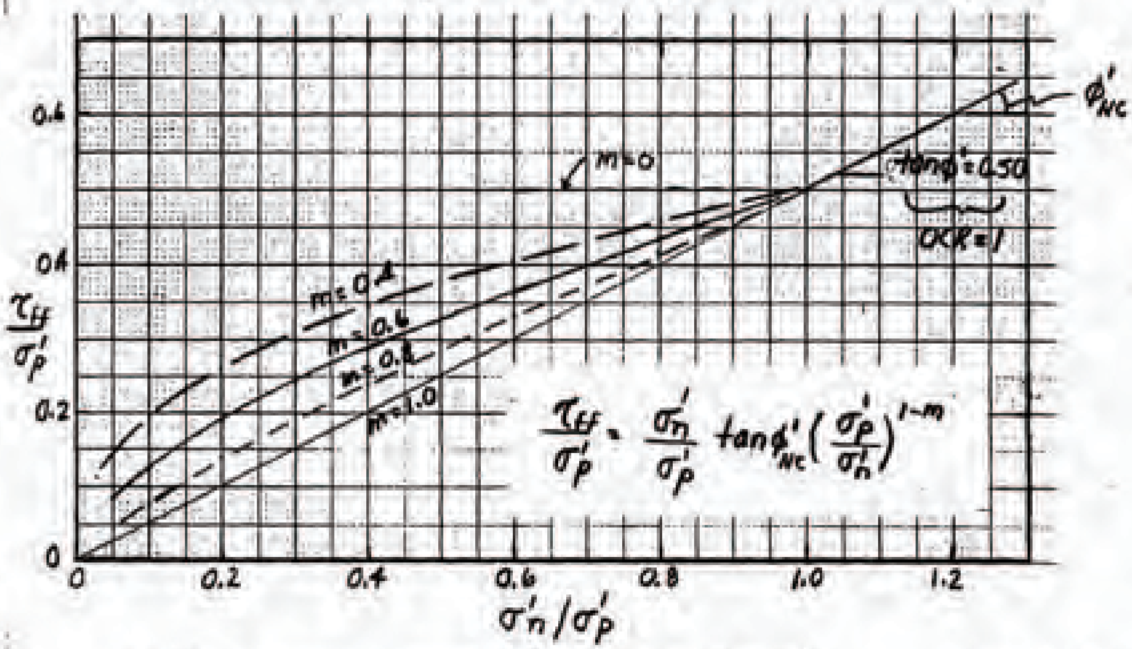
FIG. 2. Values of Friction Angle φ' for Natural Clay Compositions

ϕ' for OC clay

Data mostly from:
CIUC
CIDC
CD DS

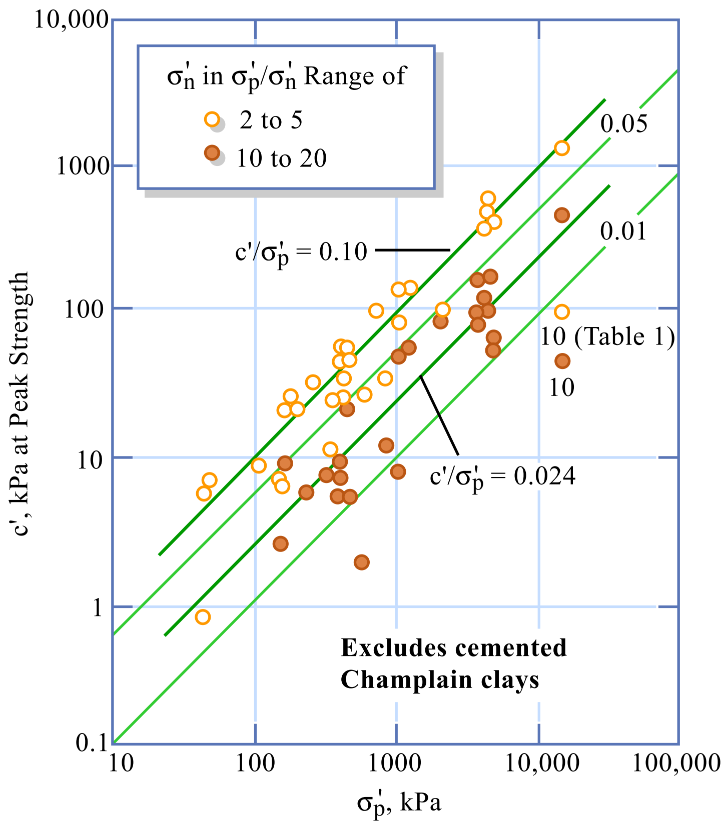
4/2/96
3/25/99

c) Magnitude of c'/σ'_p from Mesri & Abdel-Ghaffar (1993)



* Shows increasing curvature of ESE as function of $m = d\log\tau_{ff}/d\log\sigma'_n$ for $\sigma'_n \le \sigma'_p$

} Note: Fig. 8 of paper shows $m \approx 0.83 - 0.4\, I_p$ for $I_p = 20-80\%$



Relationship between Cohesion Intercept and Preconsolidation Pressure

Figure by MIT OCW.

CCL Conclusion For Mechanically OC Clays:

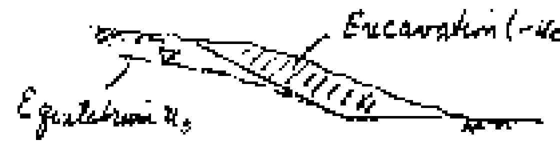
	σ'_n/σ'_p	c'/σ'_p
o	0.2-0.5	0.05-0.1
•	0.05-0.1	0.03±0.02

LONG TERM (CD) STABILITY: PROBLEM SOILS

C1 STIFF FISSURED & STRATIFIED CLAYS AND CLAY SHALES

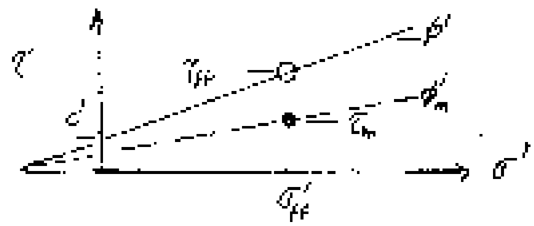
1.1 Introduction

1) Critical condition



$s_f = \tau_{ff} = c' + (\sigma - u_s)\tan\phi'$

$FS = \frac{s_f}{\tau_m} = \frac{\tan\phi'}{\tan\phi'_m}$



2) Values of c' & ϕ' to use in analysis depend on:

a) 1st time vs prior (reactivated) slide

b) Homogeneous (also partially low PI & CF = clay fraction)

vs. Non-homogeneous (NH): stiff clay & clay shales that contain

(1) Fissures = small, random oriented discontinuities (like closed cracks; some may be slickensided = "polished")

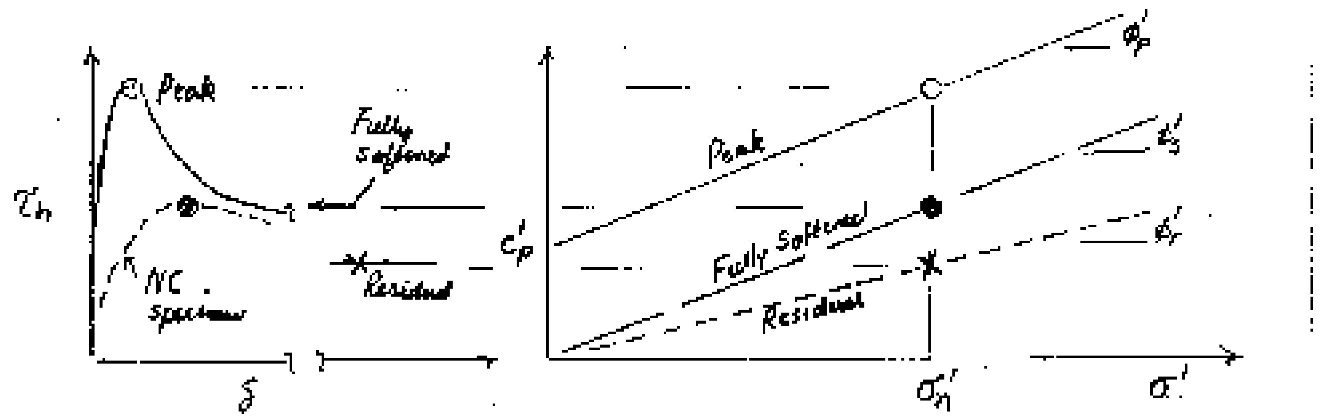
Stratified — (2) Bedding planes + laminations: these are especially important if more plastic than "bulk" soil and have a higher in situ degree of parallel particle orientation

NOTE: NH can have either or both (plus other features such as joints and faults, although these more typically associated with rock) ...

3) To appreciate problem with selection of c' & ϕ', need to understand differences in ESE as function of degree of shearing

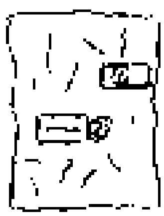
1.2 Definition of 3 Envelopes (e.g. Skempton 1964, Geot. 14(1), 77-101)

Results from CD DS tests on stiff, fissured London Clay (LL=80, PI=50, CF=55)



1) Peak Envelope (c'_p & ϕ'_p)

- Magnitude = function of size of specimen if fissured
  = parallel fr incidence to stratification
- Example for stiff, London Clay (Skempton & Hutchinson 1969, ICSMFE)



① Intact	$c'_p = 1500$ psf	$\phi'_p = 28°$
② Along fissure	" = 140	" = 18.5°

2) Residual (ϕ'_r) [Note: Will later see that actually curved]

- Shear to very large displacements leading to maximum orientation of platy particles.
- If CF > 50% & high PI, get smooth "polished" surface

3) Fully Softened (ϕ'_s) [Note: also may be curved]

- Corresponds to "critical state", i.e. peak strength from shearing OCR=1 specimen = steady state strength of OC specimen
- Skempton also postulated that have "minor shears not yet linked into a continuous surface" at fully softened state

1.3 Measurement of Residual Envelope

1) Is a Material Property.

- Same value whether test Undist. vs remolded or NC vs OC
- Little effect of strain rate (unless very fast)
- But must shear sufficiently to obtain max particle orientation. ∴ Plot τ/σ_n' vs $\log\delta$ (displacement)

← δ to reach residual
≈ 1-5 cm precut
≈ 5-50 cm intact

2) Testing methods

a) Repeated direct shear, i.e., shear to δ_i, push back, shear again, etc.

- Either precut natural (to greatly reduce δ)
- Or remold { consolidate on plate (...) }

} Both used

b) Rotational shear = Ring shear

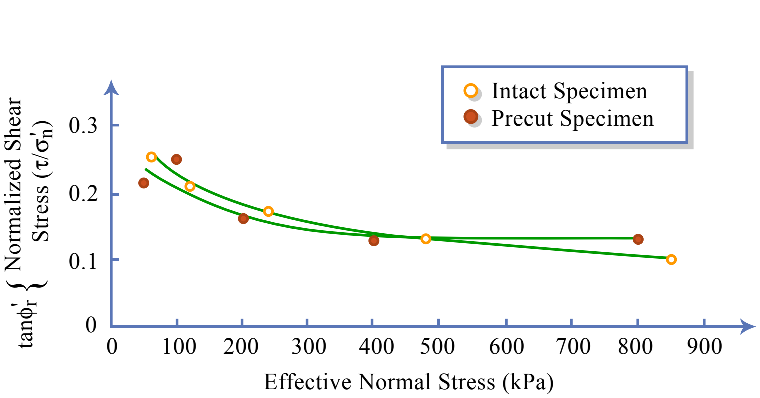
OD	ID	t (cm)	
7.1	5.1	0.75	Harvard (La Gatta 1970)
15.2	10.2	1.9	Imperial College & NGI
10	7	0.5	Stark & Eid (1993), U of I (Modified Bromhead ring shear)

} Best procedure → minimum envelope

$\boxed{\text{Typical } d\delta/dt = \dot{\delta} \approx 1 \text{ cm/day}}$

3) Some results

- See Fig 14 below → same result for precut vs intact (needed much larger δ). Also note reducing τ_r/σ_n' with increasing σ_n' = curved envelope

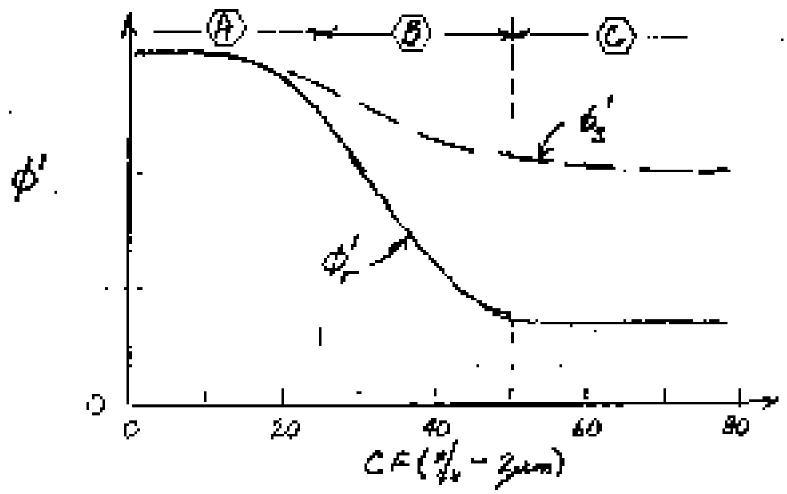


Relationship between normalized shear stress and effective normal stress for Portuguese Bend bentonitic tuff. Ca Mont. LL = 98 PI = 61 CF = 68

Figure by MIT OCW. Adapted from: Stark & Eid (1993) GTJ, ASTM 16(1)

1.4 Overview of Softened (Critical State) and Residual ϕ'

Lupini et al. [1981, Geot 13(1)] Skempton [1985, Geot. 35(1)]



Ⓐ Granular particles in contact (clay fills voids) with $\phi_r' \approx \phi_s'$ (rolling shear)

Ⓒ Granular particles "floating" in clay matrix with
- ϕ_r' due sliding shear & controlled by clay minerals
- ϕ_s' much higher

Ⓑ Transitional zone with increasing difference between ϕ_s' & ϕ_r'

1.5 Recommended Selection of c' & ϕ' Until Approx. Mid-1990s

(Mostly by Skempton, e.g. 1970 [Geot 20(2)] & 1977 [9th ICSMFE, Vol 3]

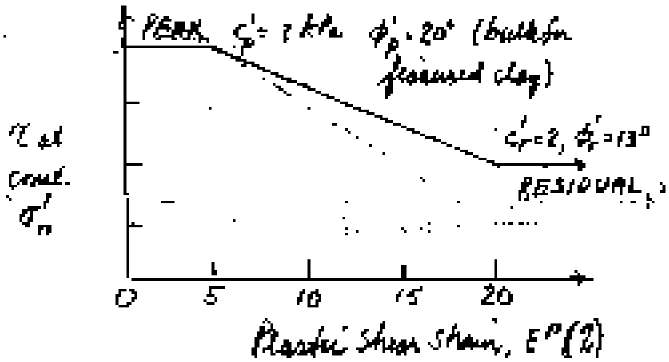
1) Along PRIOR FAILURE SURFACE having movements of ≈ 1-2 m ...
   Must use ϕ_r' independent of age of prior failure
   - CD DS tests on block sample from 10^4 yr old failure at Mangla Dam → $\tau_{max} = \sigma_n' \tan\phi_r'$

2) 1st time failure, HOMOGENEOUS CLAY (no fissures or stratification, etc)
   Can use c_p' & ϕ_p' (CCL NOTE: Only if shearing does not → strain softening after peak, which is not likely for OC clay)

3) 1st time failure, FISSURED CLAYS
   - For London clay, use $c' = 0$ & $\phi' = \phi_s'$, i.e. softening of fissures due to swelling and localized straining → fully softened condition (empirical observation from back analysis of case histories)
   - For some fissured clays, may get $\phi' < \phi_s'$,
   - e.g. Stark & Eid [1997, JGGE 123(4)] - Analysis of 14 failures involving stiff fissured clays → $\tau_m = \frac{1}{2}(\tau_s + \tau_r)$ for LL > 60%, (i.e. half way between fully softened & residual)

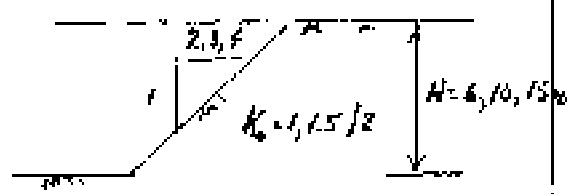
1.6 Results of Research by Potts et al (1997) "Delayed collapse of cut slopes in stiff clay" Geot 47(5), 953-982

1) Conducted coupled FE analyses (ie, included k) of cut slopes in a strain softening clay (patterned after London clay)



Boundary Conditions

* Surface suction = 10 kPa



Note: Also did analyses without strain softening → longer t_f and higher $\bar{r}_u$ = average $r_u = u/\sigma_v$ on rupture surface. Inc. suction also → much longer t_f. (ie, vegetation on slope helps)

2) Results for H = 10 m, 1V:3H as $f(K_o)$: Collapse = failure when analyses showed abrupt increase in S_h at mid-slope

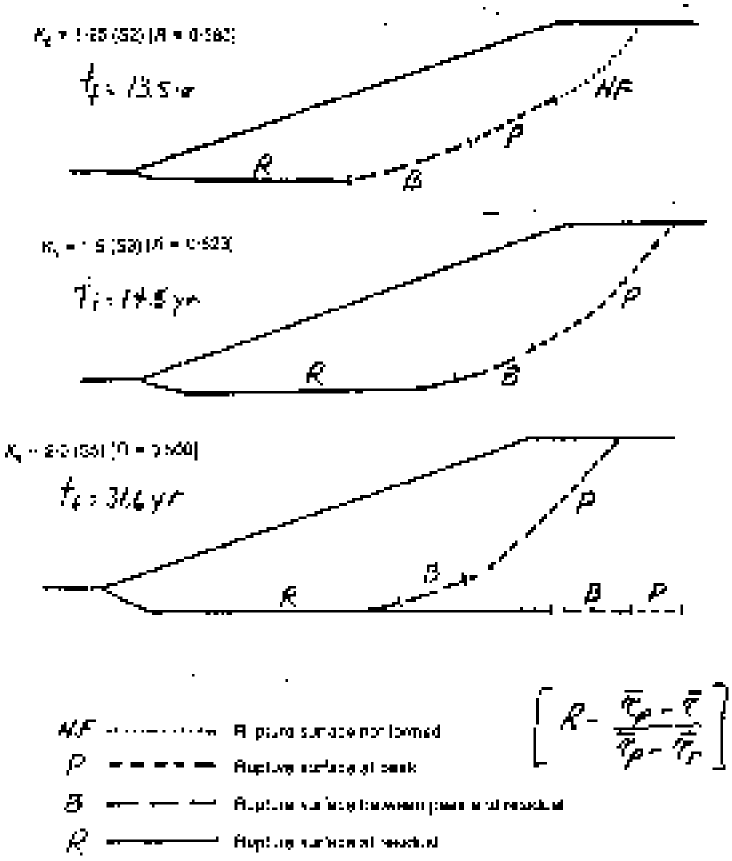
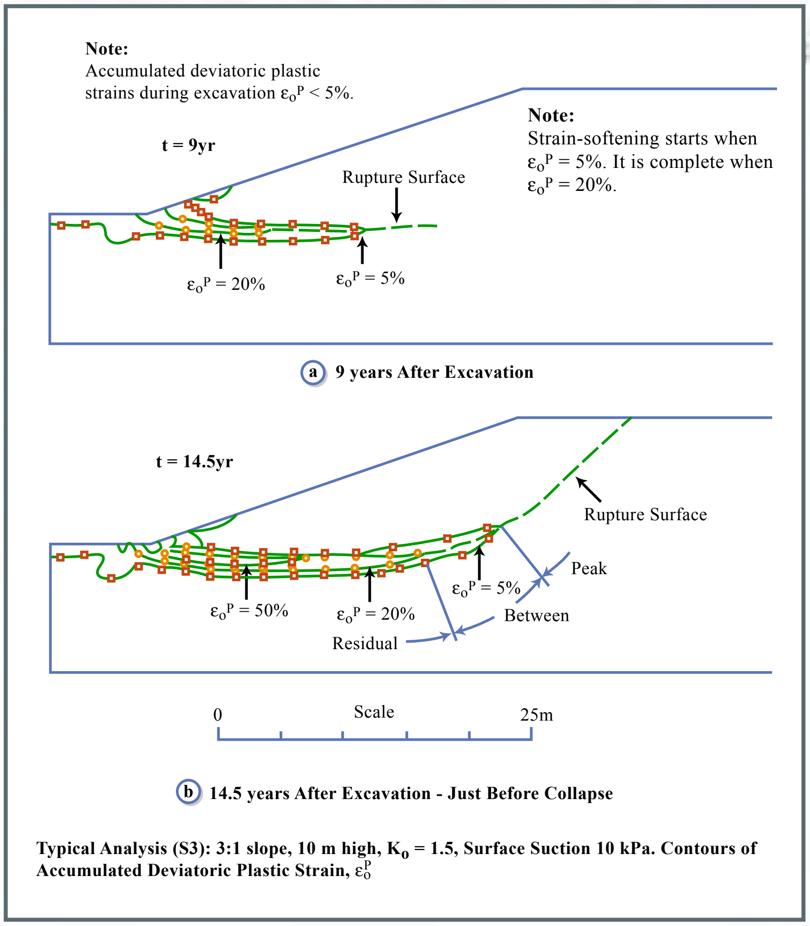


Fig. 21. Rupture surfaces predicted by the analyses on 3:1 slopes, 10 m high with surface suction 10 kPa and varying K_0

- These results show that % of slope at residual (R), at peak (P), in between (B) and not even at failure (NF) varies as $f(K_o)$.
- Varying slope height and angle also → varying percentages

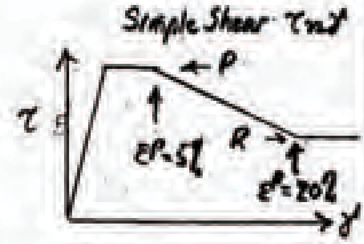
∴ No single envelope at failure

3) Results showing strain contours for $H = 10m$, $1V:3H$ and $K_0 = 1.5$ at $t = 9yr$ & $t = t_f = 14.5yr$.

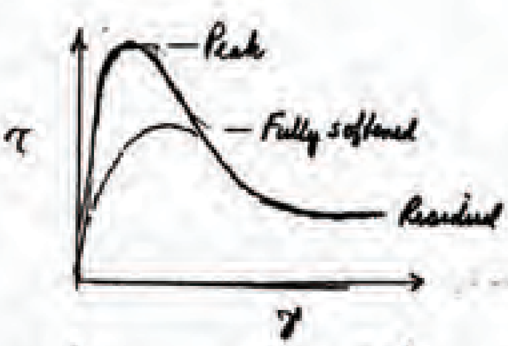


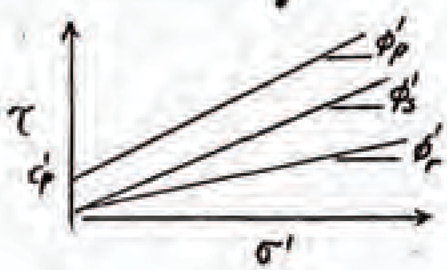
Typical Analysis (S3): 3:1 slope, 10 m high, $K_0 = 1.5$, Surface Suction 10 kPa. Contours of Accumulated Deviatoric Plastic Strain, ε_o^P

Figure by MIT OCW.



4) Conclusions





- Drained shear of all OC clays in slopes will undergo strain softening
- Increased degree of strain softening from peak to fully softened (NC) to residual will decrease t_f (less pore pressure dissipation and swelling to failure)
- Get progressive failure mechanism starting at toe and moving upslope
- No single envelope at failure.

7.1 Mesri et al. (SOA paper submitted to JGGE, 3/00)

1) Analysis of ≈ 100 case histories of failures (1st time & reactivated)

2) Principal conclusions

a) Most stiff clays & clay shales are NOT homogeneous. Rather usually stratified (bedding planes, laminations, etc.) and/or fissured

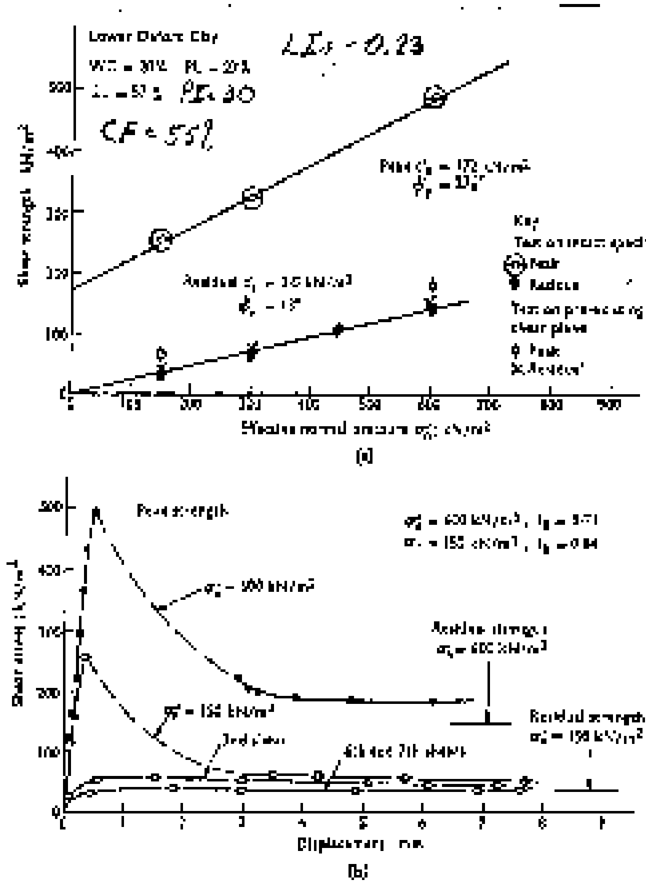
b) Stratified layers often more plastic and weaker and require less displacement to reach residual condition; may even be at residual before excavation or due to undrained shear during excavation.

c) In many cases, stratification often leads to formation of near horizontal failure surface at residual condition

d) Suggests using (I think based on Fig 17 IC7)
- Fully softened envelope along inclined failure surface
- Residual " " horizontal " "

Burland et al. (1977) Géot. 27(4), 557-591

Strongly laminated cemented (10-20% $CaCO_3$) and fissured clay shale

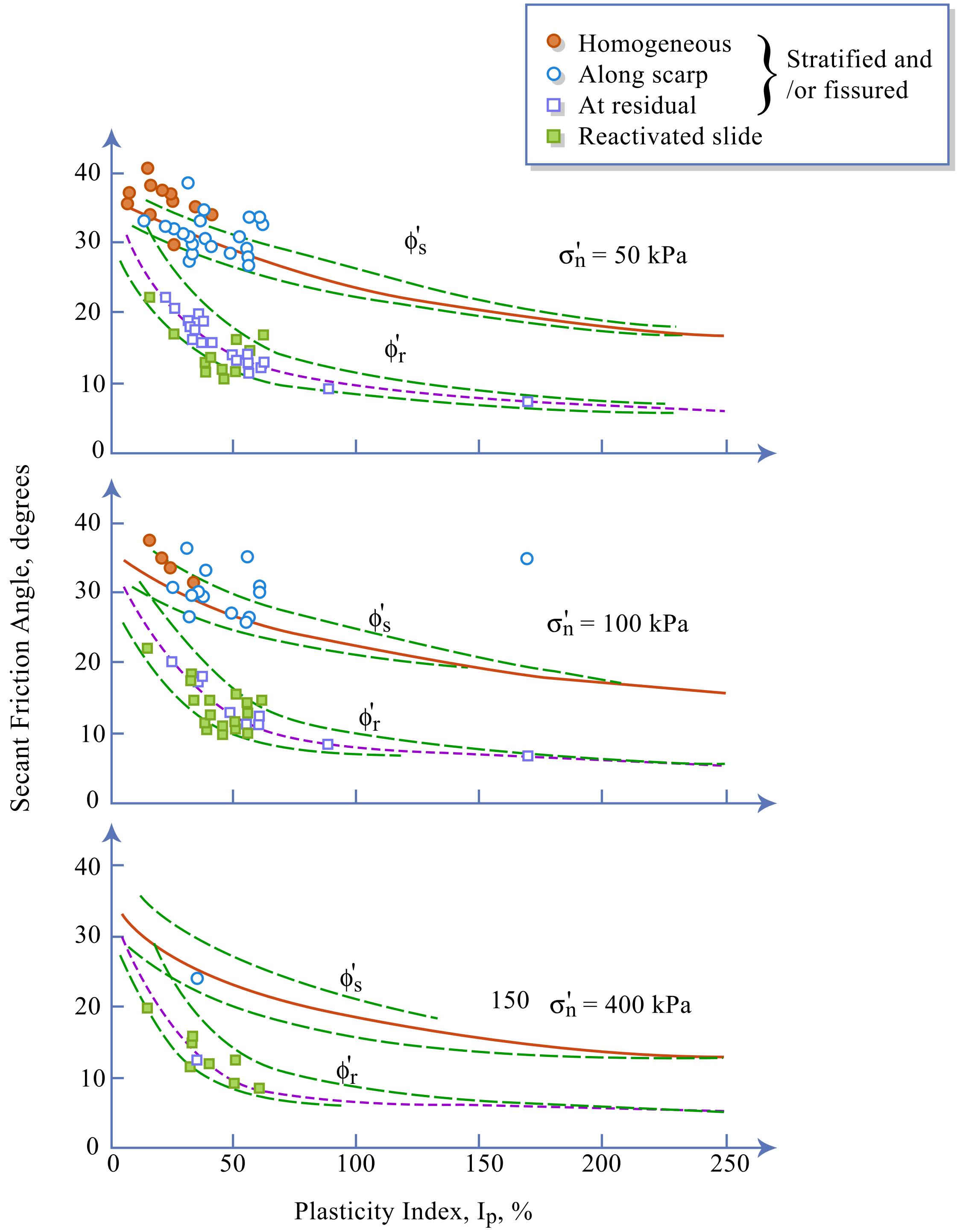


Example of large difference between peak & residual envelopes along bedding plane.

Fig. 5. Typical shear stress-displacement curves and strength envelope for direct drained shear tests on Lower Oxford Clay specimens sheared parallel to the bedding

1.7 Cont.

CCL 4/3/01



Mobilized friction angles back-calculated from reactivated and first-time slope failures compared to the range from empirical information.

Figure by MIT OCW.
Adapted from: Mesri & Shahien (3/00) DO NOT REPRODUCE

1.8 Basic Research on ϕ_r' Kenney (1967) Oslo Conf.; (1977) ICSMFE

Mineral	%-2µ	g/l Salt	w_L (%)	I_p (%)	ϕ_r'	($\tan\phi_r'$)
Quartz	100	–		0	35	(0.70)
Attapulgite	74	–	345	240	29.6	(0.57)
Na Illite	100	0	51	18	16.2	(0.29)
		30	99	___	___	( )
Na. Mont.	100	0	1325	1270	4.0	(0.07)
		30	620	___	___	( )

Mixtures of massive & clay minerals*

$$R_{\phi_r} = \frac{\tan\phi_r'(\text{Mixture}) - \tan\phi_r'(\text{Clay})}{\tan\phi_r'(\text{Massive}) - \tan\phi_r'(\text{Clay})}$$

* See IC9

1.9 Empirical Correlations

- Voight (1973) Geot: 23(2)
- Lupini et al (1981) Geot 31(2) + fundamental studies - See IC10
- Deere $\phi_r' \sim LL$ IC10
- Stark & Eid (1994) JGE, ASCE, 120(5) IC11

CF < 25% Low I_p: Relatively little particle reorientation — CCL: Higher $\bar{\sigma} \cdot a_m / \bar{\sigma}$

CF > 50% High I_p: Significant particle reorientation (Can get highly polished surface) — Lower $\bar{\sigma} \cdot a_m / \bar{\sigma}$

Skempton (1985) Geot 35(1)

CCL 4/2/87 1.322
4/89 4/96 4/01

IC9-

Residual Friction Angle of Soils

Kenney (1977) "Residual Strength of Mineral Mixtures"
9th ICSMFE Vol. I pp. 155-160

Results from repeated Direct Shear at $\sigma_n' = 1\ kg/cm^2$

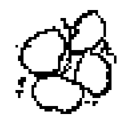
$$R_{\phi_r'} = \frac{\tan\phi_r'(\text{Mixture}) - \tan\phi_r'(\text{Clay})}{\tan\phi_r'(\text{Massive}) - \tan\phi_r'(\text{Clay})}$$

TABLE 1. RESULTS OF RESIDUAL STRENGTH TESTS ON MIXTURES.

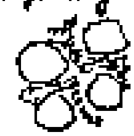
Mixture	Salinity g/l	* Mixture % dry weight	Mineral Content % dry wt. Total Massive	Total Clay	Residual State $\sigma_n' = 1.0\ kg/cm^2$ w_r %	% Volume Clay and Water	$\tan\phi_r$
A. MIXTURES CONTAINING MONTMORILLONITE							
Montmorillonite – Na and Quartz	0	50/50	[illegible]	60	300	91	0.09
		25/75	35	75	96	89	0.11
		10/90	10	90	44	69	0.45
		4/96	4	95	20	34	0.61
Montmorillonite – Na and Quartz	30	50/50	50	60	73	81	0.24
		25/75	25	75	61	66	0.38
		10/90	10	90	34	54	0.54
Montmorillonite – Na and Amorphous SiO_2	30	50/50	60	50	151	86	0.11
		25/75	18	75	87	76	0.24
Bentonite and Quartz	0	75/25	44	58	61	81	0.12
		50/50	65	33	62	76	0.24
		25/75	81	10	42	41	0.40
		15/85	89	11	36	64	0.34
Bentonite and Amorphous SiO_2	0	91/9	33	66	70	86	0.10
		82/18	58	61	64	62	0.16
		68/32	48	61	46	82	0.13
		47/53	65	35	60	78	0.16
		28/72	81	18	47	68	0.27
		10/90	92	8	61	63	0.41
B. MIXTURES CONTAINING KAOLINITE AND GRUNDITE							
Kaolinite and Quartz	0	75/25	66	75	40	86	0.52
		50/50	60	60	30	72	0.48
		25/75	75	24	23	64	0.66
Kaolinite and Amorphous SiO_2	0	75/25	21	74	44	88	0.32
		50/50	60	40	31	74	0.41
Grundite – Na and Quartz	0	75/25	25	76	67	91	0.26
		50/50	50	50	49	79	0.40
		25/75	75	26	30	63	0.62
Grundite – Na and Quartz	30	75/25	26	76	66	91	0.25
		50/50	50	50	40	77	0.40
		25/75	76	26	38	63	0.63
C. MIXTURES CONTAINING HYDROUS MICA							
Hydrous mica I – Na and Quartz	0	75/25	43	42	30	76	0.25
		50/50	85	18	31	85	0.44
Hydrous mica I – Na and Quartz	30	75/25	48	56	83	73	0.38
		50/50	61	36	23	60	0.46
Hydrous mica I – K and Quartz	0	75/25	44	62	28	74	0.41
		50/50	63	36	30	65	0.47
Hydrous mica I – K and Quartz	30	75/25	46	62	61	76	0.41
		50/50	61	35	34	47	0.60
Hydrous mica II – Na	0	75/25	32	67	43	83	0.37
		50/50	54	45	32	64	0.44
Hydrous mica II – Na	30	75/25	32	67	43	80	0.43
		50/50	54	46	57	47	0.47
Hydrous mica III – Na	0	75/25	38	67	54	85	0.44
		50/50	56	43	40	74	0.46
Hydrous mica III – Na	30	75/25	39	67	52	85	0.44
		50/50	55	43	46	76	0.50

* Think massive - clay % are reversed

"Low Clay"



"High Clay"



Sand particles inhibit reorientation of clay particles

$\eta = 60\% \rightarrow C \sim 4.5$

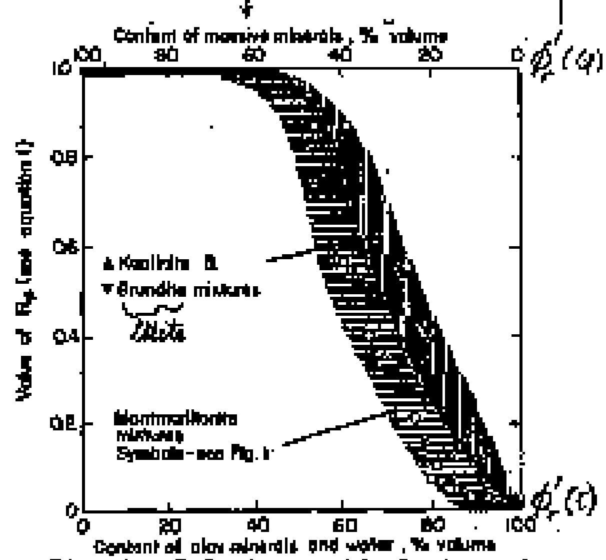


Fig. 4. Relative residual strength

CCL 4/2/87 1.322
4/89 4/3/01
1.9(Cont) Empirical Correlations : ϕ'_r (See IC7 for ϕ'_s)

I.D10

Lupini, Skinner & Vaughan (1981) Geotechnique V31 No.2, pp 181-203
(Contains alot of fundamental research on topic)

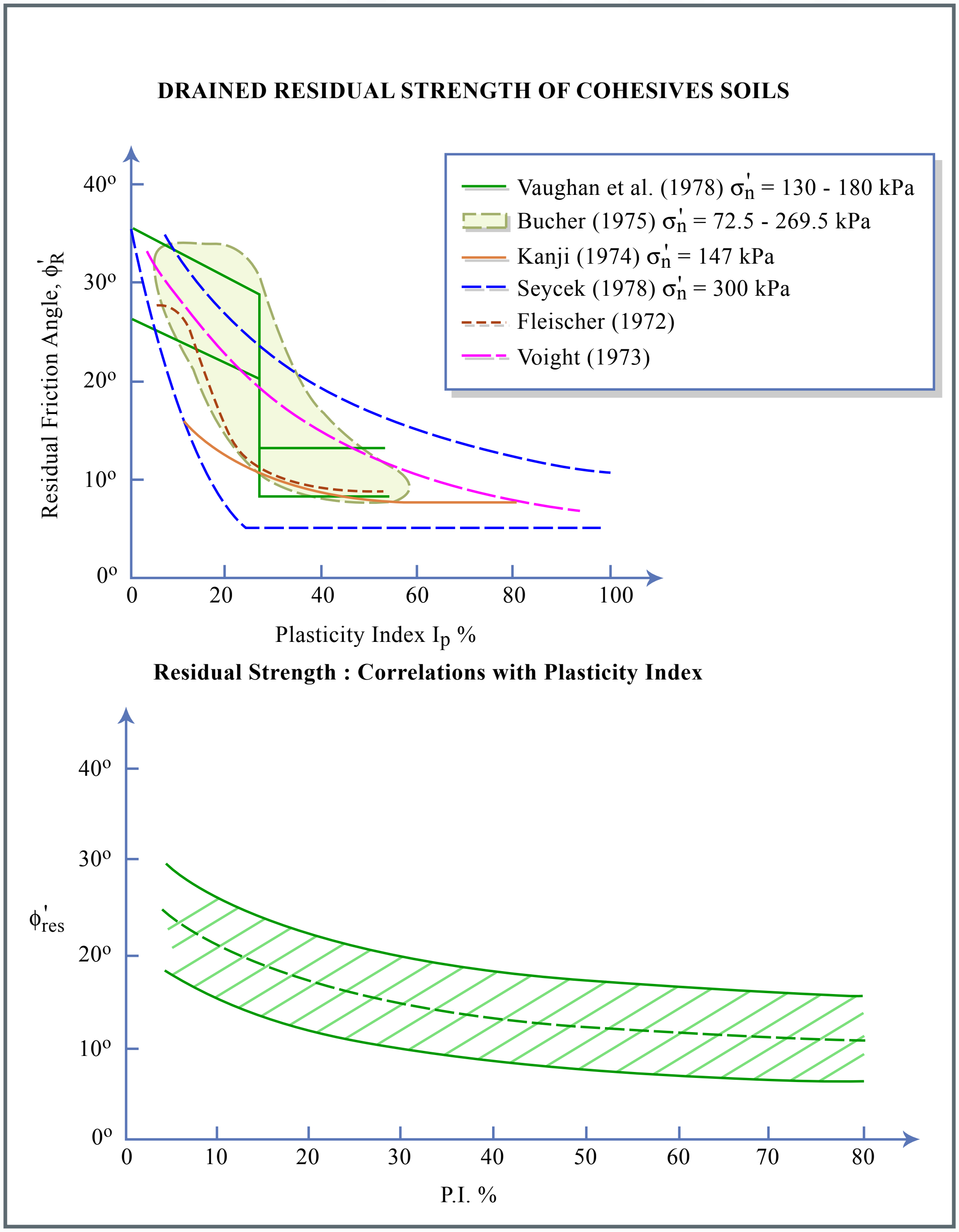


Figure by MIT OCW.

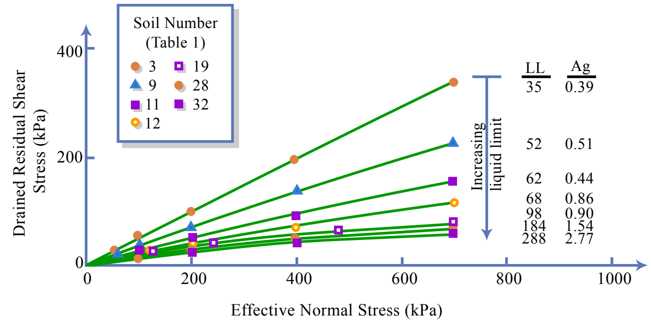
CC Ladd
4/74

CCL 7/2/90 1.322

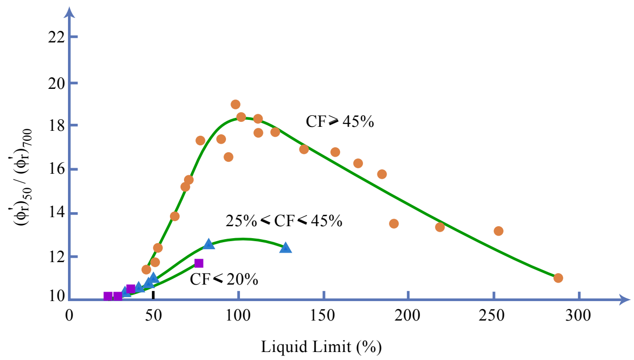
ICU

Drained Residual Strength of Cohesive Soils

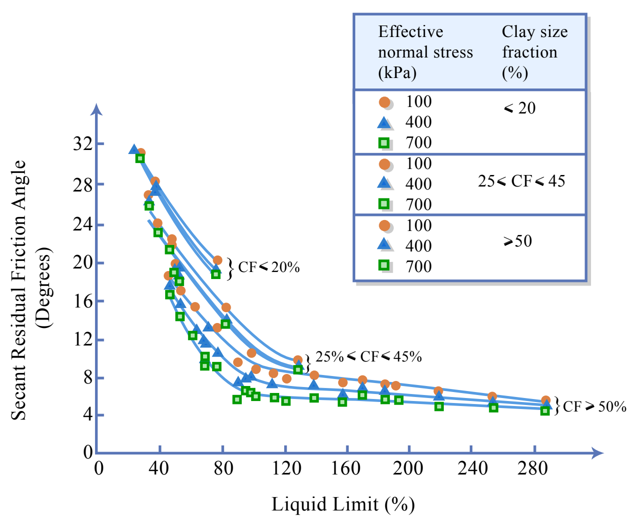
By Timothy D. Stark,[1] Associate Member, ASCE,
and Hisham T. Eid,[2] Student Member, ASCE



Effect of Clay Mineralogy on Drained Residual Failure Envelopes



Reduction in Secant Residual Friction Angle from Effective Normal Stresses of 50 kPa to 700 kPa



Relationship between Drained Residual Friction Angle and Liquid Limit

Figures by MIT OCW.
Adapted from:

JGE, ASCE 120(5) (1994)

Results of torsional ring shear tests on 32 clays and shales → Empirical correlation of ϕ'_r as function w_L, CF (%–2μm) and σ'_n. Tests on remolded specimens = mixture distilled H_2O with airdried soil after ball-milling to obtain 100% –#200 sieve (to break down aggregates).

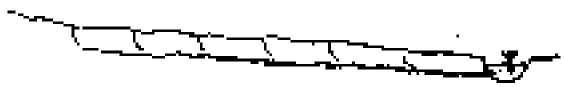
NOTE: Ball-milling essential to breakdown airdried aggregates → correct AL/CF

CCL 4/6/89 1.322 ESA CD Case
4/90 4/01

ICR

C2. HIGHLY STRUCTURED, SENSITIVE CLAYS (Quick Clays)

2.1 Background



Flake and/or retrogressive

- Almost flat slope (Norway)
- "Mini" Δ geometry → massive flow slide
  - Pictures 3rd floor
  - Russian film

- Approach = f(location)

2.2 Norway Aas (1981) ICSMFE

- Analysis 5 "flake" type slides : Treats as CU Case via USA

"Collapse" → undrained condition : $u_{sh}>0$

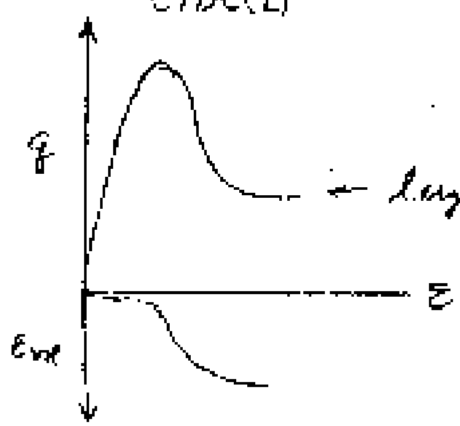
In situ $\tau/\bar{\sigma}'_{v0} = 0.18 \pm 0.035$

Lab CK0UDSS $c_u/\bar{\sigma}'_{v0} = 0.195 \pm 0.025$

2.3 Quebec Lefebvre (1981) CGJ p420

- Treats as CD Case using equilibrium u, but "large strain" values of $\bar{c}'$ & $\bar{\phi}'$

CIDC(L)



← large strain (ε ≈ 10%) used to select $\bar{c}'$ & $\bar{\phi}'$

See ICB

CCL: Seems more empirical than 2.2, but applied to circular arc type failures

CCL 1.322 4/81 4/83

SLOPE STABILITY - QUICK CLAYS (Canada)

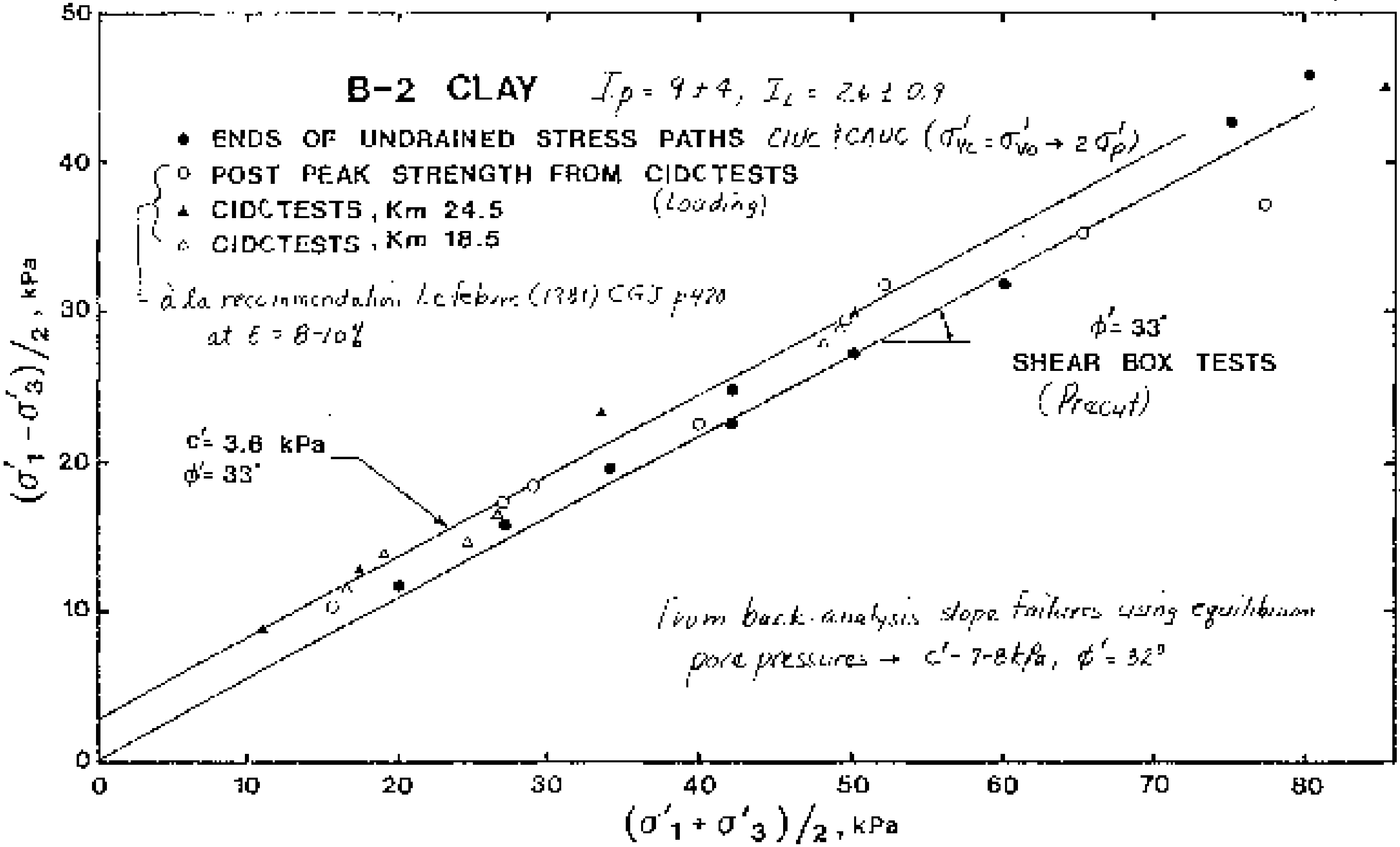


FIG. 5.5-9 TYPICAL EFFECTIVE STRENGTH ENVELOPES B-2

From SEBJ (not for publication)

ICB

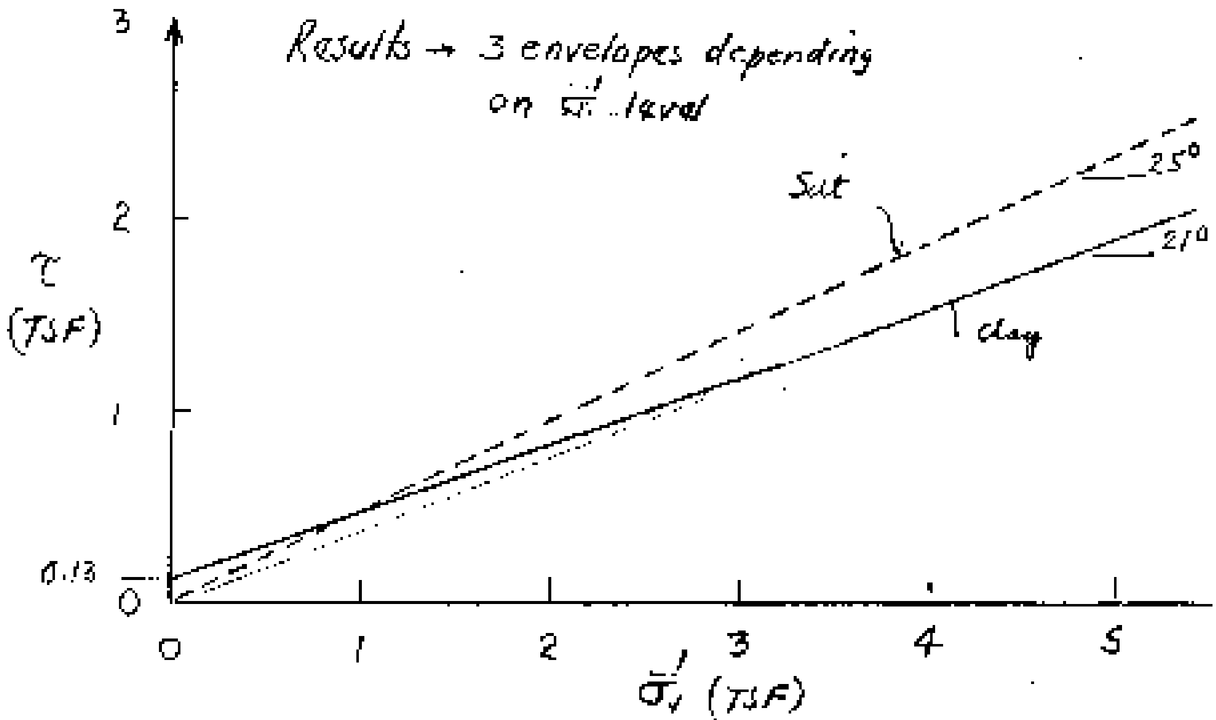
SEBJ NBR 1983

C3. Effective Stress Envelopes for Conn. Valley Varved Clay

Ladd (1975) MIT Report Ladd & Foott (1977) FHWA Report

a) CDDS Parallel to Varves (Clay w/ $\sigma'_p = 3.5$ TSF)

---- Shear through "silt" layer
—— Shear " "clay" layer



b) Summary of ESE Data Bulk I_p = 15-30% Ladd & Foott (1977) FHWA

CU Shear Across Varves
Compression & Extension SHANSEP Tests $c'/\sigma'_{vm} = 0.012$ $\phi' = 30°$

CD Shear In Clay Varves $c'/\sigma'_p = 0.025$ $\phi' = 20°$

4/97 1/99
3/98 4/01

A MEASUREMENT OF UNDRAINED STRENGTH FOR UU CASE

$\boxed{\bar{\sigma} \geq \sigma'}$

For historical perspective since B&B (60) recommendations still dominate much of current practice

Sheets: FV-1, 2, 3
CP-1, 2 & 3
PM-1 to 6

4/4/90

II: UNDRAINED STRESS-STRAIN-STRENGTH BEHAVIOR OF SATURATED COHESIVE SOILS

IIA MEASUREMENT OF UNDRAINED STRENGTH FOR UU CASE

1. INTRODUCTION

1.1 Background

- Objective - Since assume $\Delta w = 0$ for UU Case, want s_u of in situ soil to calculate $FS = s_u/\tau_m$
- Historically have used Total Stress Analysis (TSA), which for $S=100\%$ → "$\phi=0$", $c = s_u$, wherein s_u obtained by variety of both in situ & lab testing techniques that inherently assume:

  Either s_u = unique $f(w_f = w_0)$

  or s_u = " f(in situ $\bar{\sigma}_{vo}$)

1.2 Coverage of "Conventional" Testing Techniques

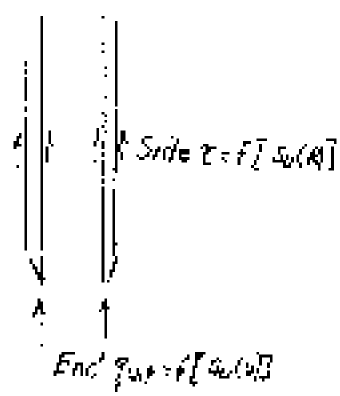
1) $s_u = f(w_f = w_0)$
   - In Situ: SPT FVT CPT SBPT DMT
   - Lab: "UU" type testing, e.g. UUC, LV, FC
2) $s_u = f(\bar{\sigma}_c)_{is}$
   - Lab CU: CIUC $\bar{\sigma}_c = \bar{\sigma}_{vo}$, "CU" DS
   - New Techniques CK_0U: SHANSEP & Recompression (Cover under IIB: Sample Disturbance)

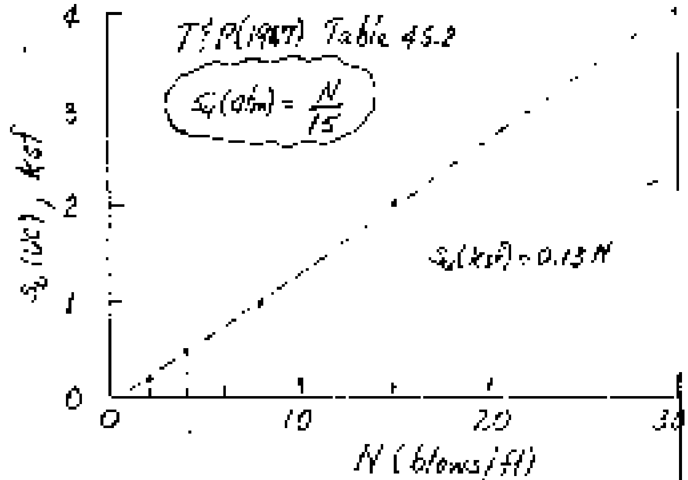
1.3 Discussion & Conclusions

1) Recommended practice à la Bishop & Bjerrum (1960)
2) Comparison
3) Need for thorough evaluation of factors affecting in situ s_u
   - Sample Disturbance
   - Stress System = $b(\sigma_2)$ & δ(anisotropy)
   - Time = strain rate effects

2. IN SITU TESTING TECHNIQUES

2.1 SPT = Std. Penetration Test (ASTM D1586)





- Tokyo 4.2.2 JHS(1995) insensitive clays, side $\tau \to 60\%$ of N
- ∴ Increasing $S_t \to$ reduced N for same $s_u(u)$ NG med-soft clays where often get "WOR" = wgt of rod "WOH" = wgt of hammer

2.2 FVT = Field Vane Test (ASTM D2573)

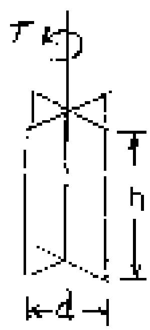
2.2.1 Test Procedures

Tokyo 77

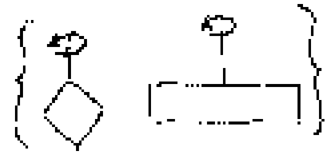
$s_u = \frac{T}{\pi\left(\frac{d^2h}{2} + \frac{d^3}{6}\right)}$

- Need gear system to obtain $\dot{\theta} = 6°/min \to t_f$ few min. (Torque wrench → ∴ t_f = few seconds)
- Need correct/eliminate rod friction Geonor vs Nilcon (Acker (aussi))
- Minimize blade t to reduce disturbance during insertion
- S_t after rotate 10 times
- Cal. s_u assuming equal τ everywhere (sides / ends)

2.2.2 Disadvantages



- No sample
- Limited to s_u < 0.5-1 TSF
- Not reliable if roots, shells, stones, etc. or if high $c_v \to$ partial drainage
- Very complex stress system with progressive failure
  - Rate probably very important
  - do NOT use to estimate s_u anisotropy

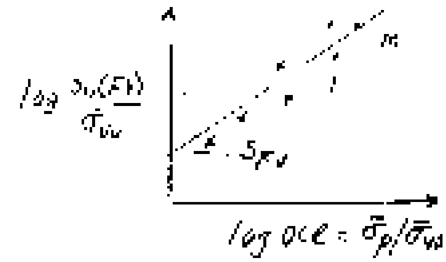


3/29/99 Update

2.8.3 Advantages & Practical Application

1) Relatively simple and inexpensive (but much slower than CPT/DMT)
Do get $S_t = s_u(U)/s_u(R)$

2) Generally good-excellent for "s_u index" profiling if "homogeneous" clay → correlations with stress history

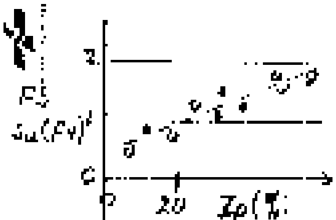


$$\frac{s_u(FV)}{\bar{\sigma}_{vo}} = S_{FV}(OCR)^m$$

+FV-3

Examples FV-1 Fig. 8
{ Also Chandler (1988) w/ Consolidation II
$OCR = \left(\frac{s_u(FV)/\bar{\sigma}_{vo}}{S_{FV}}\right)^{1.05}$ }

3) Correction Factor to obtain s_u for TSA à la Bjerrum (1972, 73)



- $s_u = \mu\, s_u(FV)$ FV-1 Fig. 51 Case histories of failures
- COV ≈ 25% if no shells, fibers, sand, etc. BUT NOT ALWAYS
  ← Smith Bay Arctic silt

2.8.4 Discussion

1) Attempts to understand - separate components of μ vs Ip
FV-2 Fig. 9 & 10 Small t_f → too high s_u $+\mu_R$
Mode shearing → too low s_u $-\mu_A$

2) Aas et al. (1986) "new" correction & effect of OCR } CCL doesn't like
- FV-2 Fig. 9 Not reliable James Bay

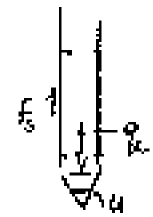
3) Incr. OCR - Why μ vs Ip maybe unsafe at higher OCR?
- Values of m from $s_u(FV)/\bar{\sigma}'_{vo}$ vs OCR correlations
SF('85) Table VI n = 9 → m = 1.03 ± 0.26 >> m from lab CK₀UDSS

2.3 Cone Penetration Test (CPT) ASTM D 3441

Piezocone Penetration Test (PCPT = CPTU)

60° $A = 10 cm^2$

1-2 cm/s

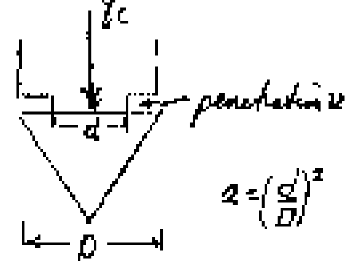


2.3.1 Testing Equipment / Correction

- Electrical essentially replaced mechanical
  More rapid + continuous data
- Some devices not reliable
  - f_s affects q_c and/or u – don't have saturated free stone
- Pore pressure area correction

Fig 27 SF 52A

$$q_t = q_c + u\underbrace{(1-a)}_{0.15-0.6}$$



$a = \left(\frac{d}{D}\right)^2$

(CP-1) Fig. 10 Example of large effect

2.3.2 Estimation s_u: Empirical Cone Factor N_k

1) Approach & Results

- $s_u = \dfrac{q_c - \sigma_{vo}}{N_k}$ Empirical correlation $N_k = \dfrac{q_c - \sigma_{vo}}{s_u \text{ Reference}}$

- ASCE "IN SITU" 75 $N_k = 5 \to 70$ Problems of Reference s_u eg UUC, FVT

- Aas et al (1986)
  (CP-1) Fig 11 $N_k = \dfrac{q_c - \sigma_{vo}}{\mu\, s_u(FV)}$ ← Not corrected
  Medium-soft clays → $N_k = 15 \pm 5$

2) Discussion

- Need correlations → $s_u = \dfrac{q_t - \sigma_{vo}}{N_{kt}}$
  - Corrected q_t
  - Ref. s_u CK₀U Tarc

- Problems Arctic silts: high OCR & low temp. (≈ 0°C)
  (CP-2) Fig 6-3
  - Conv. s_u → $N_k = 15 \pm 5$
  - SHANSEP s_u(DSS) → N_k up 50 ± 10!

