CCL 4/2/96 | 1.322

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) 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 — IC

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 correlations — 8,10,11

C.2 Highly Structured, Sensitive Clays (Quick Clays)
- 1) Background — 12
- 2) Norway — 12
- 3) Quebec — 12,13

C.3 Conn. Valley Varved Clays — 14

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
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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 <u>homogeneous</u> cohesive soils
I C Problem Soils	4/4/01	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} = 50-400kPa)? 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 u & peak ESE safe?
II A su 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 $s_u/\sigma'_{vo} = 0.22(OCR)^{0.8}$ where $OCR = (0.5K_D)^{1.56}$? 4) When would you replace unconf. compression test with a UUC test ($\sigma_c = \sigma_{vo}$)? 5) Did Bishop & Bjerrum (1960) conclude that both UUC & s_u(FV) → reliable s_u for UU Case?

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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 V3-1&2	(2+)
2) CD Case · Measurement c' & φ' · Problem Solns	IB IC					
3) UU Case: Std Practice & In Situ Testing	IIA	4.2	3.2, 3.3	4, 6	1.361 I4-1&2	(1-)
4) Sample Disturbance	II B	2.2.7	2.3	④	above	(2)
5) Stress System & Anisotropy (Combined old & new notes)	II C	2.2.2	2.4	④	–	(4±) Discuss HP#4 Q&A MTE 4/13
6) Time Effects: Strain Rate & Creep	II D	2.2.6	-	-	-	(2+)
7) CU Case	II E	–	–	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

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4/97
3/98
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Strength-Deformation Behavior of Saturated Clays and Drained/Undrained Stability (Parts D & E of Outline)

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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

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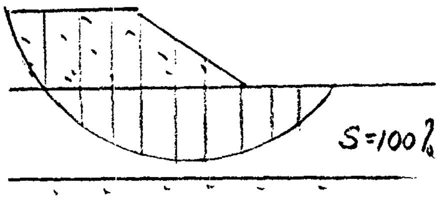
22-144 200 SHEETS
AMPAD

3/30/92 4/9/95 4/2/96

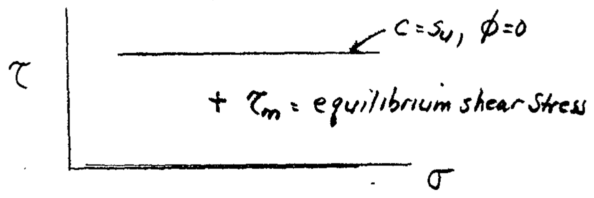
IA: CLASSES OF STABILITY PROBLEMS AND TYPES OF STABILITY ANALYSES (Sheet A2; Table 6)

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 → +Δp)

- Footings, tanks, emb, dams ⋯
- UU critical since $+u_e$
  & drainage → incr. str.
  (esp. low OCR)

Unloading (Construction → −Δp)

- Excavations, etc
- CD critical since $-u_e$
  & drainage → decr. str.
  (esp. high OCR with $-u_s$)

42 381 50 SHEETS 5 SQUARE
42 382 100 SHEETS 5 SQUARE
42 389 200 SHEETS 5 SQUARE
NATIONAL

42 381 50 SHEETS 5 SQUARE
42 382 100 SHEETS 5 SQUARE
42 389 200 SHEETS 5 SQUARE
NATIONAL

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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 ($u_e = 0$)	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.

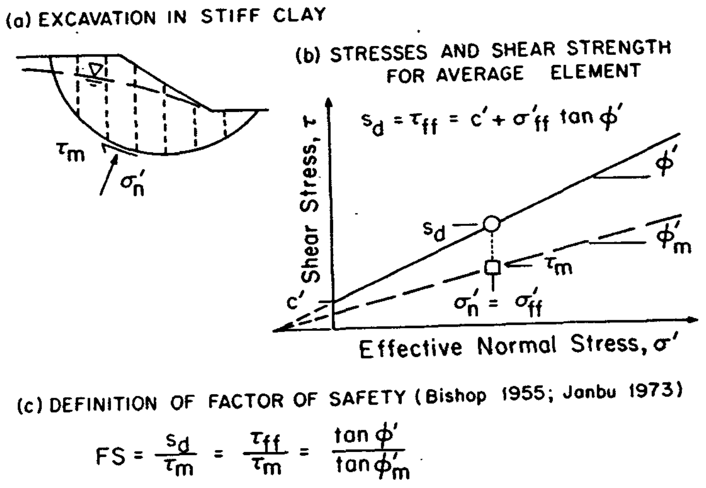


FIG. 1. Conventional Effective Stress Analysis Applied to Critical CD Case for Unloading Problem