  Effects?

3) Conclusions CPTU (CPT not recommended as want q_t)
- Less reliable than FVT based on current data base
- But more efficient than FVT + excellent for soil type
- ∴ Should consider for major jobs → local correlation

NOTE: See ISOPT-1 (1988) - 2 Vol. Proceedings

See (CP-3) $\log (q_t - \sigma_{vo})/\sigma'_{vo}$ vs $\log$ OCR data for BBC
(i.e., problems with absolute value of q_t)

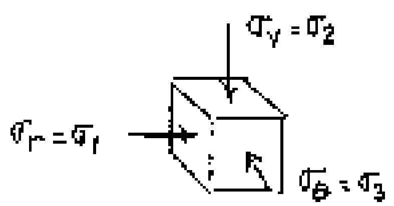
2.4 Self-Boring Pressuremeter Test (SBPT)

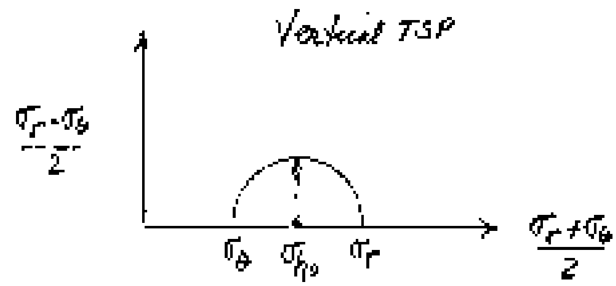
2.4.1 References

- Ladd et al. (1977) Tokyo SOA 4.2.6
- Jamiolkowski et al. (1985) SF SOA 3.2.4, 3.3.2
- Baguelin et al. (1978) The Pressuremeter & Fdn. Engr, Trans Tech Publ, Germany

2.4.2 Theoretical Interpretation: Undrained Cavity Expansion (CE)

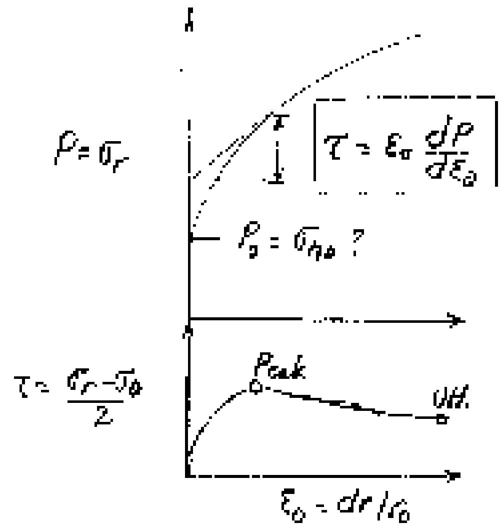
1) State of stress (CE) (Plane Strain)

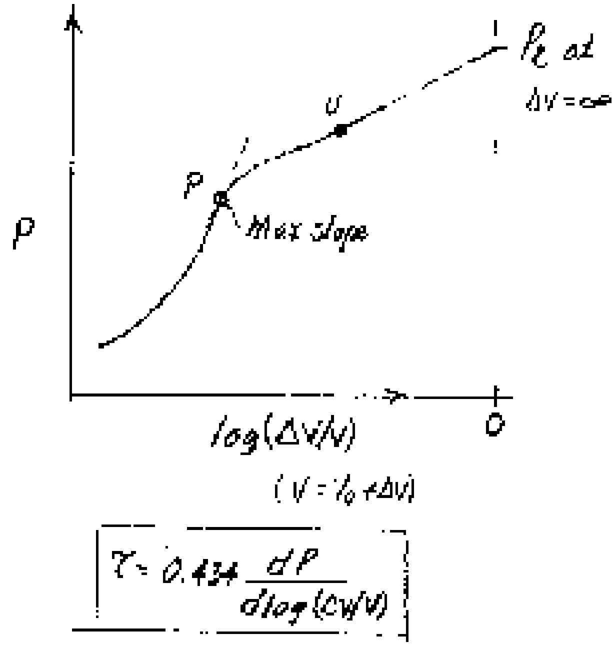




2) Derived Stress vs Strain : Assumptions
- No disturbance $P_o = \sigma_{ho}$
- No end effects, $L/D \to \infty$
- Unique τ vs γ independent of varying $\bar{\varepsilon}$
- No drainage

3) Interpretation





- Elasto-Plastic $s_u = \frac{P_L - P_0}{N_p}$ $N_p = [1 + \ln(G/s_u)]$

E_u/s_u	N_p
1000	7.0
100	5.6

2.4.3 Values of Derived s_u (e.g. Tokyo ('77) Table IV p473 9/11 soils → $s_u(P) = 1.2 - 2.5 s_u(FV)$)

1) Peak s_u usually significantly > $s_u(C)$ - i.e. for $\delta = 0$
Ult s_u " " > $s_u(Ave)$ for stability analysis

2) Possible explanations
- End effects : PAFSOR $l/D = 2-4$ Yes; CAMKOMETER $l/D = 6$ ok
- Disturbance - yes if $P_0 < \sigma_{ho}$ Poorly understood
- $\tau = f(\dot{\varepsilon})$ yes
- Partial drainage - ?
- Anisotropy . Usually expect $s_u(E) < s_u(FE) < s_u(C)$
  $\shortparallel$ $s_u(DSS)$

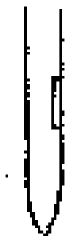
Discussion of (PMT-1) - (PMT-B)

2.4.4 Conclusions
- SBPT not reliable for s_u or τ vs ε_0
- More → Menard or pushed - Totally empirical
  (ASTM D4719 (but no eqn for s_u)

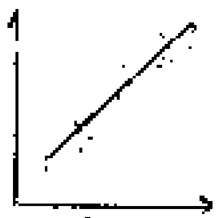
2.5 Marchetti Dilatometer Test (DMT)

(See Consolidation Notes on K_0 & OCR for references & backup)

2.5.1 Procedure



- Measure P_0 immediately after insertion
- Horizontal Stress Index $K_D = \dfrac{P_0 - u_0}{\bar{\sigma}_{v0}}$
- Empirical correlation: Marchetti (1980) Fig 11.81



$$OCR = (0.5\,K_D)^{1.56}$$

Mechanically OC Cohesive Soils

$$I_D = \frac{\Delta P}{P_0 - u_0} < 0.6$$

- Uses SHANSEP eqn to compute s_u

$$s_u / \bar{\sigma}_{v0} = S\,(OCR)^m$$

Marchetti + most computerized DMT output use $S = 0.22$, $m = 0.8$

NOTE: if poor estimate of OCR, then obviously will have poor estimate of s_u

2.5.2 Discussion

- Limited empirical data base + don't understand "fundamentals"
- Less reliable than FVT
- Less reliable than CPTU
- Worth considering if don't have FVT or CPTU

2.6 MIT AFOSR Project

- Using Baligh Strain Path Method + Whittle MIT-E3 model to evaluate FVT CPTU SBPT + Pushin DMT Iowa Stepped Blade

3. LAB TESTING TECHNIQUES

3.1 Unconsolidated-Undrained Triaxial Compression (UUC)

1) Procedures ASTM D2850 UUC
D2166 UC

UUC $\dot{\varepsilon}$ = 1%/min "plastic" materials
= 0.3%/min "brittle" " } → "t_f = 15-20 min"

UC $\dot{\varepsilon}$ = 0.5-2%/min "$t_f \le$ 15 min" 1%/min typical

2) Comments

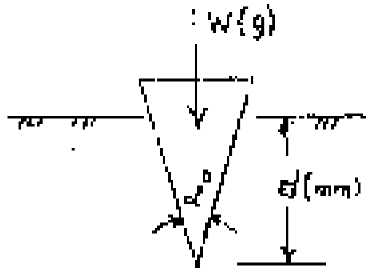
- Use $\sigma_c > 0$ if fissures, cohesionless zones (including shells) and/or if S<100% $\sigma_c \approx \sigma_{v0}$ typical
- Advantages - see sample & get σ-ε curve
- Problems - reliability depends on compensating errors (covered 1.361 + later)

3.2 Other "UU" s_u Index Testing

- Lab vane h/d = 2 d ≈ 2cm
- Torvane
- Fall cone
- Pocket penetrometer

Always use as part of test program on Undisturbed Samples

Fall Cone: Zreik, Ladd & Germaine [1995, GTJ, 18(4)] → new device to measure s_u → 0.1 kPa, (plus review of theory) using counterweight → W = 1g



Typically α = 30° or 60°, W = 10 or 100g, d = 10-70 mm

$$s_u(kPa) = 9.81\,K\left(\frac{W}{d^2}\right)$$

K from empirical correlations with lab vane

α° =	30°	45°	60°
K =	0.83	0.49	0.29

3.3 Consolidated-Undrained (CU)

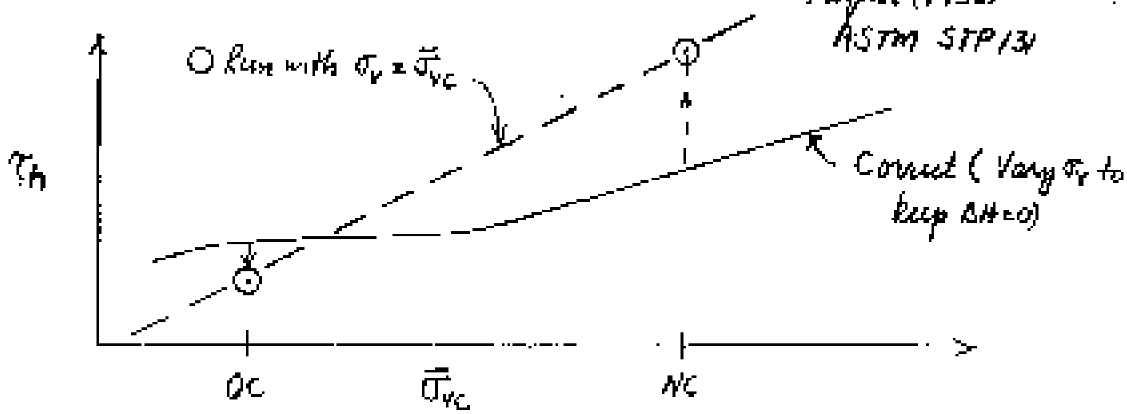
3.3.1 CIUC

- Some use as "std" test w/ $\bar{\sigma}_c \leq \bar{\sigma}_{v0}$ (e.g., MRCE use $\sigma_c' = 0.8\,\sigma_{v0}'$)
- Usually → unsafe s_u, esp. at low OCR (1.361 + later)

3.3.2 "Rapid" Direct Shear

- Assume rapid shearing → CU test, but NOT VALID

Taylor (1952)
ASTM STP131



- Strains

3.3.3 Other CU (Later)

- DM-7 & USACE QRS (CCL'91, Section 6.2)
- CK_0U Recompression & SHANSEP

4 DISCUSSION

4.1 Bishop & Bjerrum (1960) Boulder ASCE "Strength" Conf.

1) Detailed evaluation of case histories of failures falling under UU Case ("no significant dissipation of excess u" + S=100% so "$\phi=0$"), but excluding natural slopes

2) s_u from either lab UUC or FVT

3) Results

No.	Type	FS (Mean ±SD)	I_p(%) / I_L
22	Footings, load tests & emb.	1.01 ±0.06	47±27% 0.75±0.46 } n=14
4	EOC excavation failures (intact clays)	1.02 ±0.09	42±19% 0.79±0.34
7	Base failures of strutted excavations	0.96 ±0.13	—

4) Conclusions - How estimate s_u for "$\phi=0$" analyses UU Case

- Obtain s_u from UUC (or UC) and FVT (uncorrected)
- CIUC $\bar{\sigma}_c = \bar{\sigma}_{vo}$ is unsafe, especially low OCR & Ip
- (Established std. practice worldwide for next ≈ 10-15 yr)

4.2 CCL Early Experience

- Circa 1957 as MIT graduate student à la Bjerrum $s_u(UUC) = 1.11 s_u(FV)$
- 1960-62 on JWL Consulting job Kawasaki, Japan (Tanks, Tokyo Bay)

NC CL/CH
z = 20-35 m

$s_u(UUC)/\bar{\sigma}_{vo} \approx 0.28 \pm 0.03 \rightarrow F \approx 1.2$ Unsafe

$s_u(FV)/\bar{\sigma}_{vo} \approx 0.41 \pm 0.05 \rightarrow F > 1.5$ Safe

- Why difference?
  - Disturbance
  - Anisotropy
  - Shear rate

Deep BBC at I-95

Test	$s_u/\bar{\sigma}_{vo}$ (mean)
UC, UUC	0.13
FV	0.18
CK_oUC	0.33 (NC)

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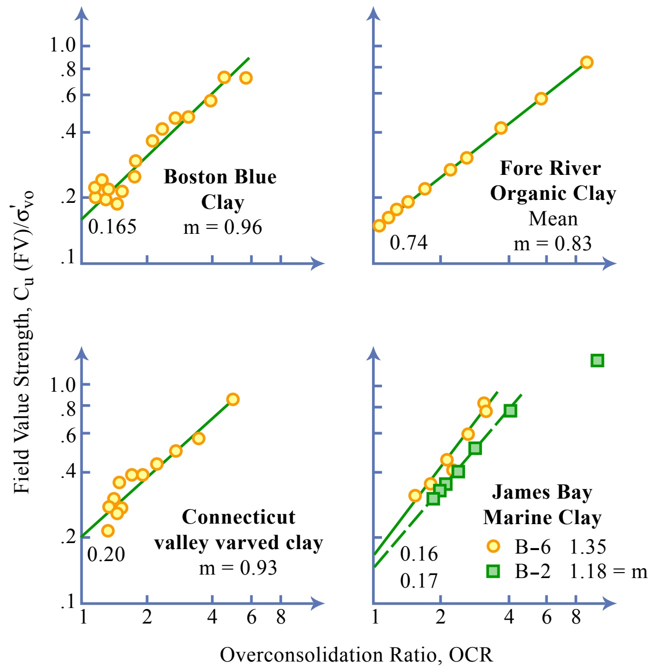
FV-1

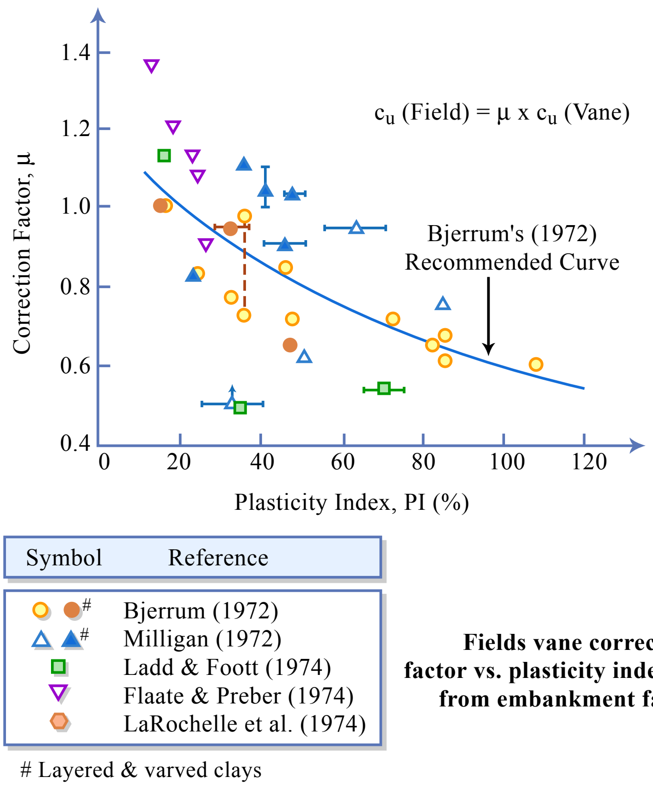


Undrained strength ratio vs. OCR from Field Vane Tests

a) Boston Blue Clay, I-95 Saugus; MA
b) Connecticut Valley Varved Clay, Amherst, MA;
c) Organic Clay with Shells, Fore River, ME;
d) Jmaes Bay B-2 and B-6 marine Clays.

Figure by MIT OCW.

$$\frac{S_u(FV)}{\bar{\sigma}_{vo}} = S_{FV}\,(OCR)^m$$



Fields vane correction factor vs. plasticity index derived from embankment failures.

Figure by MIT OCW.

Adapted from:

Ladd et al. (1977) Tokyo SOA

Bjerrum (1972) $S_u = \mu\, S_u(FV)$

PTH (1974) $\mu \approx 1.0 - 0.5 \log(I_p/20)$

1/97
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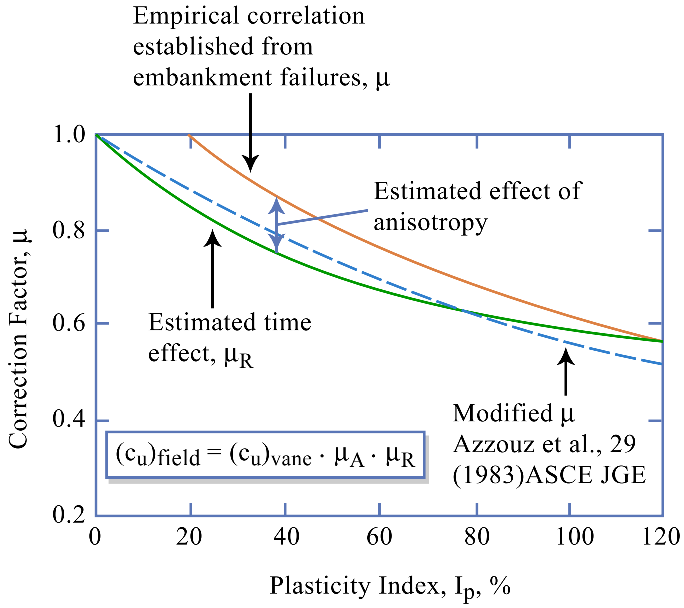
FV-2

CHANDLER ON UNDRAINED SHEAR STRENGTH OF CLAYS (1988)



Factors Relating Field Vane and Field Failure Strengths

Figure by MIT OCW.

Adapted from:

Chandler (1988)

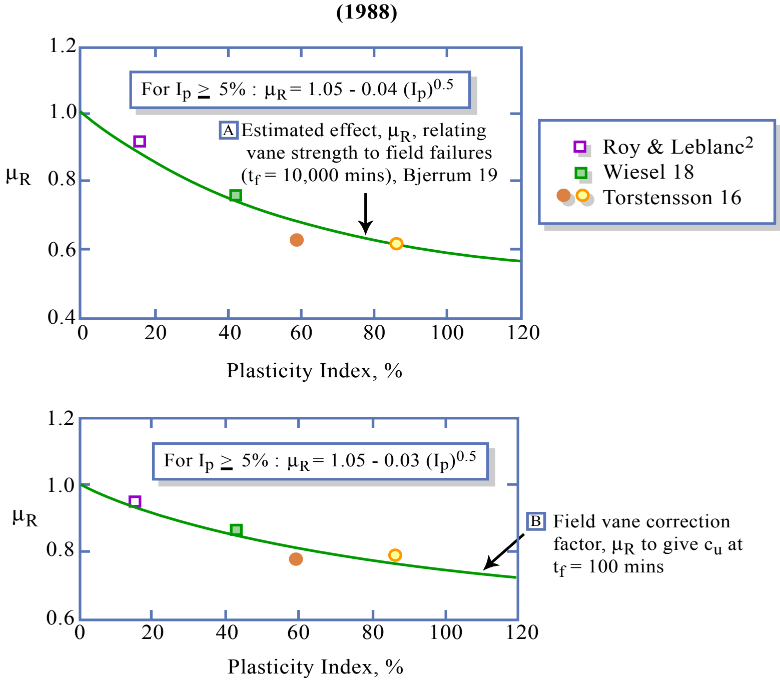
Attempts to understand FV μ factor

- μ_R = reduce $s_u(FV)$ due to rate effects
- μ_A = increase $s_u(FV)$ " " anisotropy

$\mu = \mu_A \cdot \mu_R$

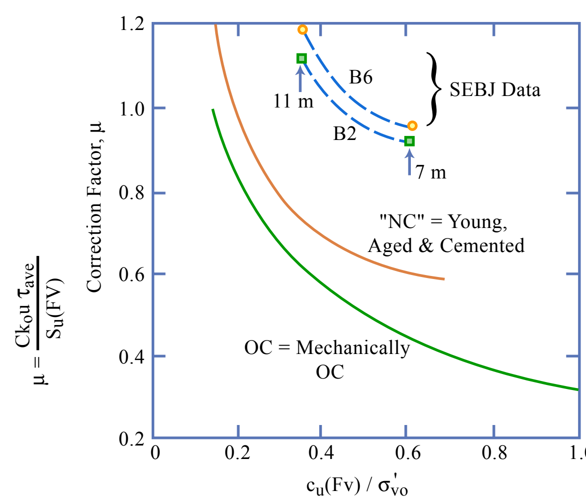
Adapted from: Bjerrum (1973)

CHANDLER ON UNDRAINED SHEAR STRENGTH OF CLAYS (1988)



Factor μ_R to Correct Field Value Strength for Strain-Rate Effects

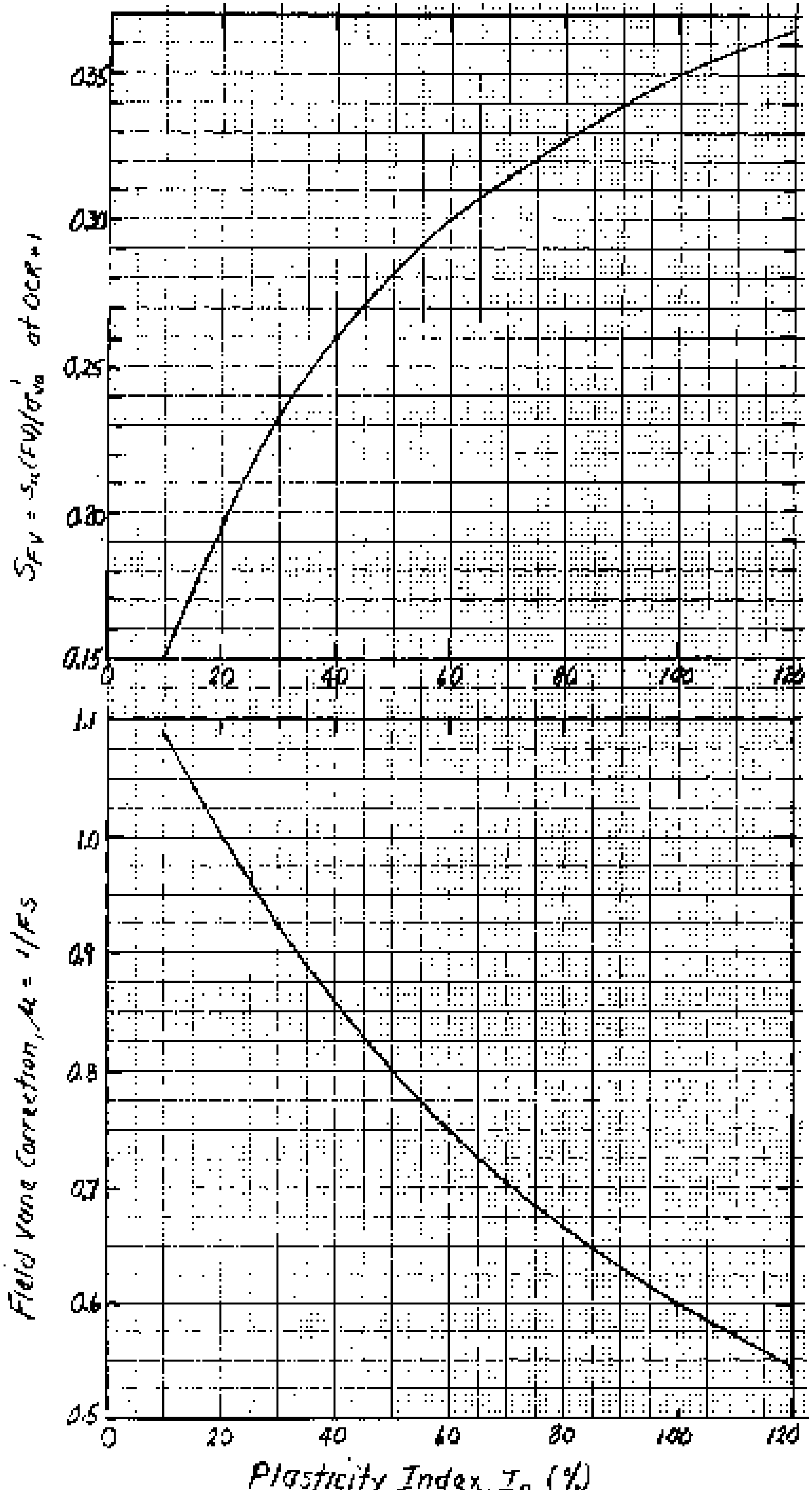
PROBLEMS WITH NEW NGI FV CORRECTION



Field Vane Correction Factor

Figures by MIT OCW.

$\mu = \frac{CK_0U\ \tau_{ave}}{s_u(FV)}$



Chandler (1988)
ASTM STP 1014

$$\frac{s_u(FV)}{\sigma'_{vo}} = S_{FV}\,(OCR)^m$$

↓

$$OCR = \left(\frac{s_u(FV)/\sigma'_{vo}}{S_{FV}}\right)^{1.05}$$

NOTE: S_{FV} = Bjerrum (1972) for OCR = 1 "young" clays

Bjerrum (1972)
Field Vane
Correction Factor
from Case Histories
of Embankment
Failures

NOTE: Drawn by CCL from linear FS vs I_p

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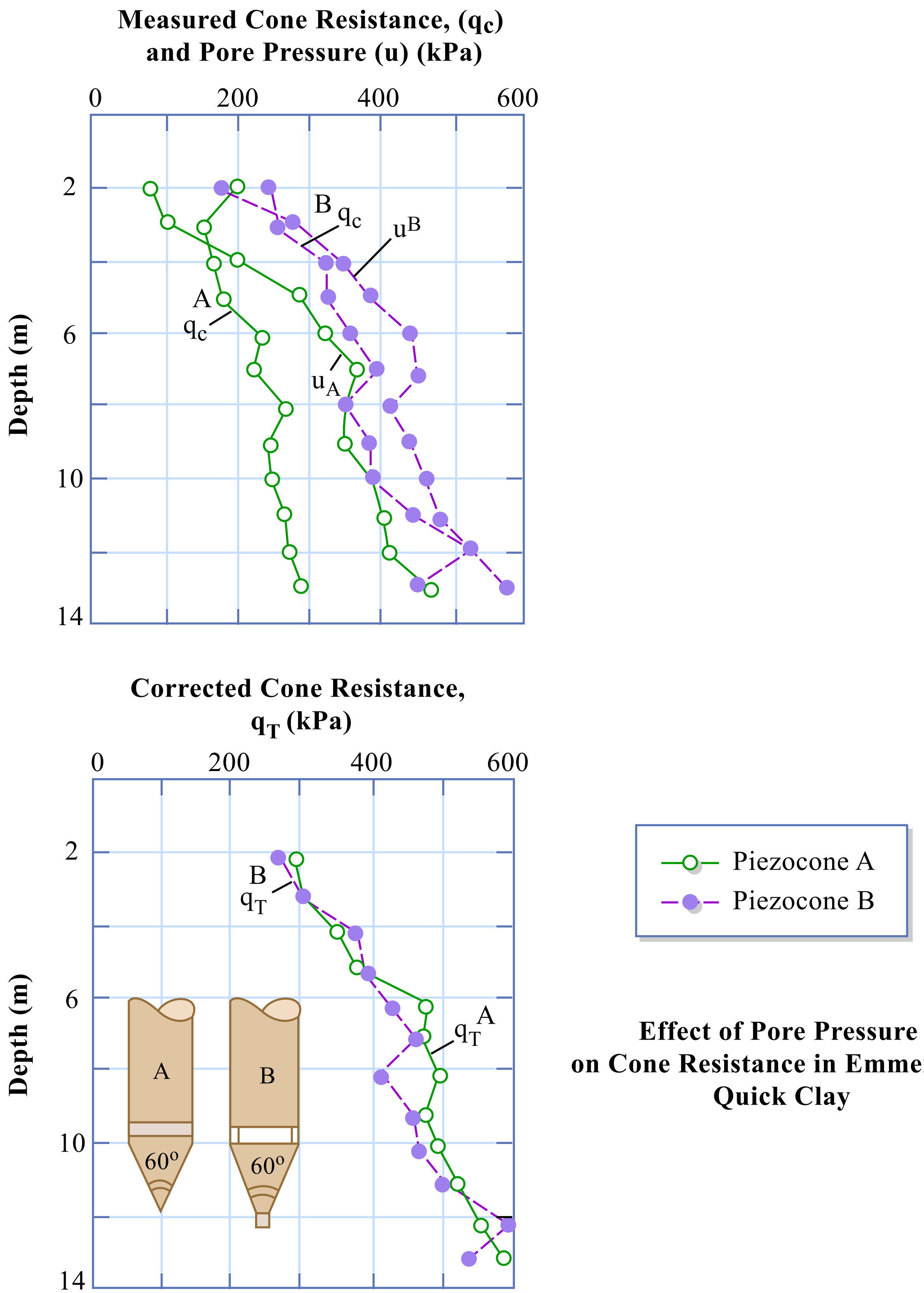
CP-1

Aas et al. (1986) ASCE INSITU '86



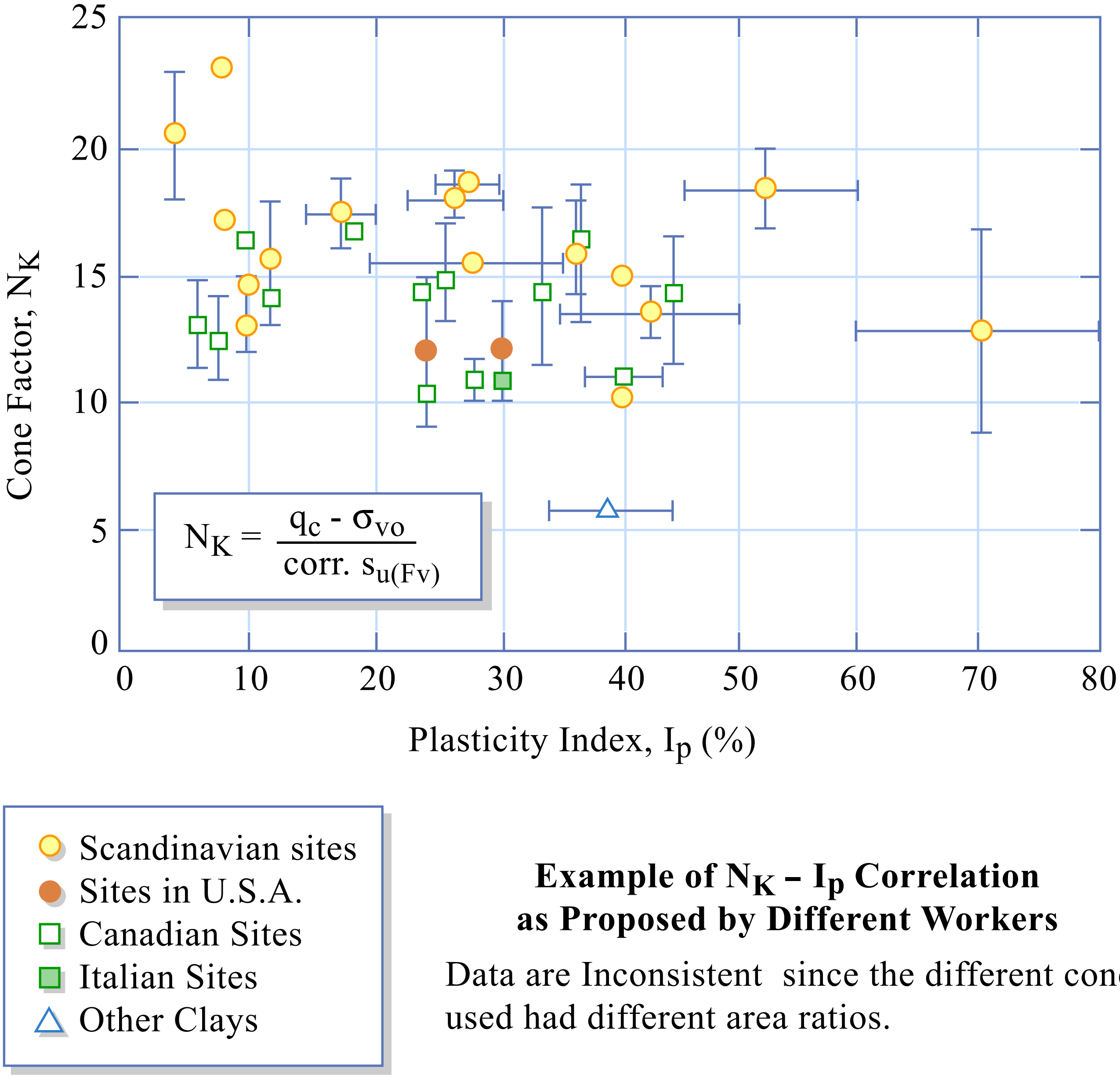
Effect of Pore Pressure on Cone Resistance in Emmerstad Quick Clay

Figures by MIT OCW.

CCL 4/89 1.322

Aas et al. (1986) ASCE INSITU '86

CP-1



Example of $N_K - I_p$ Correlation as Proposed by Different Workers

Data are Inconsistent since the different cones used had different area ratios.

Figure by MIT OCW.

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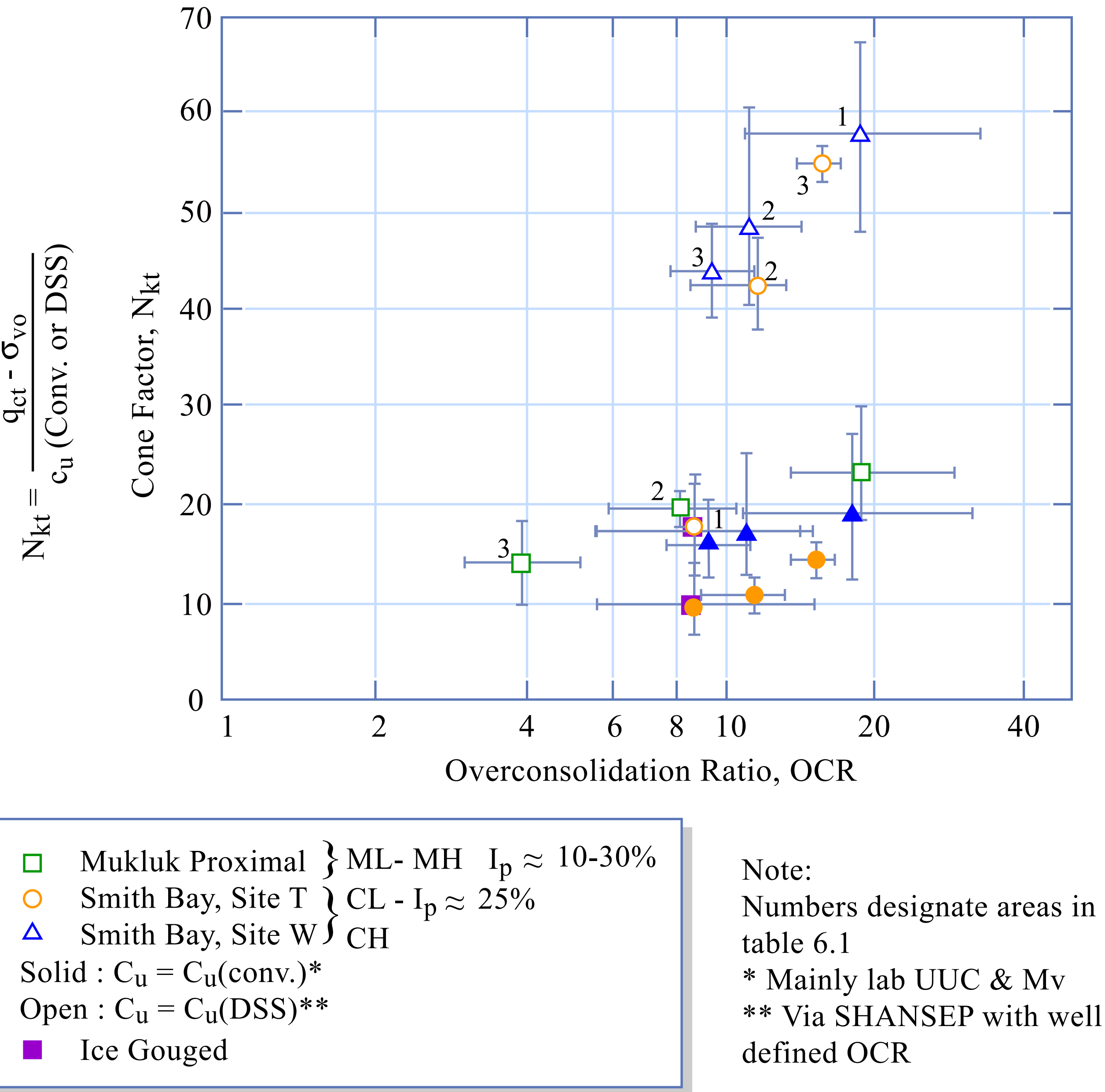
CP-2



Note:
Numbers designate areas in table 6.1
* Mainly lab UUC & Mv
** Via SHANSEP with well defined OCR

Cone Factor vs. OCR. Collective Evaluation for Harrison Bay and Smith Bay Arctic Silts. (In situ temp $\approx$ -1°C)

Figure by MIT OCW.

Adapted from: MIT SM Thesis by C. De La Huerta (6/87)

Supplement to Section 2.4

Results from MIT AFOSR sponsored research on development on rational techniques for interpretation of in situ "penetration" tests in cohesive soils by Whittle et al., namely evaluation of disturbance effects with pressuremeter testing from C. Aubeny PhD thesis (6/92)

- PM-2 Schmetatic of SBPMT, FDPMT & PIPMT
- PM-3 Shear strains from installation PIPMT & ideal SBPMT
- PM-4 Predicted expansion curves as function of amount of disturbance for OCR=1 BBC
- PM-5 Predicted vs. measured expansion curves for OCR=4 BBC
- PM-6 Predicted s_u/σ'_{v0} as function of amount of disturbance for OCR=1 BBC.

NOTE: Predictions using Baligh's (1985) Strain Path Method and Whittle's (1987, 1992) MIT-E3 soil model

Paper Whittle & Aubeny (1992) "The effects of installation disturbance on interpretation of in situ tests in clays" Wroth Symposium

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PM-2

• TYPES OF PRESSUREMETER

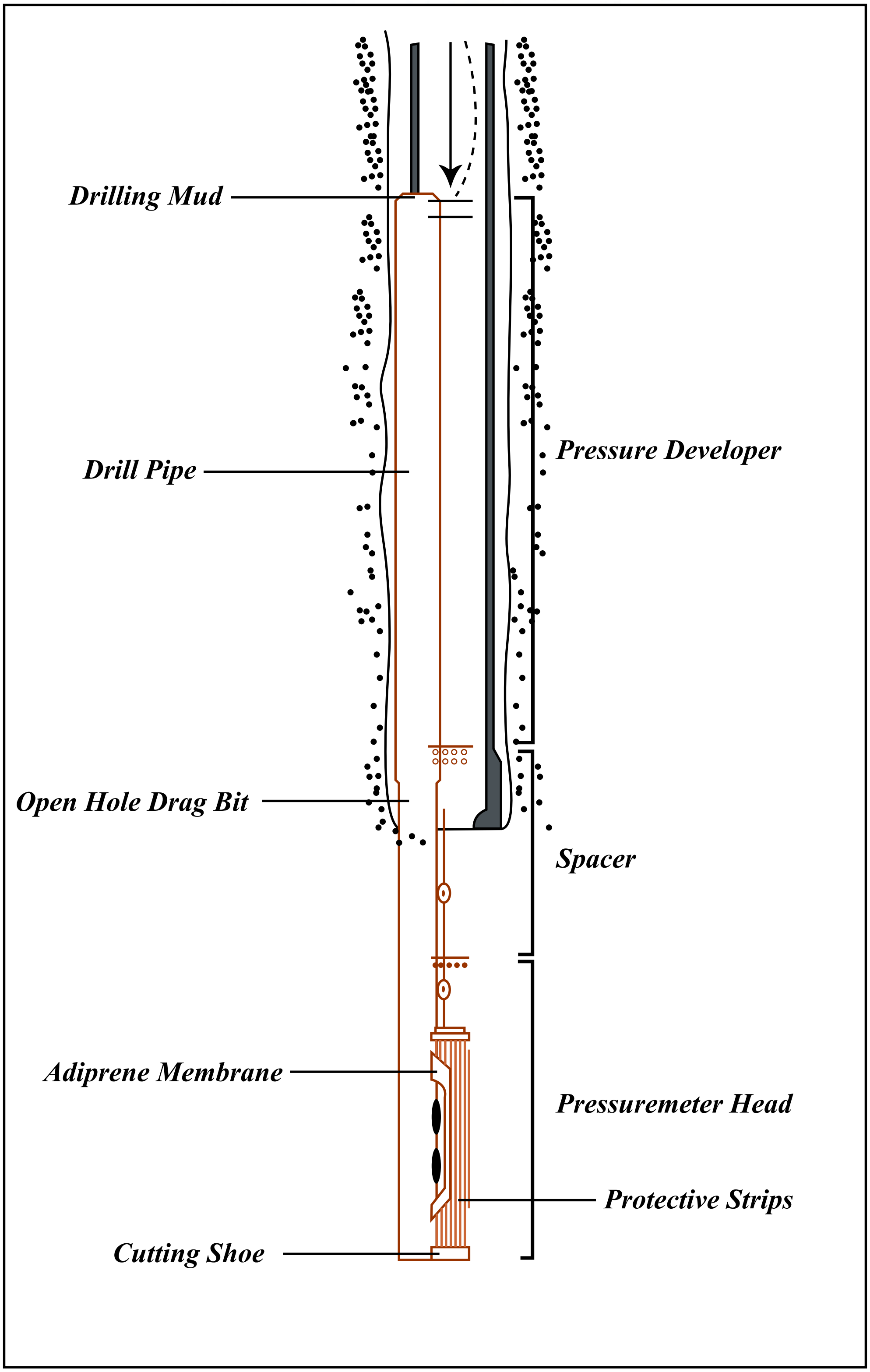

Figure by MIT OCW.

• PUSH-IN
(PIPMT)

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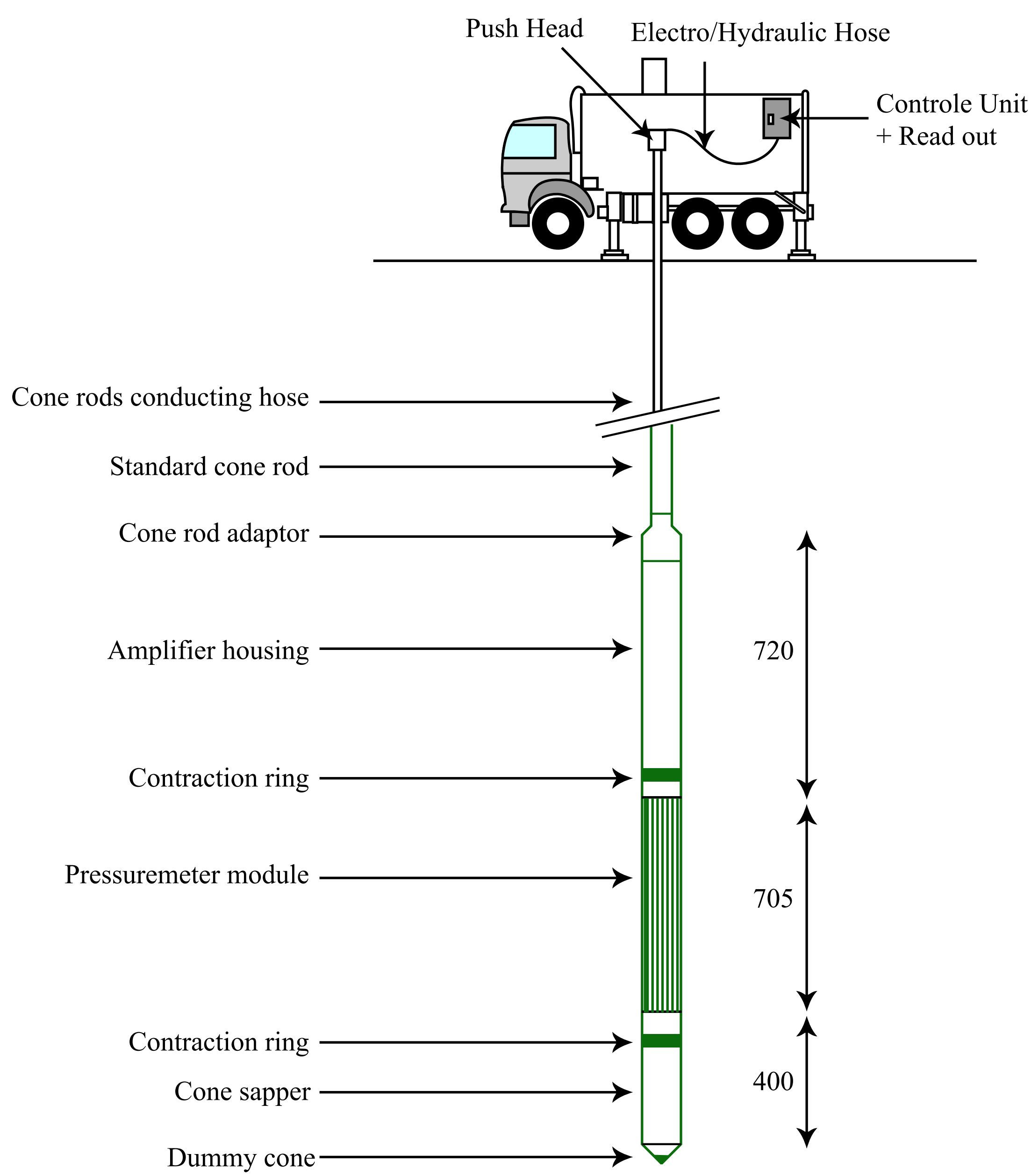
PM-2



Full Displacement or Cone Pressuremeter

Figure by MIT OCW.

• FULL DISPLACEMENT
OR
CONE PRESSUREMETER

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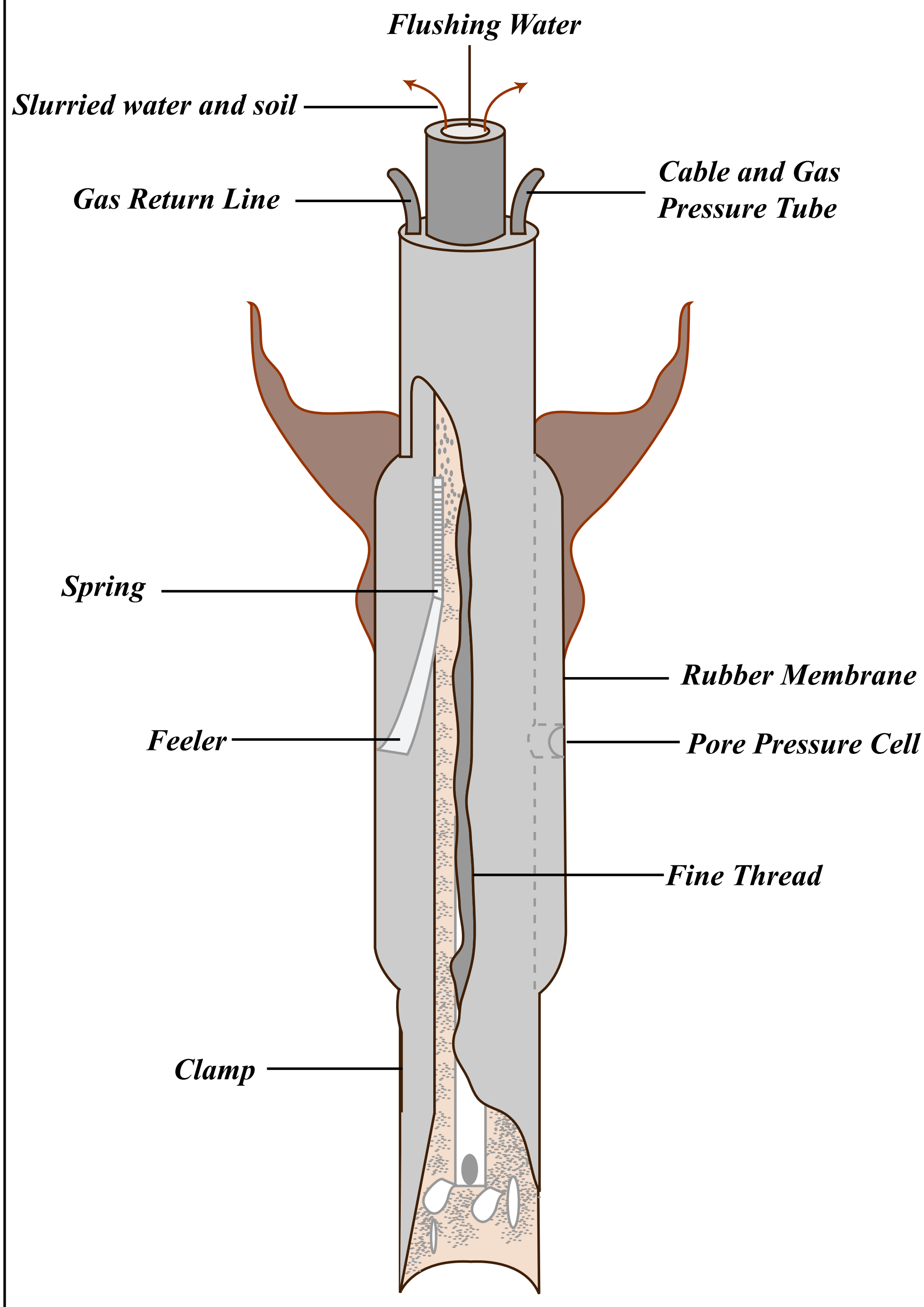
PM-2



Camkometer

Figure by MIT OCW.

• SELF BORING
(SB PMT)

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PM-3

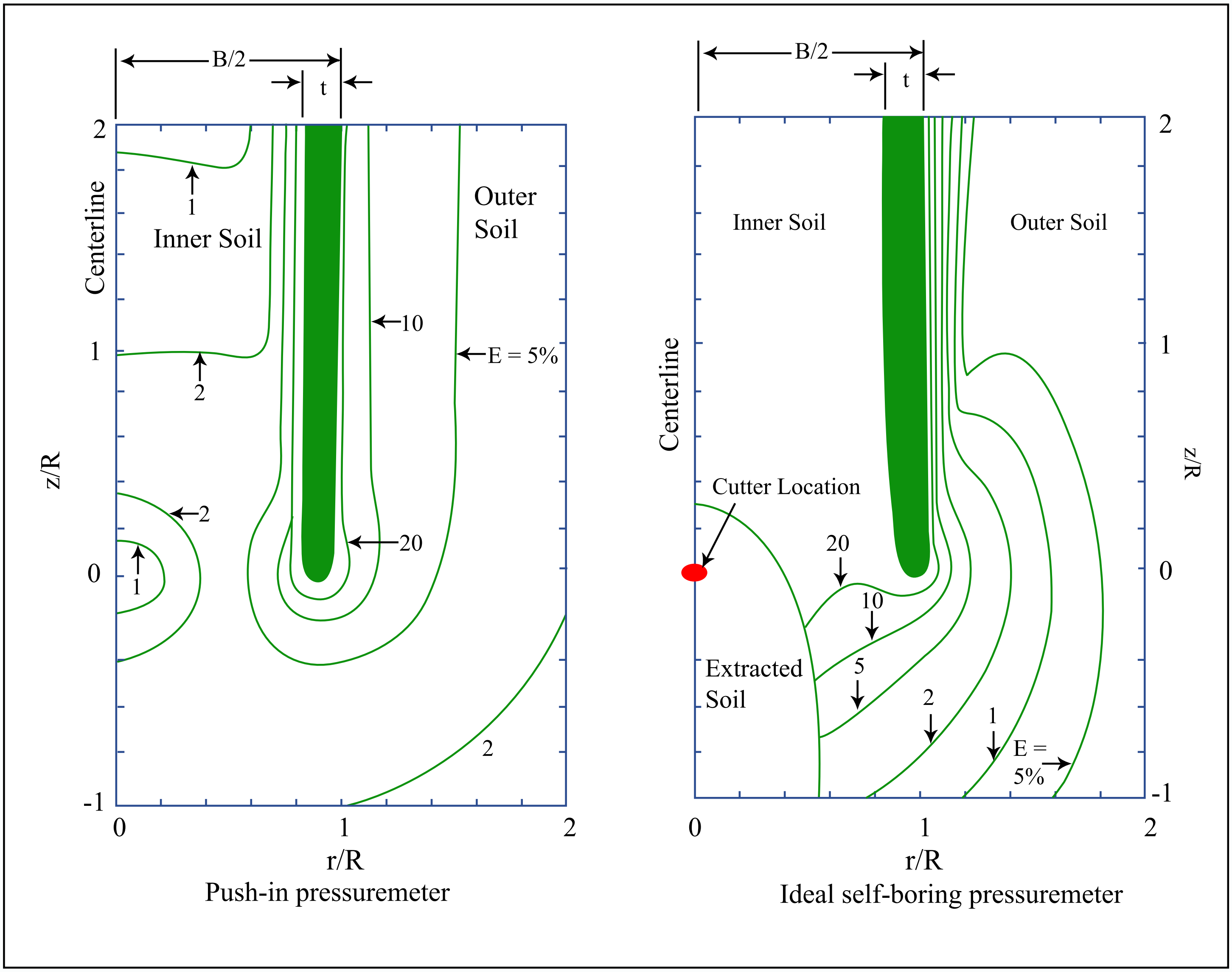

Figure by MIT OCW.

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3/99

PM-4

ASSUMPTIONS

- LONG CAVITY
  (PLANE STRAIN)
- FAR ABOVE TIP
- UNDRAINED BEHAVIOR

$$f = \frac{\text{Volume Soil Extracted}}{\text{Volume Device Inserted}}$$

DISPLACEMENT TYPE

SELF BORING TYPE

SO

$$s_u/\sigma'_{vo} = \left\{ \frac{d\left(\frac{p-u_0}{\sigma'_{vo}}\right)}{d\log(\Delta V/V)} \right\}_{max} \times 0.434$$

- EFFECT OF DISTURBANCE ON EXPANSION CURVE.

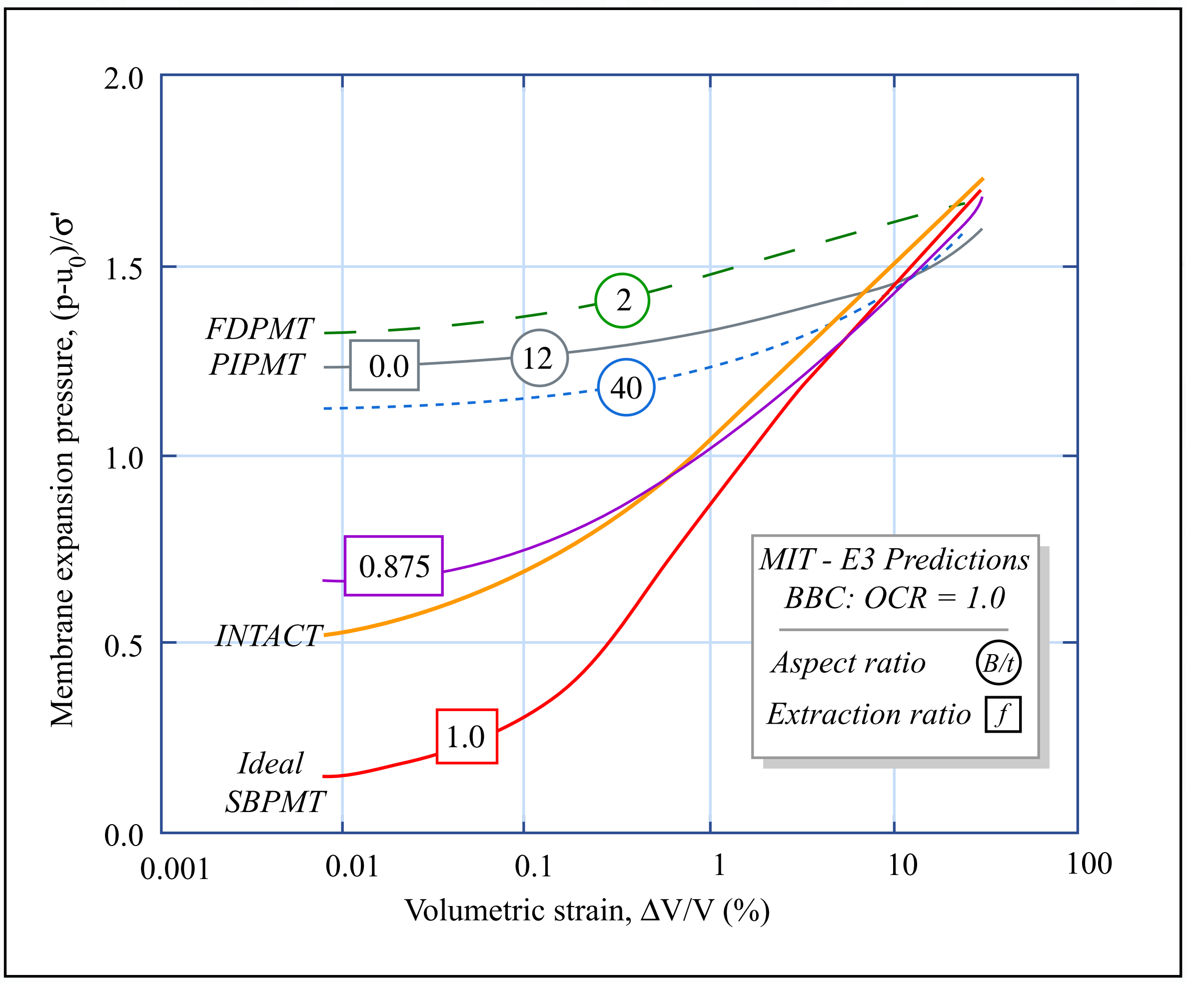

Figure by MIT OCW.

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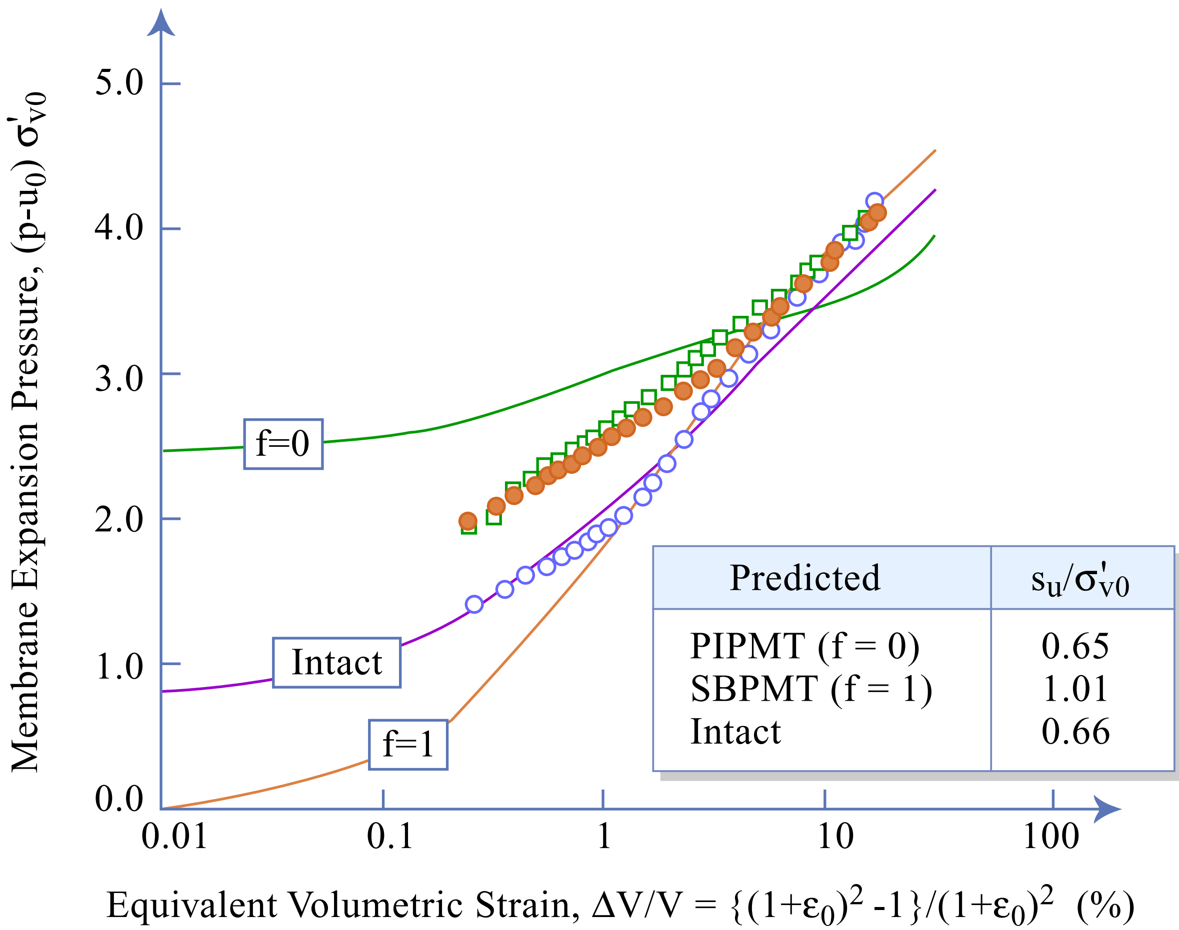
PM-5

Intact = ideal cavity expansion



Predicted	s_u/σ'_{v0}
PIPMT (f = 0)	0.65
SBPMT (f = 1)	1.01
Intact	0.66

Measured	s_u/σ'_{v0}
Arm 1 ○	1.35
Arm 2 □	0.68
Arm 3 ●	0.68

Measured Data South Boston Test Site
(Ladd, 1991)

• EVALUATION OF PREDICTIONS (OCR≈4)
SBPMT IN BOSTON BLUE CLAY

Figure by MIT OCW.

CCL 4/92 1.322

PM-6

• DISPLACEMENT PRESSUREMETERS

• SELF BORING PRESSUREMETERS

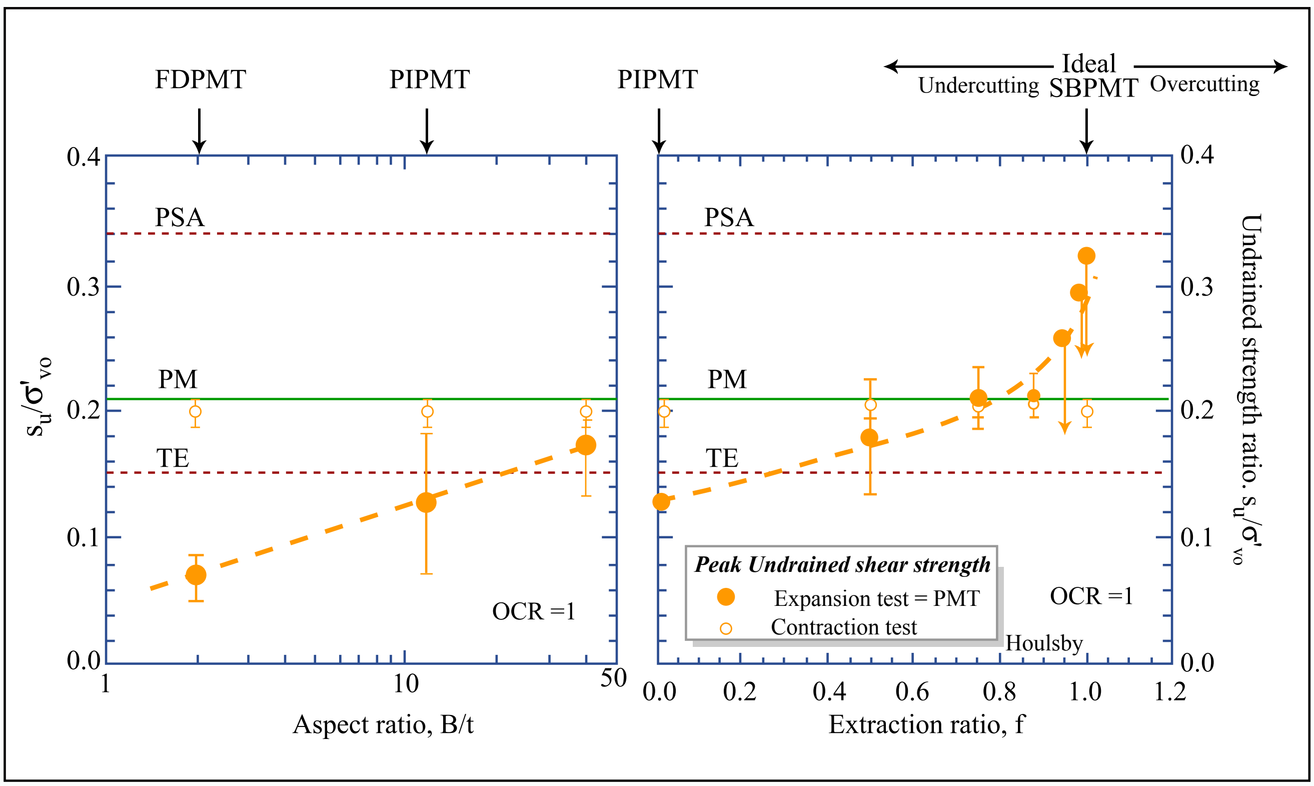


Figure by MIT OCW.

• SUMMARY OF s_u/σ'_{vc} PREDICTIONS
– BBC · OCR = 1

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 $b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3}$

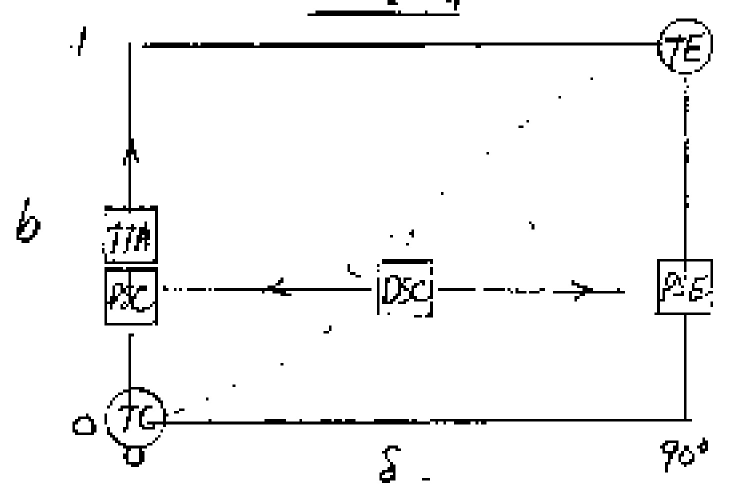
1.2 Objectives

- 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, SF 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"

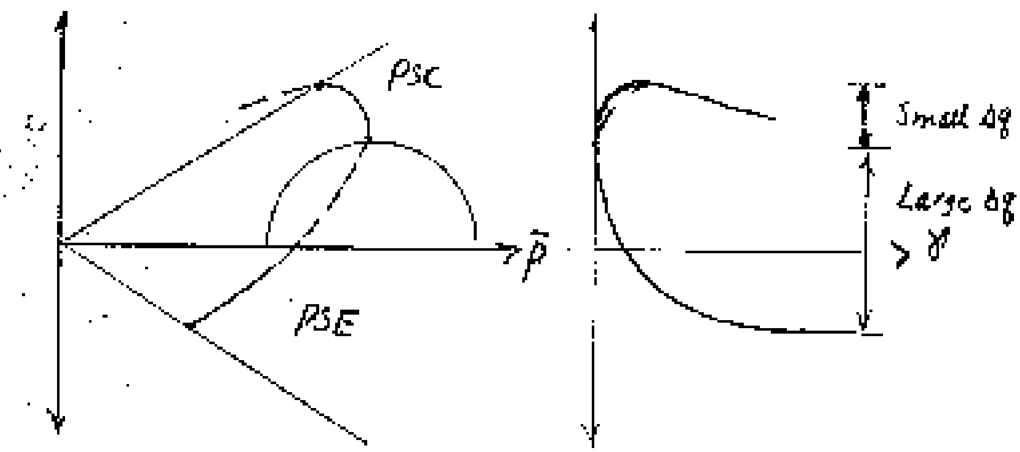
e.g. varved clay

fissures, bedding planes

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

- Hansen & Gibson (1949) (Tokyo p 432)
- CK_oUPS C/E

$q = \frac{(\sigma_v - \sigma_h)}{2}$



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

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

A wrt applied stresses

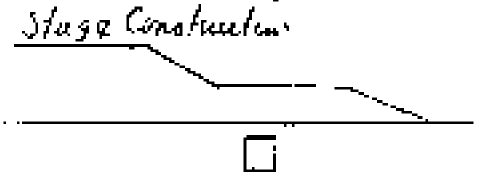
- Can produce s_u anisotropy w/o any inherent anisotropy (i.e. for same K_c, A_f & $\sin\bar{\phi}$)

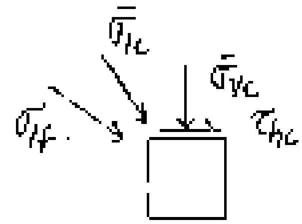
(3) Combined = Inherent + $K_o \neq 1$

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)



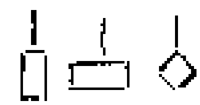


- Δ shape of yield surface (Included in Section 7.4)

3. USE OF UU TYPE TESTS TO MEASURE ANISOTROPY

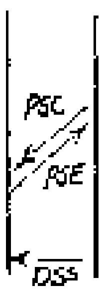
3.1 In Situ

1) FV with varying shapes (Tokyo 4.2.4)



- Disturbance + Progressive failure + Unknown stresses → unreliable results

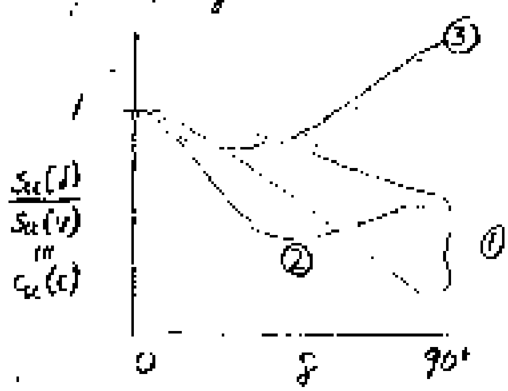
2) NGI special in situ DS device (Table 11.2 of CCL, 1971)



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

3.2 Lab UUC Cut at Varying δ

Tokyo F21



① Homogeneous sedimentary; mo. St + more effect

② Varved clay, s_u(DSS) min

③ Stiff fissured

Problems with UUC(L)

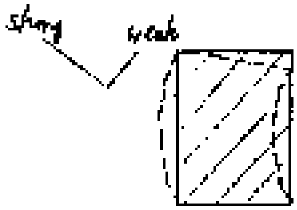
1) Neglects initial stress shear component ($K_c = 1$ vs K_o)

2) Sample disturbance .. $\therefore$ $K_S = S_u(H)/S_u(V)$ (E) (C)

	UUC(S)	CK_oUPS
Portsmouth	0.75	0.44
BBC	0.8	0.56
CVVC	0.6	0.9

separation of vane

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



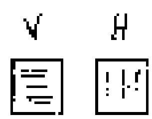
Conclusion: Need CK_oU Type testing

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 $\rightarrow K_c$ (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)
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 $\bar{\sigma}_{hc}$ / " σ_{ht}

5.2 Plane Strain

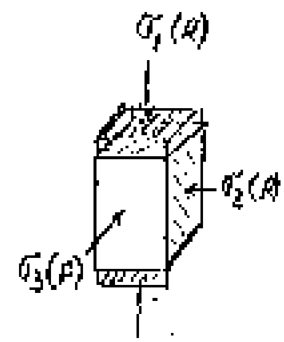
Campanella & Vaid (1974)

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

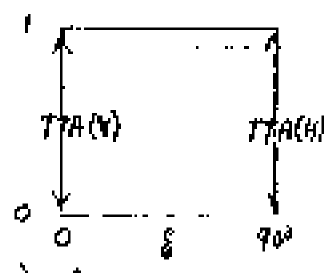
5.3 True Triaxial Apparatus (TTA)

1) Boundary conditions

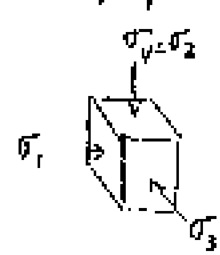
Cube
- Flexible (Rubber Bags) — Scott; UCL (MIT)
- Rigid — Cambridge Univ.
- Mixed — Lade (UCLA)



2) What can do in b-δ plot



+ Cavity Expansion (SBPT)

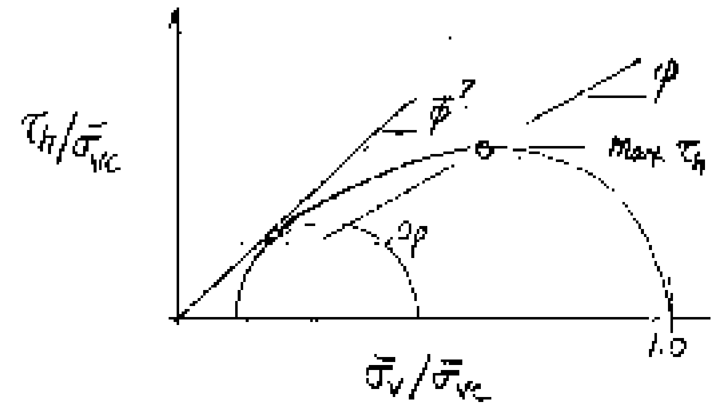


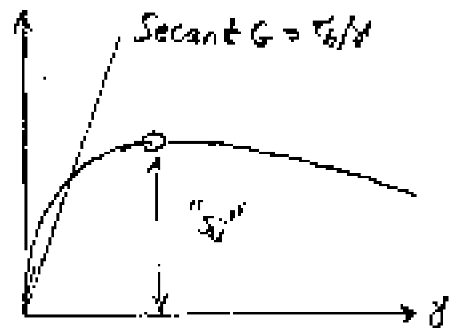
3) Conclusion → Mainly useful for studying b

NOTE: Very little CK_0U data available from TTA

5.4 Direct Simple Shear (Geonor) = DSS

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





Vucetic & Lacasse (1982) JGE No.11

(2) Problems

a) Non-uniform stresses

Lade & Edgers (1979)
Saada et al (1981) + NGI rebuttal

"worse than DS"

Elastic vs plastic

De groot et al. (1994) p60
Tests on rubber

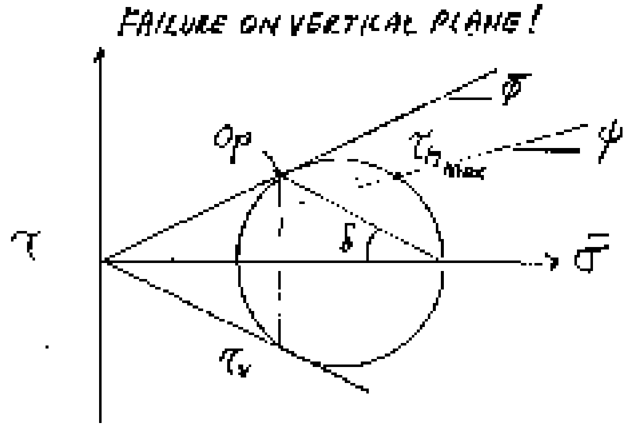
b) Indeterminate state of stress

• δ = ?   • N.G. $\tau, \bar{\sigma}, A$ • CCL opinion $G = E_u/3$

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

c) Randolph & Wroth (1981) interpretation

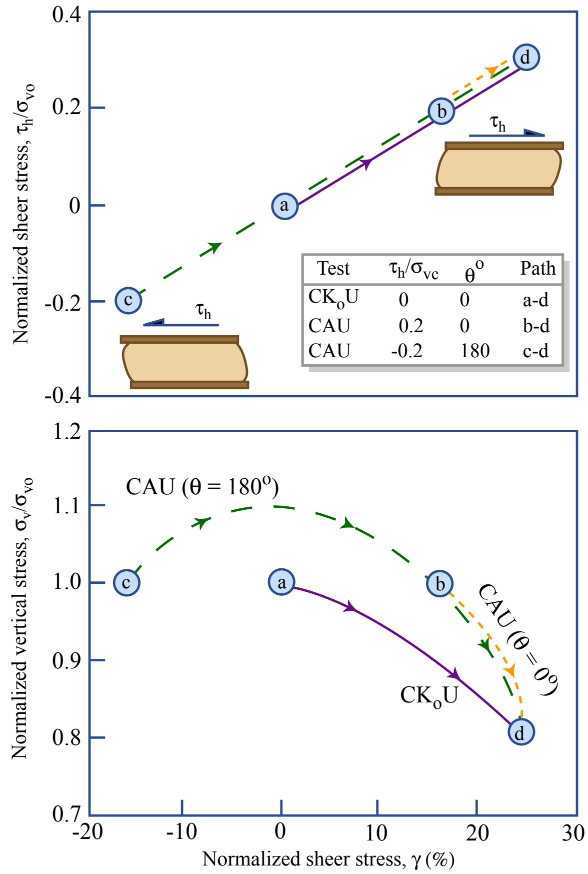
FAILURE ON VERTICAL PLANE!



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

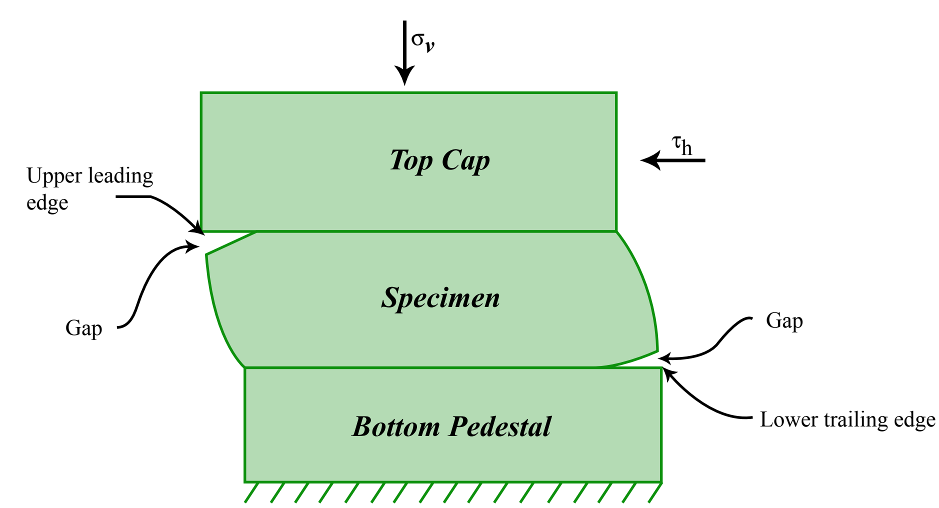
τ_v for pile capacity

4/25/95



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 ($\gamma > 10\%$)

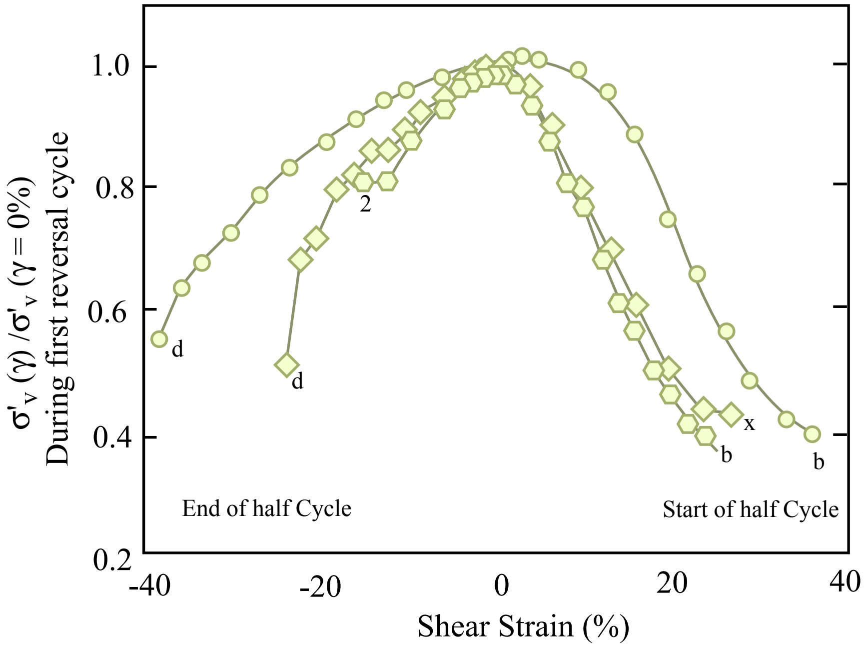
Figure by MIT OCW.

Adapted from:

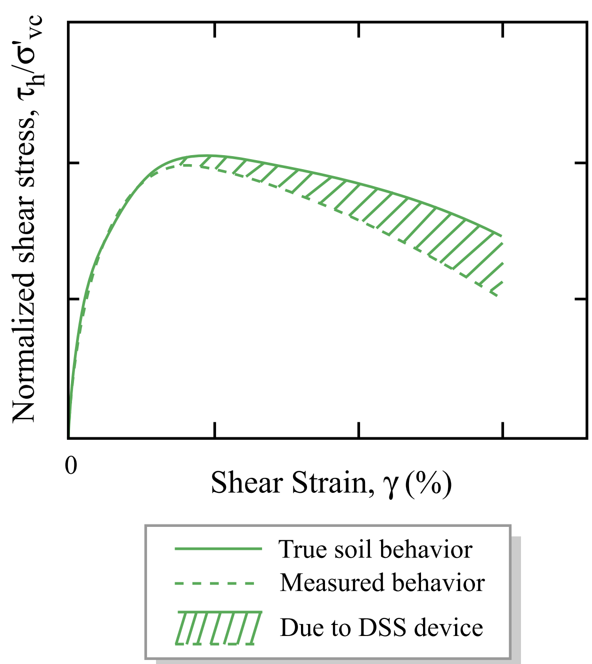
1994 JGE, ASCE 120(5)

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

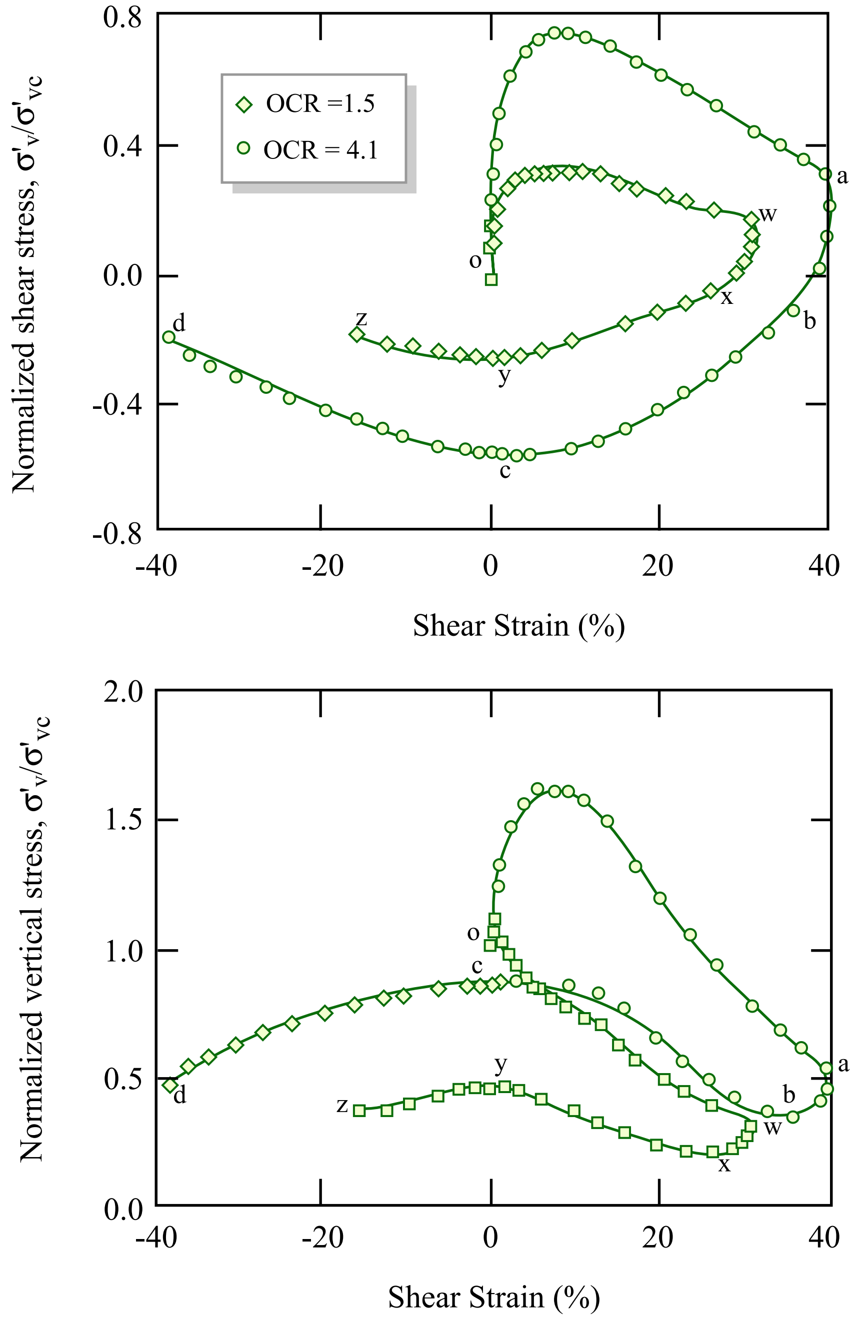


Vertical stress during first reversal stage (Normalized by σ'_v at $\gamma = 0\%$) versus shear for undrained cyclic geonor CK_oUDSS test on BBC and SFBM



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

Figure by MIT OCW.

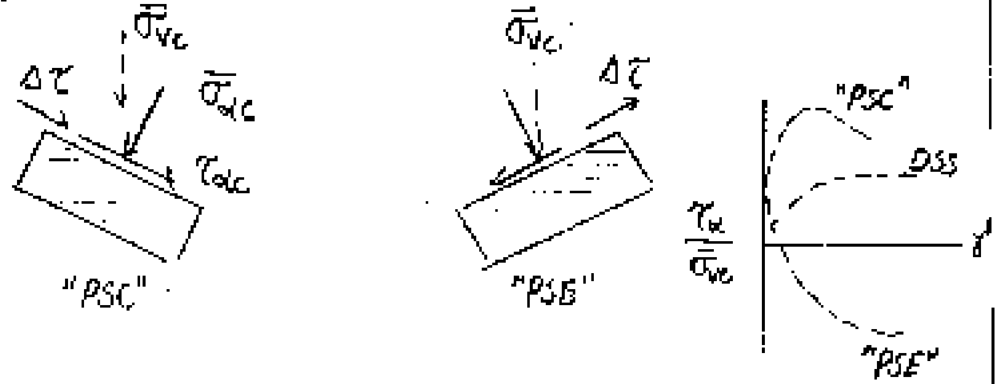


Normalized results of cyclic Geonor CKoUDSS test on SFBM with σ'_{max} = 501 k Pa: (a) Shear stress-strain curve; and (b) Vertical effective stress versus shear strain

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 Silver Device
↳ higher s_u 15±5%

(6) Cambridge SSA

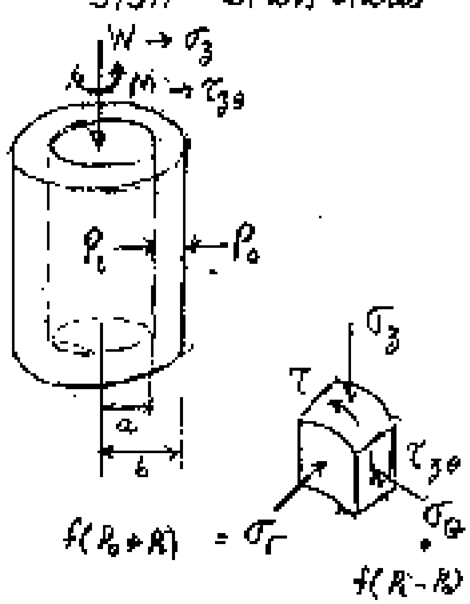
(7) CCC opinion of DSS (SHANSEP testing)
- Reasonable s_u for stability analyses & easier/cheaper CK₀UC/E
- Reasonable E_u & hyperbolic parameters for FEE(CA)

DSS-2 p7b
- Excessive strain softening at large strains, p6b

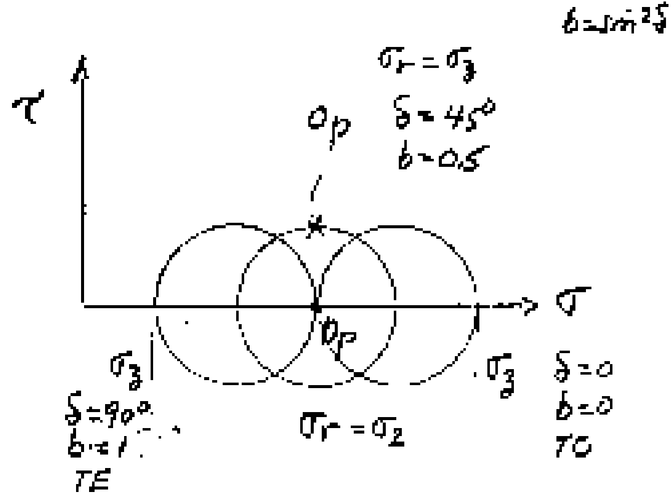
5.5 Torsional Shear Hollow Cylinder (TSHC)

SF 2.4.3

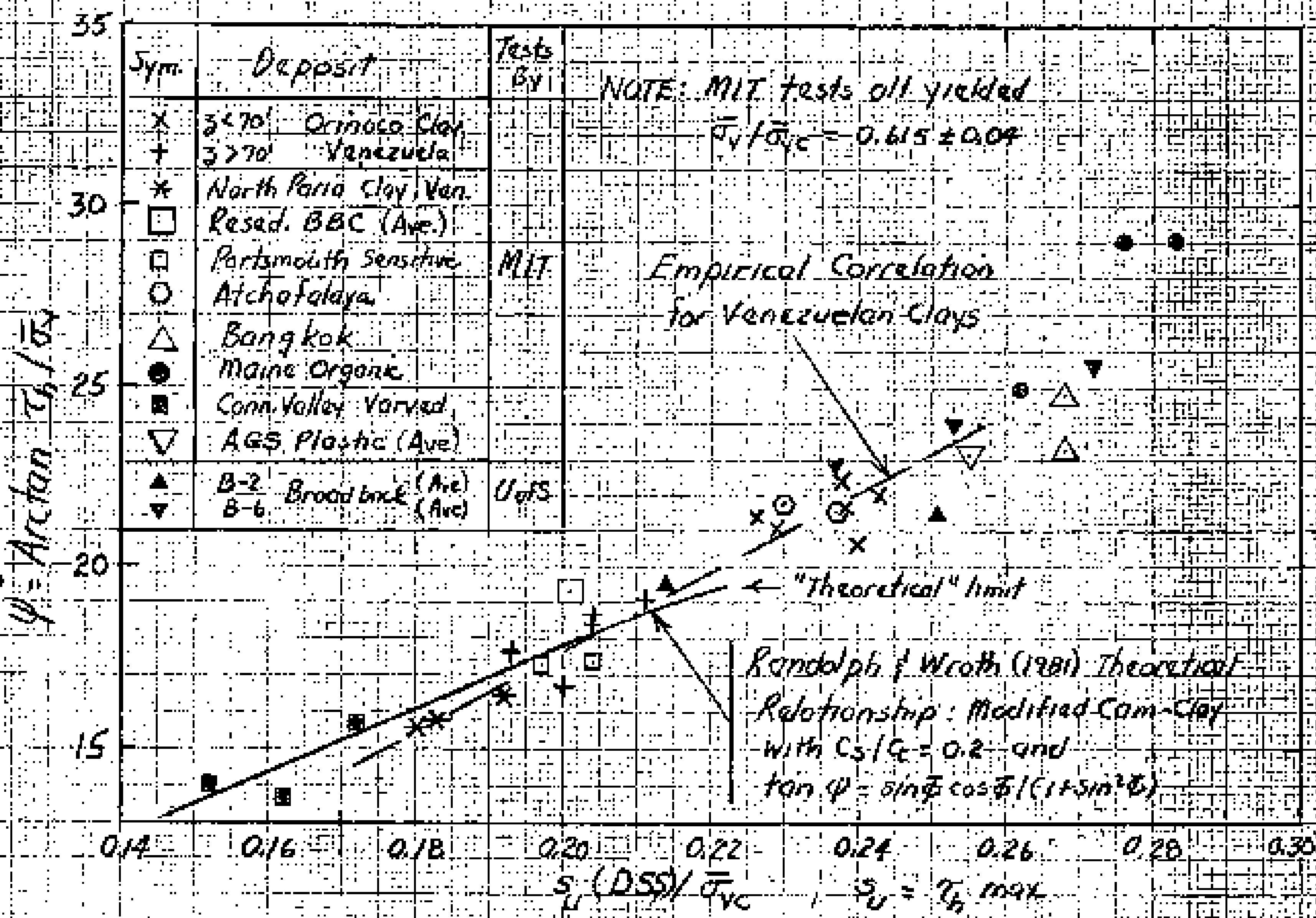
5.5.1 Stress States



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



CCL 4/22/82
1.322



CK_oUDSS Test Results on Normally Consolidated Clays

IIC

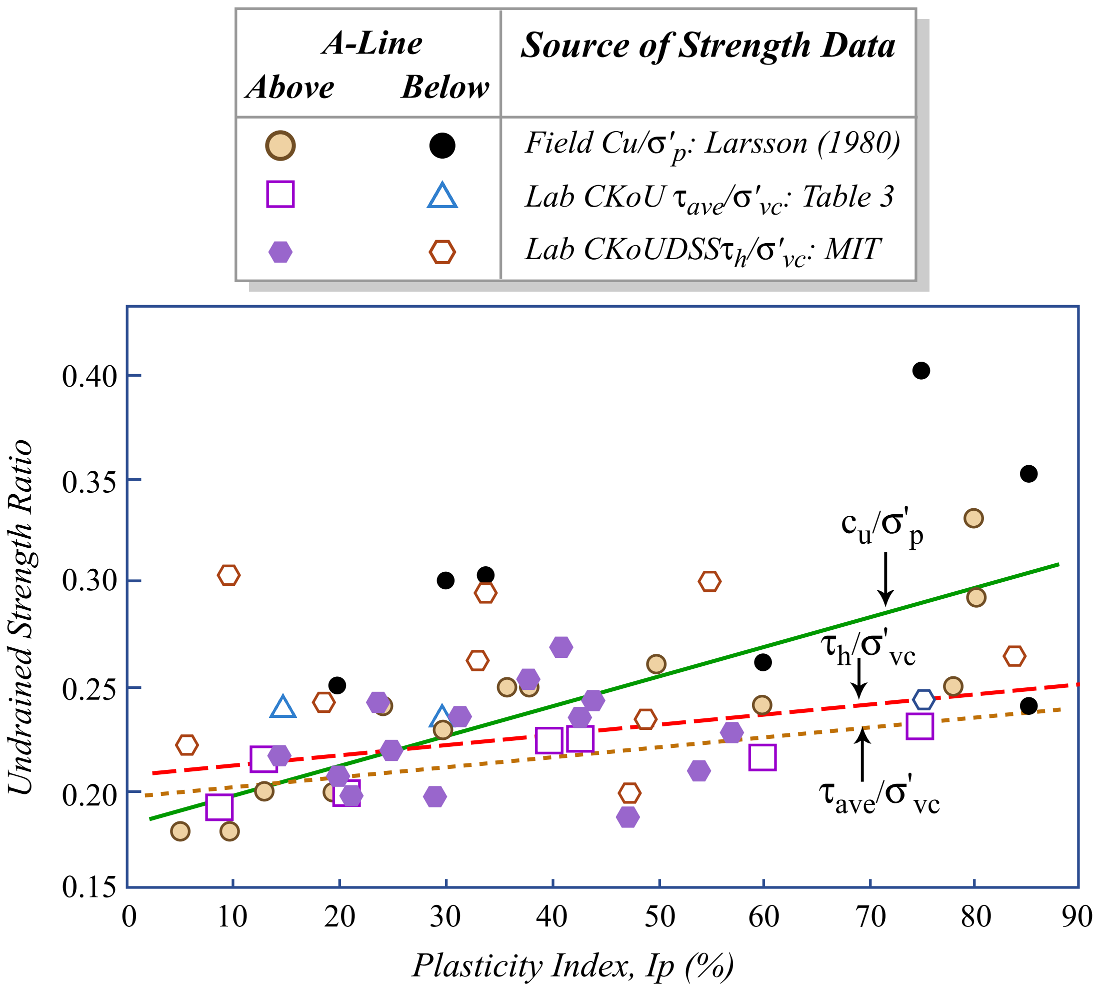
DSS-1

p7a

Ca 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)

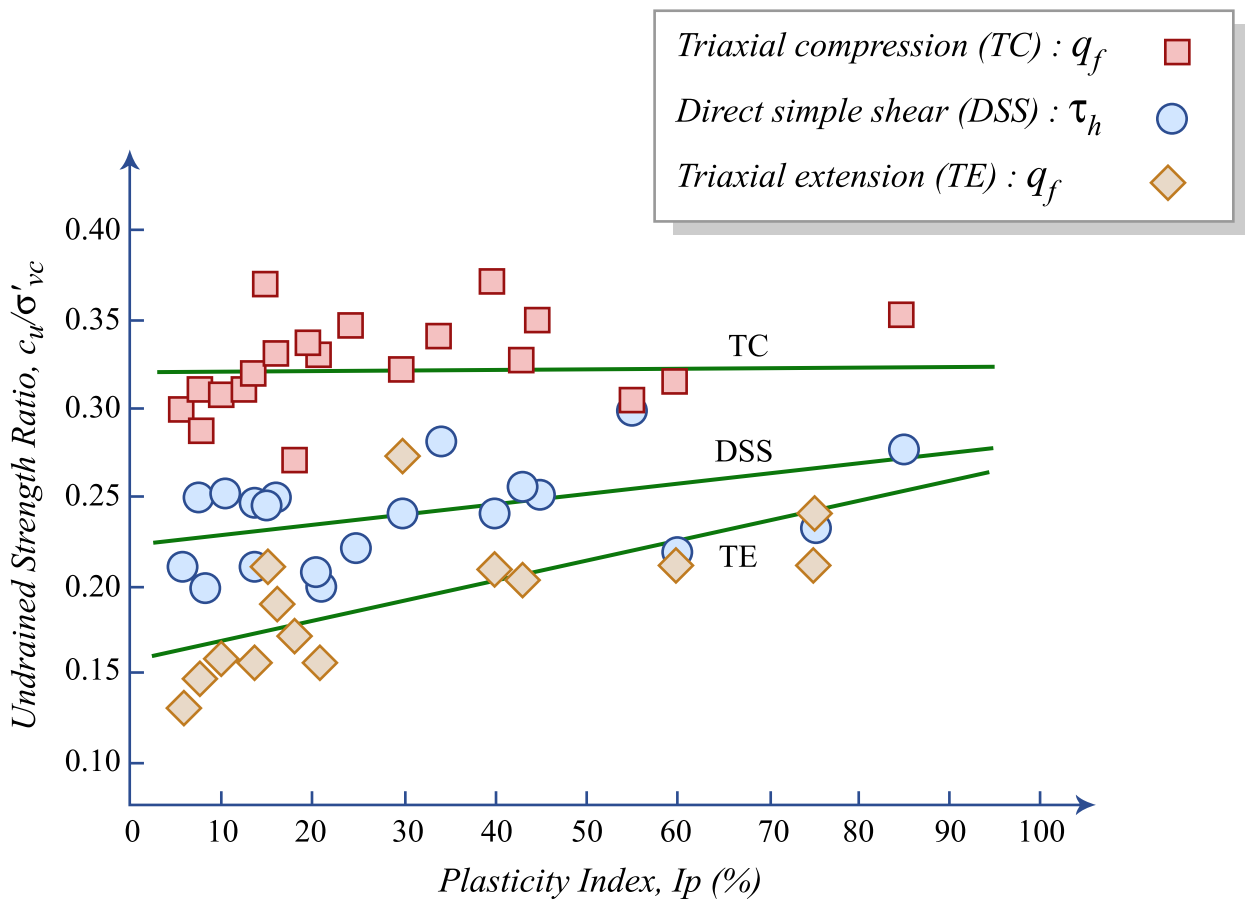
Note : Linear Regression lines for clay data

Figure by MIT OCW.

Cu 4/93 1,322 .

IIC

DSS-3
p7c .



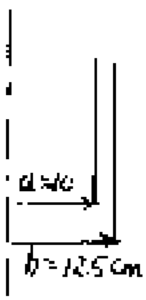
Undrained strength anosotropy from CK_0U tests on normally consolidated clays and silts.

Figure by MIT OCW.

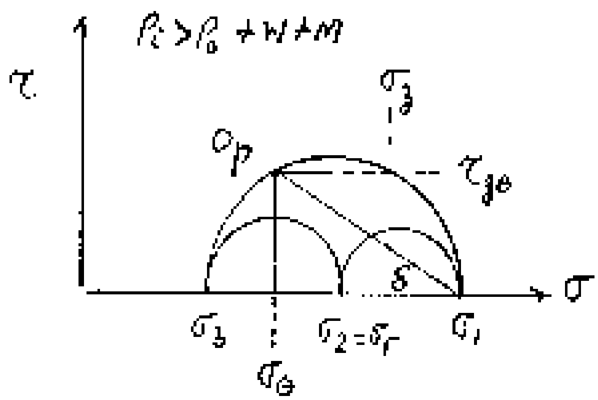
5.5.1 (a) Continued

- 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) <u>Imperial College</u> 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 $\delta \leq 45°$)
- $P_i > P_o$ to left of $b = \sin^2\delta$ line
  $P_i < P_o$ " right " " "



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

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

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

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

<u>Advantages</u>

Most versatile of any device

Data from CIU tests on sand look excellent

(Fig. 20 SF - cover later under sand anisotropy)

<u>Disadvantages</u>

Very complex & costly

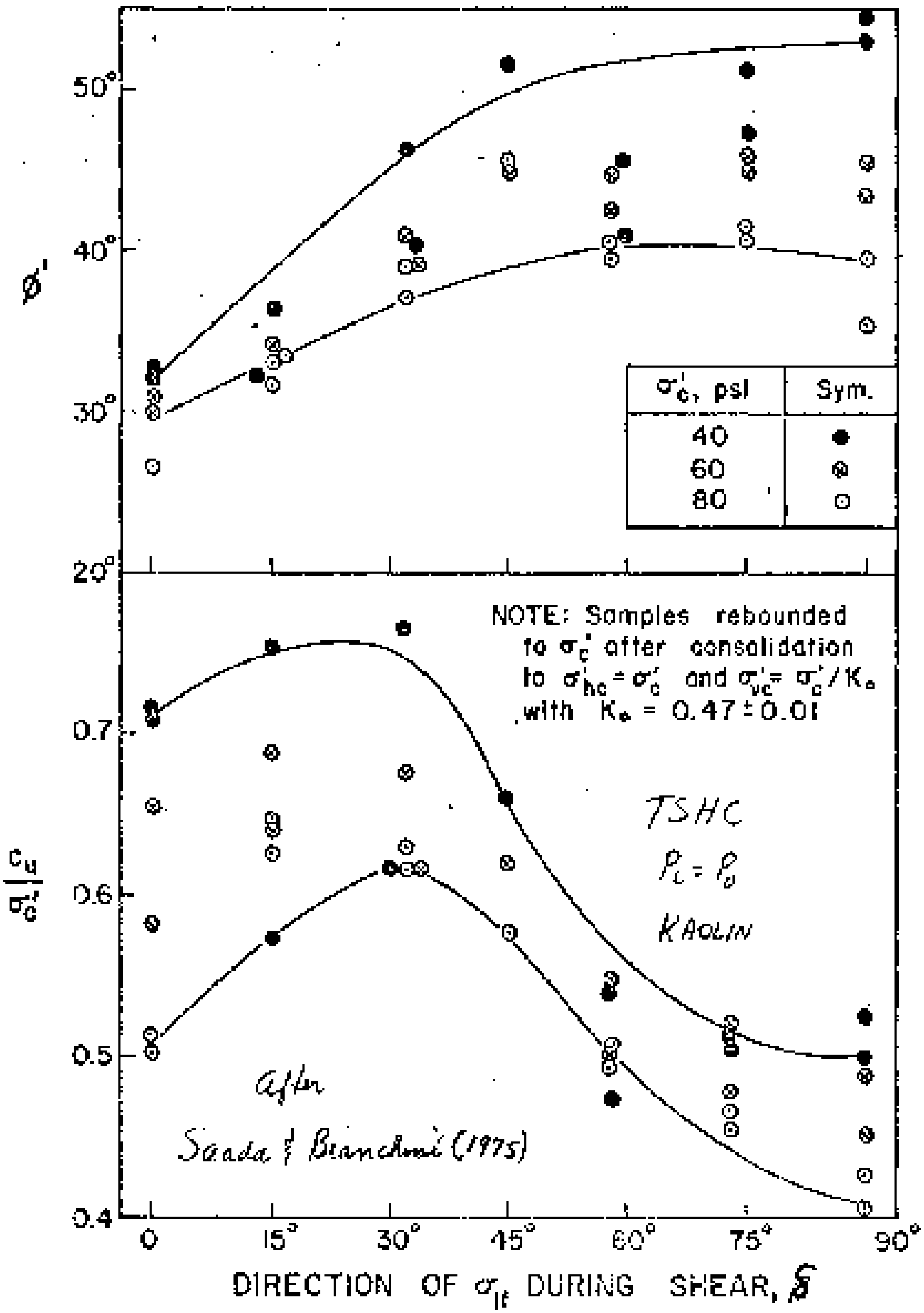
Non uniform stresses with $P_i \neq P_o$

→ End effects

Problems of testing clays

→ Need to measure strains internally

CCL 4/85 IIC 1.322 p8a

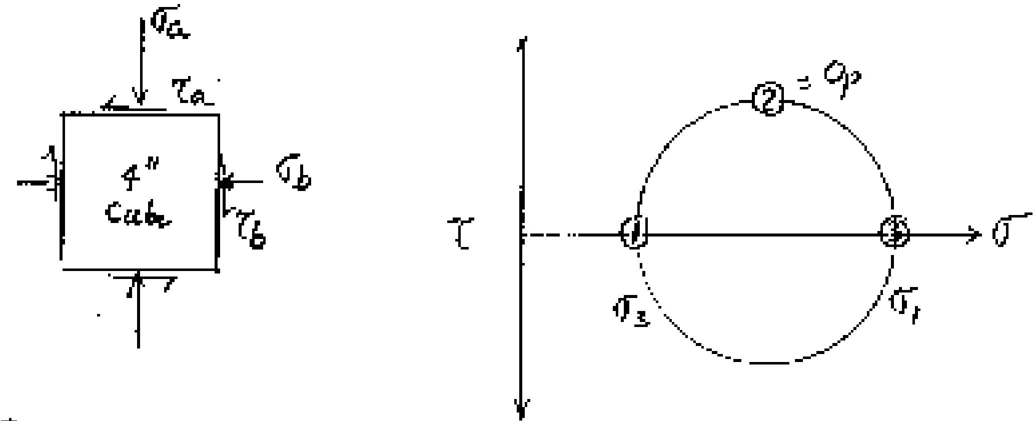


NOTE: Same basic material as Fig 19 of S.E.