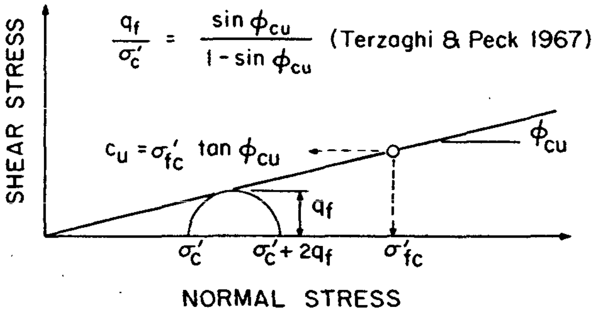


FIG. 2. Angle of Shearing Resistance ϕ_{cu} from Isotropically Consolidated-Undrained Triaxial Compression (CIUC) Tests as Defined by A. Casagrande

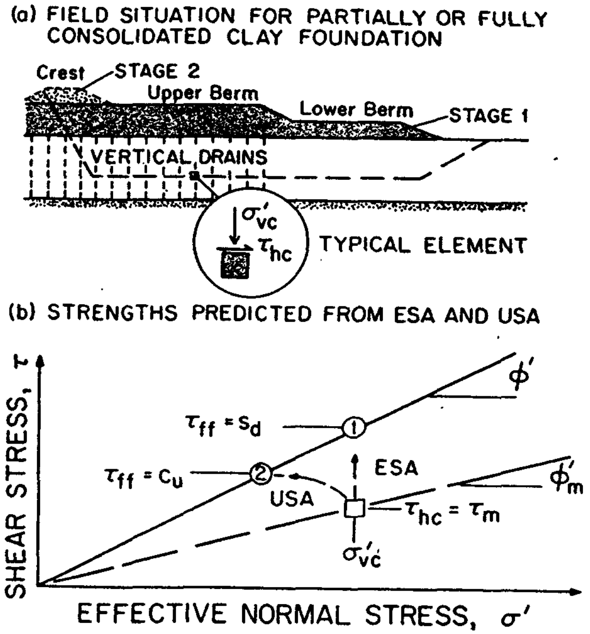


FIG. 3. Comparison of Effective Stress and Undrained Strength Analyses for Evaluating Stability during Staged Construction

IA2

IB DETERMINATION OF EFFECTIVE STRESS FAILURE ENVELOPE FOR CD CASE (Saturated Natural Cohesive Soils)

1. USE OF CD & CU TESTS

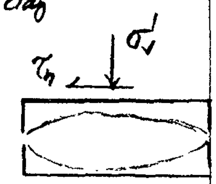
Note: Limited data on "ordinary" clays indicates that t_f has little effect on values of c' & ϕ' (i.e., using $t_f > t_{100}$ required obtain $u_s = 0$)

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

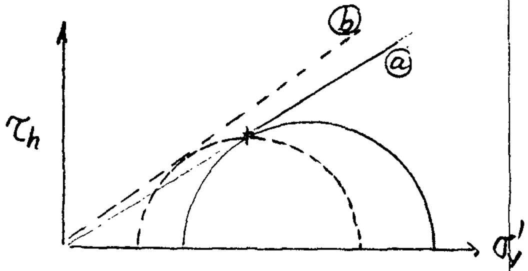
2) Unknown stress condition at failure

ⓐ $\tau_h = \tau_{ff}$; $\delta = 45 + \phi'/2$; $\tau_h/\sigma'_v = \tan\phi'$

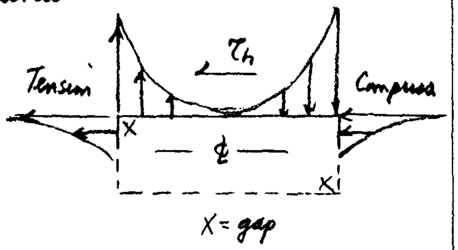
ⓑ $\tau_h = |q_f|$; $\delta = 45°$; $\tau_h/\sigma'_v = \sin\phi'$