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

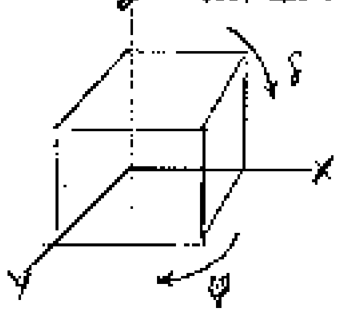
① $\sigma_a > \sigma_b$, $\tau = 0$

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 + induced 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 Computer → displacements ± 2 μm

6. INFLUENCE OF K_c AND b

6.1 Influence of K_c

4/11/88 Replaced by 6.1-1 → 6.1-5 (after p10)

6.2 Influence of b

(1) General considerations of increasing b

- Effect on s_u
- " " $\bar{\phi}$

Mohr Coulomb →
MCC →

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

Lade & Duncan (1975)
I_1^3 / I_3 = constant

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

- Handout (p 10e)
- 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

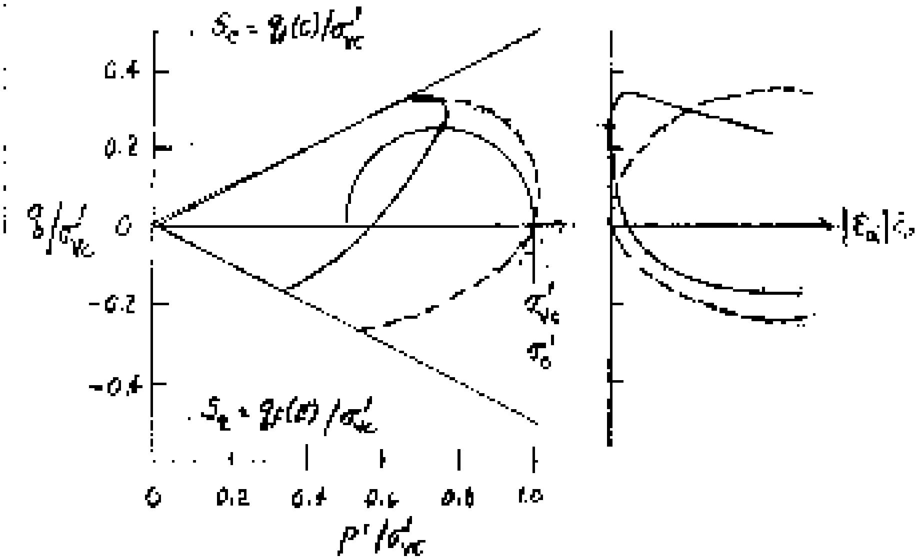
PS vs TC →
Inc. s_u
" $\bar{\phi}$
Dec. ε_f

6. INFLUENCE OF K_c AND b

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

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

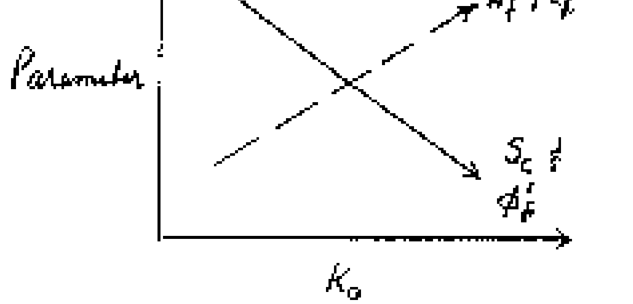
- Approx same S_c ± 10-15%
- Large decr in ε_f
- Often incr in strain softening

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

6.1.2 Influence of K_o on CK_oUC 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_oUC testing (Fig 15, CCL '91)
But NOT TRUE

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



Increasing K_o → decreasing S_c* due to:

- Lower ϕ'_f
- Higher A_f

Also increasing ε_f

* Incr K_o = 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_0UC Tests

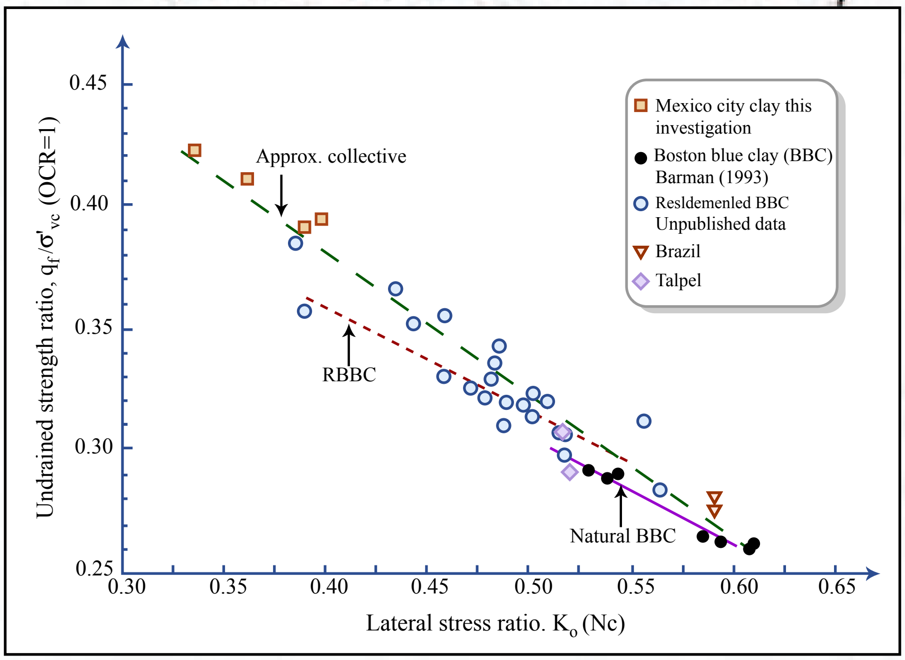


Figure by MIT OCW.

• Although collective data on ~~wide~~ wide range of soils → good correlation ($S_c = 0.62 - 0.60 K_0$) individual clays have different trends

• MCC is very plastic, but with high diatom content → high ϕ' → low K_0 → high S_c

6.1.3 Influence of K_0 on CK_0UE Behavior

1) Should expect increasing K_0 → increase in S_E since:
 • starting from higher p'_c/σ'_{vc} • smaller Δq_f $[(\varepsilon_c, q_0 - q_f(E))]$

2) Only available data (below) supports this expectation

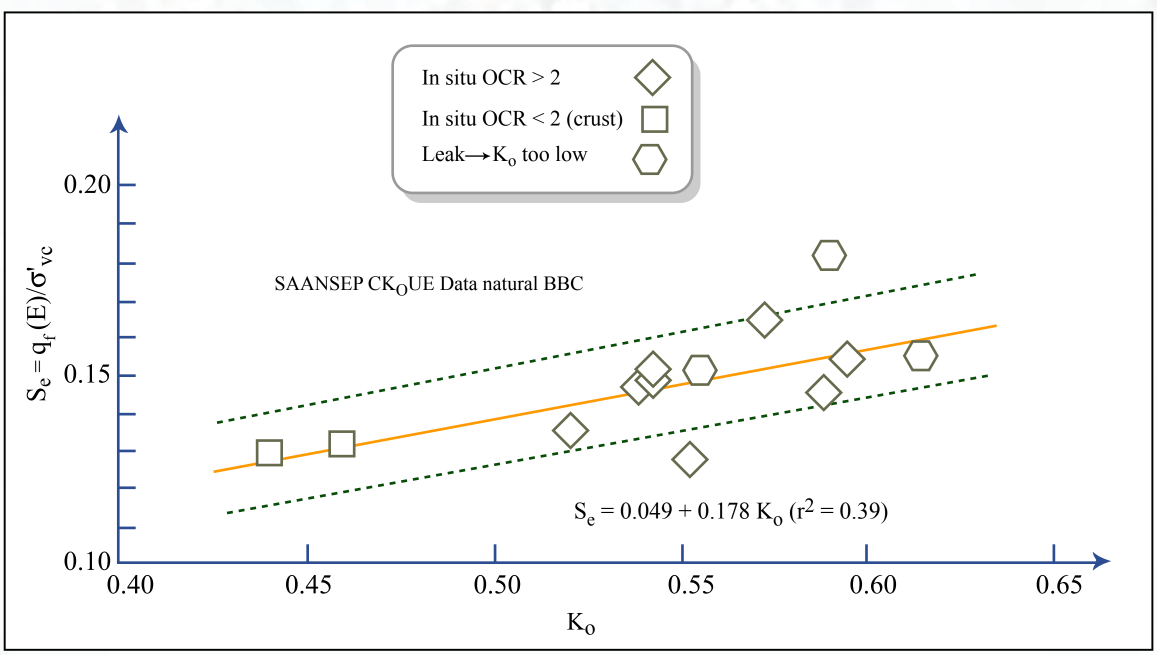


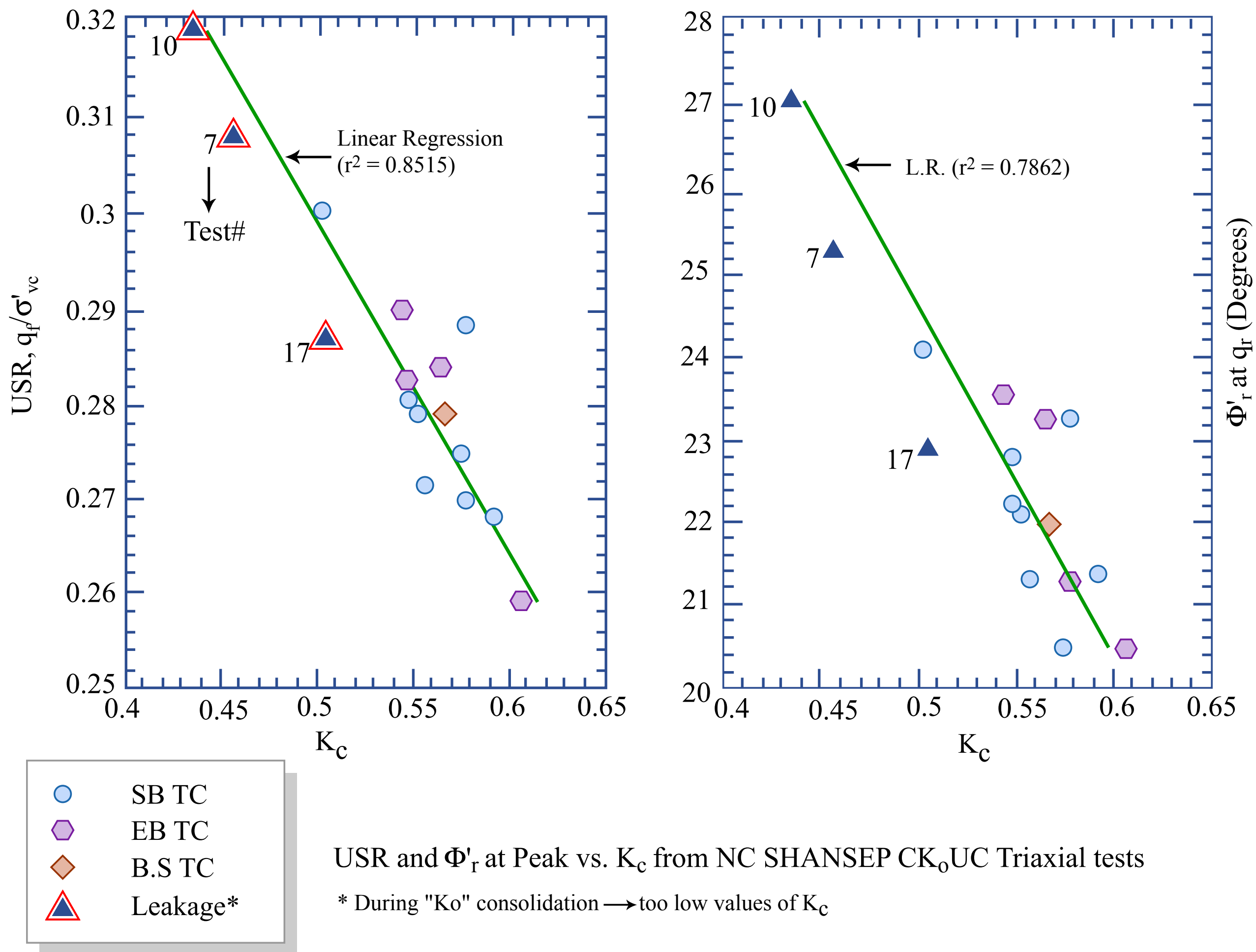
Figure by MIT OCW.

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 clear data

2) For TE, increasing $K_o \rightarrow$ significant increase in $S_c = q_f(E)/\sigma'_{vc}$. 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_oUC Triaxial tests

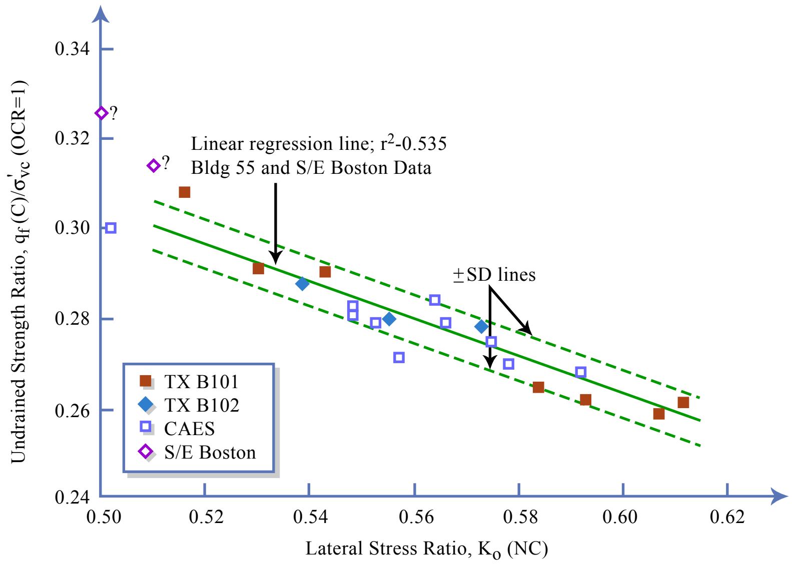
* During "Ko" consolidation ⟶ too low values of K_c

Figure by MIT OCW.

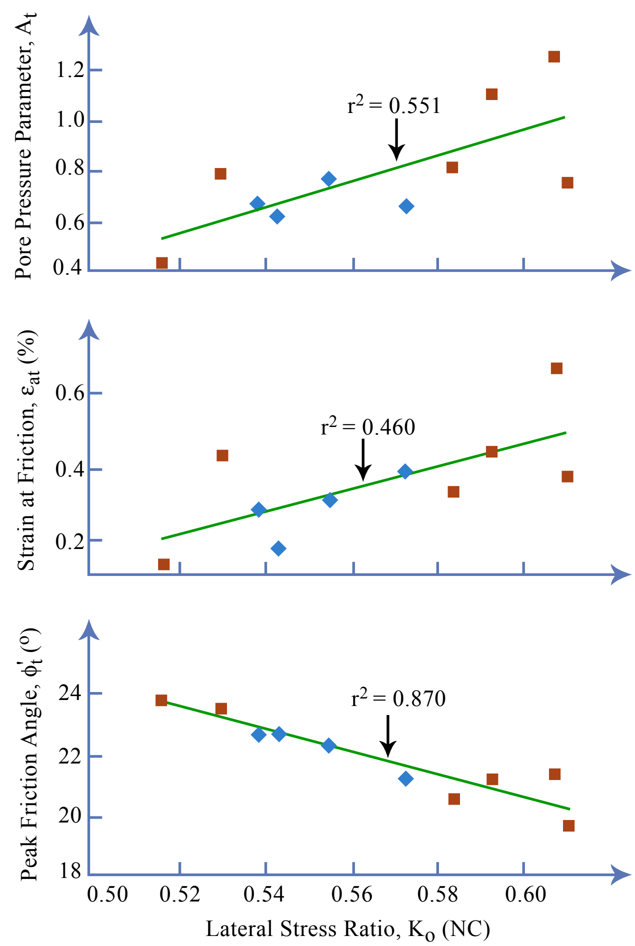
Adapted From de La Beaumelle (1991) SM thesis : HSA STP CA/T Project

- 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 $S_c = q_f(C)/\sigma'_{vc}$

K_c 1



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.

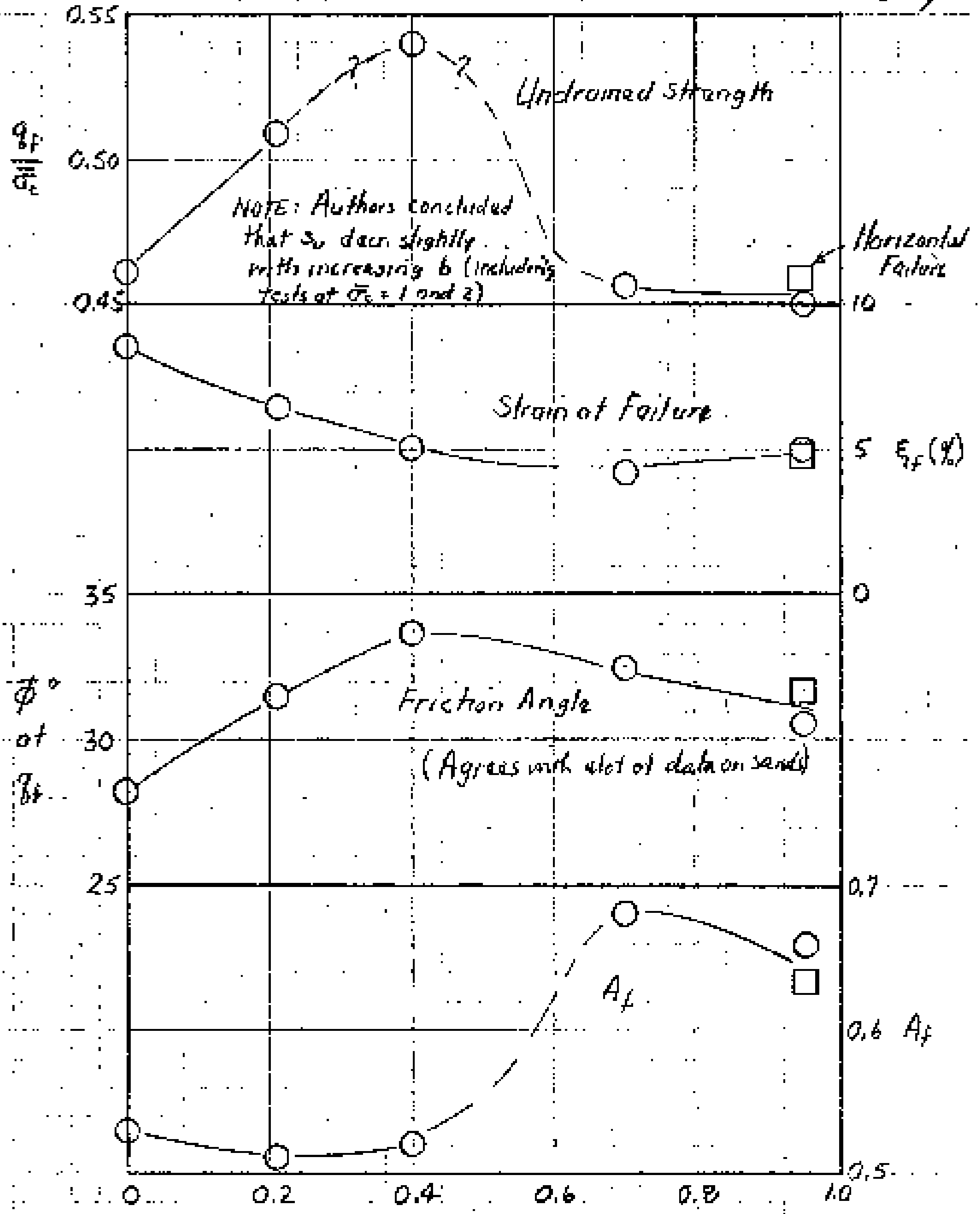
CK_0UC Data on Natural BBC (Berman 1993)

• TX B101 & B102 = MIT Bldg 68 (Biology)

• S/E Boston = CA/T STP by MIT; (excludes 3 tests with slashes)

K_02

$\overline{CIU}$ True Triaxial on Remolded Grundite (w_L=54%, PI≈31.1%)
$\overline{\sigma}_c$ = 1.5 kg/cm² (Data from Lade & Musante, 1978 JGED 672)



(3) Comparison PS vs Triaxial CK_0U Data (Table 1 Tokyo) p434

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

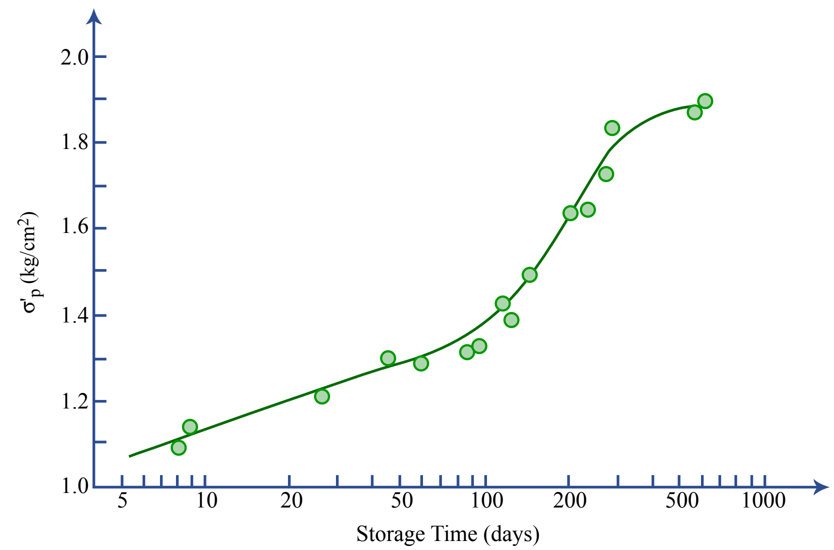
2) PSC/TE $\delta = 0$
DSS $\delta = ?$
PSE/TE $\delta = 90°$
} → $K_s = s_u(H)/s_u(V) = s_u(E)/s_u(C)$

- 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

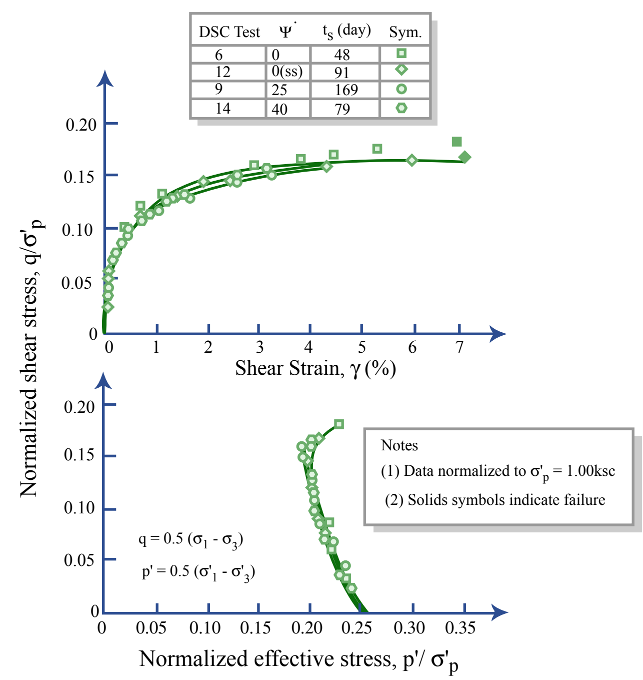


Effect of thixotropy on preconsolidation pressure of resedimented BBC (O' Neill, 1985)

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).

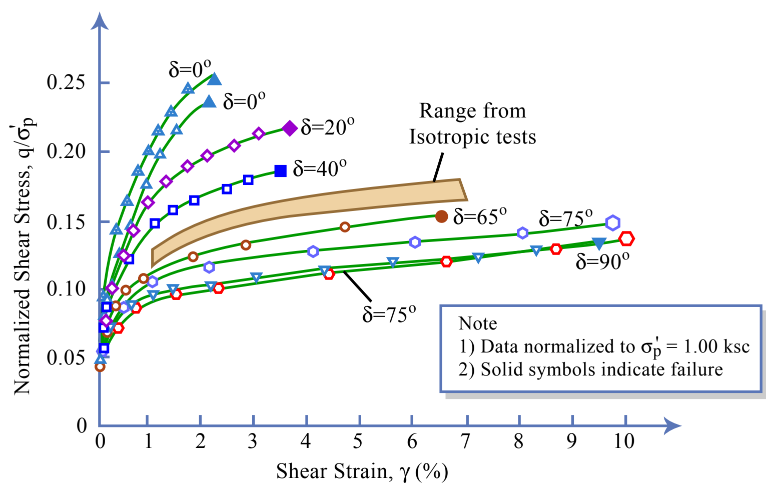
Figure by MIT OCW.

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 $K_o = 1$, 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	▽

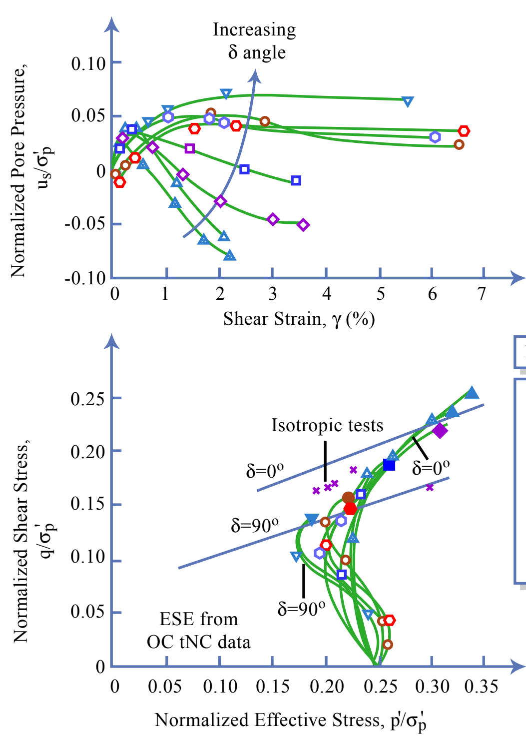
Normalized Shear Stress vs. Strain from CU DSC Anisotropic Tests on BBC at OCR = 4.

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 peaks + 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	▽

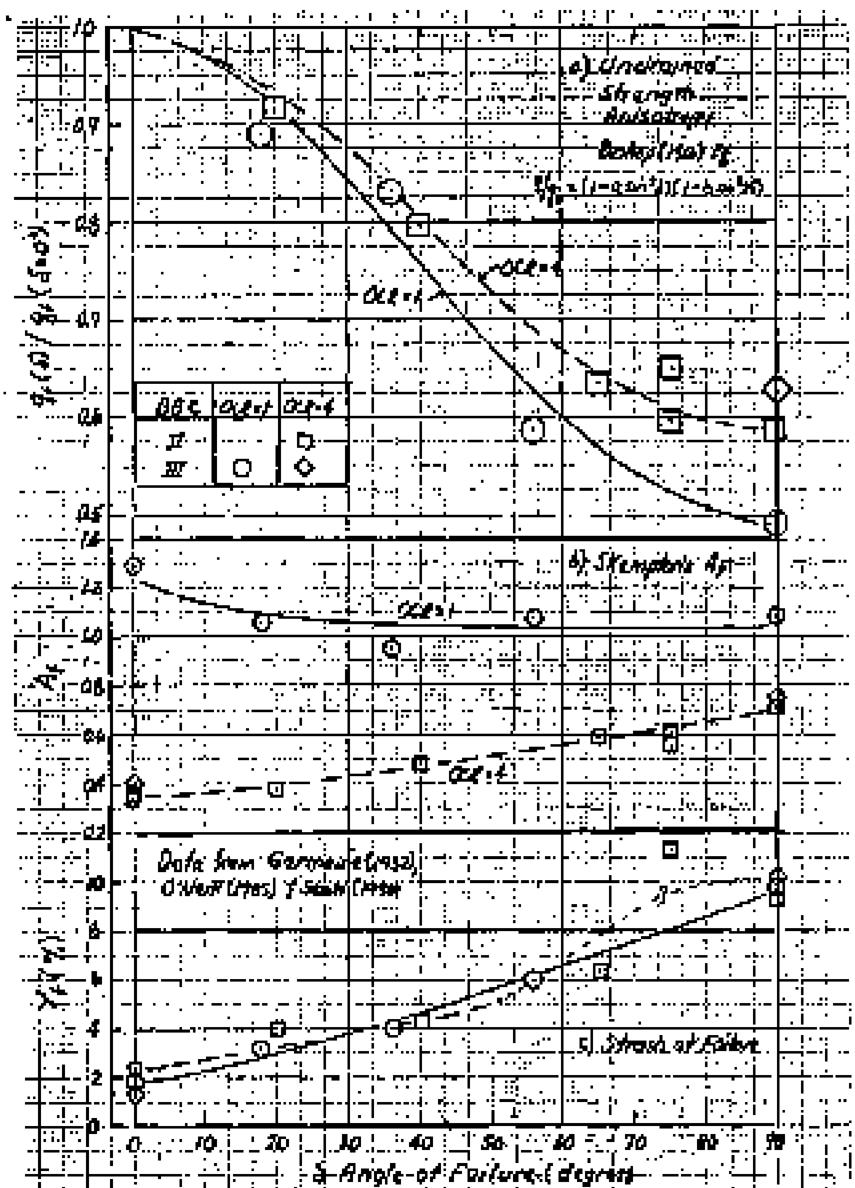
Normalized Pore Pressure and Effective Stress Paths for CU DSC Anistropic Tests on BBC at OCR = 4.

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)



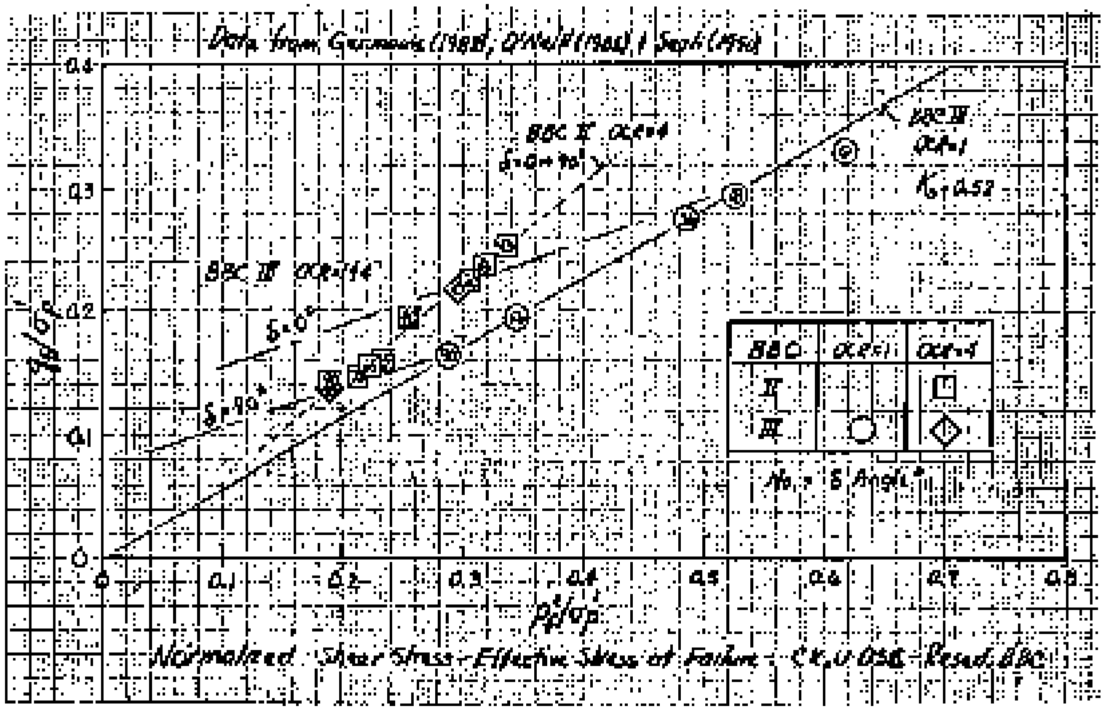
Summary of CK_0U DSC Tests on Resedimented BBC

1) Trends in $s_u(\delta)$.
   - Similar shapes w/ OCR=1 → more anisotropy since includes both intrinsic and induced anisotropy ($q_0>0$)

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

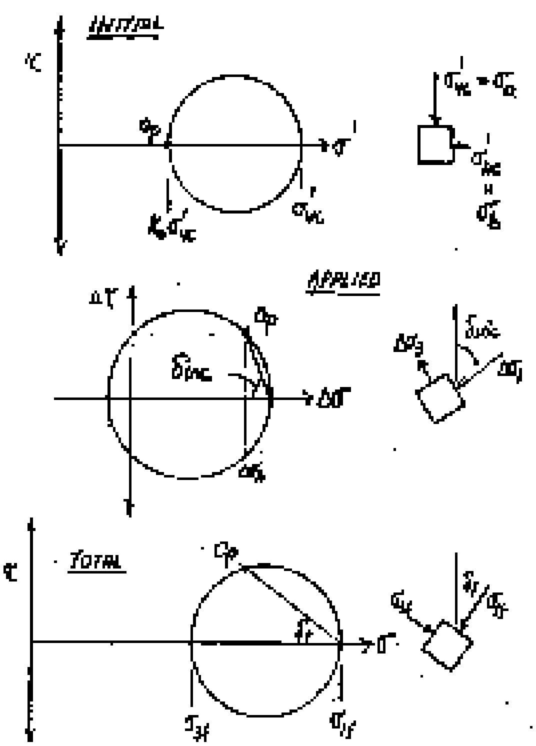
7.3.2 Continued



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

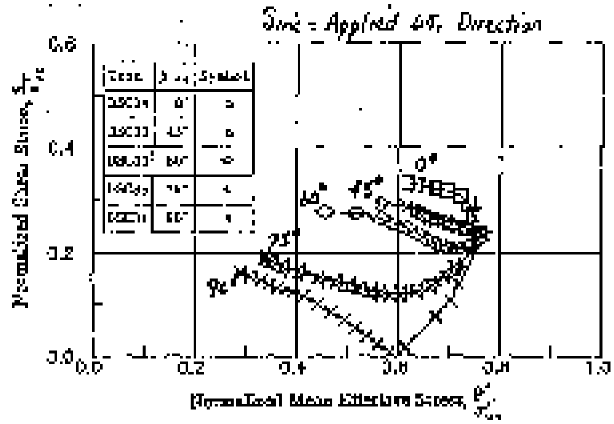
7.3.3 DSC Data at OCR=1 : Comparison of Measured vs MIT-E3 Predicted

CK0U DSC BBC OCR=1 (Seah, 1990)
[$q = 0.5(\sigma_1-\sigma_3)$, $p' = 0.5(\sigma'_1+\sigma'_3)$]



1) Experimental procedure 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

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

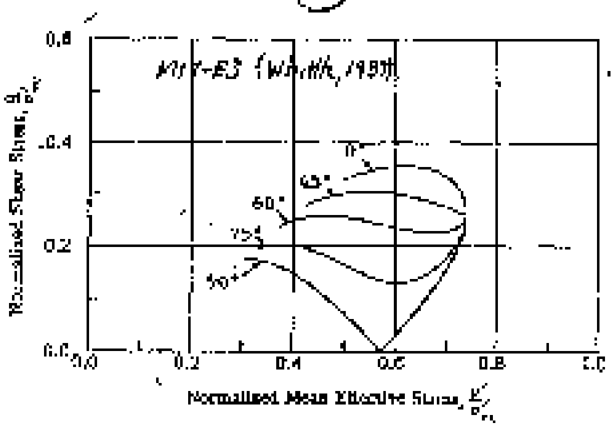


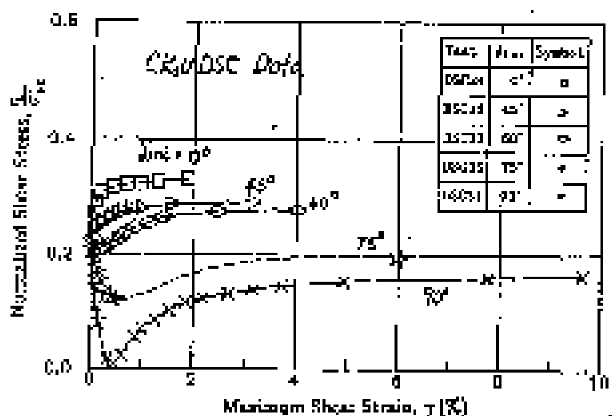
(a)

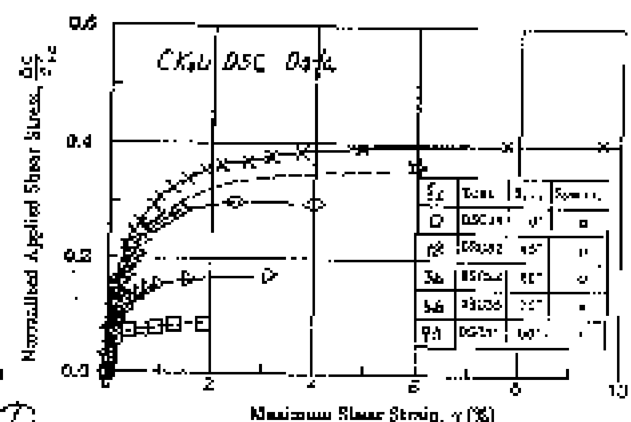
2) Predictions by Whittle (1987 ScD thesis) made *before* tests were run (Type A)

(a) Comparison of ESP

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



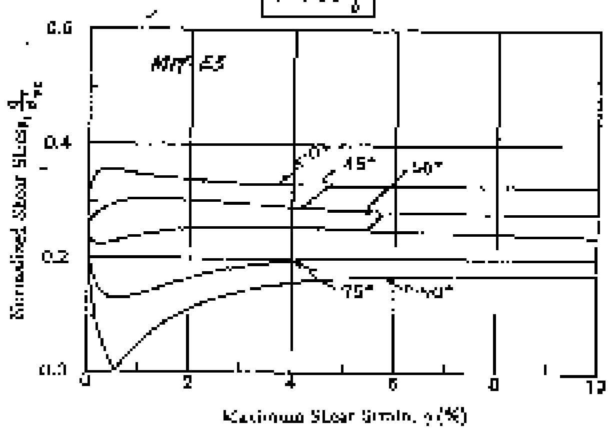


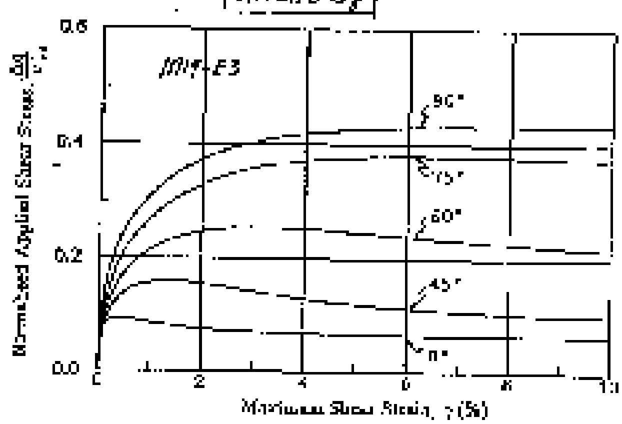


(b)

TOTAL q

APPLIED Δq

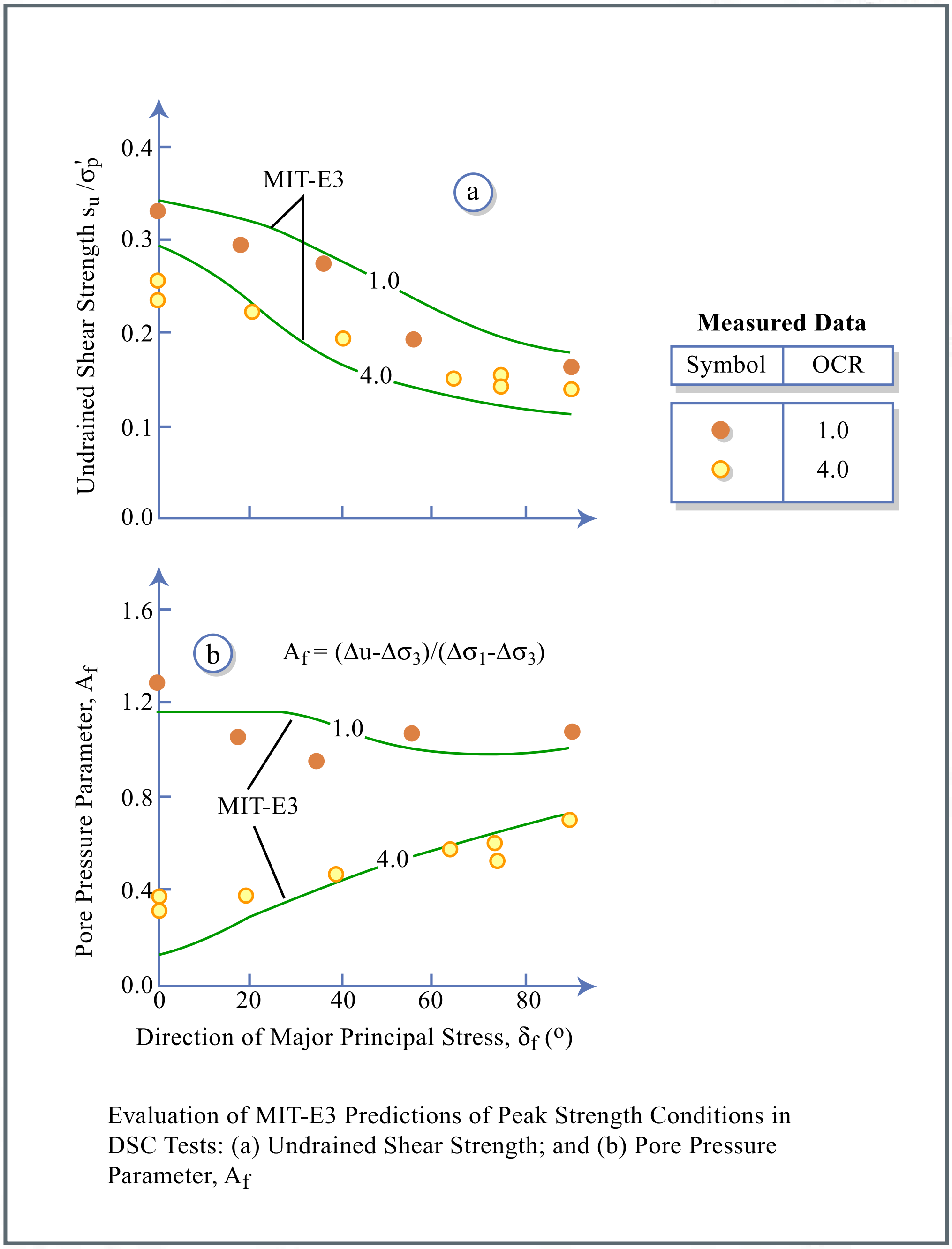




Adapted from:

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

© 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

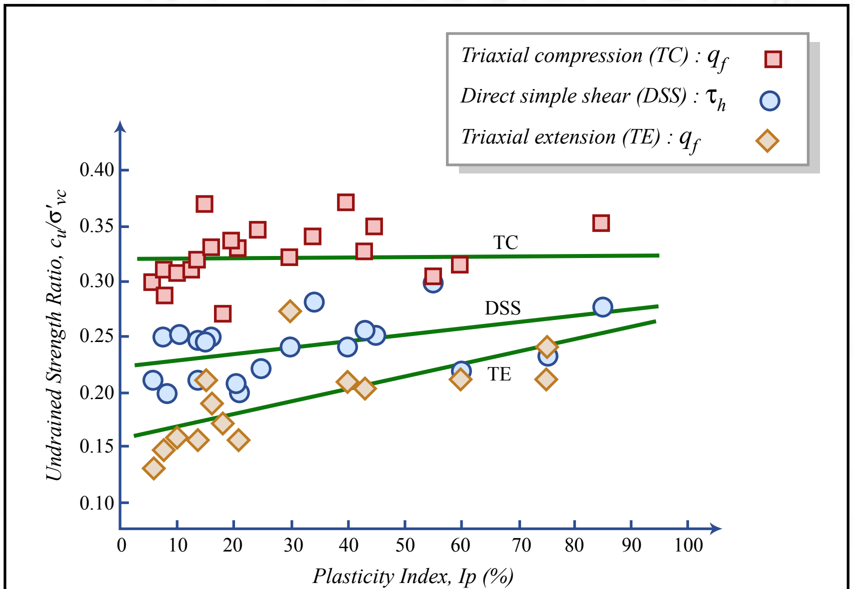
Figure by MIT OCW.

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_0U PS, TX and DSS Testing

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

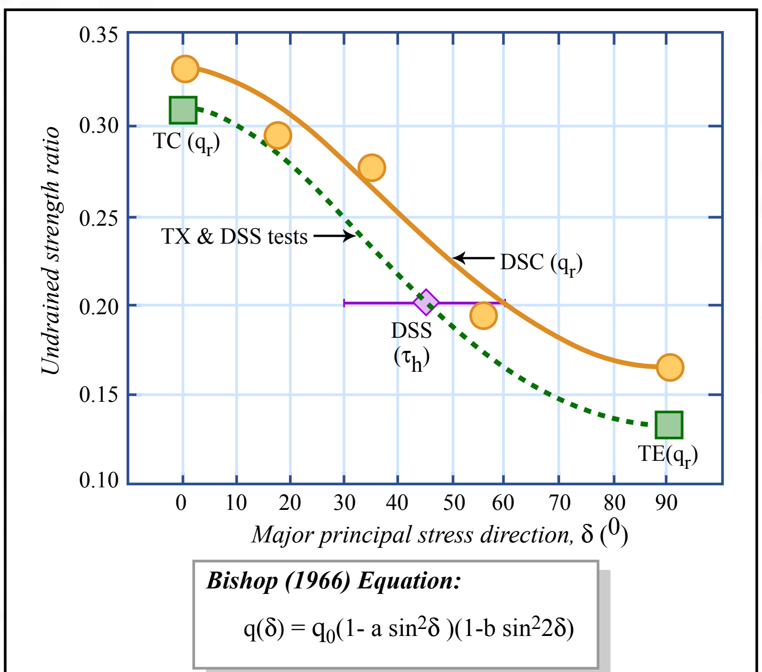


Undrained strength anosotropy from CK_OU tests on normally consolidated clays and silts.

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.

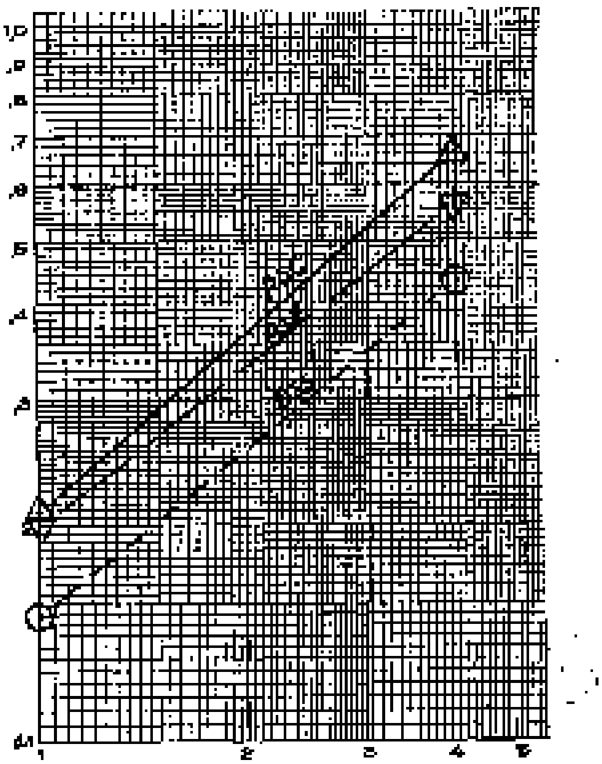
Trends with δ (Fig 3)

- 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 1978



- Varved clays are unusual since $CK_oUSS \to$ lowest s_u/σ'_{vc}, i.e., below compression & extension
- In addition s_d & NC τ_h/σ'_{vc} DSS is extremely low
- Fig. at left from Table 2 (CCL '91) where $\tau = q\cos\phi'$ (not τ_h) needed for strain compatibility

7.4.3 s_u Anisotropy of OC Clays

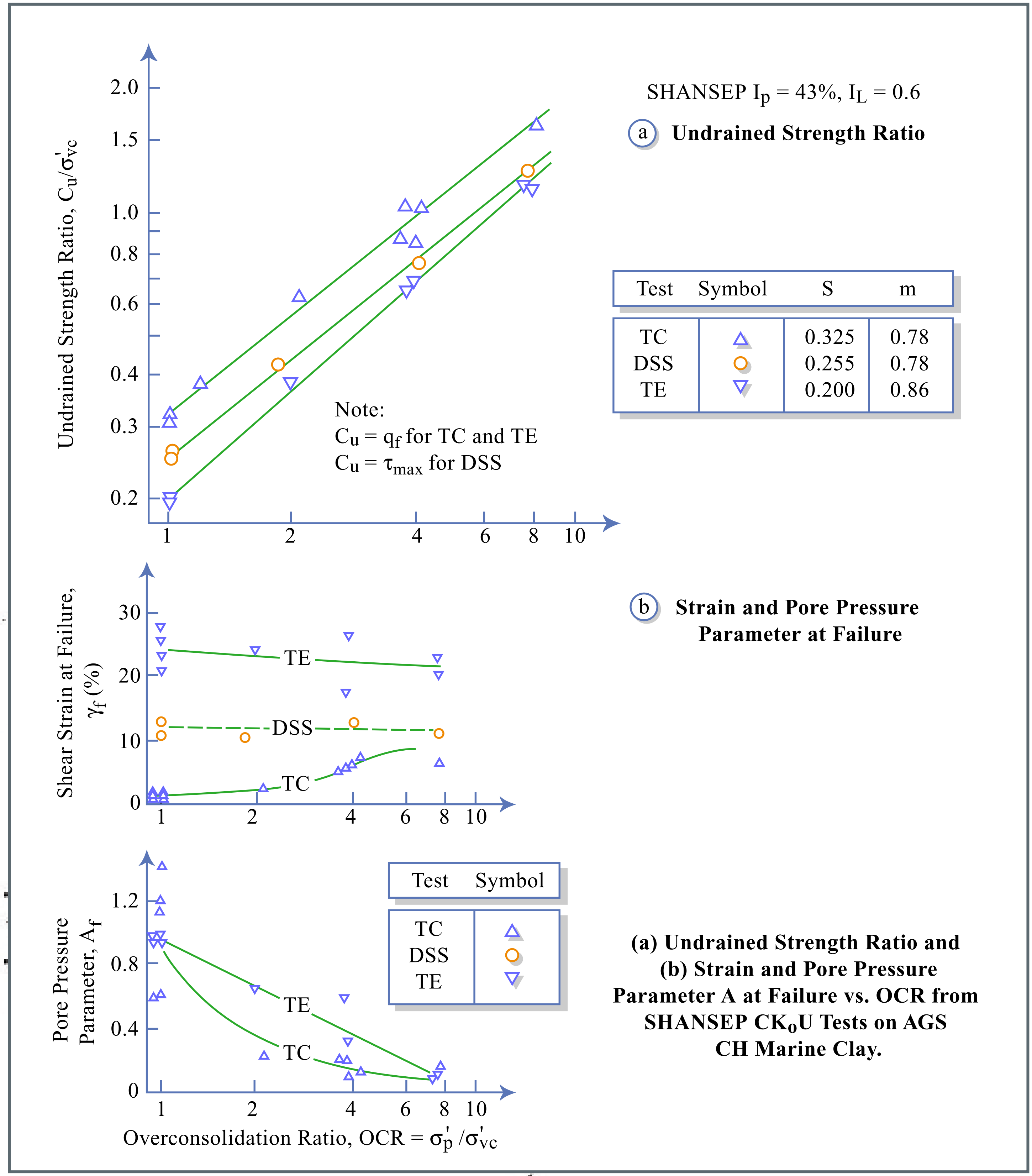
1) See p 7.4-3 for CK_oU TC, DSS & TE data vs OCR

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

2) See p 7.4-4 for $\log K_s$ vs $\log$ OCR on several clays: $\left\{K_s = \frac{s_E}{s_C}(OCR)^{(m_C - m_E)}\right\}$

- Increasing OCR → less anisotropy (except for BBC clay). Should expect since incr. OCR = incr. K_o → smaller f_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.

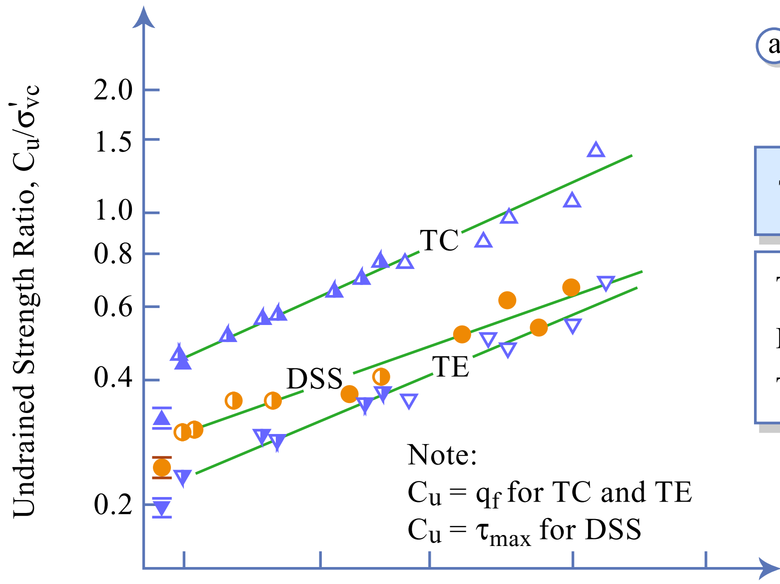
Figure by MIT OCW.

Adapted from Jamiolkowski et al (1985)

CCL 4/12/87 1.322 IIC 7.4-3
4/28/97

Recompression I_p = 13%, I_L = 1.9

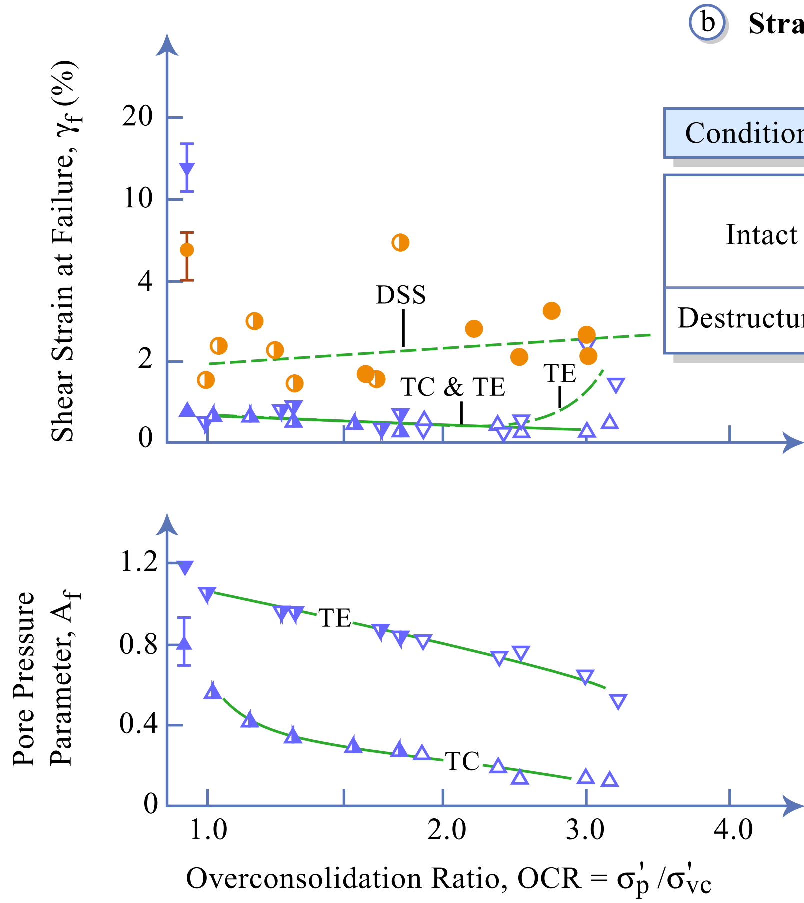
(a) **Undrained Strength Ratio**



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

(b) **Strain and Pore Pressure Parameter at Failure**

Condition	OCR	TC	DSS	TE
Intact	In situ	△	○	▽
	≥	▲	◐	▼
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.

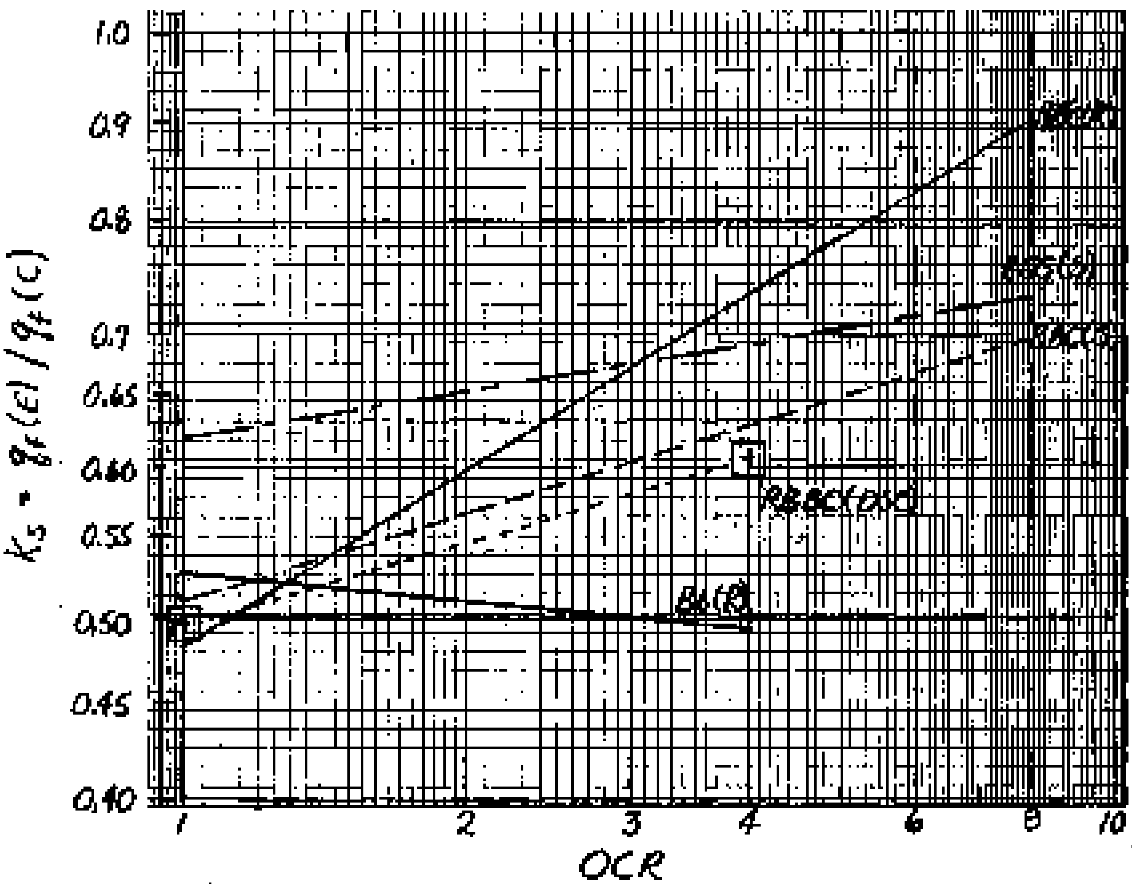
Figure by MIT OCW.

Adapted from Jamiolkowski et al (1985)

IIC

Label	Clay	Program	Reference
B6(R)	B-6 James Bay	CK_0-TX R	IIC p.120 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	I Section 7.3

R = Recompression S = SHANSEP



Variation in Undrained Strength Anisotropy with OCR

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

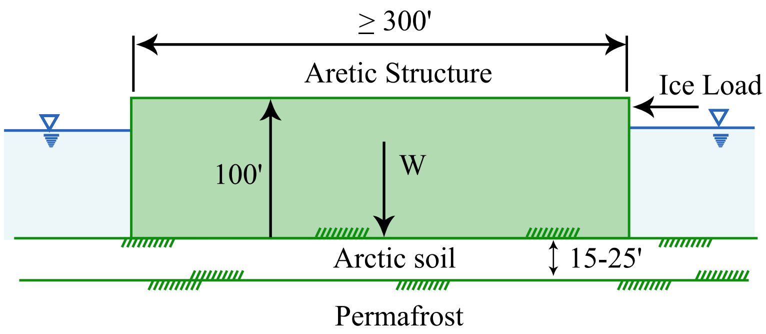
- MDSS = Multi-directional Direct Simple Shear apparatus. Same dimensions as Geonor DSS, but can apply two different horizontal shear stresses

2) Results

- MDSS-2 Schematic of problem
- " -3 " " MDSS
- " -4 Peak strength vs direction of sec. loading
- " -5 Typical stress-strain data in direction of sec. 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

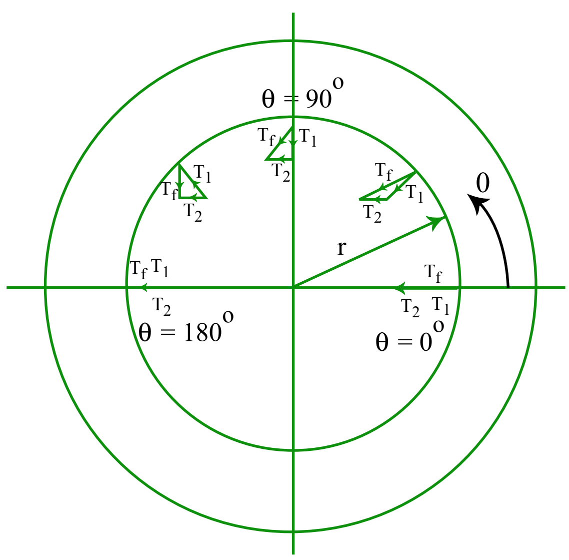


Shear stresses on soil at structure Interface (Top of foundation soil)

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 1.322

4/22/92

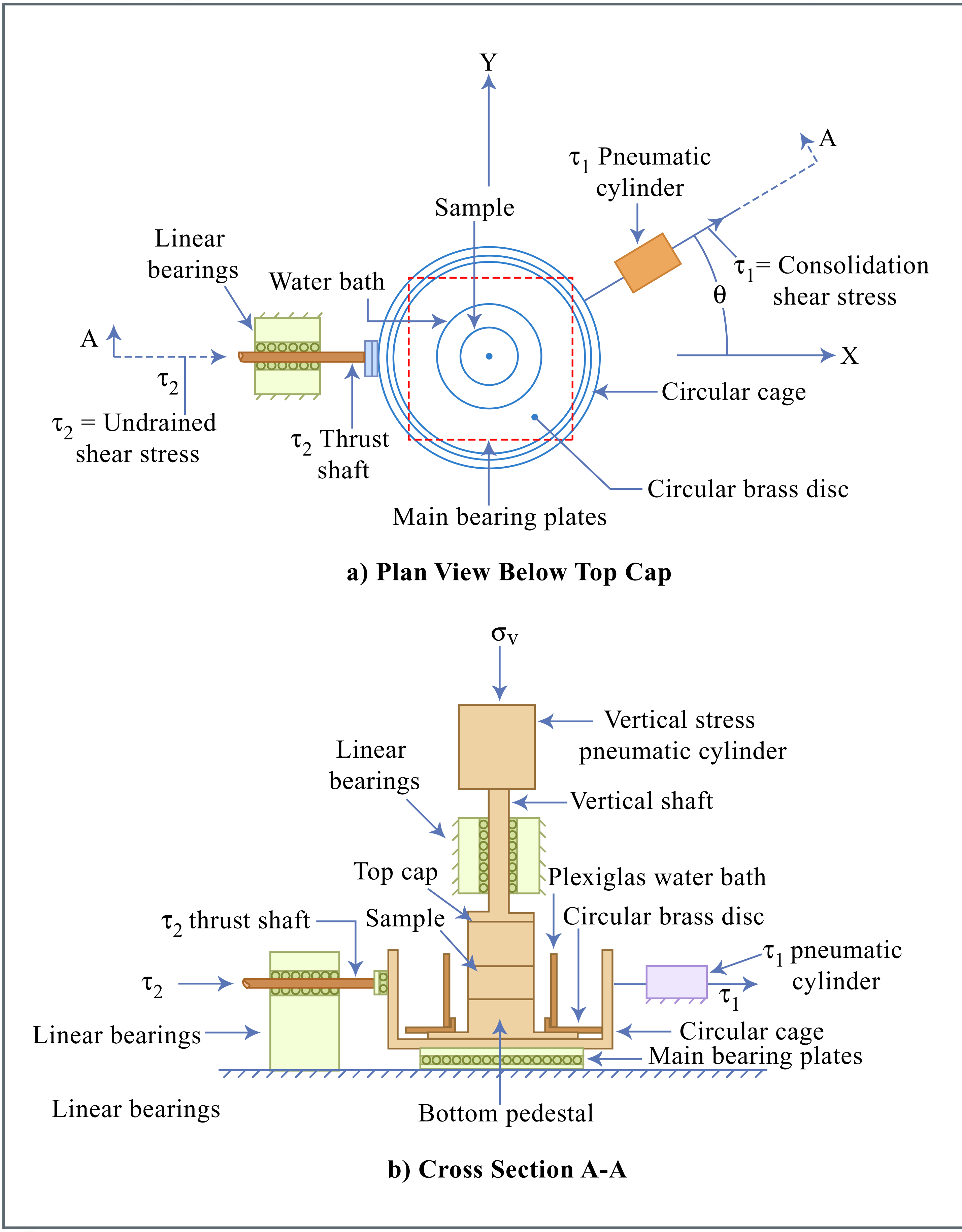


Figure by MIT OCW.

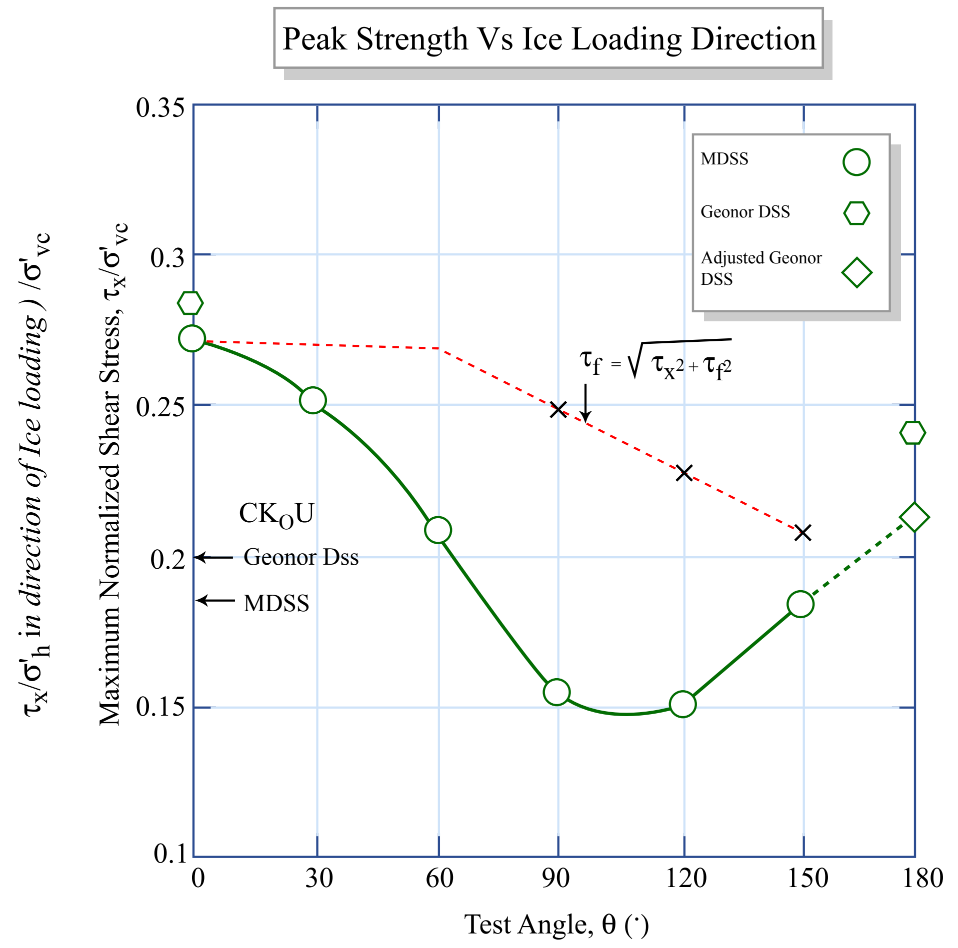
Figure 2 (from DeGroot, 1989)

10/89 CCL 4/22/92 IIC. MDSS-4 :B12

6/90 1.322 249

10/91

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:

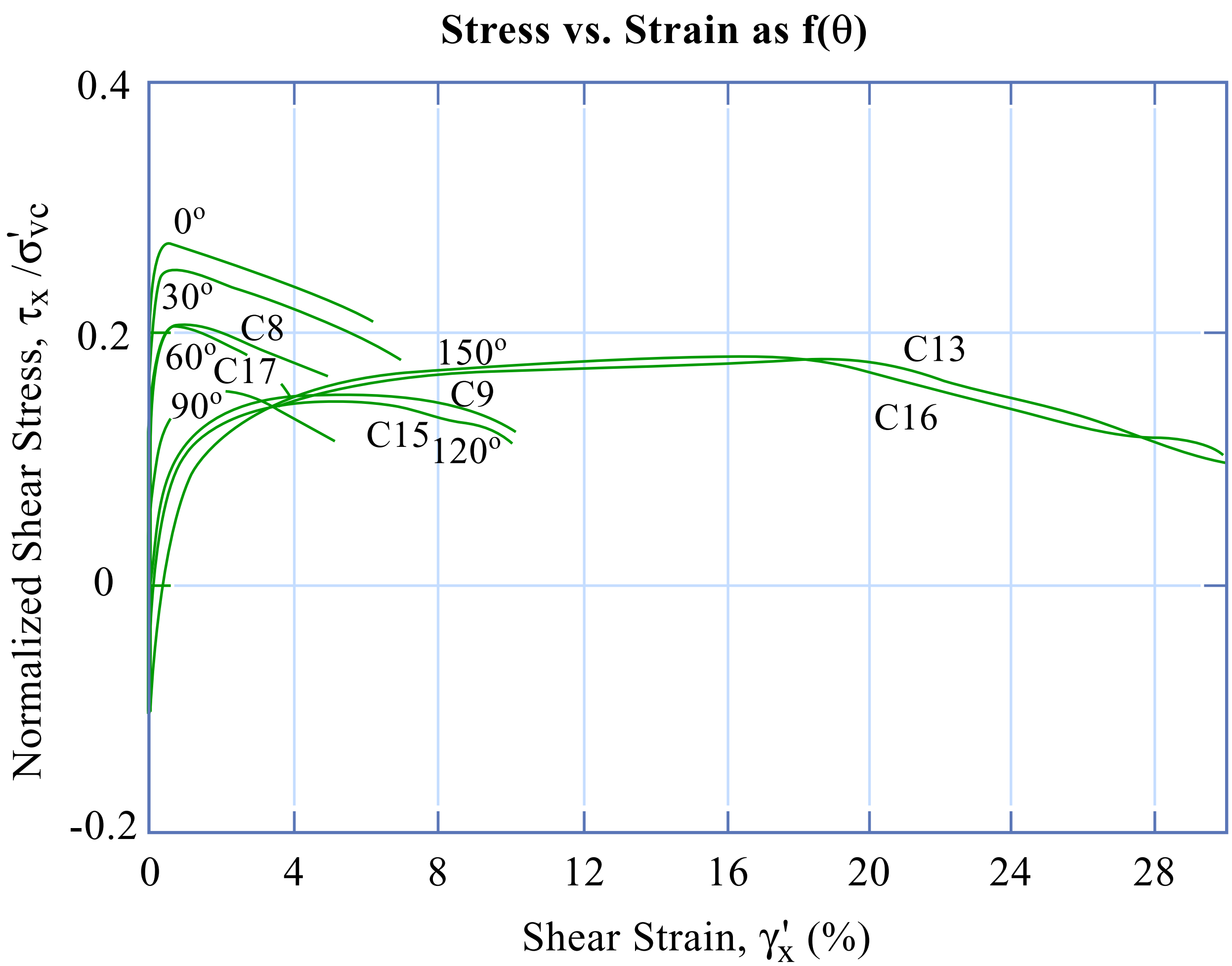
(Degroot, 1989)

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

IIC

MDSS-5

B13



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

CCL 10/89 11/91
6/90
10/90

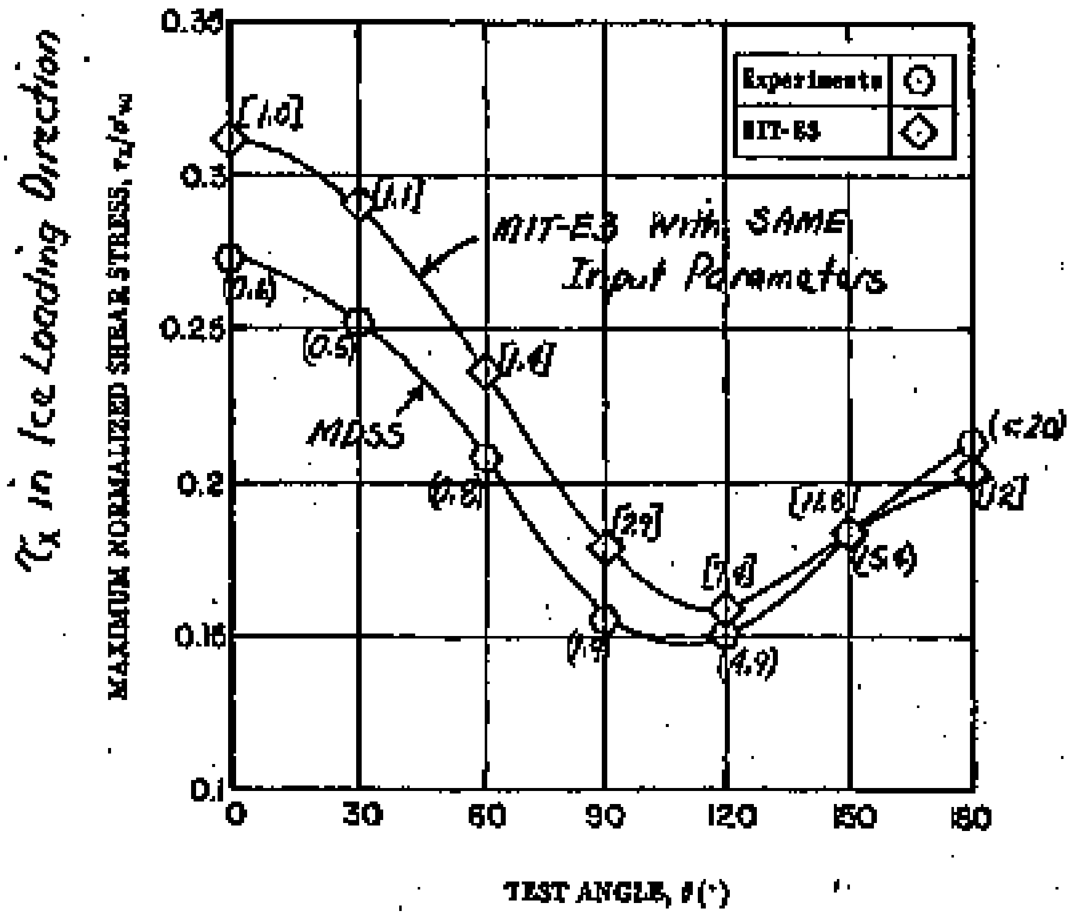
IIC

MDSS-6

B15

OCR=1 BBC τ_{hc}/σ'_{vc} = 0.20

Peak Strength Comparison



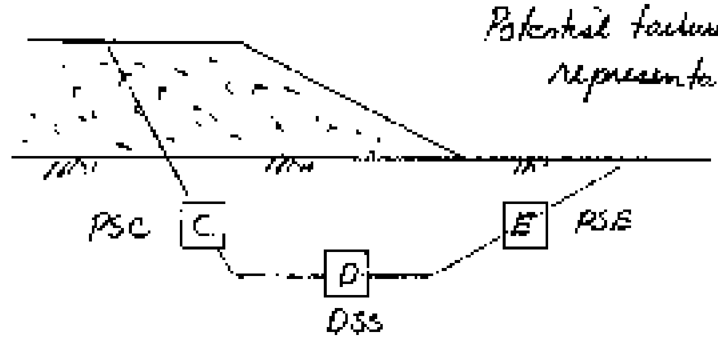
( ) = % Shear Strain in ice loading direction

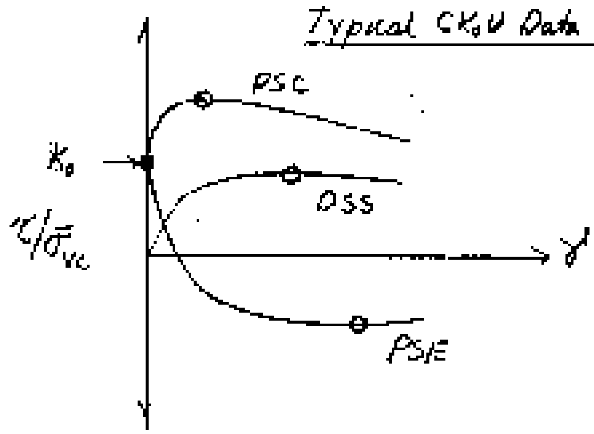
Figure 6.13: Measured and Predicted Maximum Shear Resistance τ_x/σ'_{vc} Versus Test Angle θ for CAUMDSS Behavior of BBC With $\tau_{hc}/\sigma'_{vc} = 0.2$.

(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 analysis (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 BZ OCR = 1 & 2.1 (RECOMP)

a) Plot τ (or $\tau/\bar{\sigma}_{vc}$) vs γ $\left(\begin{array}{l} = 1.5\,\epsilon \text{ for Triaxial} \\ = 2.0\,\epsilon \text{ for PS} \end{array}\right)$

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 τ_c vs τ_d vs τ_e

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

8.3 AGS Case History (KSL, 1985) – Handout

1) Background

- Breakwater for floating nuclear power plant with 3 stage construction (Fig 1, 2)
- Initial in situ OCR = 4.2 ± 0.4 (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	0.77
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 $I_p = 43\%$ & $\mu_{R3} = 0.76$ after
  consideration of end effects à la Azzouz et al (1983) ASCE JGE 109(5). (1)
  - Conclusions wrt Bjerrum μ : Unsafe for PS failure (x 1.16); ok for typical 3-D failure (x 1.03)
- Comments on c_u(UUC) data ($\dot{\varepsilon} = 10\%$/hr)
  - Increased scatter vs c_u(FV) & $\tau_{ave} \pm 1$SD : 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 (Tri 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 $\phi_{cu} = 0.26$

(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) = $[1 + 0.7(\frac{B}{L})]$

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_o vs ε_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 peak 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 $\gamma_f \approx 2\%$ OC - ≈ 0.85 for very high S_t like James Bay

Note: $q_f(TC)/\tau_{ave} = 1.4? \pm 0.10$
$\tau_h(DSS)/\tau_{ave} \approx 1.07 \pm 0.07$ (w/o CVVC)

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

9.1 Bearing Capacity (PS)

1) Davis & Christian (1971)

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

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

≈ 5.14 for $b/a = 1.0$
$= 4.00$ for $b/a = 0$
$\approx 5.0 \pm 0.14$ for typical $b/a = 0.9 \pm 0.1$

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) Koutsoftas & 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 DSC/E →
  predicted / measured q_{ult} = 1.0
- Explanation: Compensating Errors: increased q_f due to faster & offset strain compatibility

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

- $\therefore q_f$ (UUC) DEPENDS ON COMPENSATING ERRORS ($\dot{\varepsilon}$ + δ vs disturbance)
- $\therefore q_f$ (CIUC) ALWAYS UNSAFE
- μ s_u(FV)
  - For circular arc neglecting end effects → unsafe (× 1.1)
  - τ_H vs q_f → too low ($\lambda \cos\bar{\phi} \approx 0.87$)
  - $\therefore$ Compensating errors

9.2 Circular Arc Stability Analyses Using "Isotropic" strengths

1) Above comments / conclusions apply but now presumably want τ_H 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)

$\therefore$ 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

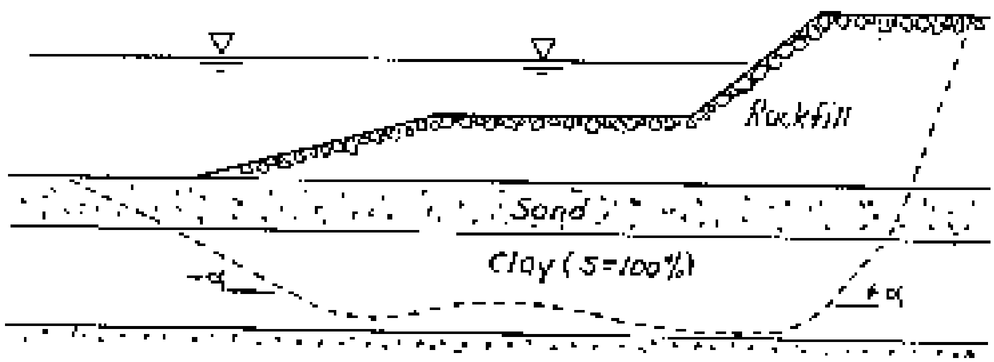
9.3 Mohr-Circular Analyses using Anisotropic Strengths (e.g. UTEXAS3)

(Note: p19-21 from Ladd (1998) Panel Discussion 13th ICSMFE, New Delhi)

CCL 11/26/93

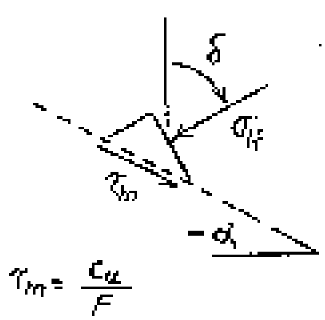
STABILITY ANALYSIS OF EMBANKMENT

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



CCL 11/26/93

TWO MAJOR QUESTIONS (Mohr-Coulomb Failure Criteria)



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?

$$\alpha = \theta - \delta$$

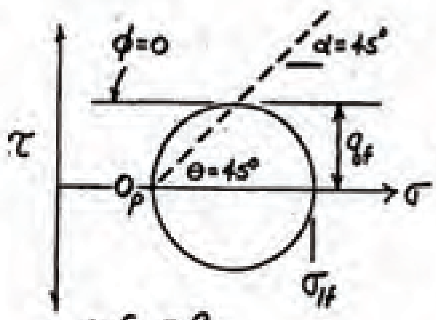
$\theta = 45 + \phi/2$ = angle between failure plane and σ_{1f} plane

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

CCL 12/14/83

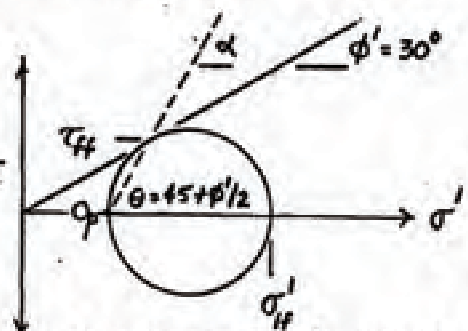
APPLICATION OF TWO HYPOTHESES (For $\delta = 0°$)

Using $\phi = 0$



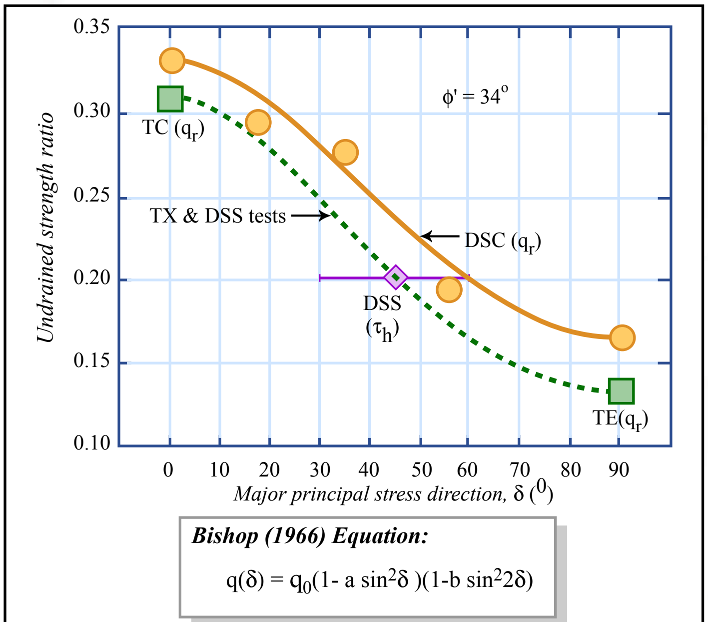
- $c_u = q_f$
- $\theta = 45°$
- $\alpha = 45° - \delta$

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 12/14/83



Undrained strength Ratio vs.Stress direction.

Figure by MIT OCW.

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 12/26/03

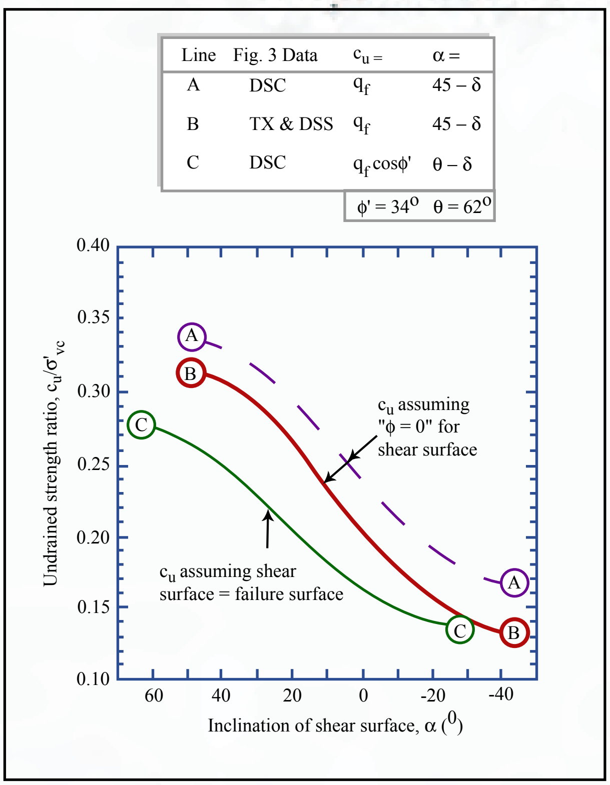


Figure by MIT OCW.

CCL 12/26/03

CONCLUSIONS

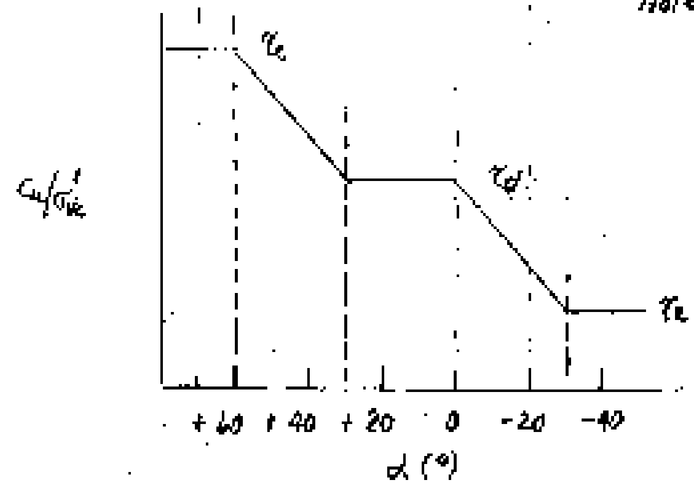
1) Run lab CK_0U tests with varying δ to measure anisotropy
   - Apply corrections to TC/TE data · Assume $\delta = 45 \pm 15°$ for DSS

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

Simplified Approach Given Uncertainty in δ / α for DSS data

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

Replacing actual variation with stepped lines



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SC-1

TABLE 3. – Normally Consolidated Undrained Strength Ratios From CK_0U 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	BADPL 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.

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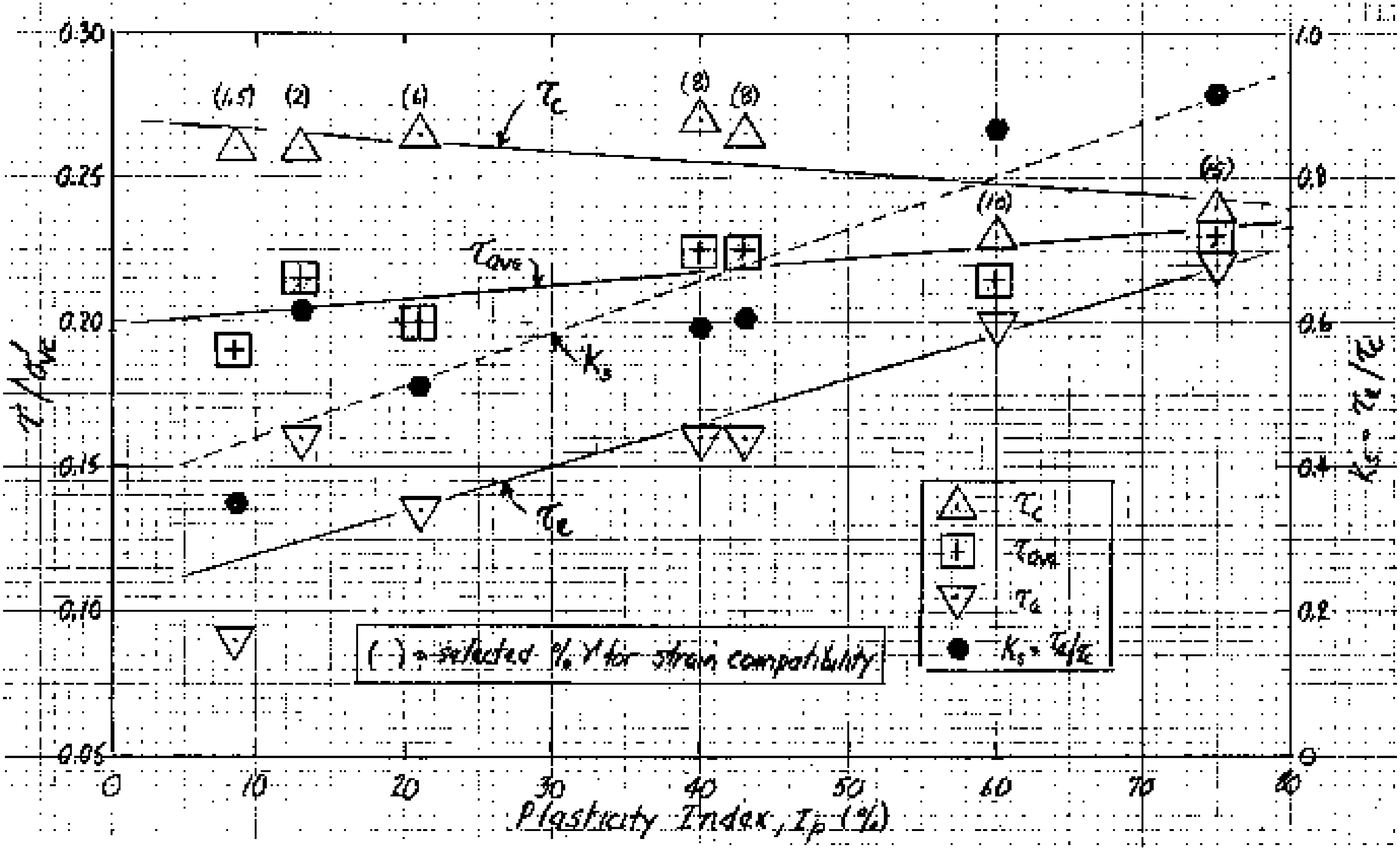
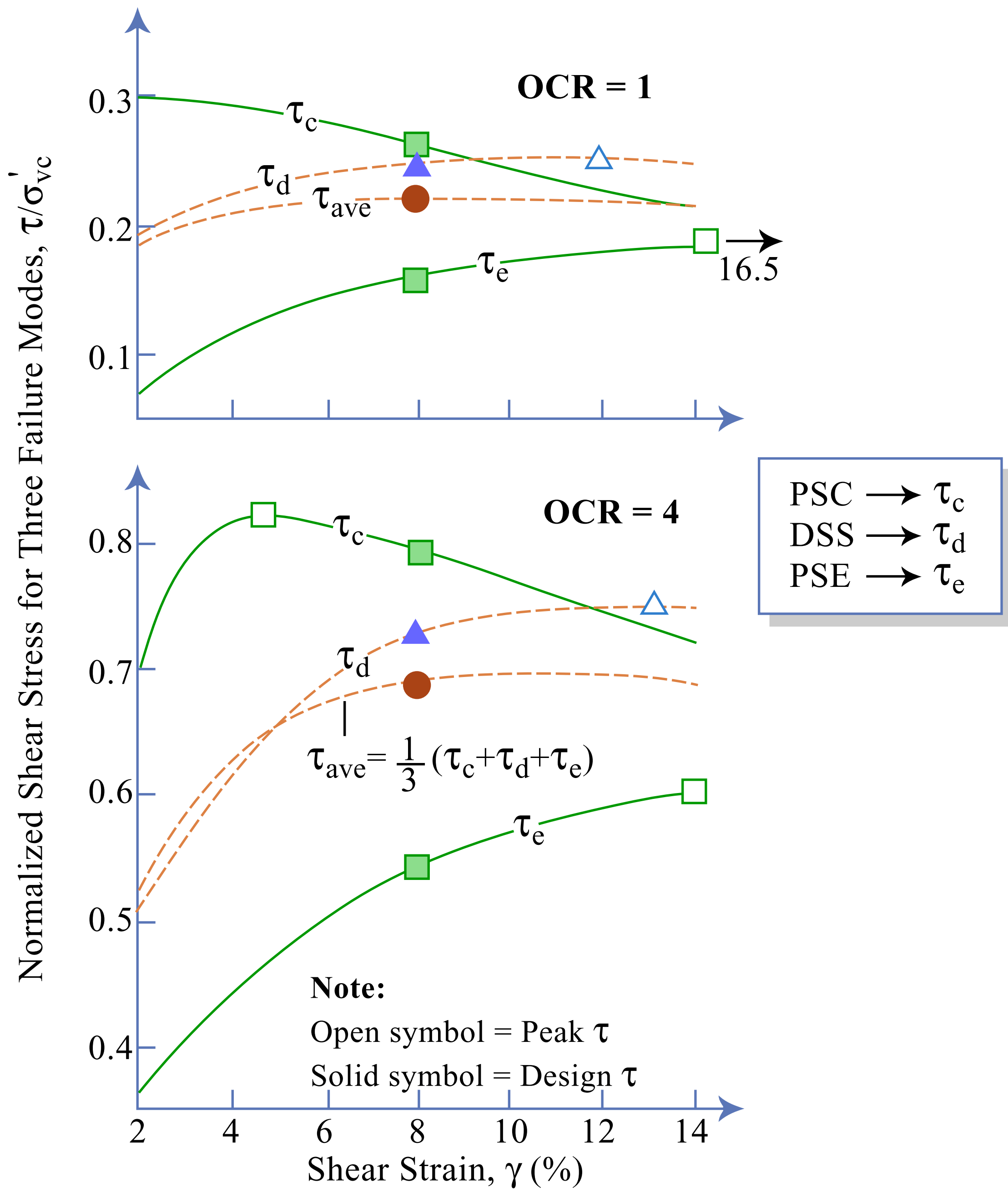


Undrained Shear Strength Ratios vs Plasticity Index for CL and CH Clays Treated for Strain Compatibility (Data from Table 4, Ladd 1991)



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

Figure by MIT OCW.

Adapted from Koutsoftas & Ladd (1985)

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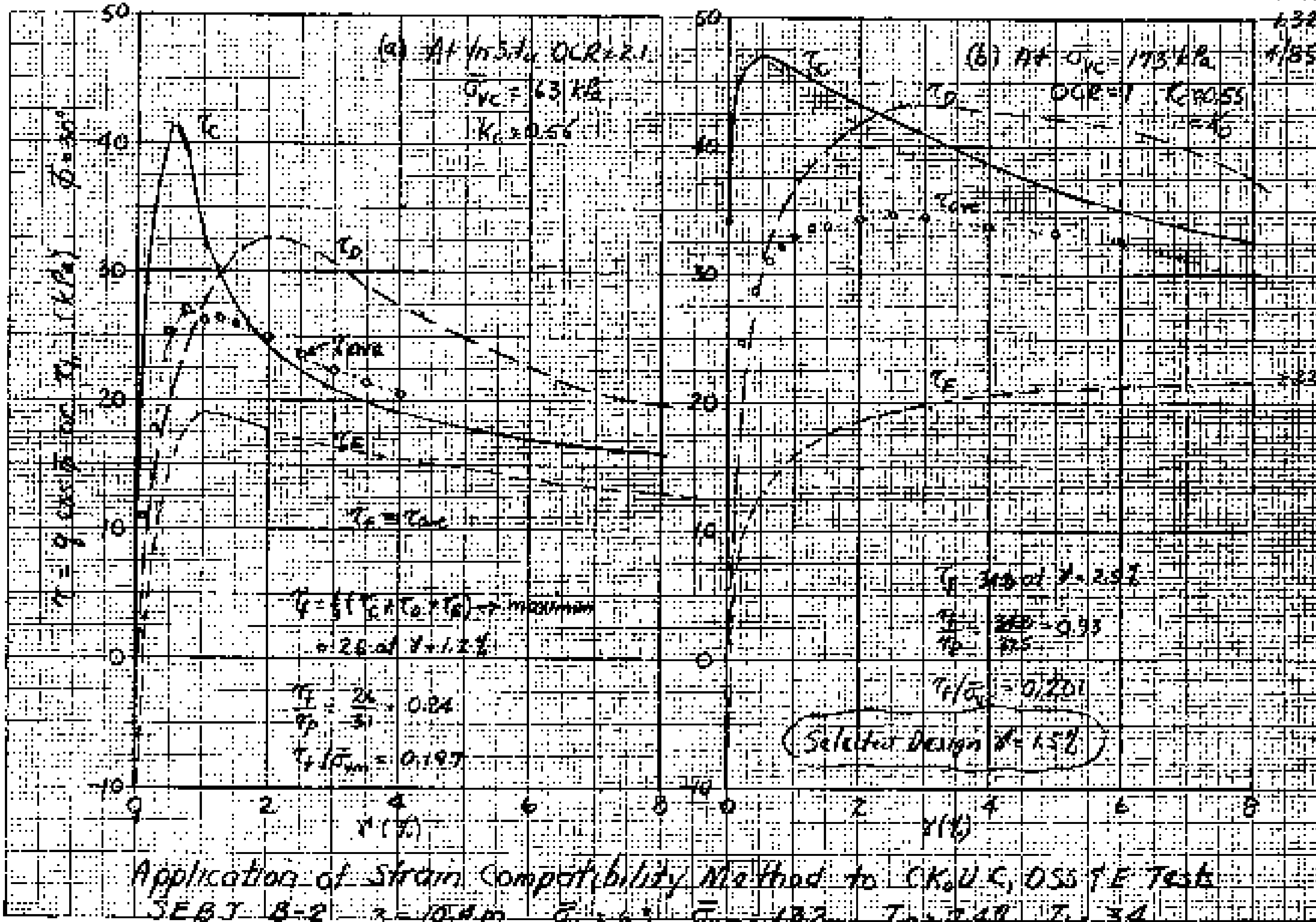
(a) At In Situ OCR=2.1
$\bar{\sigma}_{vc}$ = 63 kPa
K_c = 0.56
(b) At $\bar{\sigma}_{vc}$ = 173 kPa
OCR=1
$\tau = q$ cos $\bar{\phi}$ or τ_f (kPa)
τ_C
τ_D
τ_E
τ_{ave}
γ (%)
Selected Design γ = 1.5%
Application of Strain Compatibility Method to CK_0UC, DSS, & E Tests
SEBJ B-2 z = 10.4 m

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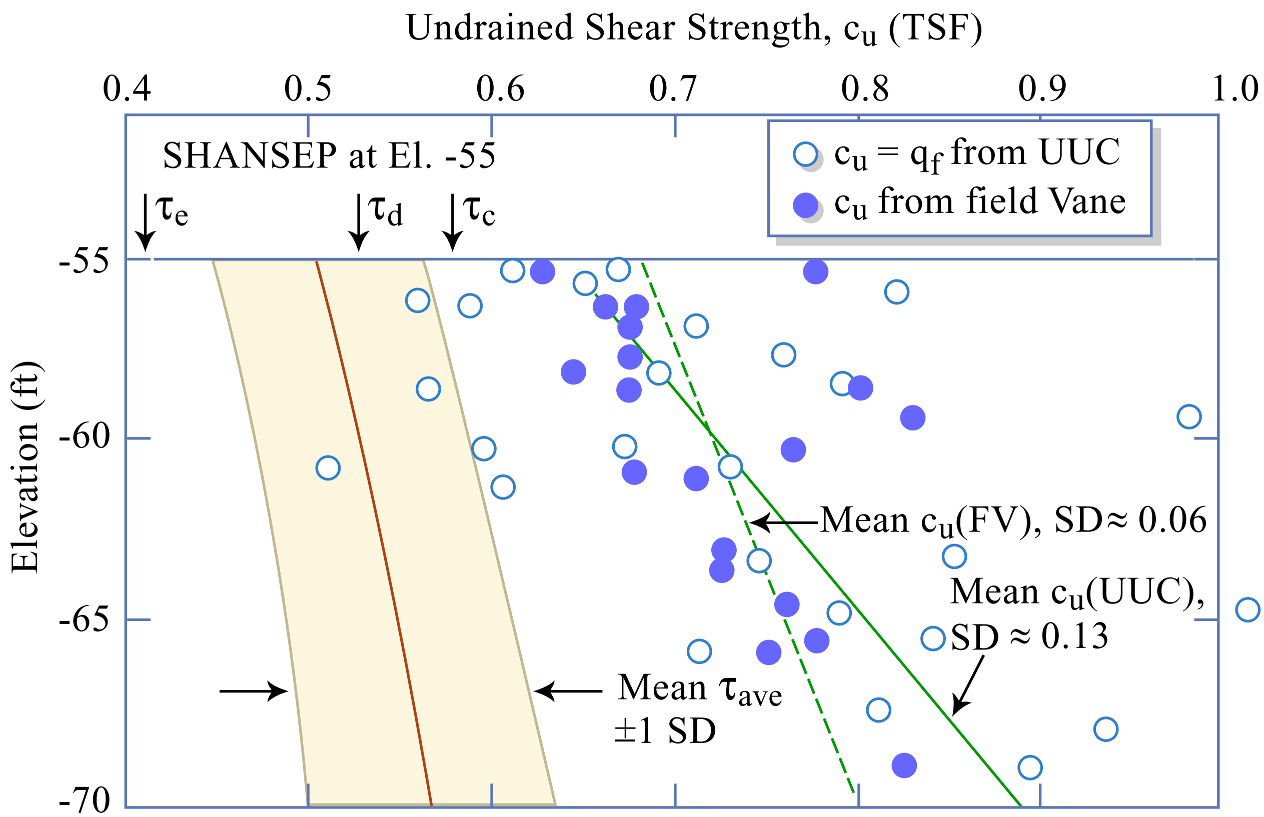
SC-4



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			Ultimate Bearing Capacity $q_{ult}/\bar{\sigma}_{vc}$	
		$s_u(ave)/\bar{\sigma}_{vc}$	$s_u(V)/\bar{\sigma}_{vc}$	$s_u(H)/\bar{\sigma}_{vc}$	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/cm^2

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

(4) $s_u = \tau_h$ maximum

Ladd (1971) Table II-4

4/97 4/01

IID INFLUENCE OF TIME ON STRESS-STRAIN-STRENGTH BEHAVIOR OF CLAYS DURING UNDRAINED SHEAR

Sheet A1,2; Sheet B — Data from Sheahan et al. (1996) from CK₀UC on $\dot{\varepsilon}$ = f(OCR) for resedimented BBC

Page No

Sheet C1, 2 Data on creep rupture and unique $\log \dot{\varepsilon}_m$ vs $\log t_m$

" D Data on Correspondence

" E1, 2 Data on relaxation

IID: INFLUENCE OF TIME ON STRESS-STRAIN-STRENGTH BEHAVIOR OF CLAYS DURING UNDRAINED SHEAR

1. INTRODUCTION

1.1 Definitions

1) Prior to shear

a) At constant w (UU): thixotropy

b) At constant σ' (CU, CD): "aging" ≡ secondary compression

2) During shear (undrained)

a) Rate of strain, $\dot{\varepsilon}$, as increase q

b) Time after applying constant q = creep

c) " " " " ε = relaxation

1.2 Comment on Drained Behavior

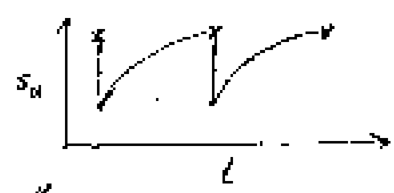
- For drained shear, $\dot{\varepsilon}$ (or t_f) believed to have little effect on $\bar{c}'$, $\bar{\phi}'$ for "ordinary" clays.

- But NOT for Highly Structured Cemented clays - see Section 4.4

2. THIXOTROPY (Mitchell, 1960 ASCE JSMFD 86(3); 1993 Book; O'Neill, 1985 MIT SM Thesis)

2.1 Definition

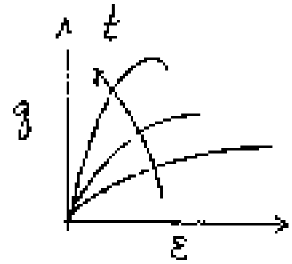
- With some clays, if remold and then store at constant composition → incr. stiffness & strength



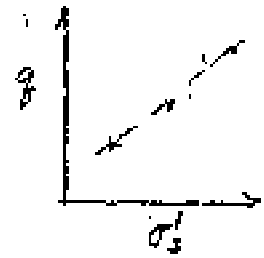
- Thixotropy = isothermal, reversible, time-dependent process occurring under conditions of constant composition & volume whereby a material stiffens while at rest and softens or liquefies upon remolding

2.2 Behavior Measured in UUC Tests

1) Based on limited published data.



plus maybe



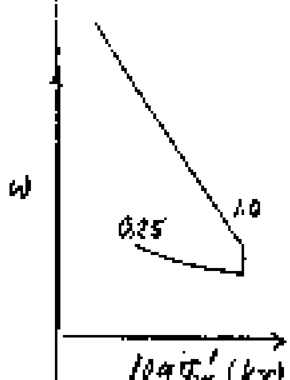
- PhD Berkeley on slurries of clay
- SM MIT UUC ~1960 ±
  (But not Ocm, 85)

2) Comments

- Storing disturbed tube samples may → incr. s_u w/ t
- No correlation TSR = $s_u(t)/s_u(R)$ and soil type, but restricted to clays and generally more important with increasing I_L

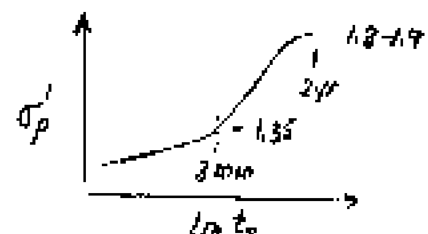
2.3 O'Neill (1985) - Behavior of Block Samples of Resedimented BBC II

Batch $\sigma_{vm}' = 1$ ksc + $t_c/t_p = 10$ cycle & rebounded to $\sigma_{vc}' = 0.25$ ksc



1) See p2a for oedometer
   σ_p' ~ log t_s (t_s = storage time)

   - 1 week → $\sigma_p' = 1.1$ (expected)
   - 3 mon → $\sigma_p' \approx 1.35$
   - 2 yr. → $\sigma_p' = 1.9$



2) During this period
   - $\Delta w \approx 0$
   - Very little increase σ_s
   - Consistent increase in q_f Recompression CK_0UC/E of same magnitude & largely due to Δu_s (see p2a)

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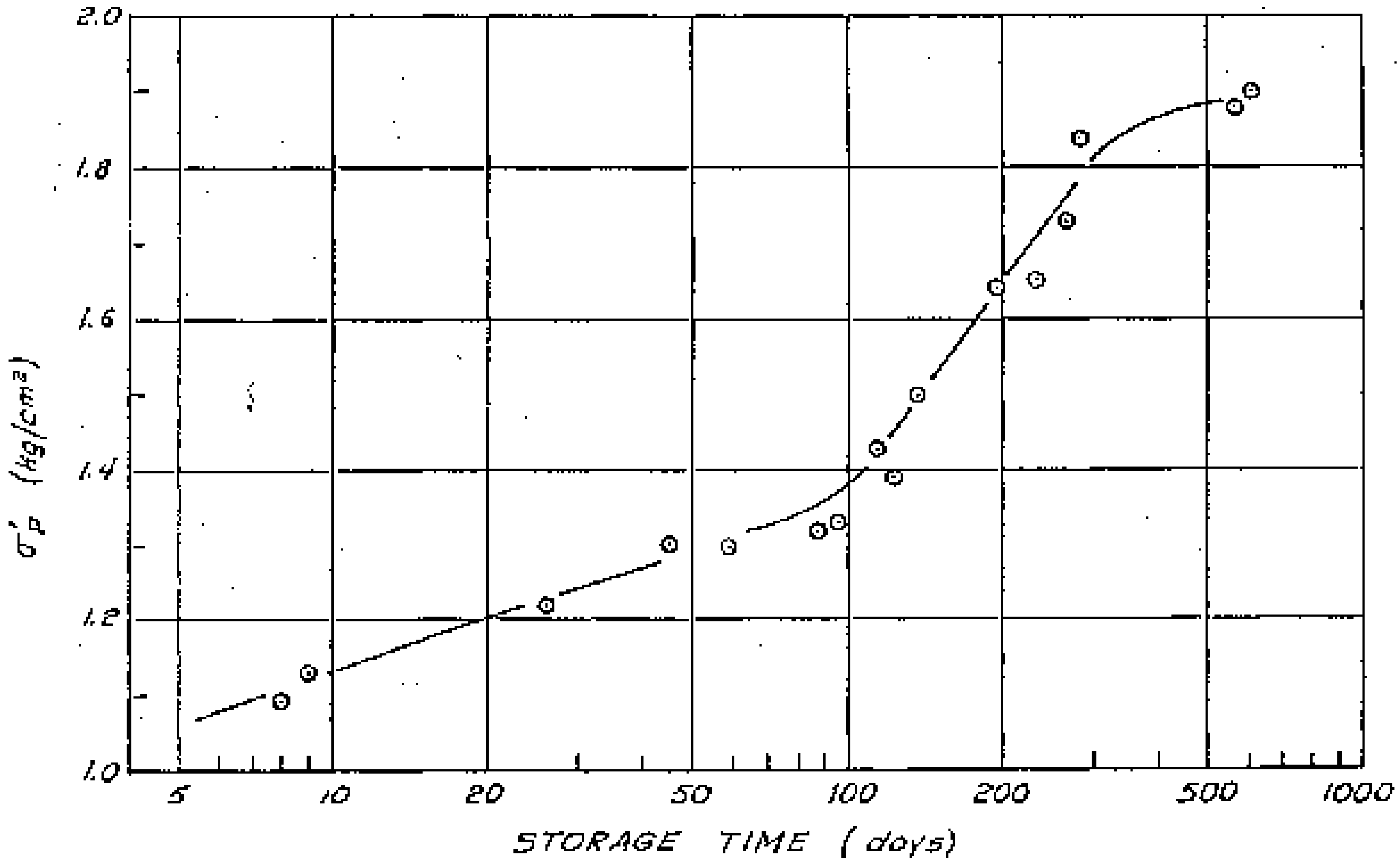
p24



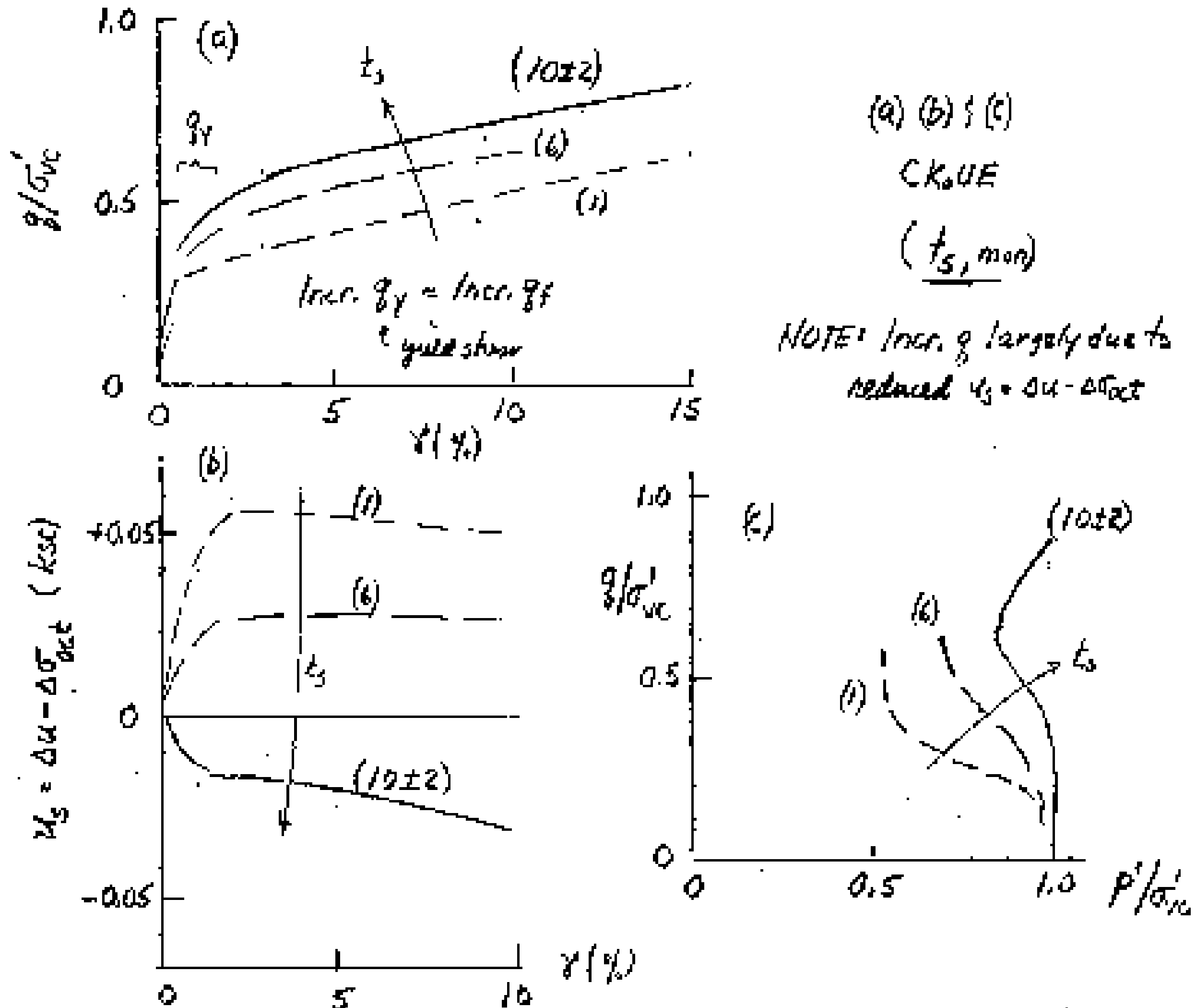
MIT Special Program (1985)

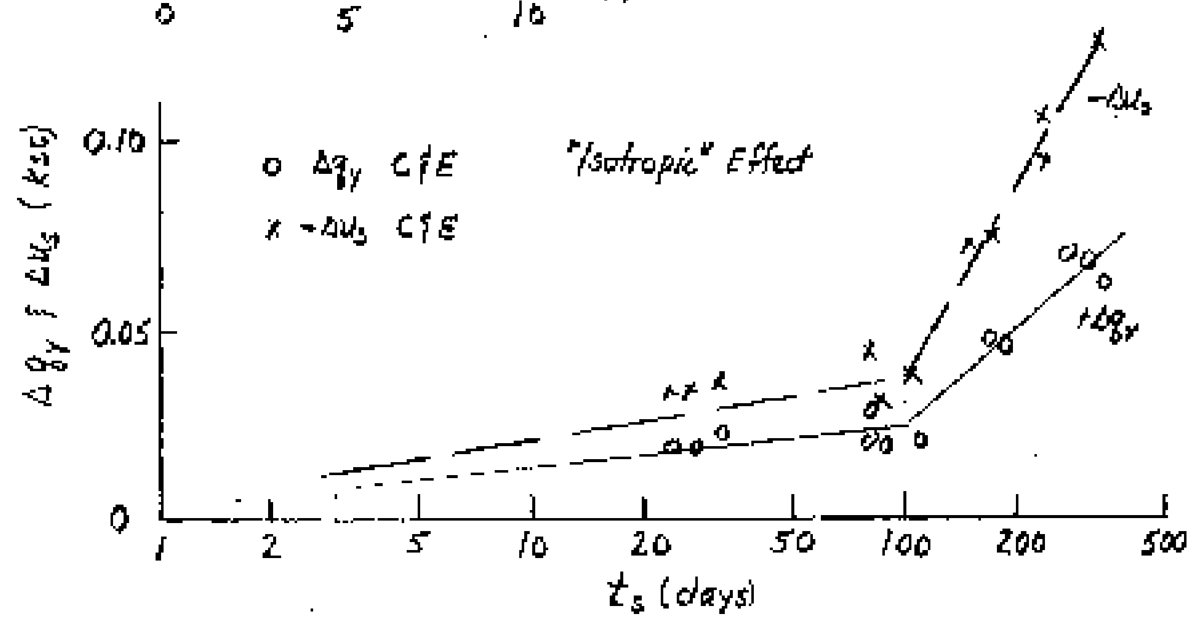
1.605

Figure 6-1 Effect of Thixotropy on Preconsolidation Pressure of Resedimented Boston Blue Clay (after O'Neill, 1985)

O'Neill (1985) Recompression CK_0U C/E Resed. BBC

(OCR = 4 prior to thixotropy) $\sigma'_{vc} = 0.25$ ksc





2.4 Possible Mechanisms

1) Reorientation of clay particles (Mitchell, 1976/1993 Book)
from "dispersed" → "flocculated" (Δ fabric and interparticle forces)
- Presumably $\bar{A} > \bar{R}$ and/or $\bar{\bar{\sigma}}_a > \bar{\bar{\sigma}}_r$
- Berkeley data supposed to show that occurs in compacted clays → large increase in k w/t!