$\tau_h/\sigma'_v = 0.5 \rightarrow \phi' = 26.6°$ ⓐ ← Std. practice

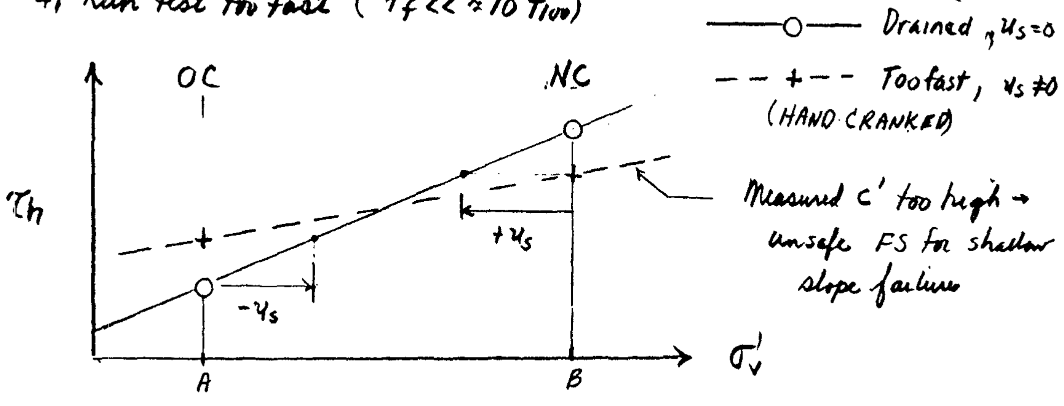
$= 30.0°$ ⓑ



3) Tilting at high τ_h/σ'_v
- Tensile σ at leading portion
· Compressive σ at trailing portion



4) Run test too fast ($t_f << \approx 10\, t_{100}$)



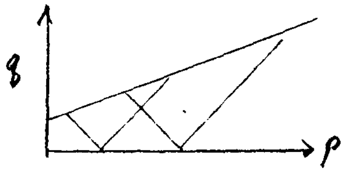
Measured c' too high → unsafe FS for shallow slope failures

1.2 CD Triaxial (Usually CIDC L/U since K_c does not affect c' & ϕ')

a) Advantages

- Know stress conditions and meaningfull stress-strain data
- Can vary ESP to define ESE at low stresses

∴ Most reliable



b) Disadvantages

- More complex equipment & harder to run
- Much longer time (1-2 weeks) & more expensive

1.3 CU Triaxial (Usually CIUC)

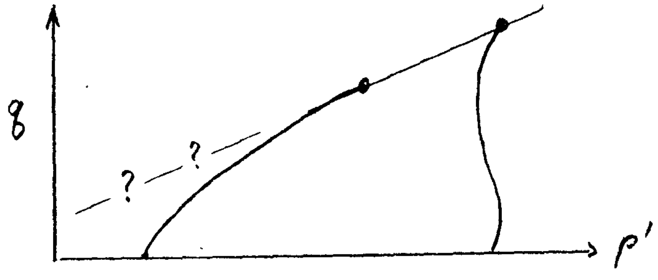
a) Advantages

- 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



(plus have curved ESE, see p IB6)