2) Water "structure": Decrease in free energy of adsorbed H_2O → decreasing u → increasing σ'_s
- CCL liked before BBC data became available

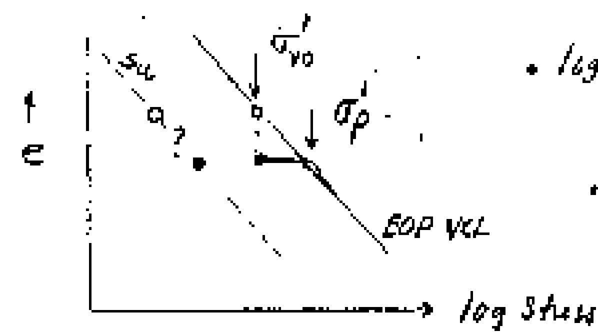
3) "Bugs" (RTM)

4) Conclusions: Mechanism(s) unknown

But some clays (especially at high I_p) certainly do stiffen with time

3 AGING = SECONDARY COMPRESSION

3.1 Review from Treatment of Consolidation



- $\log \frac{\sigma'_p}{\sigma'_{vo}} = \frac{C_{\alpha e}}{C_c} \log(t/t_p)$
- $\frac{C_{\alpha e}}{C_c} \approx \frac{C_{\alpha \varepsilon}}{CR} = 0.045 \rightarrow \approx 10\%$ increase in OCR per log cycle secondary compression. Does s_u increase by same amount?

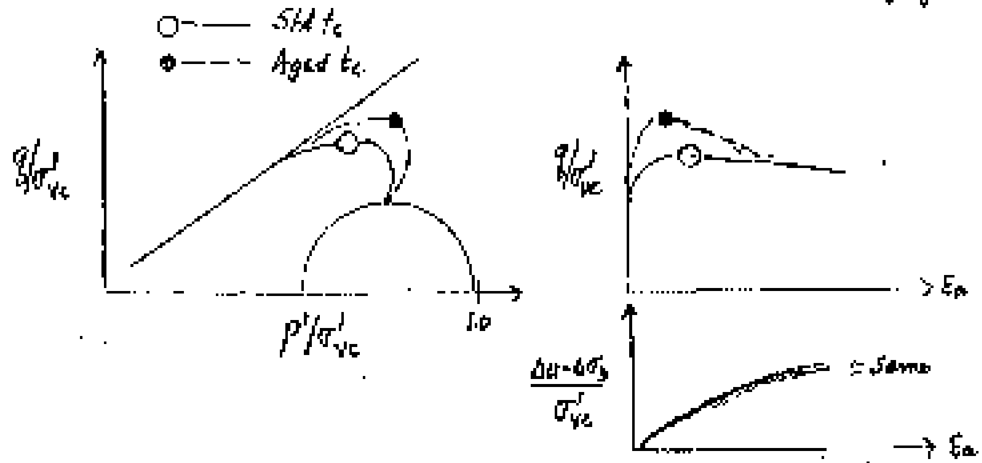
Given "fact" (à la Mesri) that $C_{\alpha e}/C = 0.045 \pm 0.015$ ($C = -\Delta e/\Delta \log \sigma'_v$)
for most cohesive soils (Both OC & NC)
- Only important at low OCR (i.e. 'high' C + high C_α)

{ Should not vary in consistent fashion with changes in I_p }
∴ Tokyo Fig. 39 very suspect

3.2 Influence on CAUC Tests (OCR=1)

(Bjerrum & Lo 1963; Ladd 1965; Vaid & Campanella 1977)

Note: Aging at constant K_c



1) Aging leads to:
   a) Modest increase in s_u/σ'_{vc}, say ≈ 5-7%/Δlog t_c
   b) Large increase in E_u/σ'_{vc} (Ladd 1965 reports 60±10%/Δlog t_c from CIUC tests)
   c) Perhaps decrease in ε_f

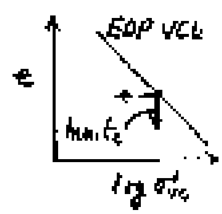
2) If same Δu vs ε_a, then:
   a) Lower shear induced $u_s = \Delta u - \Delta\sigma_{oct}$ (consistent with incr. OCR)
   b) Increased resistance at particle contacts since same Δu vs ε_a implies same displacement at contacts

3) Behavior of aged vs. mechanical precompression at same OCR?
   Aging probably → stiffer initial response (i.e., higher E_u)

3.3 MIT Standard Practice for SHANSEP Testing

Perform CK_0U tests after $t_c \approx$ 1 day (log $t_c/t_p \approx 1$ → ≈ 1 log cycle)

1) Minimize +Δu due to "stopping" secondary compression →
2) Standard t_c → more consistent data
3) To instill some "structure" in the clay (i.e., make stiffer) that was destroyed by consolidation beyond in situ σ'_p



NOTE: Empirical aspect of SHANSEP. Also actual OCR is slightly higher than reported (since aging increases σ'_{vm})

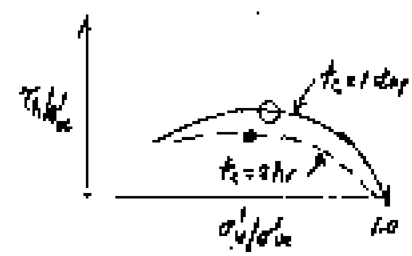
3.4 Influence of Consolidation Time on CK_0UDSS Data

1) As per 3.3, SHANSEP CK_0U tests typically use $t_c = 1$ day at test σ'_{vc} ($\neq$ test σ'_{vm}) when attempting to predict initial in situ undrained shear behavior of naturally OC clays.

2) However, during staged construction, foundation clay is still undergoing consolidation, i.e., presumably lies on t_p (EOP) compression curve. Therefore there will not be any "aging" (secondary compression).

3) Following compares $t_c = 1$ day vs $t_c = 2$ h ("EOP") for two plastic soils at Peak strength (each average of 2 tests) at $\dot{\gamma} = 5\%/hr$

Soil	t_c	γ_f (%)	τ_h/σ'_{vc}	
Fresh Kills, NY Organic Silt ($I_p = 60\%$)	1 day	14.5 ±0.5	0.296 ±0.018	
	2 hr	14.1 ±4.9	0.257 ±0.020	$\rightarrow -13\%$
Sergipe, Brazil Offshore CH ($w_n = 65\%$)	1 day	9.4 ±0.2	0.231[5] ±0.003[5]	
	2 hr	≈7.6[5] ±0.9	0.2163 ±0.0067	$\rightarrow -9.3\%$



Ave = −10.5%

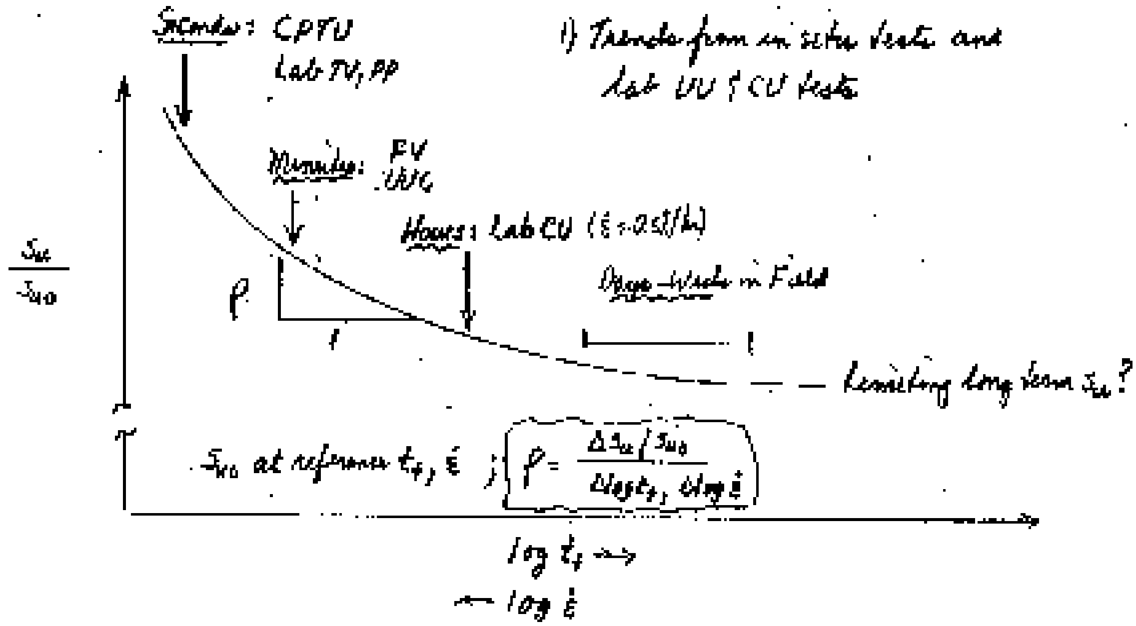
NOTE: For $\dot{\gamma} = 0.5\%/hr$, reduction $\rightarrow -11.8\%$

4) Effect is significant: Therefore should use $t_c = t_p$ (EOP) to obtain s_u/σ'_{vc} for OCR = 1 "underconsolidated" soil.

4. EFFECT OF STRAIN RATE

4.1 Overview:

1) Trends from in situ tests and lab UU & CU tests



2) Reported values of ρ and comments [also see Sheahan et al. 1996, JGE, 122(?)]

a) Most of the early (<1970) data came from UUC tests → unknown OCR

b) Most CU data with known OCR from CIUC tests

NC Typical $\rho \approx 10 \pm 5\%$; t_f = 1 min → 1 week

High OCR " " ≈ 15% ; t_f = 5 min → 1 week

[Note: Undisturbed CL-ML Haga Clay, CK_0UC at OCR = 10-40 → ρ ≈ 30-35%!
(Andersen & Stenhamar, 7/82, JGE)]

3) Implications when comparing s_u data having different t_f / $\dot{\varepsilon}$

a) Compare UUC at $\dot{\varepsilon}$ = 1%/min vs CK_0UC at $\dot{\varepsilon}$ = 0.5%/hr → 2 log cycles

b) $\Delta q_f \approx 20 \pm 10\%$ for $\rho = 10 \pm 5\%$ at low OCR

" ≈ 30-60% for ρ ≈ 15-30% at high OCR

4) Extreme case: Offshore Alaska at Smith Bay; CL-CH Pleistocene clay

s_u(UUC, FV & CPTU [N_k=15]) ≈ 3 × s_u(DSS)
from SHANSEP with well defined
σ'_{v0} & σ'_p profiles.

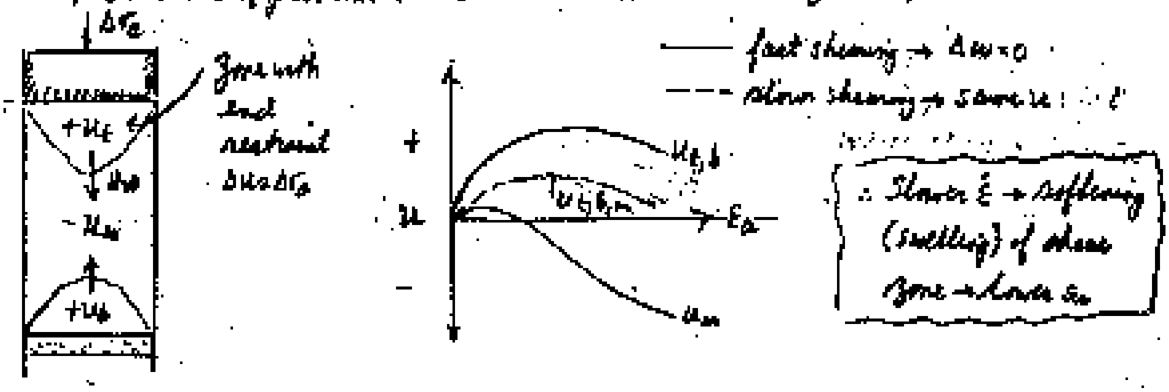
I_p ≈ 25%, T ≈ -1°C
z = 1-5.5 m
OCR = 40-8

Young (1986) MIT SM thesis

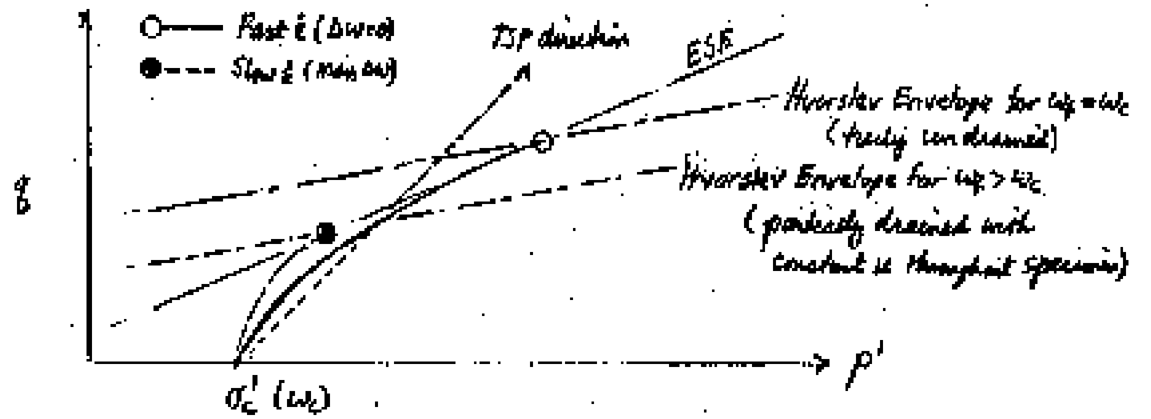
4.2 Results from CIUC Tests at High OCR: Fixed End Caps

NOTE: Also applies to UUC testing at varying $\dot{\varepsilon}$

1) Overview of problem (also see Notes on measurement of c' & φ', I B)



2) Results on OCR=16 CH clay [Richardson & Whitman 1963, Test 13(4)] → CCL reinterpretation
CIUC with u_m (at middle → correct ESP)



3) Conclusions

a) Regular UU/CU tests at varying $\dot{\varepsilon}$ on high OCR clay are partially drained; hence decr. in s_u at slower rates due in part to softening of shear zone

b) Need lubricated end caps to measure correct Δs_u vs $\Delta\dot{\varepsilon}$

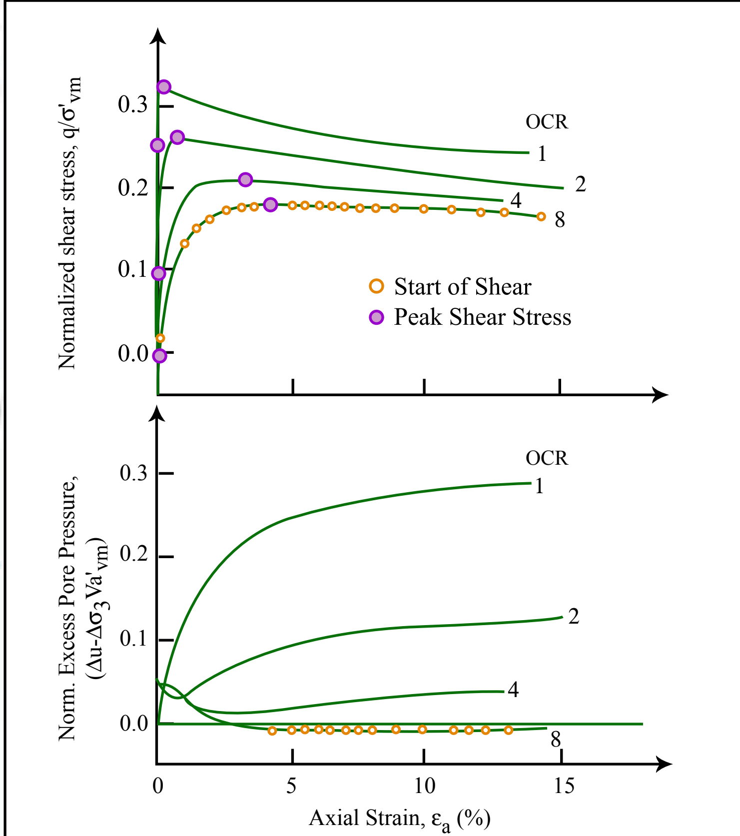
c) Will in situ shearing of high OCR clay also → softening of potential shear zone, and hence lower s_u?
CCL doesn't know, but probably possible (need to study lubrication)

4.3 Results from CK_0UC Test on RBBC at OCR = 1, 2, 4 & 8: Lubricated End Caps and Mid-Specimen u Data

[Sheahan 1991 MIT ScD thesis; Sheahan et al. 1996, JGE, 122(2)]

1) Test Program

- 1st MIT automated TX cells, $\dot{\varepsilon}$ = 0.05, 0.5, 5 & 50%/hr (Slow, Std, Fast, V.Fast)
- Cell Fluid = oil to prevent membrane leakage
- $\rho_{as} = (\Delta s_u/s_{u0})/\Delta \log \dot{\varepsilon}$, where s_{u0} at $\dot{\varepsilon}$ = 0.5%/hr.



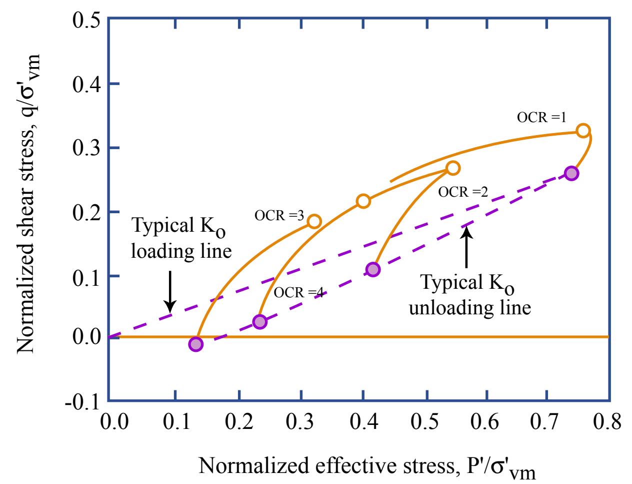
Typical normalized shear stress and excess pore pressure versus strain for CK_0UC tests on resedlmented BBC at reference strain rate (ε_a = 0.5%/h)

- Measured u both at base & [illegible] probe

2) Behavior at reference $\dot{\varepsilon}$ = 0.5%/hr

- See Fig 1 & 2
- Note that data normalized to σ'_{vm}

OCR	q_f/σ'_{vm}	p'_f/σ'_{vm}	ε_f(%)
1	0.322	0.761	0.15
2	0.261^5	0.552	0.7
4	0.213^5	0.418	3.0
8	0.180	0.328^5	4.2



Typical normalized effective stress paths for CK_0UC tests on resedlmented BBC at reference strain rate (ε_a = 0.5%/h)

Peak shear stress
Start of Shear

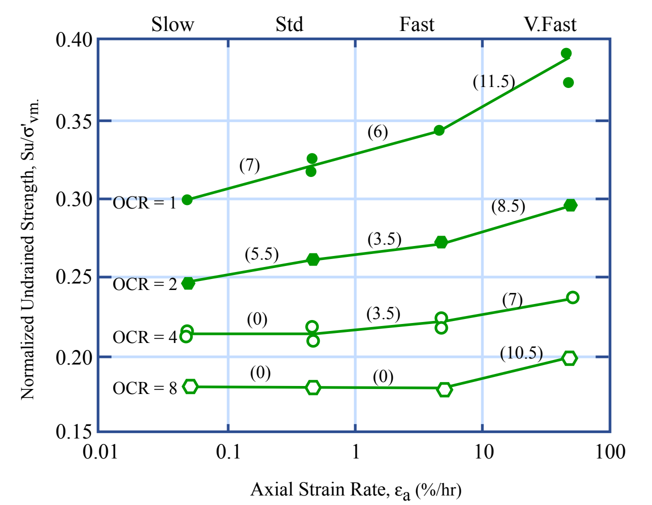
Figures by MIT OCW.

4.3 Cont

3) Overview of effects on s_u

a) s_u/σ'_{vm} vs $\log \dot{\varepsilon}$ as $f(OCR)$: Fig 11

$\dot{\varepsilon} = 0.05$ |— Zone A —| 5 — Zone B —| 50%/hr



Normalized sheer strength versus strain rate CK_0UC tests, resedimented BBC

Figure by MIT OCW.

Zone B (Fast → Very fast)

- get effect at all OCR that is ≈ constant
- $\rho_{a5} = 9.4 \pm 2.0\%$

Zone A (Slow → Fast)

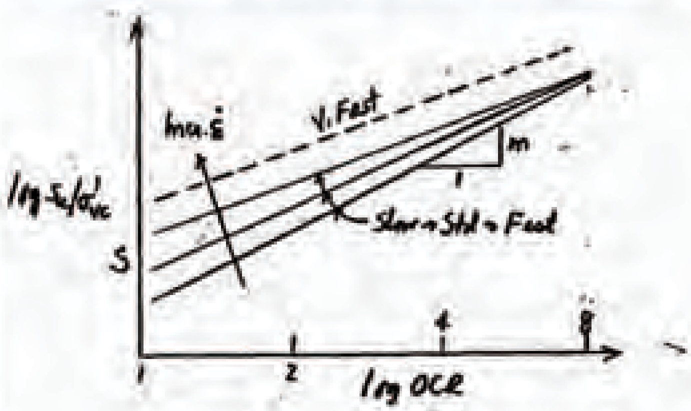
- ρ_{a5} decreases with increasing OCR (1st data to show this!)
- ρ_{a5} goes to zero with increasing OCR more rapidly at lowest $\Delta \log \dot{\varepsilon}$ range.

b) $\log s_u/\sigma'_{vc}$ vs $\log OCR$ as $f(\dot{\varepsilon})$: Table 4

TABLE 4. SHANSEP Parameters for BBC at Different Strain Rates

Strain rate $\dot{\varepsilon}_a$ (%/h) (1)	S^a (2)	m^b (3)	r^2 (4)	Number of observations n (5)
0.05	0.298	0.757	0.9997	6
0.5	0.320	0.714	0.9993	6
5	0.340	0.689	0.9997	5
50	0.373	0.686	0.9984	8

$^a S$ = value of s_u/σ'_{vc} at OCR = 1, based on regression analysis.
$^b m$ = strength increase exponent [refer to Eq. (3)].



Complex trends

- Slow → Fast $\dot{\varepsilon}$ → increasing S / decreasing m
- Fast → Very Fast $\dot{\varepsilon}$ → increasing S at constant m

4.3 Cont.

4) Overview of effects on stress-strain & ESP behavior

a) Shear stress vs ε_a

- See Sheet A1 & A2 for q/σ'_{vc} vs ε_a → very consistent trends
- Fig. 12 of Sheet B shows that normalized $\Delta q/\Delta q_{max}$ vs ε_a is unique at OCR = 2, 4 & 8 (very important for soil modeling). Post peak behavior at OCR = 1 is scattered

b) Pore pressure vs ε_a

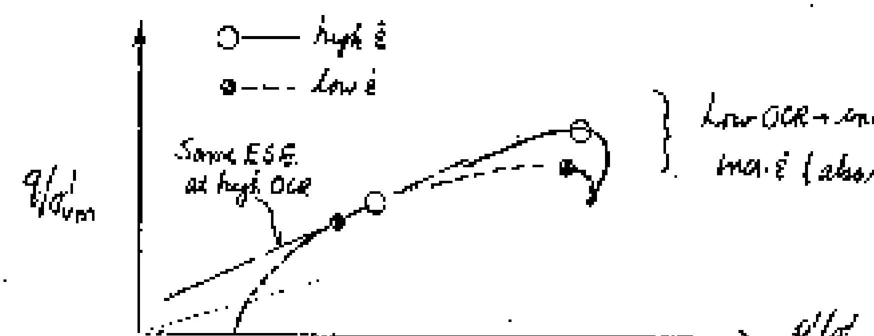
- Look at shear induced $\Delta u_s = \Delta u - \Delta \sigma_{oct} = \Delta u - \frac{1}{3}\Delta\sigma_1$ vs ε_a
- on Sheets A1 & A2
- $\boxed{\text{Increases in } s_u \text{ are always accompanied by lower } \Delta u_s}$ (also see Fig 13a, Sheet B), e.g.

  OCR = 1, increasing $\dot{\varepsilon}$ → incr. s_u & decr. Δu_s

  OCR = 8, $\dot{\varepsilon}$ = 0.05 to 5, $\Delta s_u = 0$ → no change in Δu_s

c) Effective stress paths and failure envelopes

- See Sheets A1 & A2 for ESP → consistent trends



Low OCR → incr. ϕ'_p with incr. $\dot{\varepsilon}$ (also see Fig 13b, Sheet B)

$\boxed{\begin{array}{ll} \text{OCR} = 1 \text{ \& } 2 & \text{• Low OCR: Increased } s_u \text{ due to both lower } \Delta u_s \text{ \& higher ESE } (\phi'_p) \\ \text{OCR} = 4 \text{ \& } 8 & \text{• High OCR: Increased } s_u \text{ due only to lower } \Delta u_s \text{ (same ESE)} \end{array}}$

4.3 Cont

5) Summary of CK_0UC testing program on RBBC (& practical implications for non-structured clays)

a) Very fast shearing → increased s_u that is ≈ constant at all OCR ($\rho_{0.5}$ ≈ 10%). Applies to in situ testing & Lab UC, TV etc.

b) At slow shear rates, shear rate sensitivity ($\rho_{0.5}$ > 0) decreases with increasing OCR.

Hence for field loading, would not expect design s_u of moderate to high OCR clays to be < measured lab. CK_0U testing

c) Shear rate sensitivity (incr. in s_u with incr $\dot{\varepsilon}$) is caused by two mechanisms:

1) Increasing $\dot{\varepsilon}$ → decreasing Δu_s : Occurs at all OCR

2) Increasing $\dot{\varepsilon}$ → increase in ESE at peak strength : Occurs only at low OCR

- at OCR ≤ 2, ↑ $\rho_{0.5}$ due to both decreased Δu_s & incr. ϕ'
- at OCR ≥ 4, ↑ $\rho_{0.5}$ due only to decreased Δu_s

d) At OCR > 1, obtain unique $\Delta q / \Delta q_{max}$ vs ε_a independent of $\dot{\varepsilon}$ (simplifies modeling)

4.4 Behavior of Highly Structured (Cemented-Sensitive) vs "Ordinary" Clays

1) Observations from 1-D Consolidation Data

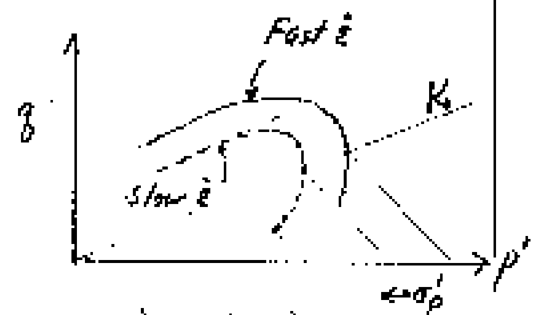
Hypothesis A
- Unique ε vs $\log \sigma'_{vc}$ during primary, i.e. independent of $\dot{\varepsilon}$
- Appears reasonable for saturated clays of low-moderate S_t
- Same mechanisms cause creep as occur during primary, e.g. slippage at particle contacts

Hypothesis B
- Unique $\varepsilon - \dot{\varepsilon} - \sigma'_{vc} \rightarrow$ same ε vs $\log \sigma'_{vc}/\sigma'_p(\dot{\varepsilon})$
- Better model for high I_L-S_t Canadian clays
- "Structural Viscosity" due to time dependent strength of cementation bonds (true cohesion) } CCL

2) CU Test Programs at Varying $\dot{\varepsilon}$ by Lefebvre & LeBoeuf (1987)
- 5 block samples from 3 sites, $I_p = 10 \pm 3$ (40%), $I_L = 2.3 \pm 0.4$, $\sigma'_p = 140 \pm 45$ kPa

a) CU Tests on INTACT Clay, $\sigma'_{vc} = \sigma'_{v0}$ (see p12a)
- Approx same Δu vs ε up to peak s_u at $\varepsilon_f < 1\%$, and hence $\approx$ same prefailure ESP

∴ Lower s_u with decr. $\dot{\varepsilon}$ due to lower yield stress = failure envelope
- attributed to rate dependent cementation bonds ("structural viscosity" of cohesion component à la Bjerrum, 1973)



b) CU Tests on DESTRUCTURED Clay, $\sigma'_{vc} > \sigma'_p$ (see p12b)
- Decreasing $\dot{\varepsilon} \rightarrow$ higher Δu ∴ lower s_u due to lower p'_f (and also lower ϕ'_p for CAUC tests à la RBBC)

NOTE: Both test series $\rightarrow$ same ρ (Fig 16, p12b), but due to different mechanisms

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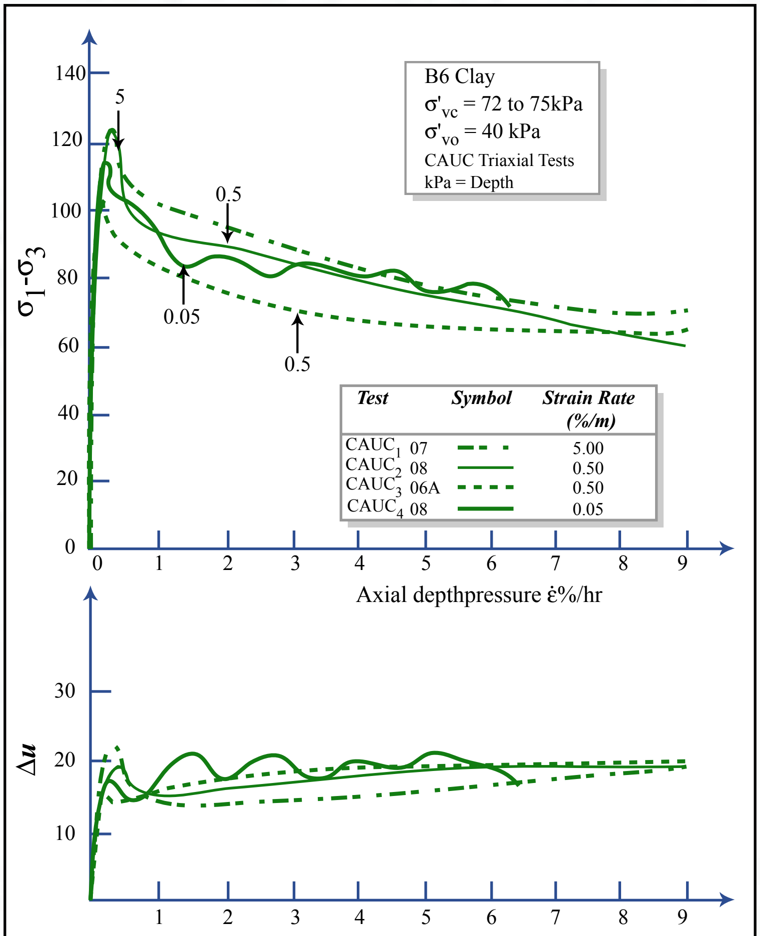
Lefebvre & LeBoeuf (1987) JGE, ASCE 113(5)

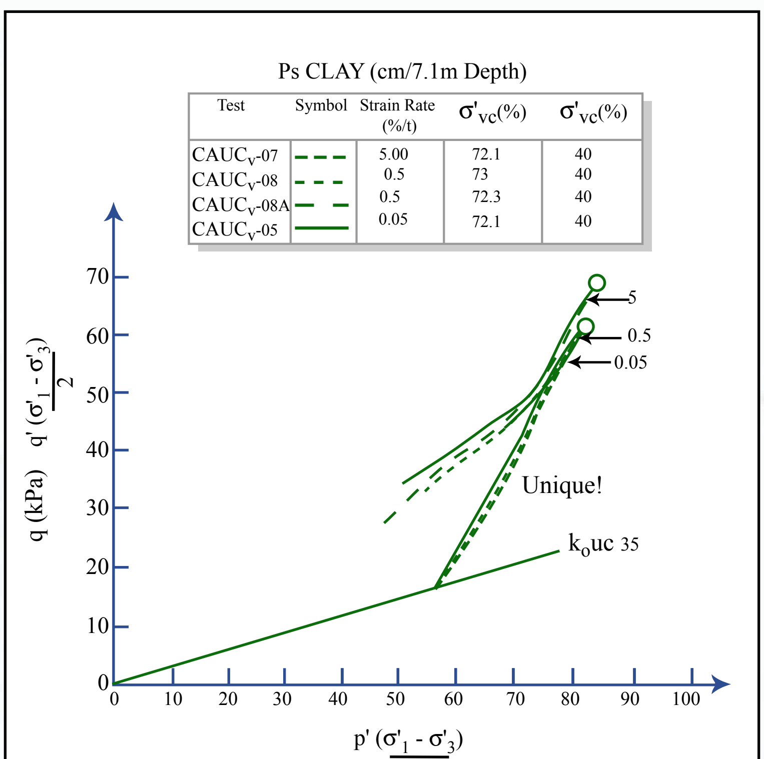
CU Test Data on "Intact" Highly Structured Clay ($\sigma'_{vc} \approx \sigma'_{vo}$)

Note: Vaid et al (1979, CGJ, 16(1)) showed similar behavior for Canadian clay; σ'_p = 920 kPa; I_L = 1.6, S_t = 10; CIUC testing



Stress strain and pore pressure strain curves for B6 Clay

Figures by MIT OCW.

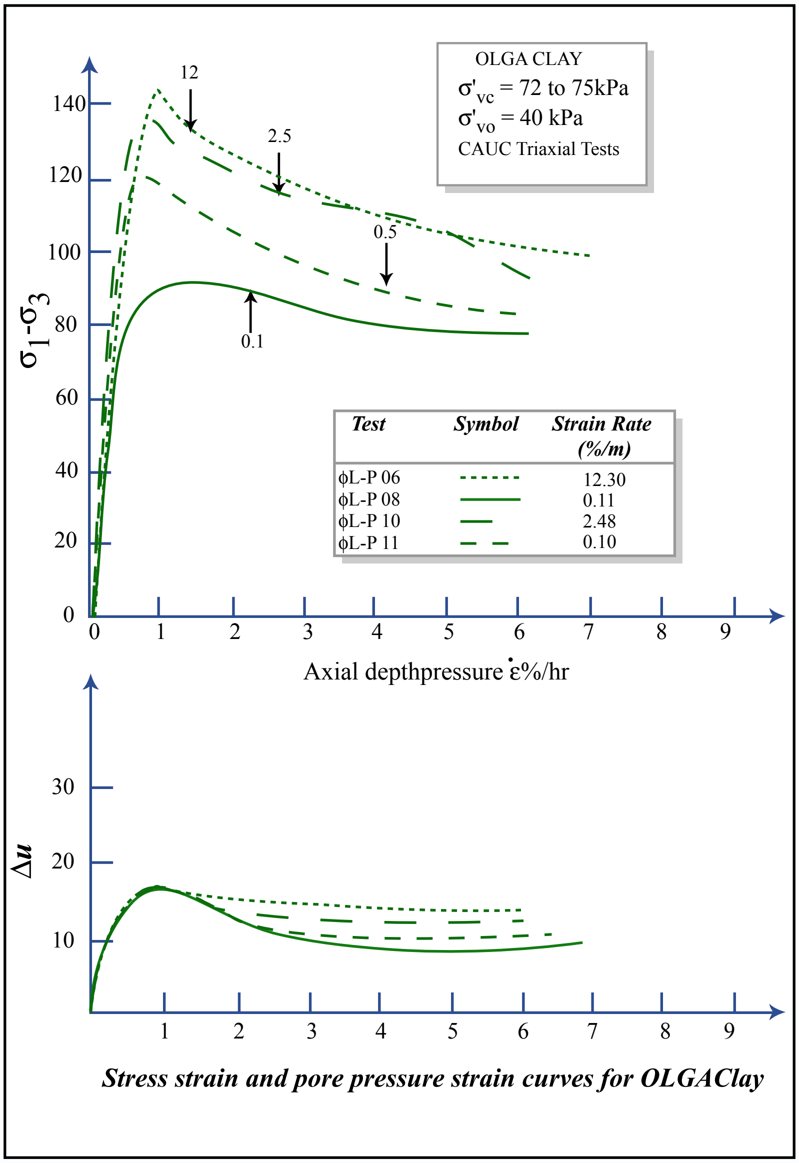


Stress Paths for Structured B6 Clay

CAUC I_p = 10% I_L = 2.5

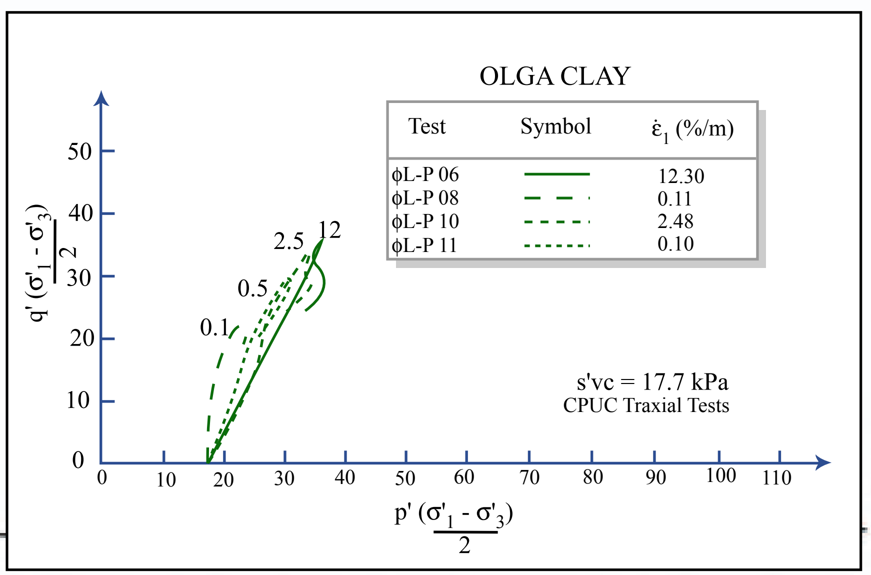
B = Broadback σ'_p = 175 kPa $\dot{\varepsilon}$ %/hr

(SEBJ NBR Project)



Stress strain and pore pressure strain curves for OLGAClay

Figures by MIT OCW.



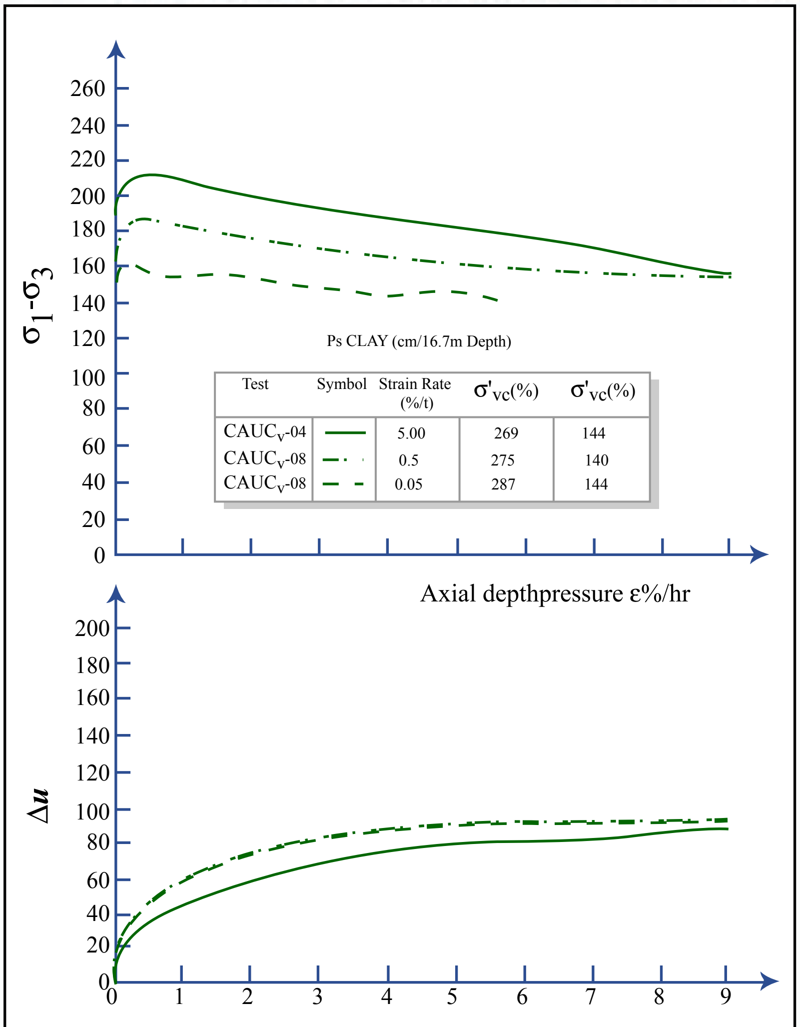
CIUC $I_p = 40\%$ $I_L = 1.55$

$\sigma'_p = 78$ kPa

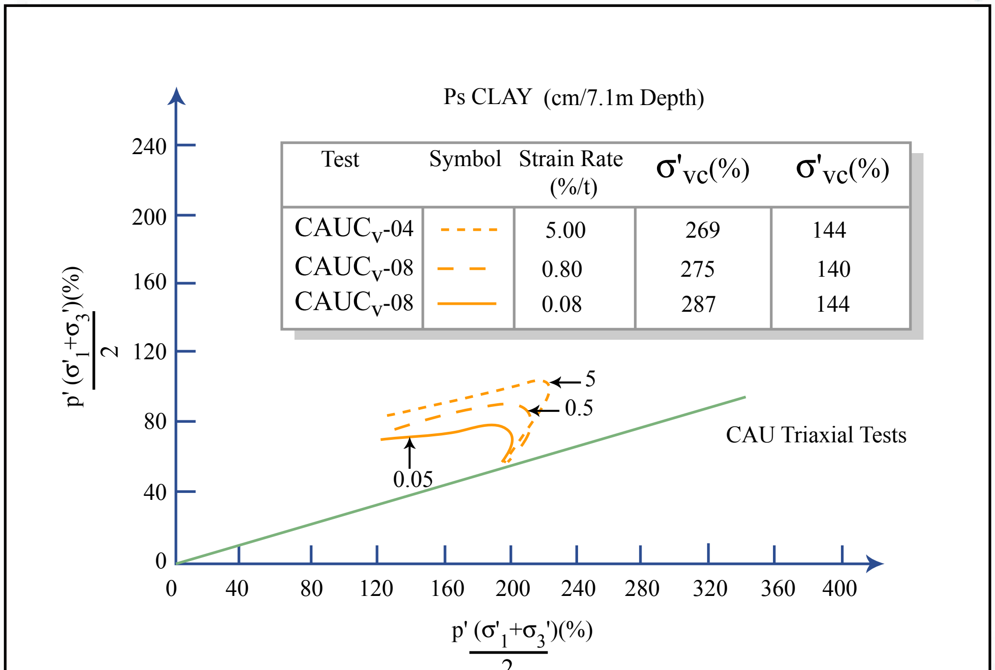
$\dot{\varepsilon}$ %/hr

- Approximately same Δu vs ε up to ε_f at peak undrained q_f (especially B6 → unique ESP independent of $\dot{\varepsilon}$)
- Therefore decrease in s_u due to lowering of failure envelope, i.e., brittle cementation bonds exhibit "structural viscosity"

CU Test Data on "Destructured" Clay ($\sigma'_{vc} > \sigma'_p$)



Stress-strain and pore pressure strain for normally consolidated B6 Clay

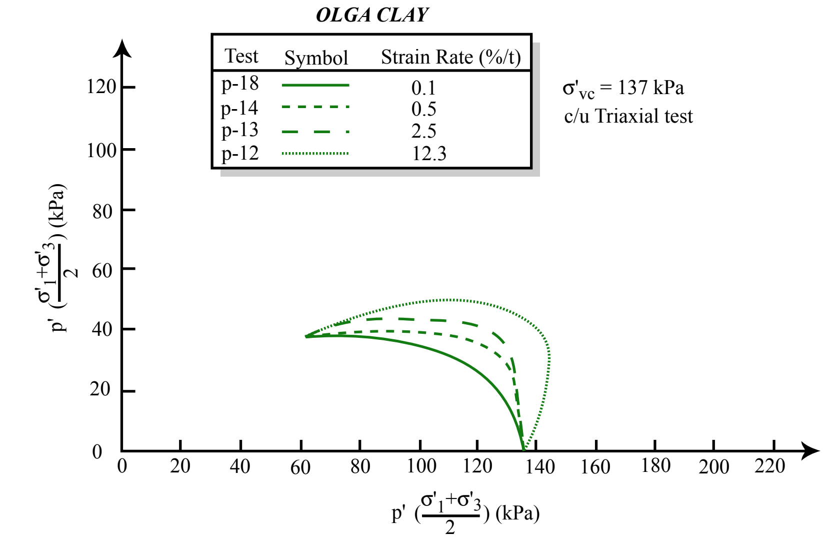


Stress paths for normally consolidated B6 clay

Figures by MIT OCW.

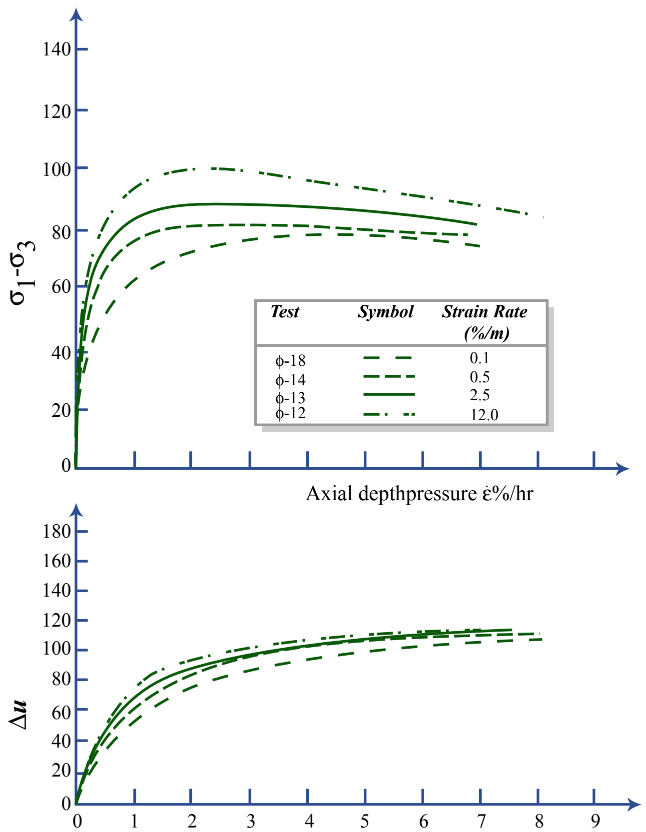
CK₀UC $I_p = 14\%$ $I_L = .68$

$\sigma'_p = 145$ $\sigma'_{vc} = 270$



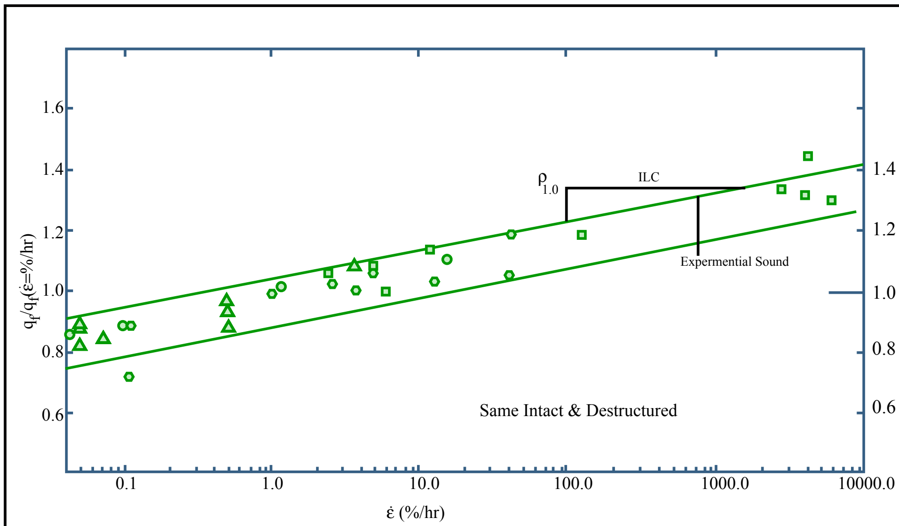
Stress paths for Normally consolidated Olga Clay

CIUC $\sigma'_{vc} = 1.7\,\sigma'_p$



Stress-strain and pore pressure-strain curves for normally consolidated Olga clay

Figures by MIT OCW.



CHANGE OF UNDRAINED STRENGTH RATIO, NORMALIZED TO UNDRAINED STRENGTH RATIO AT $\dot{\varepsilon}_1$ = 1.0%/h, WITH STRAIN RATE FOR ALL INVESTIGATED CLAYS.

4.5 Concluding Remarks

1) Be aware of most published data on $\dot{\varepsilon}$ effects due to experimental problems

   a) membrane leakage → incr. Δu → decr. s_u

   b) End restraint at high OCR → softening in shear zone → decreasing s_u

   c) If very low t_c, associated ↑ σ_{vc}' due to secondary compression may → confusing Δs_u vs $\Delta \dot{\varepsilon}$ (e.g. Holzer et al. 1973 CGJ, OCR tests on SFBM)

2) It is likely that all cohesive soils will exhibit strain rate sensitivity at very fast strain rates (say $\dot{\varepsilon}$ > 5-10%/hr) that will affect interpretation of in situ tests and lab UUC, TV, etc. tests. Can get very high ρ = 10-30%

3) For non-structured clays similar to Resedimented BBC, Sheahan et al. (1996) present the only good CK₀UC vs $\dot{\varepsilon}$ data as f(OCR). Principal conclusions are (for $\dot{\varepsilon}$ ≤ 5%)

   a) At low OCR ≤ 1:

      - $\rho_{0.5}$ due to both lower Δu_s and increased ϕ'_p
      - Should expect strain rate effects in field

   b) At medium to high OCR:

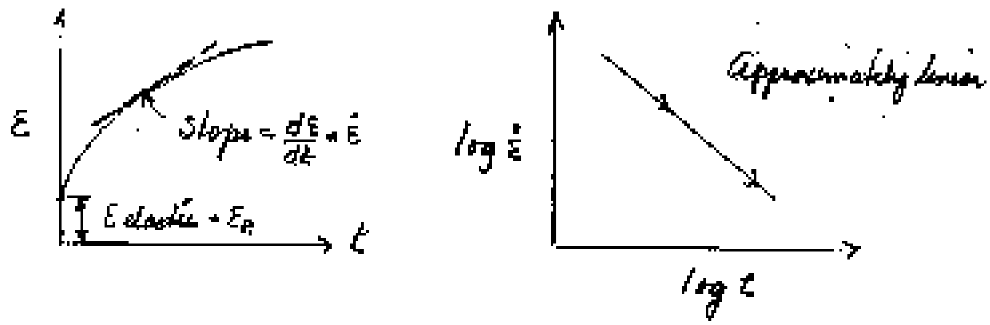
      - $\rho_{0.5}$ due mainly or only to lower Δu_s
      - Strain rate effects in field may be very small since ρ → 0 w/ incr. OCR at low $\dot{\varepsilon}$

4) For Canadian cemented clays, expect significant strain rate effects in both lab and field over entire $\dot{\varepsilon}$ range (for both consolidation and drained/undrained shear)

5. UNDRAINED CREEP

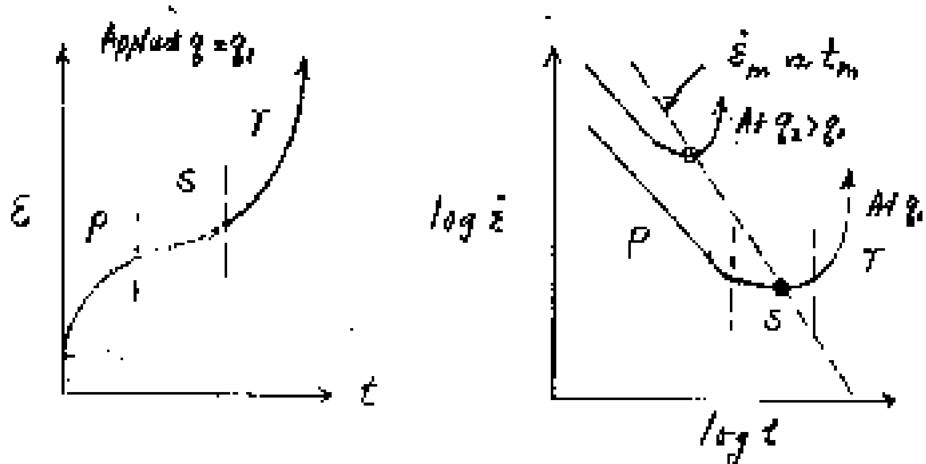
5.1 Introduction (Constant q testing)

1) "Low" Stress Level (Only "primary")



- Will model w/ Singh-Mitchell 3 parameter eqn.