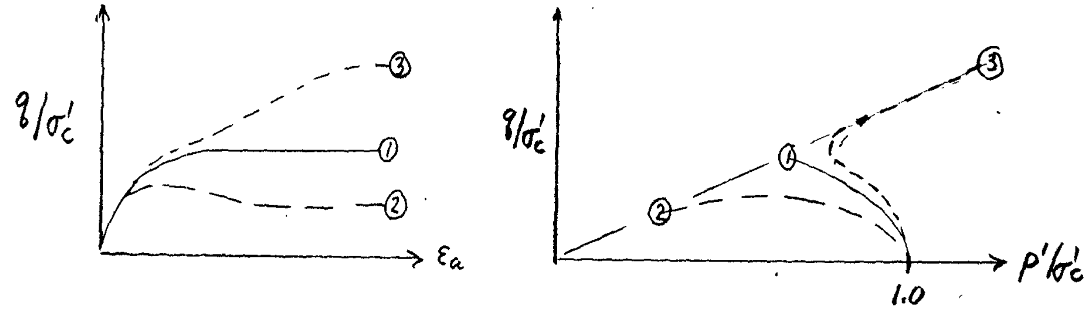
3) Varying ESE at q_f, max obl. & tangency (See 2.1 & 2.2)
Note: generally use max obl. or tangency to estimate c' & ϕ' for CD case

4) Potential large unsafe error in c' if do not obtain pore pressure equalization (See 2.3)

2. MISCELLANEOUS

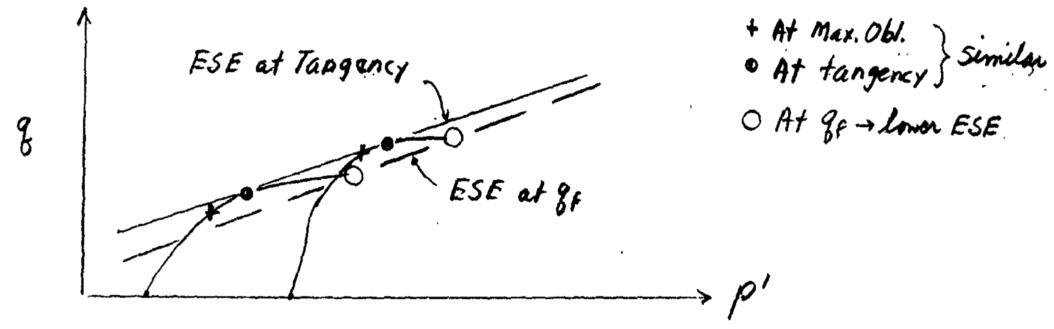
2.1 Variation in ESE : OCR=1 (CIUC Tests)

① Simple clay type behavior : $\phi'_{qf} = \phi'_{mo}$ ② Sensitive clay ; $\phi'_{qf} < \phi_{mo}$ ③ Arctic silt : not really NC
Cohesionless (small +ψ)



- Value of $q = f$(mobilized ϕ') × (magnit. of p') ; High S_t, $\phi'_{qf} < \phi'_{mo}$ 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)

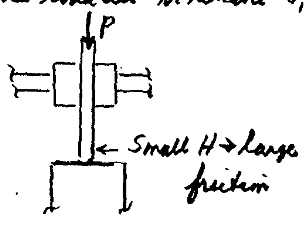


- Usually select ESE at Max. Obl. or tangency to estimate ESE for CD Case. But extrapolated envelope is TOO HIGH at low p'_f (See IB6)

2.3 Common Triaxial Testing Problems (Germaine & Ladd, 1988 ASTM STP 977) + 1.37

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

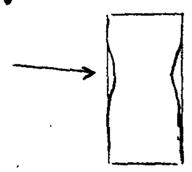


b) Filter Strips (CU & CD)

- Compression : 10cm² x 8cm Typical correction $\Delta q \approx 100$ psf = 5kPa
- Extension : Need spiral + precut notches → maximum flexibility

c) Area Correction (CU & CD)

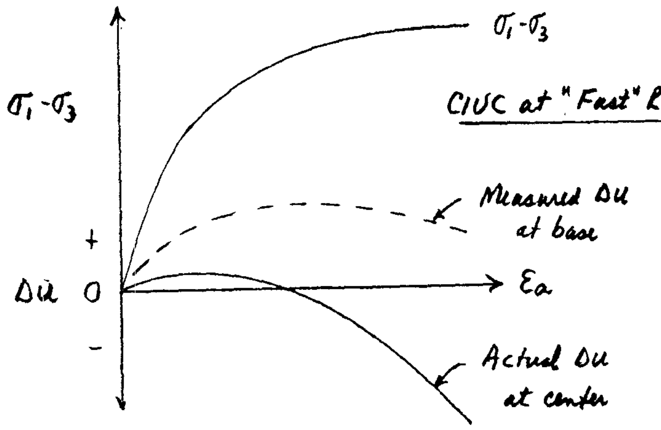
- Compression : See G&L (88) for cylinder, parabolic & bulging
- Extension : Discount data when noticeable necking occurs →