2) "High" Stress Level → Creep Rupture



Transient ≡ Primary = $\dot{\varepsilon}$ decreasing

Viscous ≡ Steady State ≡ Secondary = $\dot{\varepsilon}$ "Constant" → $\dot{\varepsilon}_m$ (inflection point if creep rupture)

Tertiary = $\dot{\varepsilon}$ increasing → creep rupture

3) Questions

- Physical explanation of behavior (if possible)
- Mathematical models of behavior, at least for primary
- How to estimate q that will not → creep rupture (limiting long term q_f)
- "Correspondence" between creep & constant $\dot{\varepsilon}$ test data

5.2 Singh-Mitchell 3 Parameter Eqn. (1968, 1976, 1993 book) : t JSMFD 94(1)

1) Eqn

$$\dot{\varepsilon} = A e^{\bar{\alpha}\bar{D}} (t_1/t)^m$$

- "Derived" from Rate Process theory
- Restricted to Primary creep

t_1 = reference t

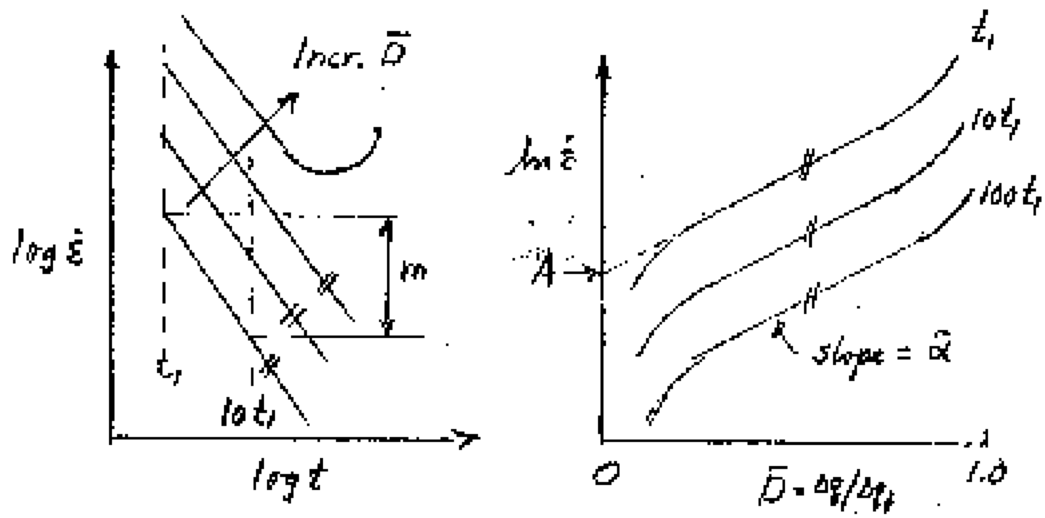
$\bar{D} = \Delta q/\Delta q_f$ (Δq_f at reference $\dot{\varepsilon}$)

$m = -d\log\dot{\varepsilon}/d\log t$

$\bar{\alpha} = d\ln\dot{\varepsilon}/d\bar{D} = 2.3\, d\log\dot{\varepsilon}/d\bar{D}$

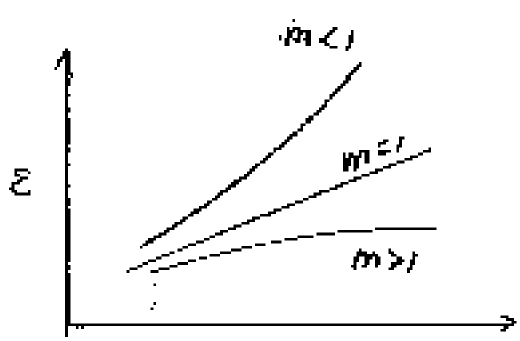
$A = \dot{\varepsilon}$ at $t = t_1$, $\bar{D} = 0$

2) Basic plots



$\bar{\alpha}$ presumed constant for $\bar{D} \approx 0.3 - 0.7$

3) Significance of m (Presumed basic soil property by S-M; lower m → more creep susceptible)



$m = 1 \rightarrow \dot{\varepsilon}t$ = constant (like constant C_α)

$m < 1$ more "creep susceptible" i.e. S-M w/ $\dot{\varepsilon}t$ increasing with t

$$\left.\varepsilon = \varepsilon_1 - \frac{A e^{\bar{\alpha}\bar{D}} t_1}{(1-m)} + \underbrace{\frac{\dot{\varepsilon}t}{(1-m)}}_{*}\right\} m \neq 1$$

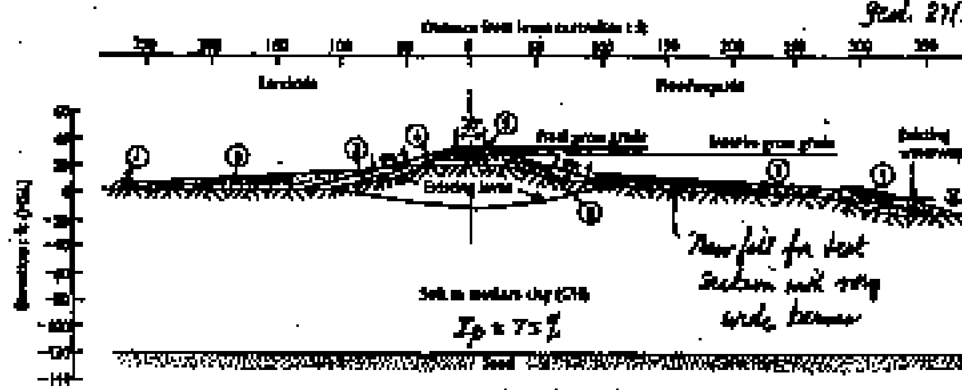
Slope $= d\varepsilon/dt = \dot{\varepsilon} = \dfrac{0.434\,(d\varepsilon/d\log t)}{t}$

* Is negative for $m > 1$ → substraction of less neg. no. w/ incr. t → incr. ε w/ time

4) Results from MIT research for flood control levees along Atchafalaya River in Louisiana next to Gulf of Mexico (Edgers et al. 1973 MIT report)

a) Problem (EABPL)

R. FOOTT AND C. C. LADD ('77) Geot. 27(2)



- "Existing" levees: construction 1930 to 1970 to maintain net grade of 15 ft caused accumulated ¢ ρ of up to 35 ft!
- Very expensive test sections (above) did not perform much better, i.e. excessive lateral deformations (undrained creep) + excessive ¢ settlement

b) Results from creep testing on EABPL clay

	Test	$\bar{D}$	m	$\bar{\alpha}$
Sl. OC	UUC	0.75-0.9	0.95	4.7
NC	CK_0UC	0.5-0.9	0.55±0.1	4±0.2
NC	CK_0UPSE	0.5-0.85	0.85-0.9	4±0.4

} $m = 0.55$ to 0.95 for this highly creep susceptible clay

5) Campanella & Vaid (1974 CGJ) tests on low I_p NC Haney Clay

- CIUC → $m = 0.6$ • CK_0UC → $m = 0.35$ • CK_0UPSC → $m = 0.5$

∴ $m = 0.35 - 0.6$ for clay that is not very creep susceptible

6) Conclusions

d) However eqn. still useful for modeling given set of primary creep data

a) m & $\bar{\alpha}$ are not material properties since they vary with mode of shearing

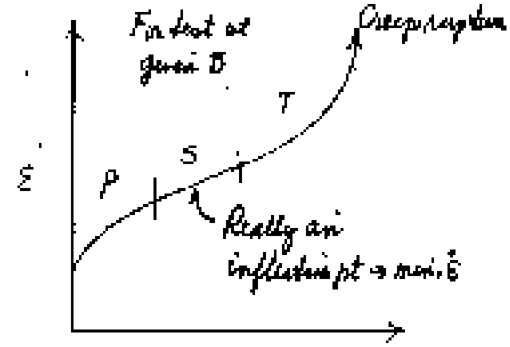
b) $\boxed{\text{Value of } m \text{ is not valid criterion for creep susceptibility, i.e. lower } m \text{ does not mean more highly creep susceptible}}$

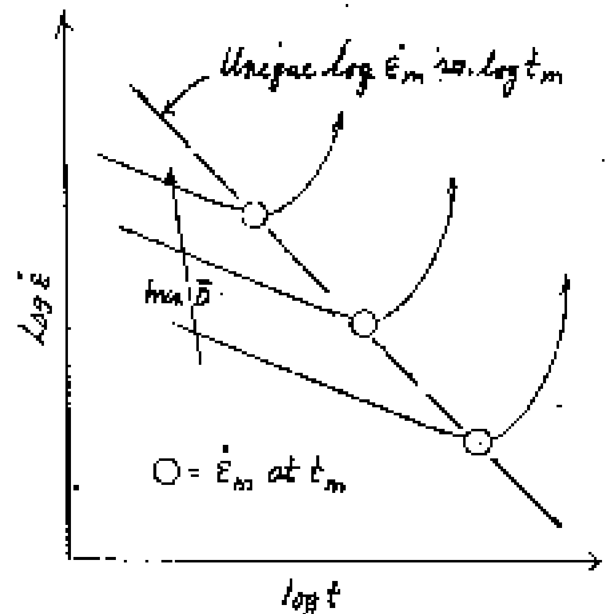
(see Section 5.6 for new criteria)

c) Techniques do NOT exist to predict undrained creep in the field (even though S-eqn. has been added to "MCC" to do this, e.g. Borja et al. 1990, JGE, 116(8))

5.3. Creep Rupture

1) General Behavior: Results from creep tests run at varying $\bar{D} = \Delta q/\Delta q_f$





∴ Creep tests (of given type) run at varying $\bar{D}$ yield unique relationship between minimum strain rate ($\dot{\varepsilon}_m$) and the time (t_m) to reach $\dot{\varepsilon}_m$

2) Strain at Minimum Strain Rate (ε_m)

- Experimental data show approximately constant strain (ε_m) along the $\log \dot{\varepsilon}_m$ vs $\log t_m$ relationship
- $\dot{\varepsilon}$ is decreasing before reaching ε_m, and then accelerates after reaching ε_m. This suggests that "damage" starts to occur near ε_m, leading to a weakened material that eventually fails in creep rupture.

3) Creep Rupture Data on Haney Clay (Sheet C1)

- Fig 2 shows $\log \dot{\varepsilon}$ vs $\log t$ data from CIUC, CK_oUC and CK_oUREC tests on NC clay → 3 different $\log \dot{\varepsilon}_m$ vs $\log t_m$ relationships
- Fig. 3 shows constant ε_m for each test series. However, ε_m decreases from $\varepsilon_m = 2.8\%$ for CIUC tests to $\varepsilon_m = 0.3\%$ for CK_oUC tests

4/99 5/01 5.3 continued

4) Log $\dot{\varepsilon}_m$ vs log t_m and ε_m Data for Other Materials (Sheet C2)

- Figs. 1 and 2 show log $\dot{\varepsilon}$ vs log t data from unconfined compression tests on frozen Manchester Fine Sand (ice saturation S_i > 40%) and polycrystalline ice, respectively, and their unique log $\dot{\varepsilon}_m$ vs log t_m relationships.
- Fig. 4 summarizes the unique log $\dot{\varepsilon}_m$ vs log t_m relationships for frozen MFS (S_i = 20%, 40% [100%]), ice and CIUC/CK$_0$UC tests on Haney clay.
- Note the shift to the right in log $\dot{\varepsilon}_m$ vs log t_m with increasing ε_m (i.e., increasing ε_m → longer time to reach critical strain at which "damage" → increasing $\dot{\varepsilon}$)
- log $\dot{\varepsilon}_m$ vs log t_m can be modeled by $\dot{\varepsilon}_m = \beta t_m^{\gamma'}$; data in Fig. 4 show $\gamma' = -1.0 \pm 0.2$

5) Summary of Main Points

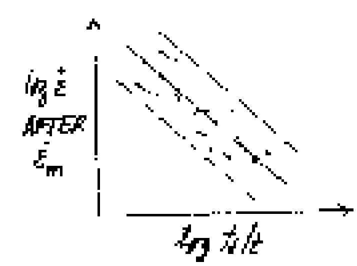
a) Should plot log $\dot{\varepsilon}$ vs log t from creep tests in order to identify the minimum strain rate ($\dot{\varepsilon}_m$) at $t = t_m$

b) Creep data from different types of tests (e.g., UC → CIUC → CK$_0$UC) and different materials (clay → ice) each show unique log $\dot{\varepsilon}_m$ vs log t_m (with slope $\gamma' = -1.0 \pm 0.2$) relationships having a constant ε_m

c) This ε_m represents onset of "damage" that → increasing $\dot{\varepsilon}$ and eventual creep rupture

6) Prediction of Creep Rupture

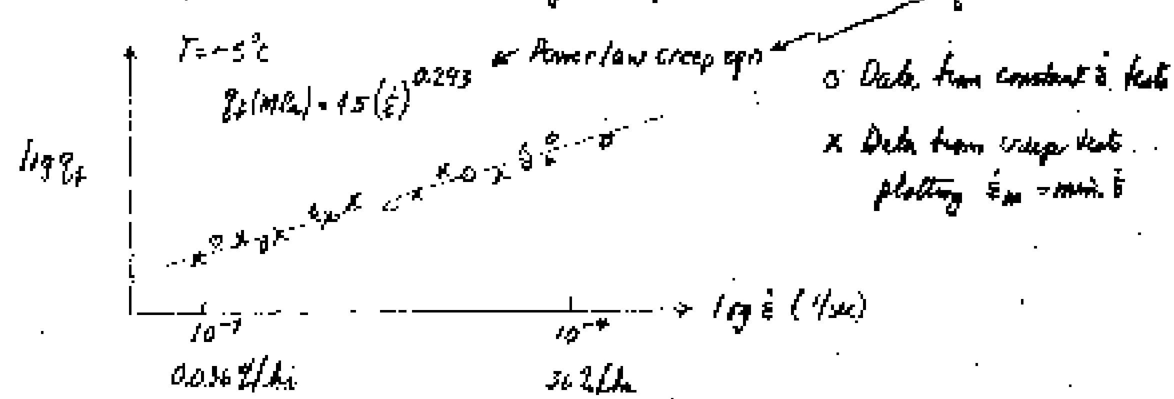
- The literature contains equations & plots to predict when creep rupture will occur.
- However, the scatter in data for different soils and different types of tests is so large that eqns/plots have little practical significance.



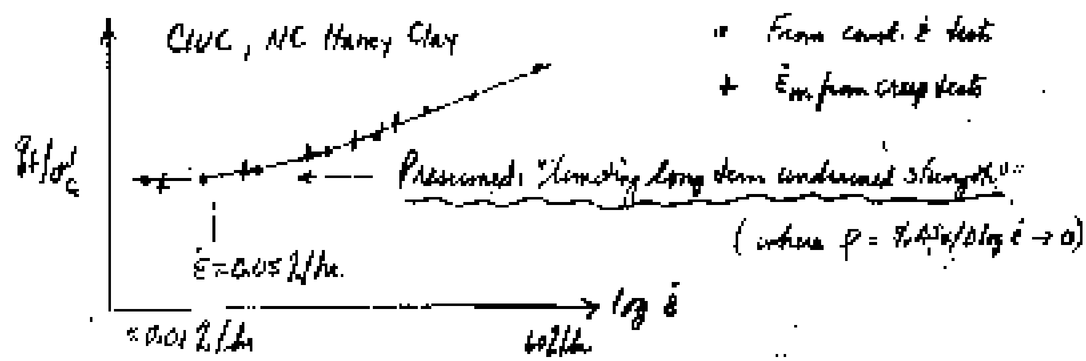
5.4 "Correspondence"

1) This topic addresses the issue between results from constant $\dot{\varepsilon}$ tests and from creep tests that eventually rupture.

2) Extensive data on polycrystalline ice* (within so-called ductile region with minimal cracking) show a unique relationship between strength and strain rate using $\dot{\varepsilon}_m$ for the creep tests, i.e. $q = C(\dot{\varepsilon})^{1/N=3.41}$



3) There are little data on clays comparing constant $\dot{\varepsilon}$ and creep testing. However, results for Haney clay (see sheet D) also show correspondence when use $\dot{\varepsilon}_m$ for creep tests.



4) Conclusion:

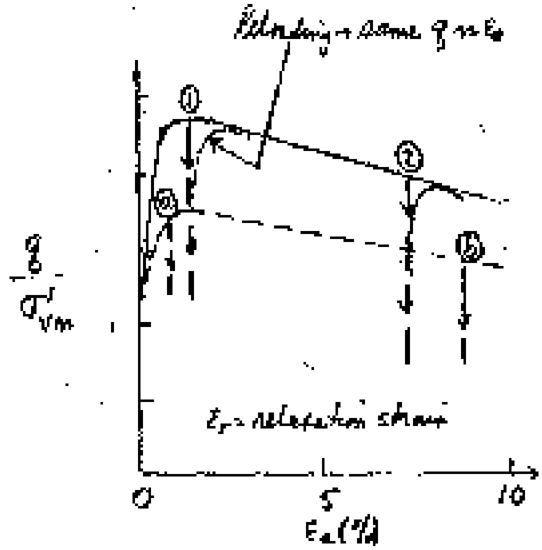
Use $\dot{\varepsilon}_m$ from creep tests for comparison with s_u vs. log $\dot{\varepsilon}$ from constant $\dot{\varepsilon}$ tests

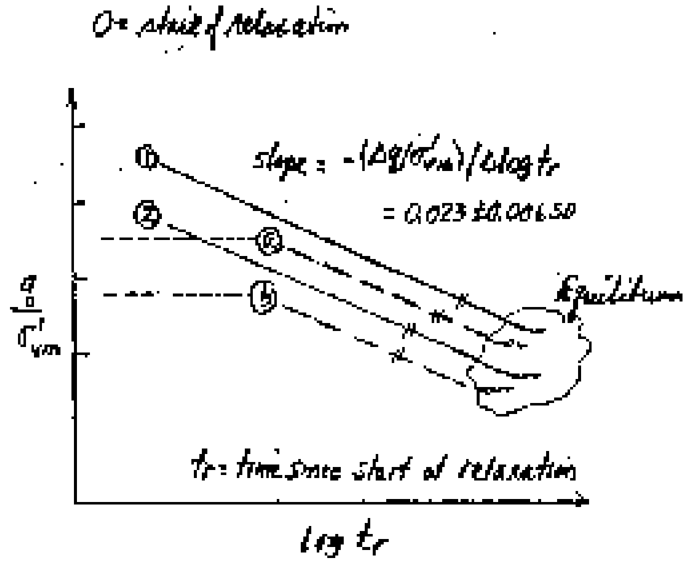
* Mellor & Cole (1983) Cold Regions Science & Technology, p 207-230

5.5 Relaxation

1) Refers to decrease in q (relaxation) at constant strain after shearing at constant $\dot{\varepsilon}$ up to the relaxation strain level (ε_r)

2) Overview of CK_0UC data from Sheahan et al. [1994; ASTM, GTJ 17(4)] on resedimented BBC (part of test program discussed in Section 4.3)





- For fast shearing to ε_r: relaxation starts quickly
- For slow " " " : start of relaxation is delayed

} Approx same slope independent of $\dot{\varepsilon}$, ε_r and OCR

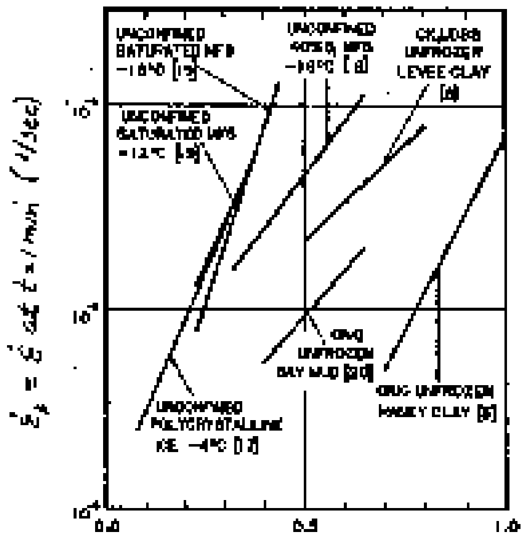
- For relaxation from relatively small strain levels ($\varepsilon_a \leq 1.5\%$), equilibrium stresses ended up close to NC K_0 line; Relaxation from larger strain level ($\varepsilon_a \geq 2.5\%$ → Δ structure of clay) → equilibrium at higher K.

3) See Sheets E1 & E2 for actual test data (mostly for OCR=1)
   - Fig. 1: q/σ'_{vc} vs ε_a
   - Fig 2: ESP data
   - Fig 3: q/σ'_{vc} vs $\log t_r$
   - Fig 3: Equilibrium stresses

5.6 Criteria For High Creep Susceptibility

Fig. 3 Ting et al. (1983) JGE 109(10)

1) m value is not reliable criterion



$\bar{D} = \Delta q / q_f$ (q_f at reference $\dot{\varepsilon}$)

2) Fig 3 plots $\dot{\varepsilon}$ at $t = 1$ min vs. $\bar{D}$ from creep tests on variety of materials.

- Haney clay of low creep susceptibility plots to lower right (low $\dot{\varepsilon}_1$ at high $\bar{D}$)
- Ice & frozen sand of high creep susceptibility plot to upper left (high $\dot{\varepsilon}_1$ at low $\bar{D}$)

3) Therefore, materials with high creep susceptibility have a high initial strain rates at low shear stress levels.

4) However, value of m is still relevant since:

- high m → rapid decay in $\dot{\varepsilon}$ with time
- low m → slow " " " " "

6. SUMMARY AND CONCLUSIONS

6.1 Measurements of s_u

1) Strain rate sensitivity of saturated cohesive soils ($\rho = (\Delta s_u/s_{uo})/\Delta \log \dot{\varepsilon}$)
   - All soils have significant ρ at very fast strain rates (say $\dot{\varepsilon} > 5$-$10\%/hr$). ρ may range from $\approx 10\%$ to $> 35\%$/logcycle
   - Cemented, sensitive Canadian clays probably have significant ρ at all strain rates
   - For "unstructured" clays (like RBBC), value of ρ at slow to moderate $\dot{\varepsilon}$ probably decreases with increasing OCR (Section 4.3)

2) Must consider $\Delta\dot{\varepsilon}$ (or Δt_f) when comparing s_u data from different types of shear tests
   - For in situ tests, CPTU $t_f \approx$ seconds & FVT $t_f \approx$ minutes
   - Lab UUC, Std $\dot{\varepsilon} = 1\%/min \rightarrow t_f \approx$ minutes
   - Lab CK_0UTX, $\dot{\varepsilon} \approx 0.5\%/hr \rightarrow t_f \approx$ hours

3) For SHANSEP/Recompression CK_0U testing programs, many major labs use

   $\underbrace{\cdot\ TX\ \ \dot{\varepsilon} \approx 0.5\%/hr}_{\text{Should be ok for most clays at OCR > 1}} \qquad \underbrace{\cdot\ DSS\ \ \dot{\gamma} = 5\%/hr}_{\text{May be somewhat unsafe for low OCR clays}}$

6.2 Predictions of Undrained Creep

1) The Singh-Mitchell 3 parameter eqn. (Section 5.2) is widely used to model primary creep data from lab tests. However, its use to predict in situ creep is suspect (due to variation in parameters with different modes of shearing; plus problems with its incorporation in an effective stress soil model)

2) One probably can assume correspondence between constant $\dot{\varepsilon}$ tests and creep rupture tests (using $\dot{\varepsilon}_{min}$) when developing s_u vs $\log \dot{\varepsilon}$ correlations

6.2 Cont.

3) It is still difficult to predict when undrained creep is likely to cause "excessive" deformation in the field.

- Refer to Foott & Ladd (1981) → loading at low FS of low K_c soil with long consolidation time
- Might also run some CU creep test for comparison with data in Fig 3, p21

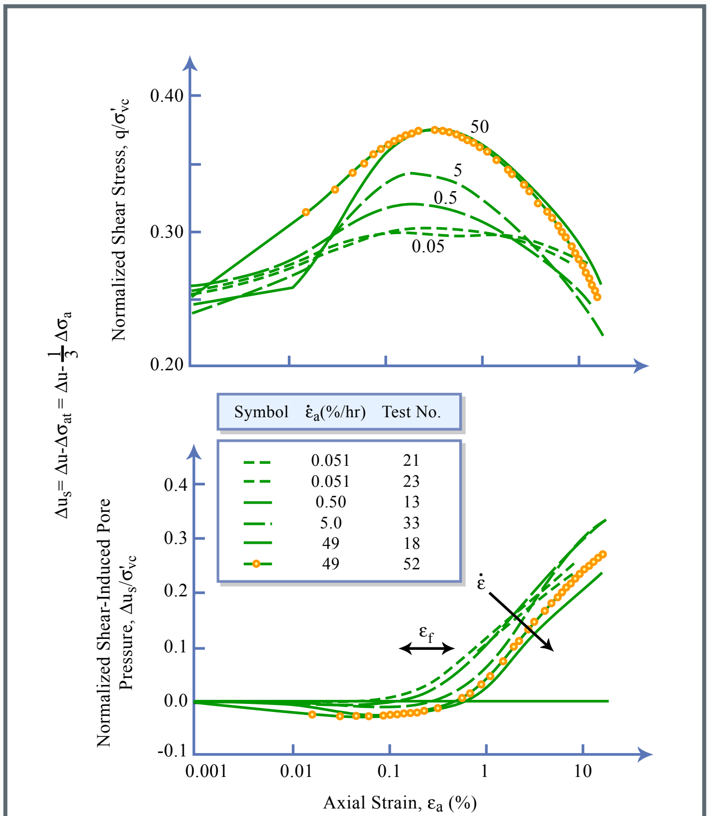
4) Will a loaded clay undergo undrained creep rupture in the field at a significant time after the end of loading?

CCL thinks highly unlikely, but many others will disagree

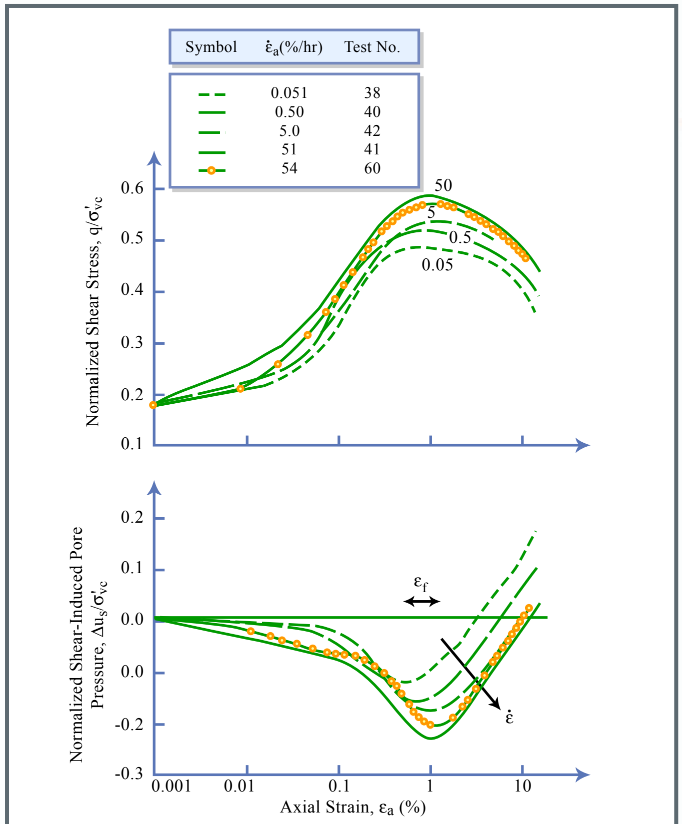
6.3 Miscellaneous

1) The importance of thixotropy in situ remains unclear

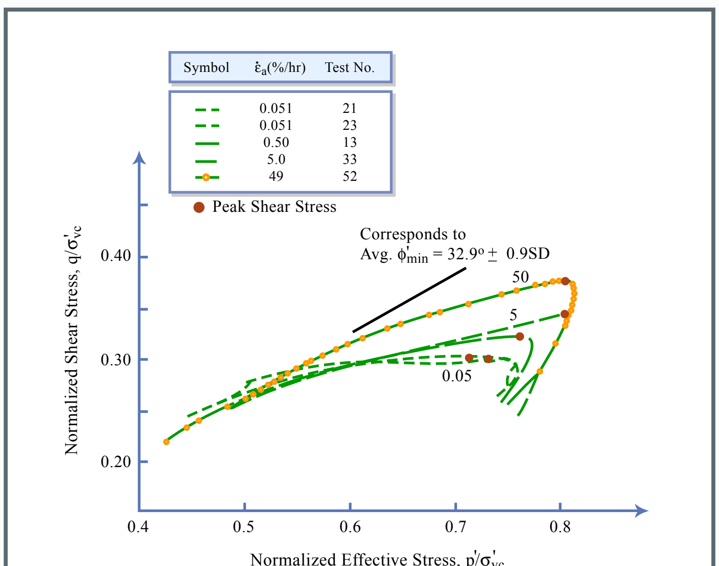
2) For NC clays that are still consolidating, use $t_c = t_p$ (vs. std. t_c = 1 day) for CK_0U testing



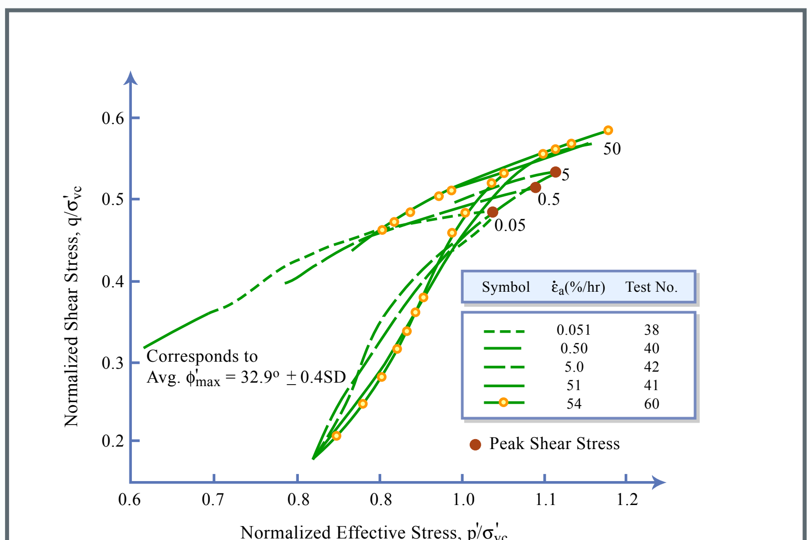
Normalized Shear Stress and Shear-induced Pore Pressure vs. Strain, OCR = 1 CK₀UC Tests, Resedimented BBC



Normalized Shear Stress and Shear-induced Pore Pressure vs. Strain, OCR = 2 CK₀UC Tests, Resedimented BBC



Normalized Effective Stress Paths, OCR = 1 CK₀UC Tests, Resedimented BBC



Normalized Effective Stress Paths, OCR = 2 CK₀UC Tests, Resedimented BBC

Figures by MIT OCW.

CK₀UC Tests on Resedimented BBC as f(ε̇) : OCR = 1 & 2

Adapted from: Sheahan et al. (1996)

A1

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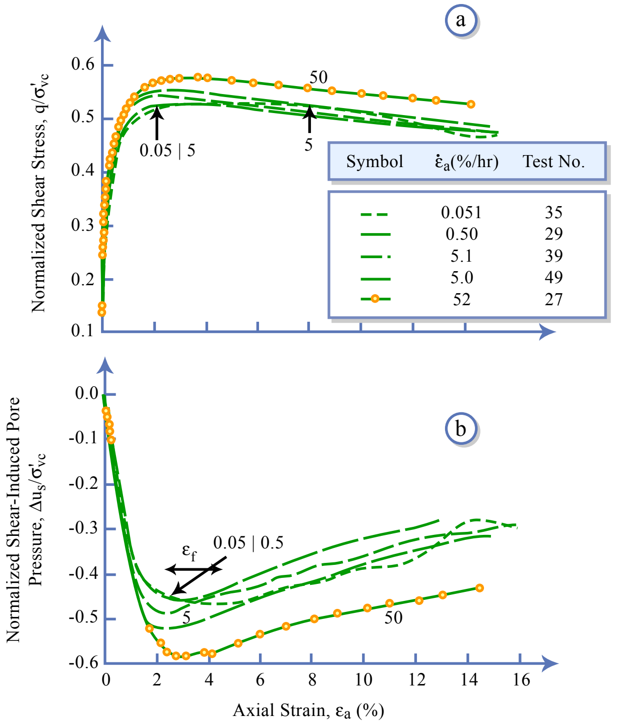
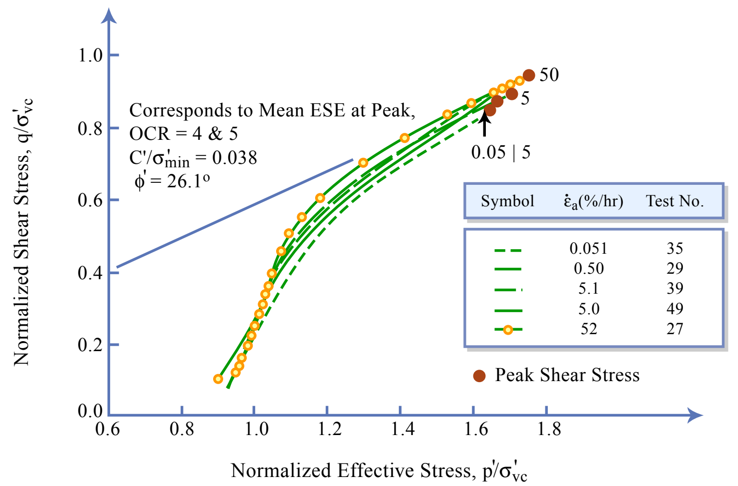
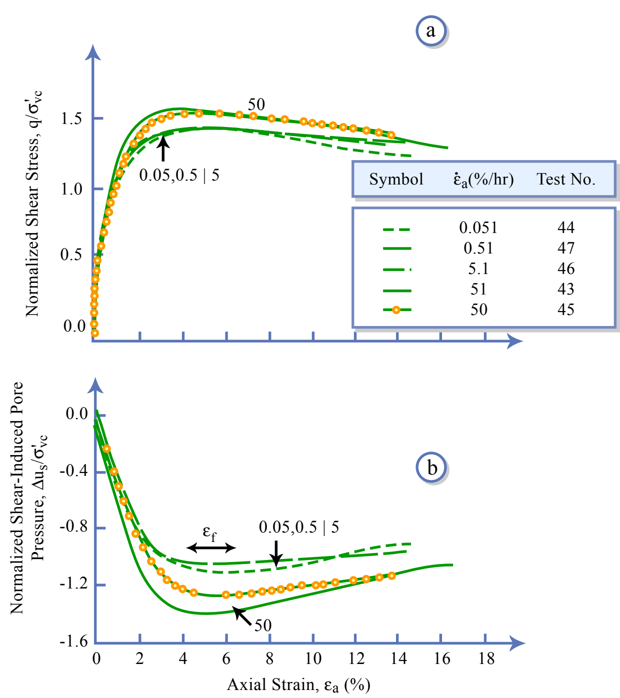
IIB



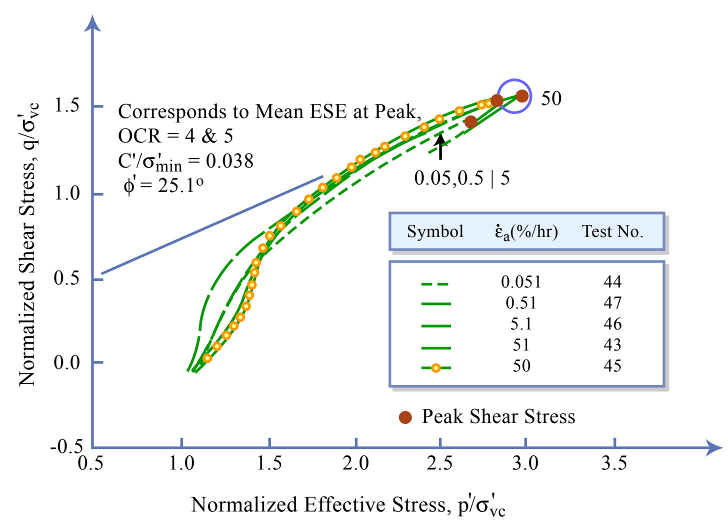
Normalized Shear Stress and Shear-induced Pore Pressure vs. Strain, OCR = 4 CK_0UC Tests, Resedimented BBC



Normalized Effective Stress Paths, OCR = 4 CK_0UC Tests, Resedimented BBC



Normalized Shear Stress and Shear-induced Pore Pressure vs. Strain, OCR = 8 CK_0UC Tests, Resedimented BBC



Normalized Effective Stress Paths, OCR = 8 CK_0UC Tests, Resedimented BBC

Figures by MIT OCW.

CK_0UC Tests on Resedimented BBC as $f(\dot{\varepsilon})$: OCR = 4 & 8

A2

Adapted from: Sheahan et al. (1996)

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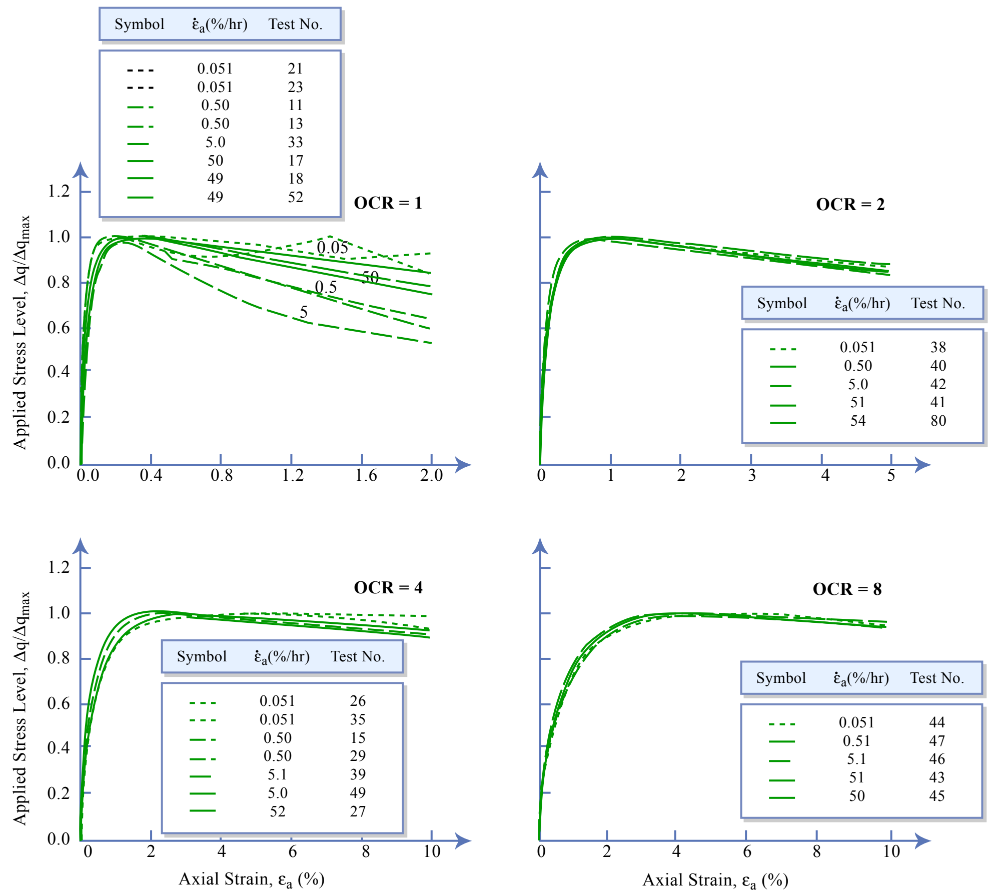
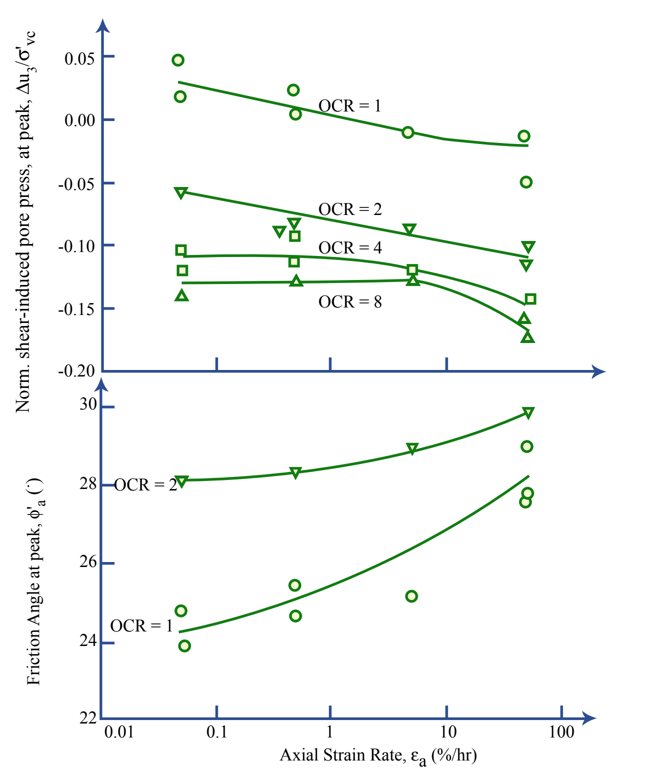
II B



Axial Shear Stress Level vs. Strain, CK_oUC Tests, Resedimented BBC

Unique NORMALIZED Shear Stress ($\Delta q/\Delta q_{max}$) vs ε_a (especially OCR = 2, 4 & 8)

Increasing s_u always accompanied by decreasing Δu_s



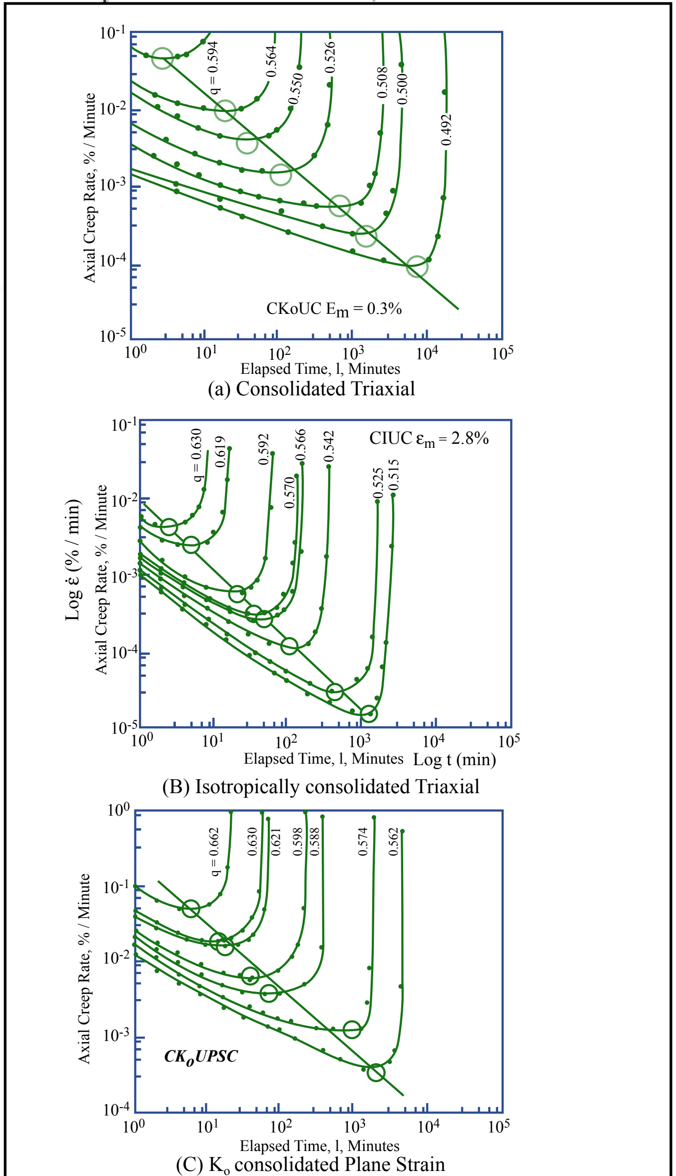
Summary plots of Mechanisms: (a) Normalized Shear-induced pore pressure and (b) Friction angle at peak versue strain rate, CKoUC tests, resedimented BBC

Increasing s_u at low OCR also accompanied by increasing ϕ'

Figures by MIT OCW.

Adapted from: Sheahan et al. (1996)

B



Creep rate behavior of normally consolidated undisturbed Haney clay.

Figure by MIT OCW.

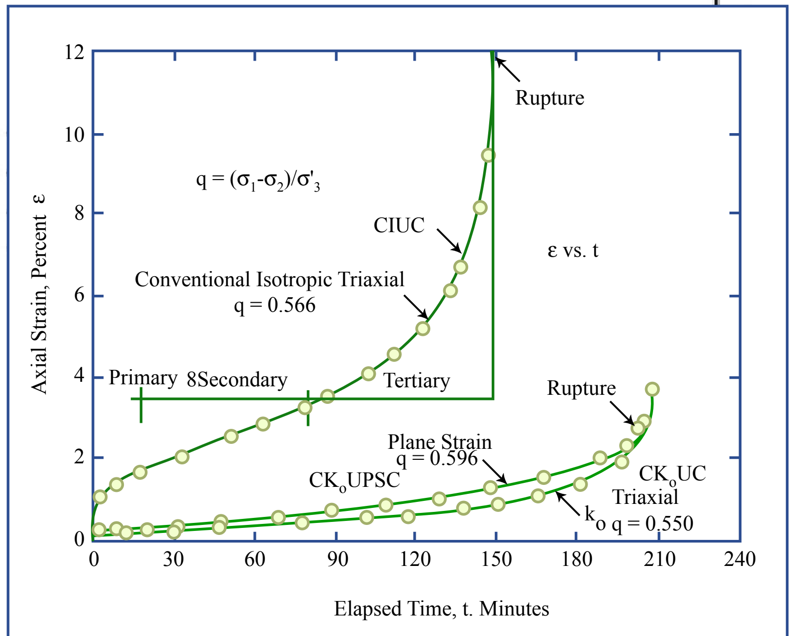


Figure by MIT OCW.

Campenella & Vaid (1974) CGJ, 11(1)

N.C. Haney Clay $w_L = 44$, $I_p = 18$

Lab $\sigma'_{vc}/\sigma'_p = 1.5$

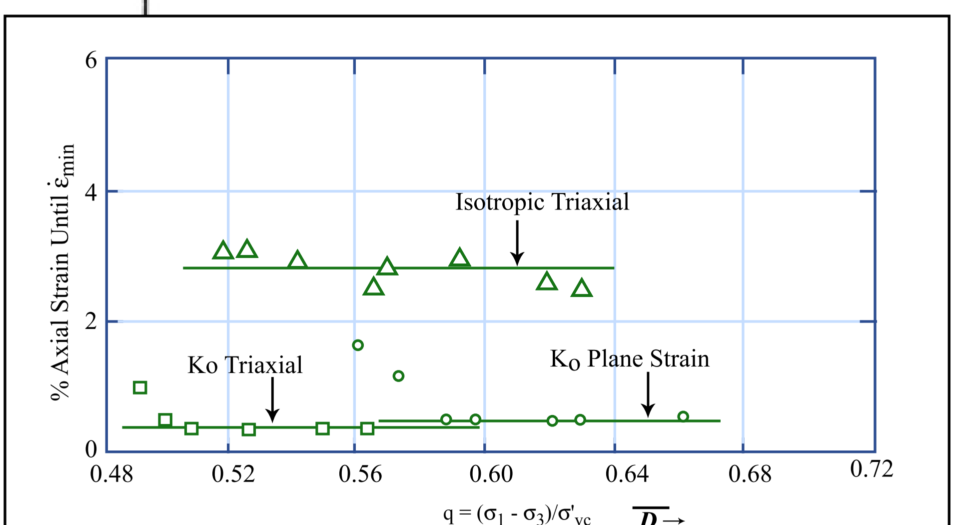
• Fig. 2 = log $\dot{\varepsilon}$ vs log t at varying $\bar{D}$

O = location of min $\dot{\varepsilon} = \dot{\varepsilon}_m$ at $t = t_m$

(lines through $\dot{\varepsilon}_m - t_m$ by CCL)

• Fig. 3 Axial strain at $\dot{\varepsilon}_m = \varepsilon_m$

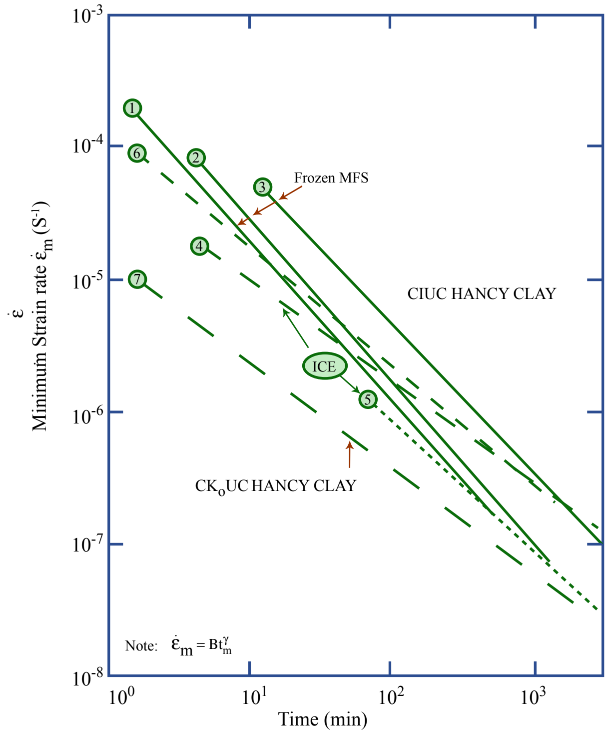
Test	ε_m (%)
CIUC	2.8
CK_oUPSC	0.5
CK_oUC	0.3



Axial strain until minimum strain rate as a function of creep stress.

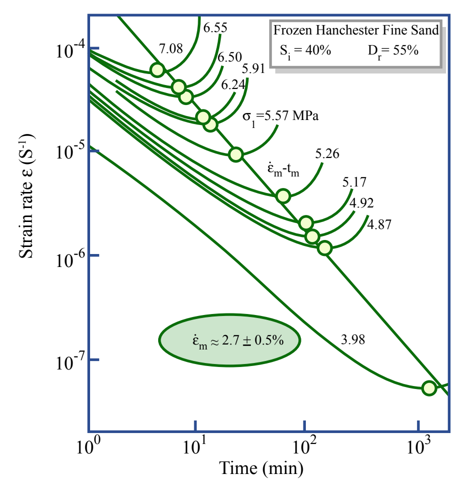
Figure by MIT OCW.

C1

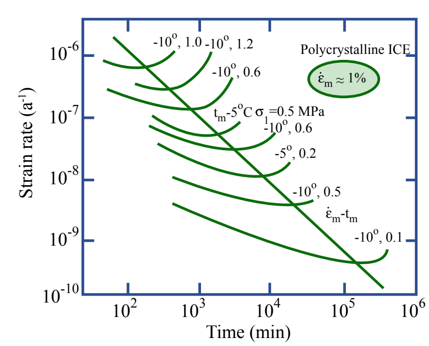


No.	Material	Testing	Reference	B°	γ°	r^2	No. Tests	ε_m (%)
1	20% S_i MFS	Uniaxial	Martin Ting & Load (1981)	2.8×10^{-4}	-1.2	0.987	7	2.1
2	40% S_i MFS	Uniaxial		4.2×10^{-4}	-1.2	0.993	40	2.7
3	100% S_i MFS	Uniaxial		8.1×10^{-4}	-1.2	0.991	28	4.6
4	Ice	Uniaxial	k_{uo} (1972)	7.9×10^{-5}	-0.8	0.987	7	≈ 1
5	Ice	Uniaxial	Jacka, in Lue (1979)	6.5×10^{-5}	-1.0	0.996	8	
6	Unfrozen Haney Clay	CIUC	Campanella & Void (1974)	1.3×10^{-4}	-0.9	0.997	8	2.8
7		CK_oUC		1.5×10^{-5}	-0.8	0.987	7	0.3

Summary of minimum creep rate: Correlations of time to minimum for various materials



Results of unconfined (uniaxial) compressive creep testing of 40% saturated, 55% relative density Manchester fine sand at - 18.8°C (data from Martin et al. 1981)



Results of unconfined (uniaxial) compressive creep testing of polycrystalline ice. (data by Jacka, see Lile 1979).

Figures by MIT OCW.

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Figures by MIT OCW.

Adapted from:

Vaid & Compenella (1977) JGED 103 GT

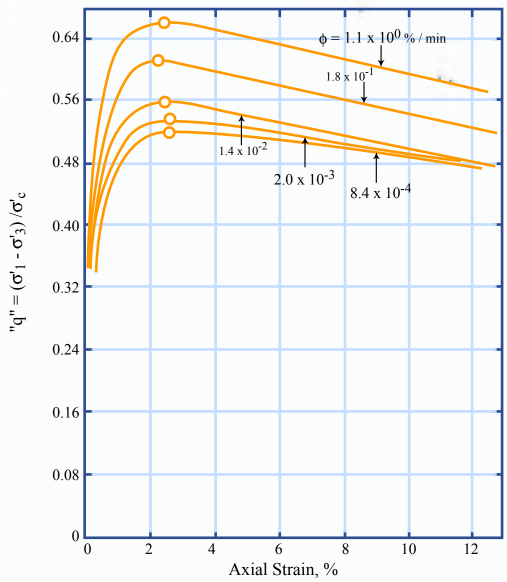
CIUC OCR=1 Haney Clay

Fig. 2 Constant $\dot{\varepsilon}$, take peak •

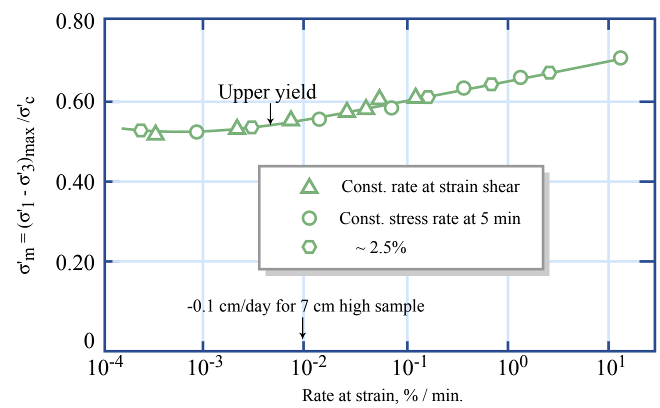
" 5 Constant stress, use $\dot{\varepsilon}_m$ •

&

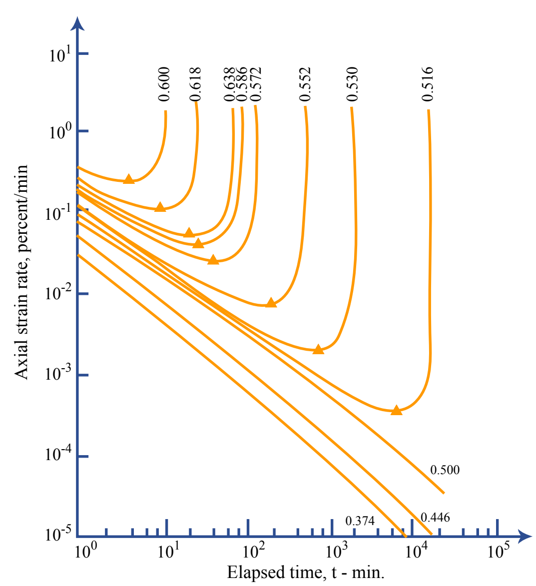
Fig 3 "Correspondence"



Influence of rate strain on undrained stress-strain behavior in constant rate of strain shear.

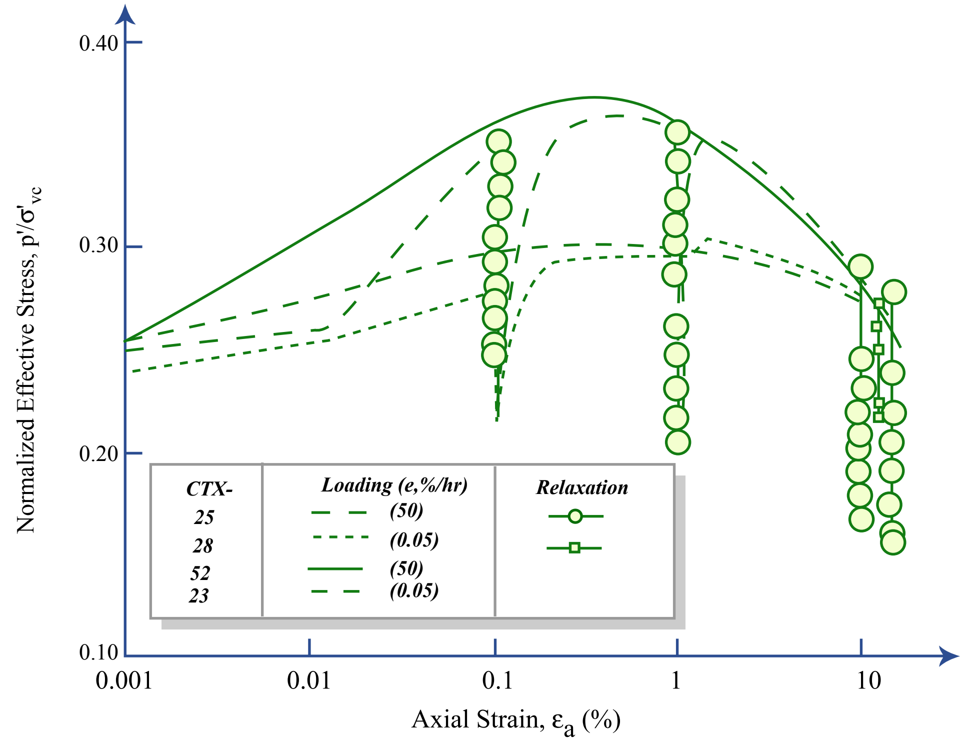


Strain rate dependence of undrained strength in constant rate of strain Shear and constant stress creep

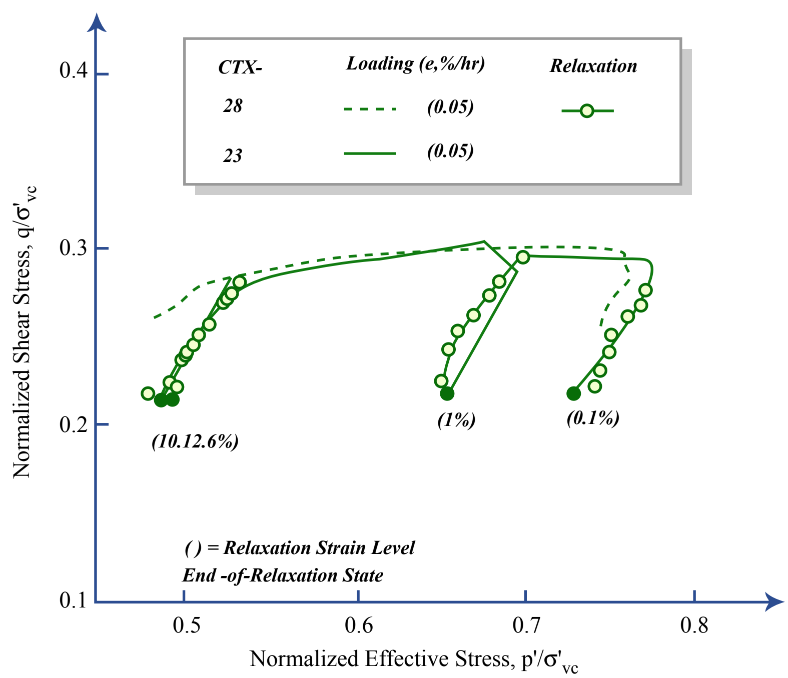


Variation of creep rate with time in constant stress creep

D



Stress-strain curves, relaxation versus constant strain rate CKoUC tests, resedimented BBC: OCR =1



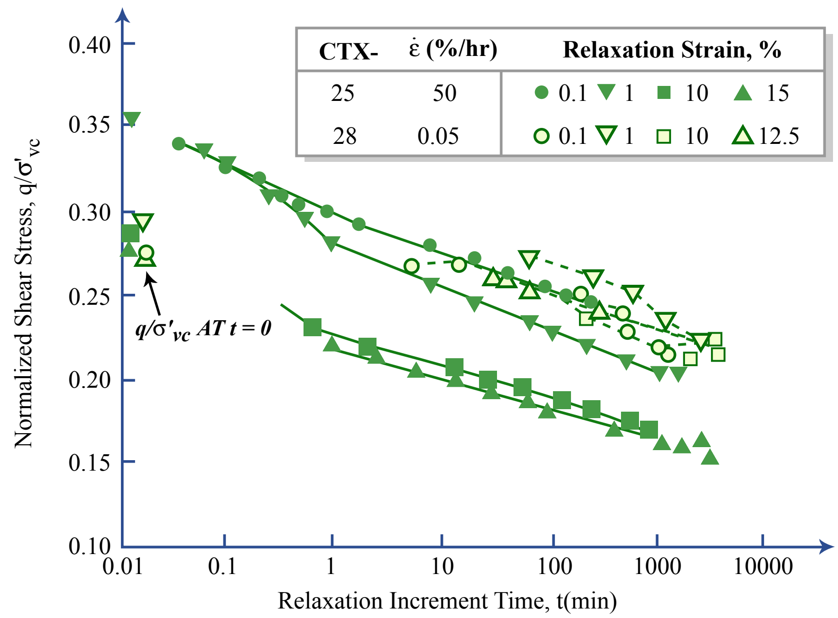
Effective stress paths, relaxation versus constant strain rate CKoUC tests, resedimented BBC.OCR = 1,0.05%/h

Figures by MIT OCW.