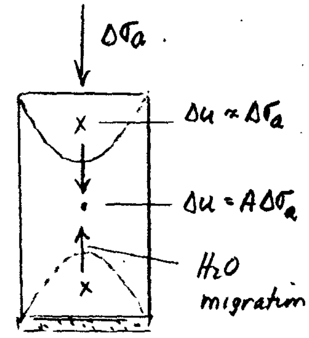


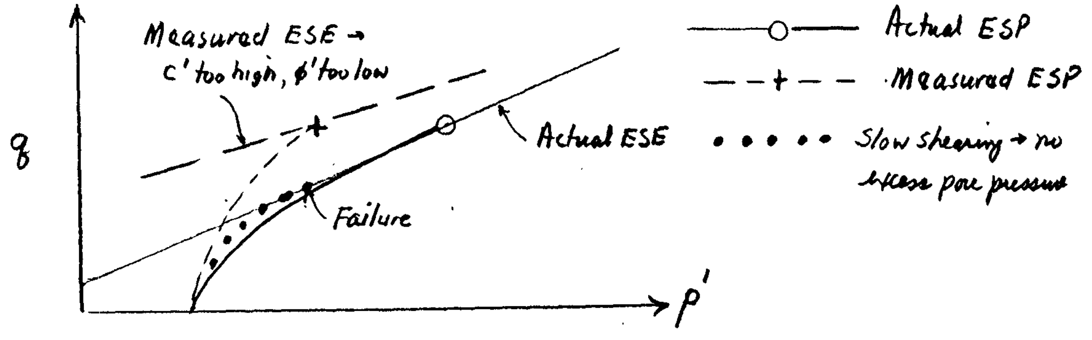
d) Saturation (CU)

- Need min. $u_b \approx$ 2-3 atm · Always check that $B \geq \approx 95\%$ few minutes

e) Frictional Ends - Pore Pressure Equalization at "High" OCR (CIUC)







Correct ESE requires : 1) Either u measured at ℄ if fast (+ correct s_u)
(with frictional ends) 2) Or very slow if u measured at base (s_u too low)

2.4 Comparison of ESE and Correlations

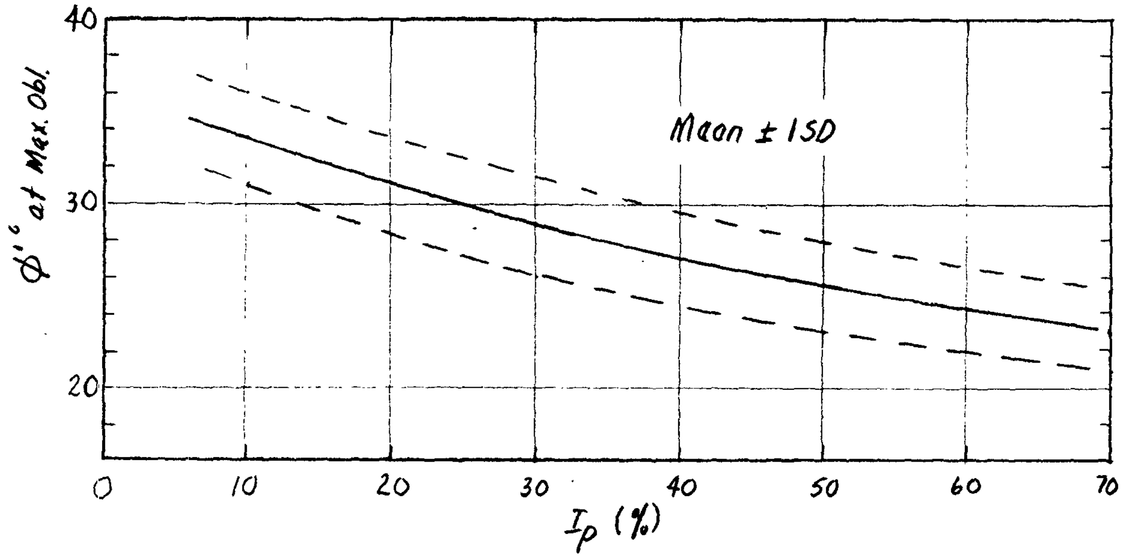
a) Natural BBC : CK_0UC/E ; OCR = 1.5-6 ; Max. Obl. (CA/T Project)