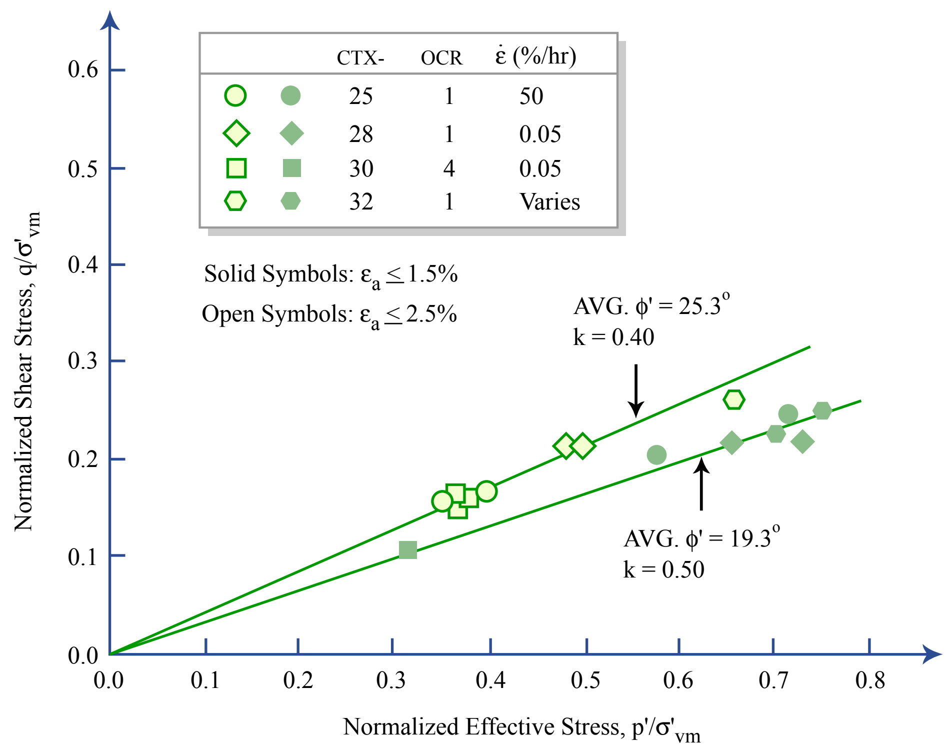
Relaxation data from CK₀UC tests on RBBC

Adapted from: Sheahan et al (1994)

E1



Shear stress decay with time, CKoUC relaxation tests on resedimented BBC: OCR =1



Stabilized stress states at the end of relaxation phases, CK_0UC relaxatin tests on resedimented BBC.

Figures by MIT OCW.

Relaxation data from CK_0UC tests on RBBC

Adapted from: Sheahan et al. (1994)

IIE STAGED CONSTRUCTION (Mostly abstracted from Ladd (1991) = Terzaghi Lecture)

(Mini-Questions - 1p)

CCL 4/30/96 1.322 IIE
4/01

Mini Questions on IIE: Staged Construction

1) Does this material apply both to stability during construction and to the "long term" case where $u_c = 0$ ($\bar{U} = 100\%$)?

2) Regarding comparison of ESA & USA stability analyses:

a) Altho. both require a knowledge of the in situ σ' values, how do they differ in use of this information?

b) Which type of analyses is easier to use and why (assuming extensive u data from piezometers)?

c) Does the above answer depend upon whether your USA analysis follows Ladd's recommendations or uses the QRS methodology?

3) Regarding the three case histories

a) Do any of these "prove" that ESA → very unsafe values of FS?

b) For the two embankments, what are the major limitations of the USA stability estimates? In particular, what would you do in order to obtain more reliable estimates of FS(USA)?

4) How would you use CK_0U TC/TE data on NC clay to develop a best estimate of $c_u = f(\alpha)$ for UTEXAS3 stability analyses (non-varved clay)?

IIE STAGED CONSTRUCTION (TL = Ladd (1991) Terzaghi Lecture)

1 INTRODUCTION

1.1 Background (Sections 1 & 2)

1) Controlled rate of loading → increased consolidation → faster strength gain to improve foundation stability of embankments, landfills, tanks, etc. & slope stability of tailings waste storage dams

2) Controversial issue: What TYPE of stability analysis to use
   - For design of project (need to predict u)
   - Check stability during construction
   - " " of an existing structure (either shortly or long after end of construction)

   {Use measured u}

1.2 Types of Stability Analysis & Definition of Factor of Safety (Sections 2 & 6)

1) $FS = \dfrac{\text{Available shear strength of soil} = s}{\tau_m = \text{shear stress required for equilibrium} = \text{mobilized } \tau}$

IMPORTANT NOTE: s must be consistent with assumed in situ drainage conditions during potential failure

2) **TSA** = Total Stress Analysis

$FS = s_u/\tau_m$
- s_u from "UU" type testing, e.g. FVT, UUC
- Generally only applied to UU Case

3) ESA = Effective Stress Analysis = Drained Strength Analysis

$FS = s_d/\tau_m = \tan\phi'/\tan\phi'_m$ (since same FS applied to both c' & $\tan\phi'$)

- Correctly applied to CD Case for unloading problems
- But also widely used for staged construction
- Treats in situ σ' = σ' at failure

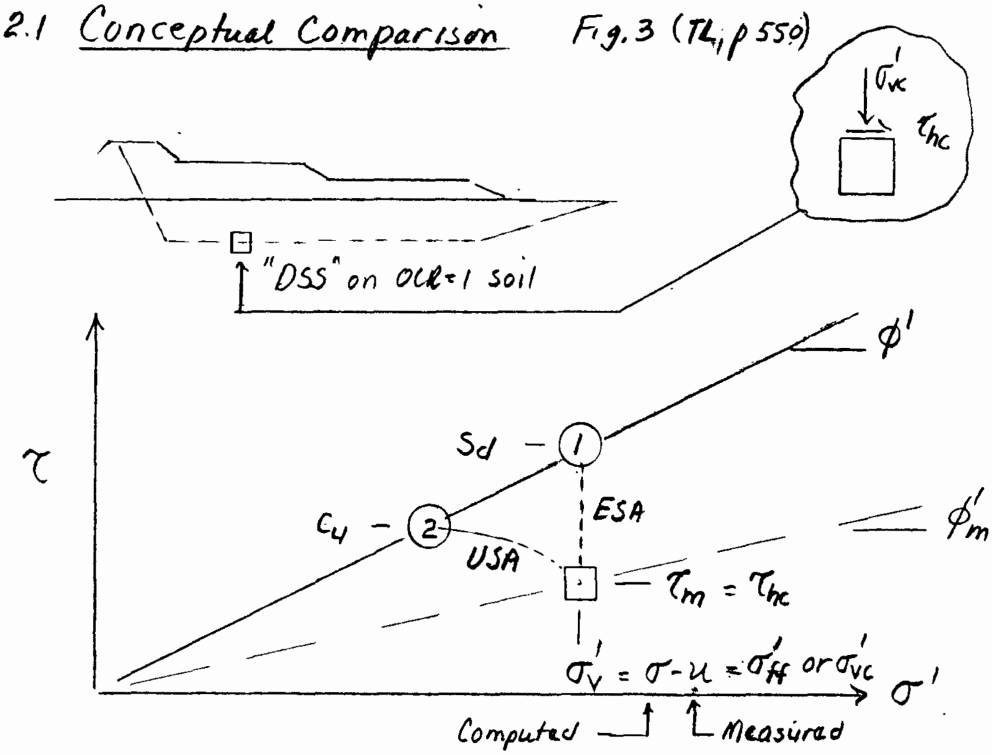
4) USA = Undrained Strength Analysis

$FS = c_u/\tau_m$

- Treats in situ σ' = consolidation stresses prior to undrained failure $[c_u = f(\sigma'_c)_{ij}]$
- Can be applied to both UU & CU Cases
- Different methodologies
  CCL vs QRS

2. COMPARISON OF ESA vs USA FOR STAGED CONSTRUCTION

2.1 Conceptual Comparison   Fig. 3 (TL, p 550)



[Recommended by Bishop & Bjerrum (1960) to account for dissipation of excess pore pressures during/after construction. [Before SHANSEP invented to predict $\Delta S_u = f(\Delta\sigma'_{vc})$]

(1) ESA Treats $\sigma'_v = \sigma'_{ff} \rightarrow s_d = \tau_{ff} = \sigma'_v \tan\phi' \rightarrow FS = s_d/\tau_m = \tan\phi'/\tan\phi'_m$
Inherently assumes $u_s = 0$ (altho users may select u > measured and/or ϕ' < measured) corresponding to CD Case

(2) USA Treats $\sigma'_v = \sigma'_{vc} \rightarrow c_u = S\,\sigma'_{vc} \rightarrow FS = c_u/\tau_m$
Inherently assumes undrained failure corresponding to CU Case

3) Simplified prediction à la p 562 of TL

- $$\frac{FS(ESA)}{FS(USA)} = \frac{\tan\phi'/\tan\phi'_m}{c_u/\tau_m} = \frac{\tan\phi'/(\tau_m/\sigma'_{vc})}{(c_u/\sigma'_{vc})/(\tau_m/\sigma'_{vc})} = \frac{\tan\phi'}{c_u/\sigma'_{vc}} = \frac{\tan\phi'}{S(DSS)}$$

- $\phi' = 25°$ & $S = 0.20$ ..., $\phi' = 30°$ & $S = 0.25$ $\rightarrow \frac{FS(ESA)}{FS(USA)} \approx 2.3$!

2.2 Case Histories (Section 3)

Table 2 (p 561)

Example	Condition	$FS\left(\frac{ESA}{USA}\right)$	Sheet
1) Embankment on CVVC (Design)	$\bar{U} = 100\%$	$1.9\ (\frac{2.8}{1.5})$	A1*
2) Embankment on Quick Clay (Design)	$\bar{U} = 100\%$	$2.3^5\ (\frac{5.2}{2.2})$	A2*
3) Upstream Tailings Dam (Construction)	During construction with meas. u	$1.9\ (\frac{2.4}{1.25})$	A3

* See Table 2, Sheet A3 for values of S & m

NOTES 1) & 2) design studies
3) Real problem where adjacent dam failed during construction under similar conditions

2.3 Conclusions

1) Experience and common sense tell us that actual failures of loads on soft, cohesive soils occur rapidly (hence preclude significant dissipation of shear induced pore pressures, u_s).
2) Therefore should treat staged construction as CU Case and obtain FS via Undrained Strength Analysis (USA) wherein $c_u = f$ (in situ consolidation stresses)
3) Moreover, an undrained failure will occur whenever in situ $\tau_m \rightarrow$ in situ c_u
4) Since an ESA inherently assumes a <u>slow, drained failure</u> (CD Case), it is highly UNSAFE (even though many practitioners still use: see Section 3.8 of TL)

3. USA METHODOLOGY

3.1 Recommended Approach (Section 5 & Table 5, p580)

1) Establish initial stress history, i.e. profiles of σ'_{vo} & σ'_p

2) Establish changes in vertical stress history via stress distribution analyses plus
   - Consolidation analyses for design
   - Piezometers during construction

   $\}$ for $u \rightarrow \sigma'_{vc} = \sigma_v - u$

3) Develop $c_u/\sigma'_{vc} = S(OCR)^m$ relationships for fdn soils
   - A CK₀U C, DSS & E + strain compatibility using SHANSEP or Recompression
   - B CK₀UDSS à la SHANSEP

   $\}$ anisotropic c_u (τ_c, τ_d & τ_e)
   - C Empirical correlations for S & m

4) Use 1) + 2) + 3) → computed c_u profiles for USA analyses

3.2 Discussion of 3.1

1) Simplifications & errors in estimation of c_u

   a) Use of σ'_{vc} that can be significantly less than σ'_{1c} → predicted c_u too low
   - How/when use CAUDSS testing à la Fig 19 (Sheet B)

τ_{hc}/σ'_{vc} =	0	0.1	0.2	} 3 clays
% inc. in c_u =	0	5±5%	30±10%	

   b) Should have used S from CK₀U tests with $t_c = t_p$ (not t_c = 1 day) for NC clay à la Section 3.4 of IID

2) Simplifications in stability analyses for two embankment case histories

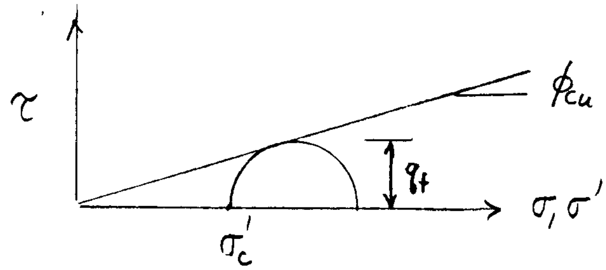
   a) Used active wedge at α = + 50-60° with τ_c
      " passive " " α = - 30° with τ_e
      ] + horizontal surface with τ_d

   b) More sophisticated analyses with UTEXAS3 would → lower FS using non-circular search for more critical failure surface (Also see Section 4 of IIE).

3.3 QRS Methodology (Section 6)

1) Initial c_u from UUC - You should know problems

2) Gain in strength from CIUC Fig. 2, 20 (p588)
(Sheet B)



$$c_u = \sigma'_{fc} \tan\phi_{cu} = \sigma'_n \tan\phi_{cu}$$

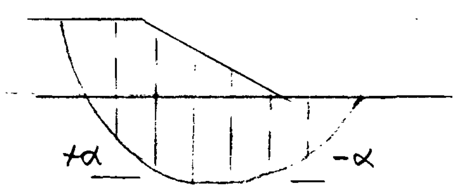
- What is physical significance of ϕ_{cu}? (Answer = NONE)

- $c_u/\sigma'_{fc} = \tan\phi_{cu} = \dfrac{q_f/\sigma'_c}{\sqrt{1+2q_f/\sigma'_c}}$

q_f/σ'_c	$\tan\phi_{cu}$
0.25	0.204
0.30	0.237
0.35	0.268

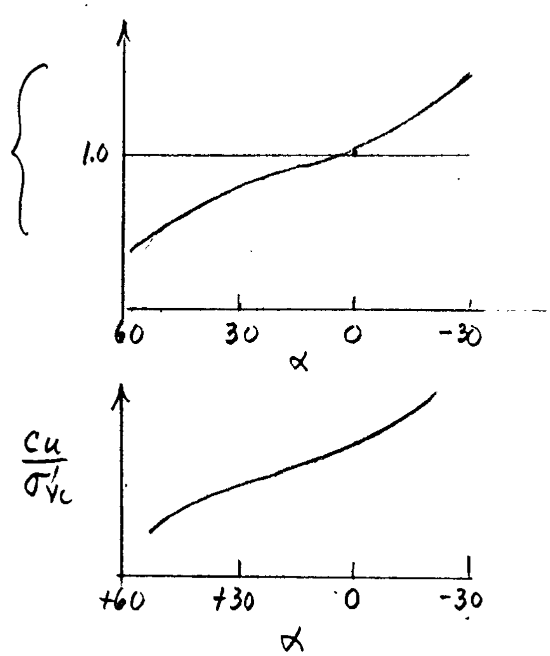
↑ Reasonable range

3) Computed c_u/σ'_{vc} Fig. 21 (Sheet B)



$$\frac{\sigma'_n}{\sigma'_v} = \frac{\sigma'_{fc}}{\sigma'_{vc}} = \frac{1}{1 + \dfrac{\tan\alpha \tan\phi_{cu}}{FS}}$$

Simplified Bishop (Remember 1.361 plot)

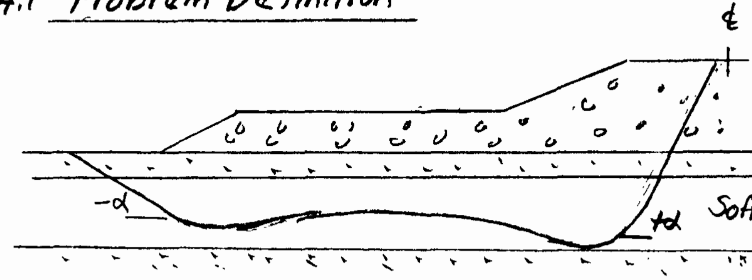


How compare with your understanding of c_u anisotropy?

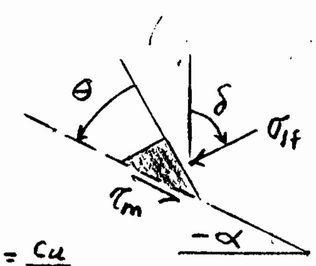
4. NON-CIRCULAR STABILITY ANALYSES WITH ANISOTROPIC UNDRAINED SHEAR STRENGTHS

4.1 Problem Definition

Patterned after UTEXAS 3 analyses of TPS



Soft Clay ($S=100\%$ → "$\phi=0$", $c=c_u$)



$\tau_m = \dfrac{c_u}{FS}$

Require input of $c_u = f(\alpha)$

Two major questions $[\,q_f = 0.5(\sigma_1-\sigma_3)_f\,]$

1) Definition of $c_u = q_f \cos\phi$

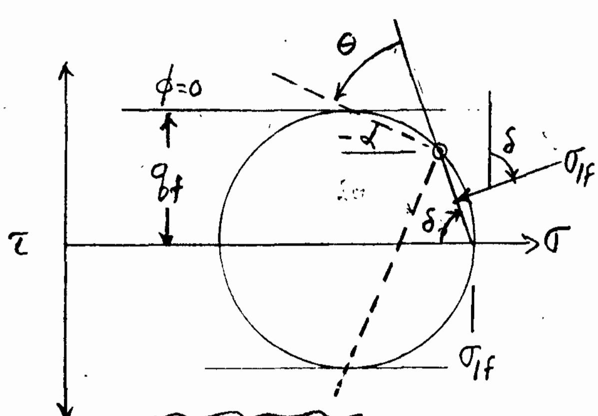
   Use $\phi=0$ or $\phi=\phi'$?

2) Value of θ = angle between failure plane and σ_{1f} plane $= 45+\phi/2$ leading to $\alpha = \theta - \delta$

4.2 Theoretical Relationships

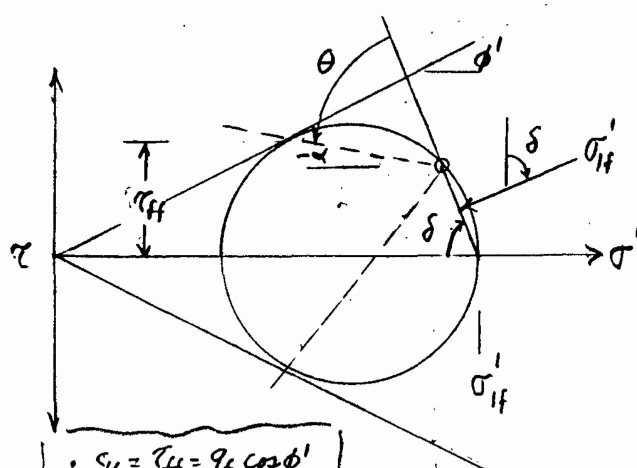
- From CK_0U test like DSC, have known q_f vs. δ ; $\alpha = \theta-\delta$

Using $\phi=0$



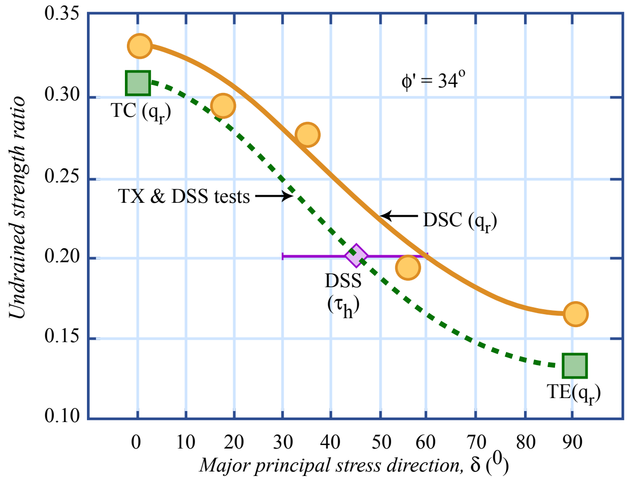
- $c_u = q_f$
- $\theta = 45°$
- $\alpha = 45° - \delta$

Using $\phi=\phi'$



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

4.3 Application to CK_oU Data on Resed. BBC



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.

1) Measured data (Ladd 1994)

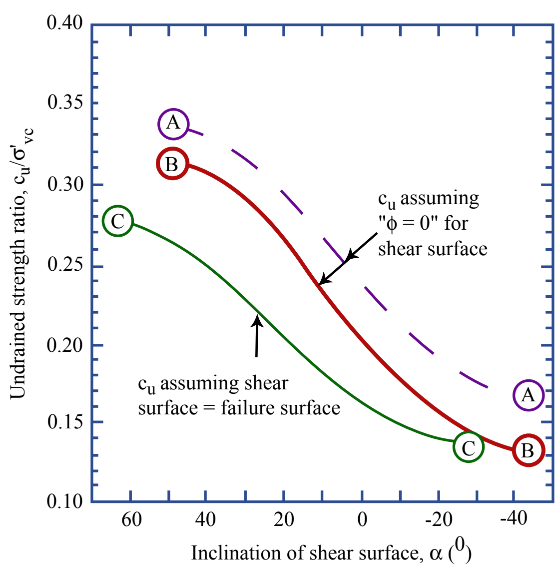
Will interpret using:

$\phi = 0 \rightarrow c_u = q_f$
$\alpha = 45^\circ - \delta$

$\phi' = 34^\circ \rightarrow c_u = 0.83 q_f$
$\alpha = 62^\circ - \delta$

NOTE: 34° = measured θ in NC DSC test

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



Slection of design strengths from experimental data in fig. 3.

Figure by MIT OCW.

2) Interpreted data for use in UTEXAS 3 stability analyses

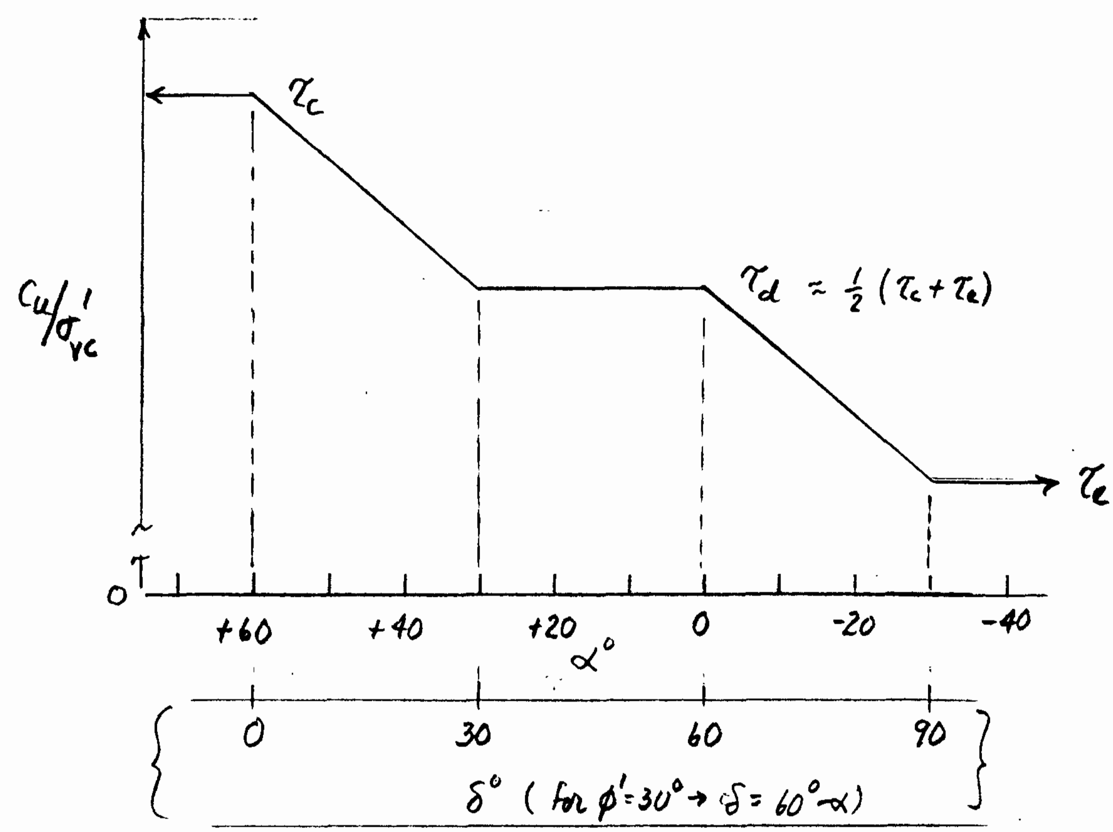
Curve	Data	ϕ	Mean*
A	DSC	$\phi = 0$	0.255 (+38%)
B	TX & DSS	$\phi = 0$	0.225 (+22%)
C	DSC	$\phi = 34^\circ$	0.185

* For $\alpha = +45^\circ$ to -30°

3) Conclusion: If searched failure surface close to actual potential failure surface, then "$\phi = 0$" assumption $\rightarrow c_u = q_f$ & $\alpha = 45^\circ - \delta$ is very UNSAFE (by ≈ 40%) for PS data & ≈ 20% for TX data

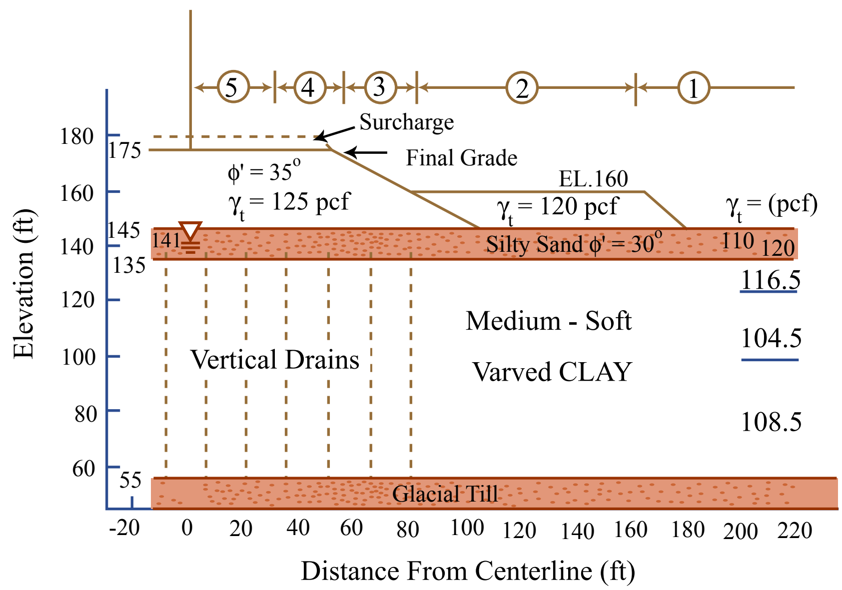
4.4 Simplified Approach

Given uncertainty in how s_u varies with $\alpha = 45 + \phi'/2 - \delta$, especially regarding interpretation of $s_u(DSS)$ (i.e., $\delta = 45 \pm 15°$), CCL has often used the following approach for UTEXAS anisotropic stability analyses for non-varved clays.

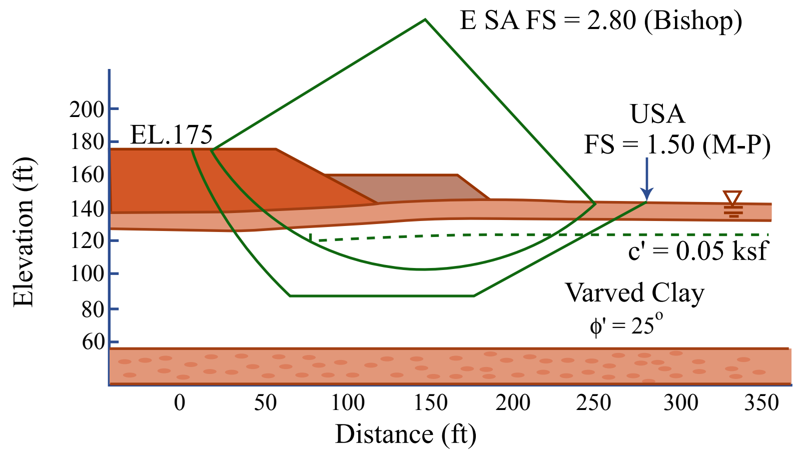


Comments:

1) If have TC, TE & DSS data: adjust TX → PS & apply strain compatibility using $c_u = q \cdot \cos\phi'$
2) If have only DSS data: use Sheet C to estimate τ_c & τ_e
3) If only $s_u = \mu \, s_u(FV)$: set $\tau_d = \mu \, s_u(FV)$ & estimate τ_c & τ_e via Sheet C



Design problem for highway embankment on connecticut valley varved (1 ft = 0.305 m; 1 pcf = 0.157 kN/m³)



ESA and USA factors of safety for embankment on Connecticut valley varved clay at U = 100% [from Ladd and Foott (1977)] (1 ft = 0.35 m; 1 ksf = 47.9 kpa)

Figure by MIT OCW.

CCL 4/30/96 1.322 IIE

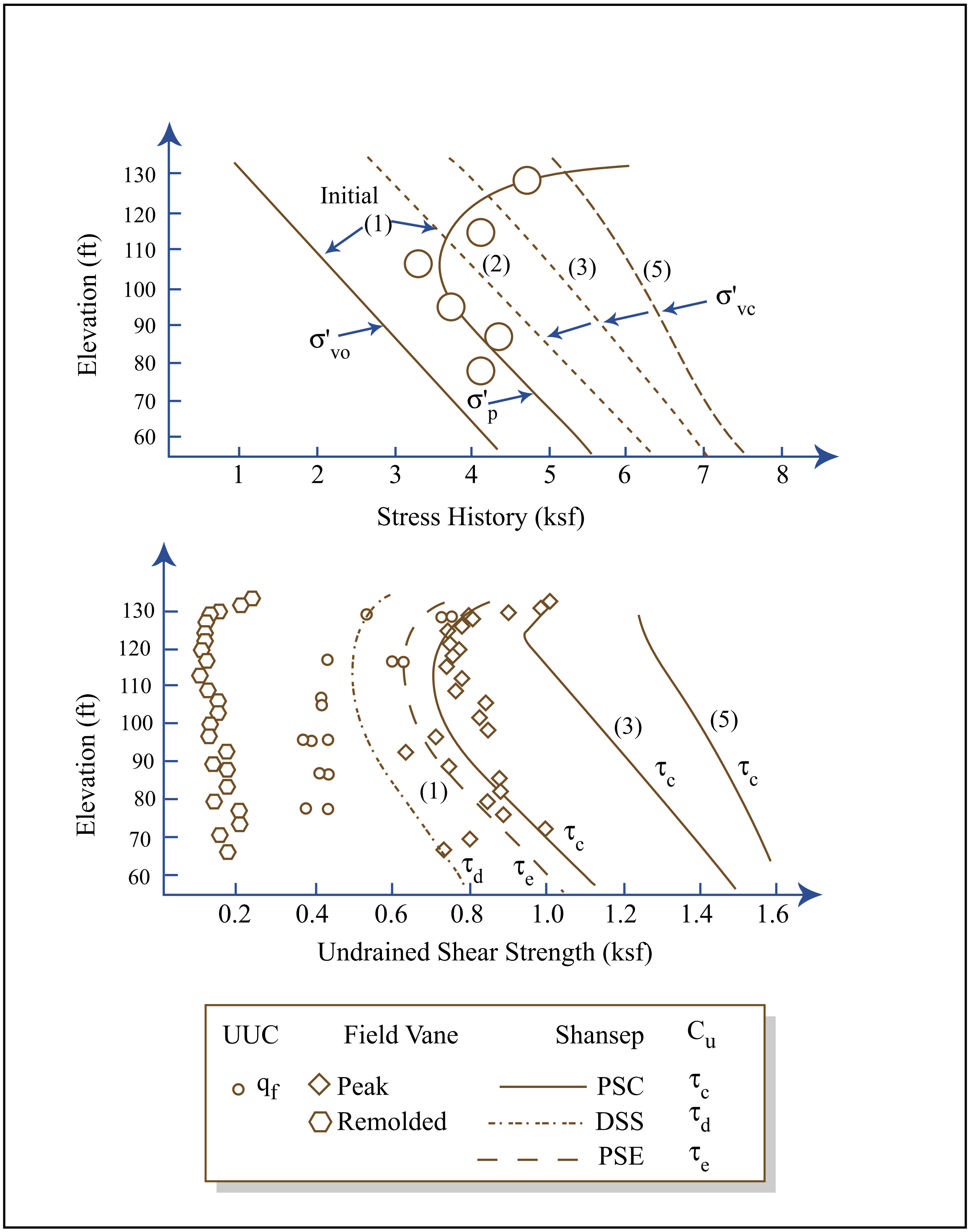
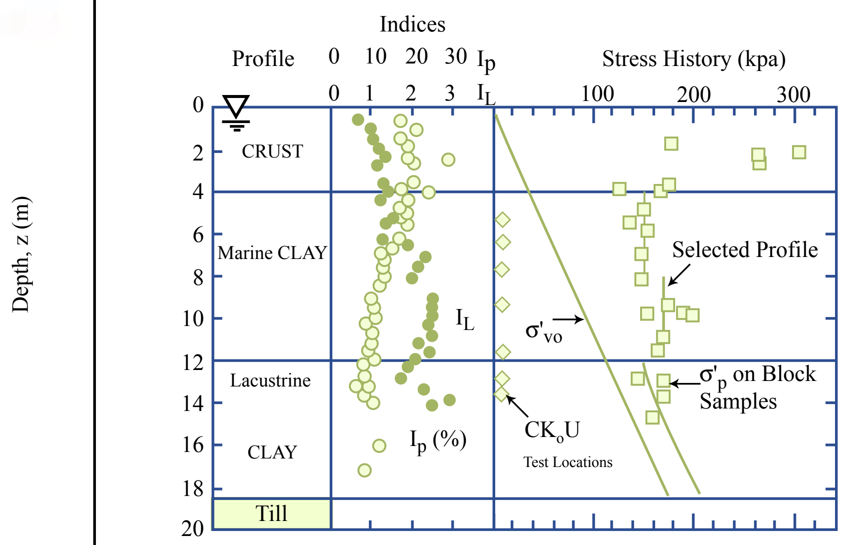
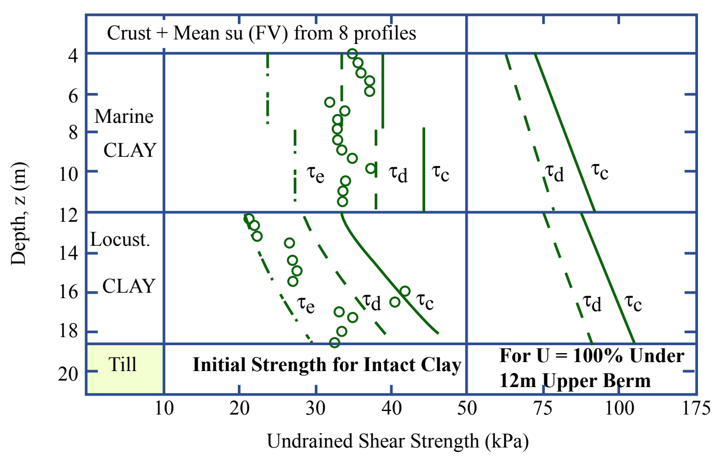


Figure by MIT OCW.

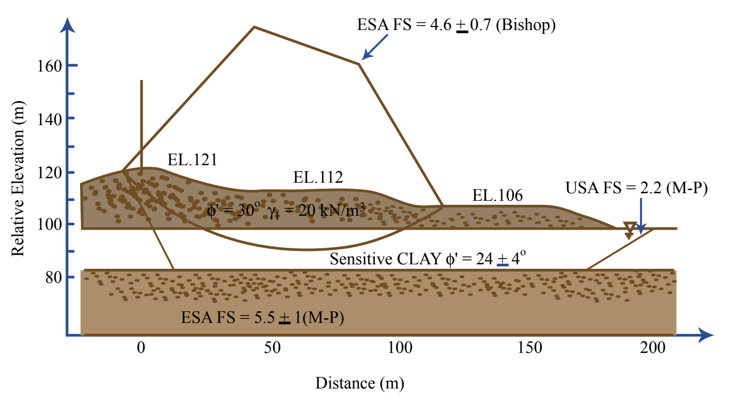
Embankment on Varved Clay: $\bar{U}$ = 100%
(Ladd 1991)



Soil profile, Index properties, and stress history at James Bay Site B-6



Field vane and anisotropic undrained strength profiles at James Bay Site B-6



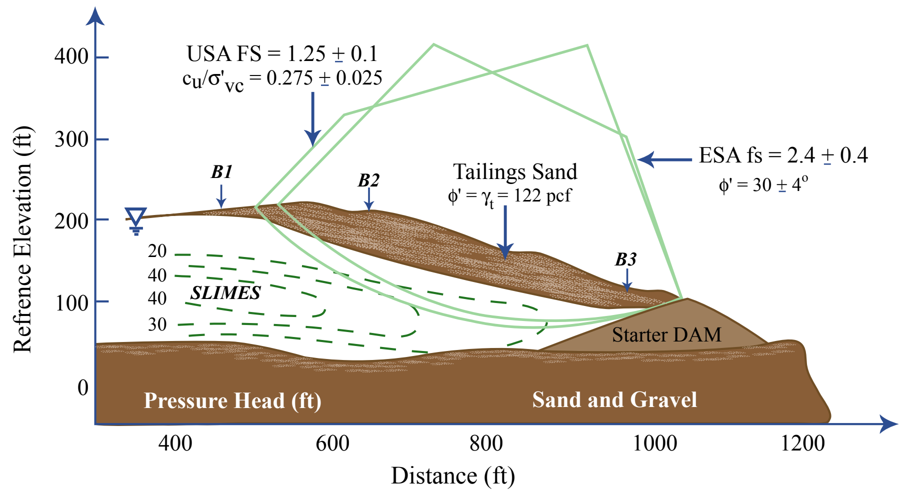
ESA and USA Factors of safety for Embankment Dam on James Bay sensitive clay at Ū= 100%

Note: *For Circular arc from crest to toe of berm, FS = 5.2 ± 0.7*

Figures by MIT OCW.

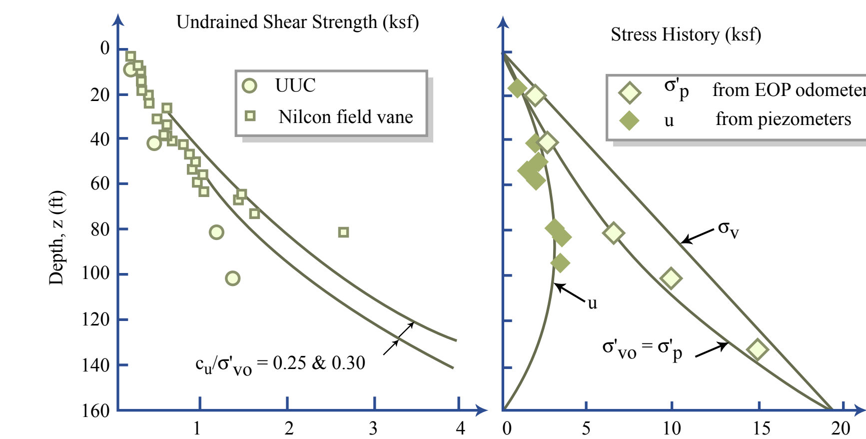
Adapted from: Embankment on James Bay Quick Clay: Ū=100% (Ladd 1991)

CCL 4/30/96 1.322 II G



ESA and USA Factors of safety upstream tailings Dam (1 ft = 0.305 m; 1 pct = 0.157 kN/m^2)

Figure by MIT OCW.



Undrained strength and stress history data at Boring B1 of Upstream fallings Dam (1 ft = 0.305 m; 1 ksf = 47.9 kPa)

Figure by MIT OCW.

Upstream Tailing Dam During Construction

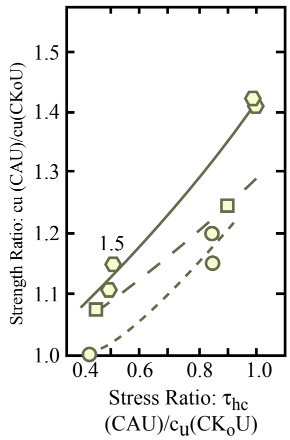
TABLE 2. Undrained Strength Parameters for Connecticut Valley Varved Clay and James Bay Sensitive Clay

Ladd (1991)

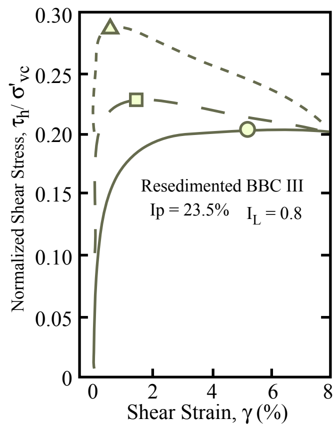
Clay deposit (1)	MODE OF FAILURE: Compression S (2)	m (3)	Direct Simple Shear S (4)	m (5)	Extension S (6)	m (7)
Connecticut Valley	0.21	0.83	0.15	0.775	0.20	0.74
James Bay Marine						
(1) Intact[a]	0.26 ±0.015	1.00	0.225 ±0.02	1.00	0.16 ±0.015	1.00
(2) Normally consolidated	0.26	—	0.225	—	0.16	—
James Bay Lacustrine						
(1) Intact[b]	0.225 ±0.03	1.00	0.19 ±0.00	1.00	0.14 ±0.01	1.00
(2) Normally consolidated	0.25[c]	—	0.215	—	0.12[c]	—

[a]Mean ± one standard deviation from five test series.
[b]Mean ± one standard deviation from two test series.
[c]Estimated from data on other clays.

A3



Soil		CK_oU c_u/σ'_{vc}	Sym
BBC	I	0.201	⬡
	III	0.203	⬡
Toilings		0.223	□
EABPL		0.235	○



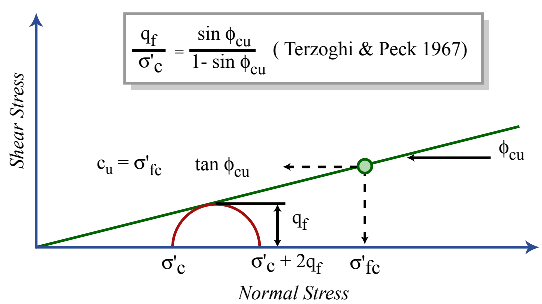
τ_{hc}/σ'_{vc}	Symbol
0	-○-
0.10	-□
0.20	-△-

Effect of consolidation shear stress on undrained direct simple shear behavior of normally consolidated clay: (a) Increases in peak strength for Boston blue clay, Copper tailing and Atchafalaya clay; and (b) Shear stress versus shear strain for Boston blue clay.

Note: Copper Tailing data by R.S. Ladd; Other data by MIT

CAUDSS Data
NC Clays

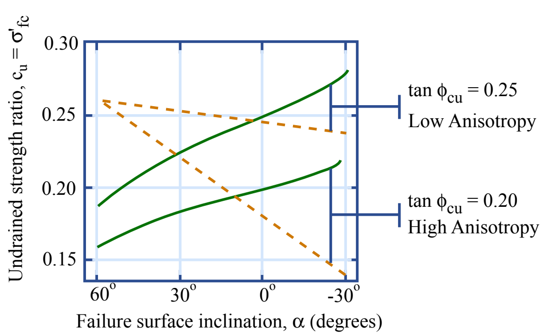
$$\frac{q_f}{\sigma'_c} = \frac{\sin\phi_{cu}}{1-\sin\phi_{cu}} \quad \text{(Terzoghi \& Peck 1967)}$$



Angle of shearing resistance ϕ'_{fc} from isotropically consolidated - undrained triaxial compression (CIUC) tests as defined by A. Casagrande

QRS
Methodology

Definition of ϕ_{cu}



Undrained strength ratios from QRS methodology compared to trends from CK_OU *testing for normally consolidated clay.*

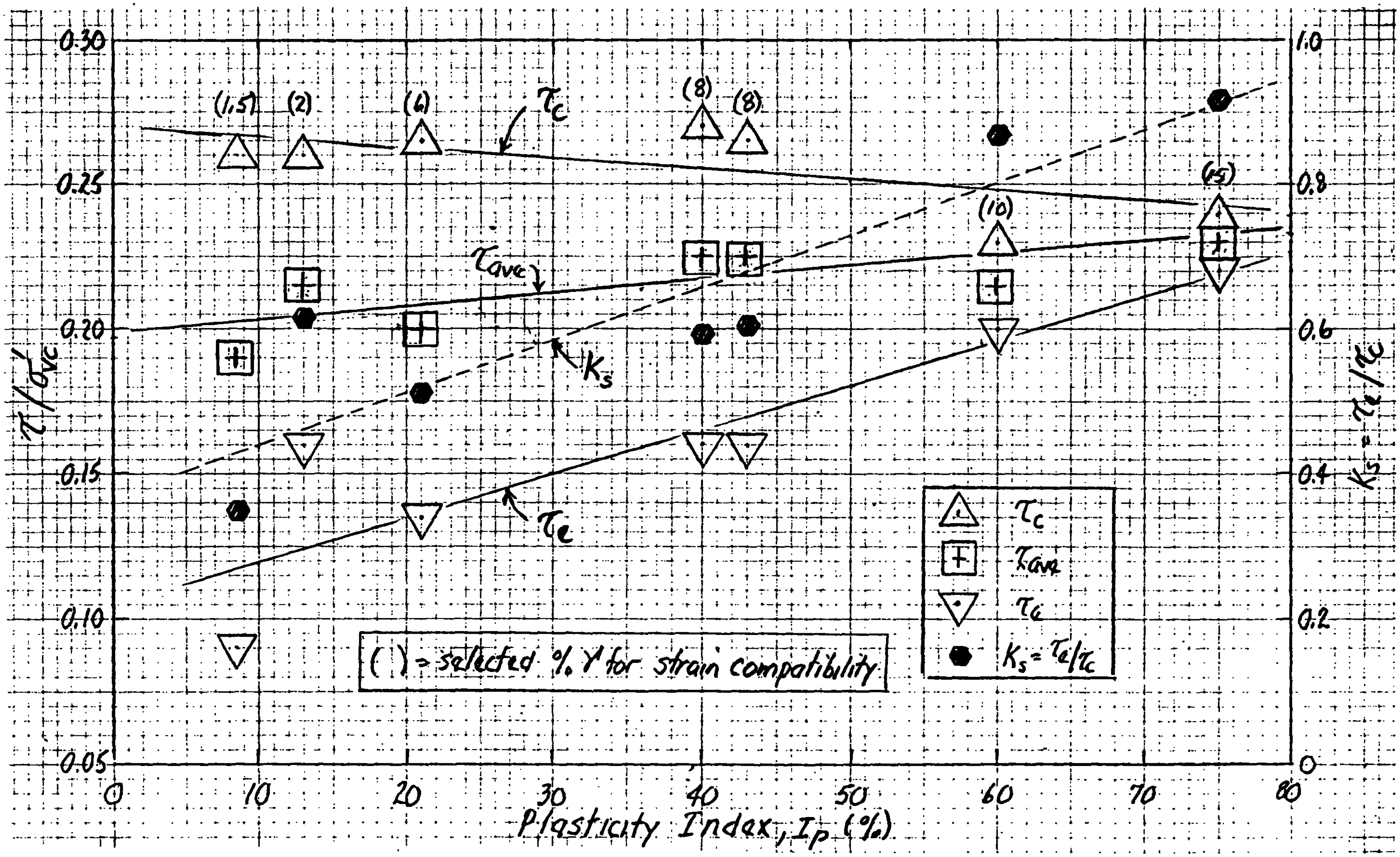
Computed c_u/σ'_{vc} and
· via QRS
· Actual trends

(Ladd 1991)

B

Figure by MIT OCW.

CCL 8/10/90 4/27/97 4/98 4/01



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

C

IIF SUMMARY: ESTIMATION OF s_u FOR UNDRAINED STRENGTH ANALYSES (USA)

IIF SUMMARY: ESTIMATION OF s_u FOR UNDRAINED STRENGTH ANALYSES (USA)

Note: TL = 22nd Terzaghi Lecture

1. INITIAL STABILITY (UU Case)

1.1 In Situ Tests (NOTE: FVT, CPTU & DMT also useful for stress history profiling)

1) FVT + Bjerrum μ vs I_p: Typical COV ≈ 20 → 10% for PI = 20 → 100% for sat. sedimentary cohesive soils without shells, sand lenses, fibers, etc.

* 2) CPT & CPTU with N_K = 15±5: Smaller data base suggest COV ≈ 35% for medium-soft clays. Some evidence of much larger N_K for stiff clays (e.g. N_K → 50 for Smith Bay)

4) Menard Pressuremeter: too empirical & costly

5) SBPT: not much better & far more costly (+ derived s_u very unsafe)

* 3) DMT ("std" application uses S = 0.22 & m = 0.8, but can be altered):

- Lacks extensive data base on variety of soil types (Note that empirical correlation is with OCR, not s_u)
- Growing popularity, e.g. ISOPT-1 (1988)

* NOTE: CPTU & DMT both also applicable to granular soils & good-excellent for soil profiling (stratification)

1.2 Lab "UU" Tests

1) TV, LV, PP, FC ...: Serve as "s_u index" tests, but recommended due to simplicity & low cost

2) UUC: s_u value depends on 3 compensating errors:

- Incr. s_u from δ = 0 & fast $\dot{\varepsilon}$ vs decr. s_u due disturbance
- Net error can easily be ± 25-50%

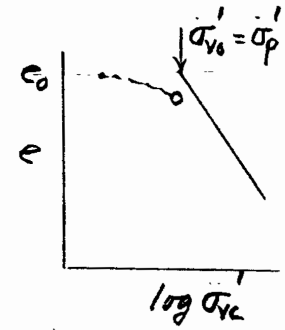
1.3 Lab C"K_o"U Tests

NOTE: CIUC only applicable as part of Recompression test program when in situ OCR ≈ 4 ($K_o \approx 1$) and should not be used as sole CU test type since → UNSAFE s_u (e.g. neglects s_u anisotropy, plus $K_c = 1$ falls below K_o compression curve)

(Also see QRS in 2.3)

1) Recompression ($\sigma'_{vc} = \sigma'_{vo}$)

- Preferred technique
  - When have block samples
  - Highly "structured" soils (high S_t & I_L)
  - Very high OCR
- Unsafe results when in situ OCR ≈ 1



- Need variation in s_u index and/or SH to know when to run tests and for interpolation / extrapolation of "point" data
- Need OCR to check if s_u/σ'_{vo} is reasonable

2) SHANSEP

- Requires well defined SH and more testing → USR vs OCR, but can use NSP on area wide basis, plus subsequent jobs.
- Preferred technique for tube samples of "ordinary" clays and must be used when OCR ≈ 1
- Probably → underestimate of s_u/σ'_p for highly structured soils (and E_u much too low)
- Underestimates of stiffness of OC clay, esp. in extension
- Automated CK_o-TX & DSS → excellent 1-D compression curves for values of σ'_p, CR & K_o (for TX) → very cost effective

3) <u>BOTH Recompression & SHANSEP</u>

a) Have empirical component regarding "time effects", e.g., assume using $\dot{\varepsilon} \approx 0.5$-$1\%/hr$ for TX & $\dot{\gamma} \approx 5\%/hr$ for DSS → reasonable values compared to in situ shearing rates

b) Explicitly consider effects of s_u anisotropy and can evaluate effects of "progressive failure" via strain compatibility technique

(1) PS testing → complete data à la DSC (future?)

(2) PSC/E + DSS (few PS devices)

(3) TC/E + DSS

- Can use τ_{ave} or τ_c, τ_d & τ_e
- See TL Table 4 & Fig. 18 for results that agree quite well with collective data from case histories
- See IIE Sheet C for anisotropy vs PI

(4) DSS
- Less soil & easier to run than TC/E
- Geomor preferred
- See TL Table 4 & Fig. 18

(5) TC/E (see p3a) → $s_u = \frac{1}{2}[q_f(C) + q_f(E)]$ → use for $\phi = 0$ bearing cap. / MOS w/o 3-D correction

c) Should always be accompanied by detailed evaluation of SH (σ'_{vo} & σ'_p → OCR)

- Oed.-CRSC testing ESSENTIAL
- Use in situ testing to help assess spatial variability, e.g. FVT, CPTU, DMT
- Evaluate s_u data via $\log s_u/\sigma'_{vo}$ vs $\log OCR$ → S & m

INSERT: Discussion of Use of CK_oU TC & TE Data

Sources of Compensating Errors

(1) TX vs PS q_f: $\left.\begin{array}{l} TC/PSC = 0.92 \pm 0.05 \\ TE/PSE = 0.82 \pm 0.02 \end{array}\right\} \approx 0.87$

(2) Strain compatibility: $\dfrac{q_f(\text{Ave}) \text{ at design } \gamma}{q_f(\text{Ave}) \text{ of peaks}} \approx 0.90$ NOTE: Assumes that s_u of crust (high OCR) will be reduced to design γ selected for "soft" clay

(3) Shear stress on failure surface: $\tau_f / q_f = \cos\phi' \approx 0.88$ for $\phi' = 25\text{-}30°$

(4) "Slope" stability, "end-effects": $\dfrac{FS(3D)}{FS(2D)} = 1.11 \pm 0.06$ SD

Stability Evaluations Using $c = 0.5 \times$ Peak $[q_f(C) + q_f(E)]$ from CK_oU TX *

(a) Bearing capacity, UU Case, $S = 100\%$

- $\phi = 0$, $c = q_f = 0.5(\sigma_1 - \sigma_3)_f$
- (1) & (2) compensate, i.e. $\times \frac{1}{0.87} \times 0.90 = 1.03^5 \approx 1.0$ ∴ OK to use

(b) Slope stability analyses with method of slices assuming that predicted location of critical shear surface ≈ actual failure location

- Although $\phi = 0$, $c = \tau_f = q_f \cos\phi'$
- For true plane strain failure: Σ (1), (2) & (3) → $\times \frac{1}{0.87} \times 0.90 \times 0.88 = 0.91 \approx 0.9$ [i.e., FS(2D)] ∴ unsafe by ≈ 10%
- For typical failures, Incl. (4) → $0.91 \times 1.11 = 1.01 \approx 1.0$ ∴ OK to use without correction for "end effects"

* For $S = 100\%$ and approximately linear $q_f(S)$ vs. S relationship

1.4 For ALL Approaches

1) Check measured/computed s_u from 1.1, 1.2 and/or 1.3 using $s_u/\sigma'_{v0} = S(OCR)^m$, which obviously requires some knowledge of in situ STRESS HISTORY.

> NOTE: CCL view that good oedometer test & AL single best approach for estimating s_u via Level C prediction

2) Plane strain failure $L \to \infty$ (TL Table 4 & Fig. 18)

- Sensitive marine clays ($I_p < 30\%$, $I_L > 1$) — $S_p = s_u/\sigma'_p = 0.20 \pm 0.015$ SD

Above A-line
- CL & CH sed. clays, low-moderate S_t ($I_p = 20$–80%) — $S = 0.215 \pm 0.015$ ($S = 0.20 + 0.05 I_p$)

NOTE: Varved $S \approx 0.16$ (N.E. US)

Below A-line
- Sedimentary silts & organic soils + clays w/ shells — $S = 0.25 \pm 0.05$

3) Typical "end effects" à la Azzouz et al. (1983)

$F(3\text{-}D)/F(2\text{-}D) \simeq 1.1 \pm 0.06 \rightarrow S \simeq 0.23^5 \pm 0.02$

CL & CH low-moderate S_t

(Compares well with Larsson (1980) case histories non-layered low OCR clays, $I_p < 60\%$ $s_u/\sigma'_p = 0.23 \pm 0.04$)

4) Value of m

- Mechanically OC $m \approx 0.88(1 - C_s/C_c) \pm 0.06$
  or simply $m = 0.8 \pm 0.1$

- Cemented, high S_t $m \approx 1 \rightarrow S_p(3\text{-}D) = 1.1 \times 0.20 = 0.22$

(Mesri, 1989, CGJ 26(1), 162-164: $s_u = 0.22\,\sigma'_p$)

2. STAGED CONSTRUCTION (CU Case)

Includes "long term" loadings

2.1 General

1) TL treats in detail + Section IIE

2) Stress history most important design parameter

- Controls initial su
- Generally small Δs_u until $\sigma'_{vc} > \sigma'_p$
- Combinated lab. oed.-CRSC + in situ for spatial variations (and/or auto. CK_o-TX & DSS)

2.2 Recompression vs SHANSEP

- See 1.3, but since will have some NC foundation soil, must run some CK_oU tests with $\sigma'_{vc} >> \sigma'_p$

2.3 QRS Approach

- TL Section 6
- IIE, Section 3

Empirical approach that depends on compensating errors

2.4 Non-Circular Anisotropic Analyses

- IIE, Section 4