	SHANSEP		Recompression	
	c'/σ'_{vm}	ϕ'	c'/σ'_p	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) NAVDOCKS DM-7 (1961)



2) Mesri & Abdel-Ghaffar (1993) JGE, ASCE, 119(8), 1516-1249

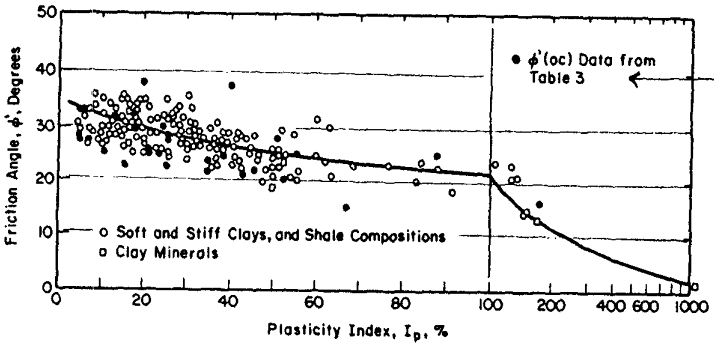


FIG. 2. Values of Friction Angle φ' for Natural Clay Compositions

φ' for OC clay

Data mostly from:
CIUC
CIDC
CDDS

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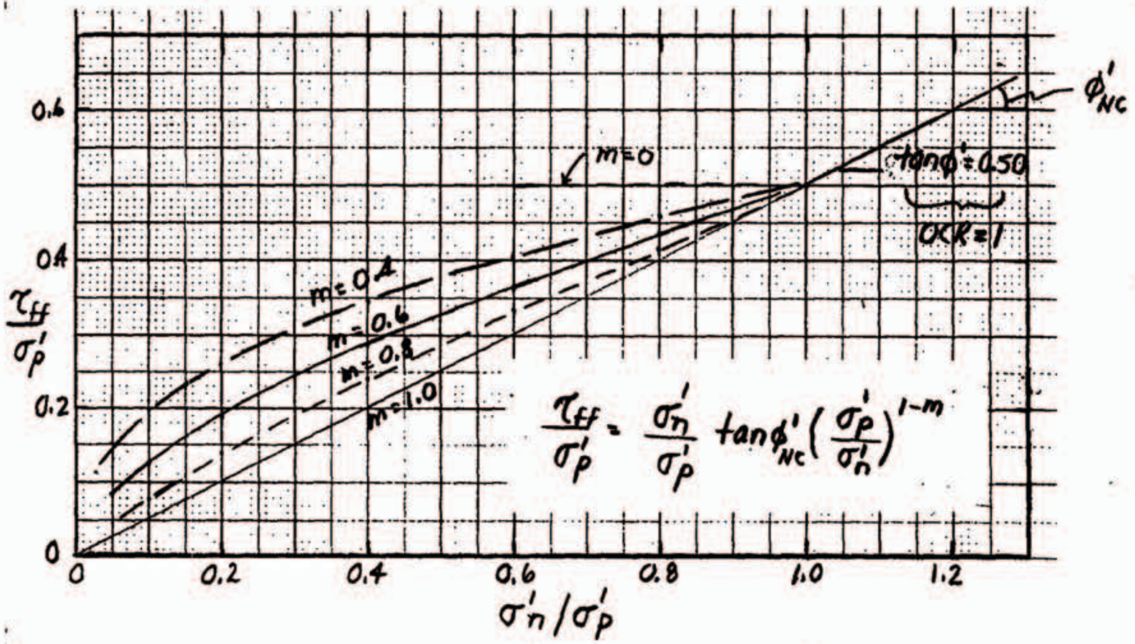
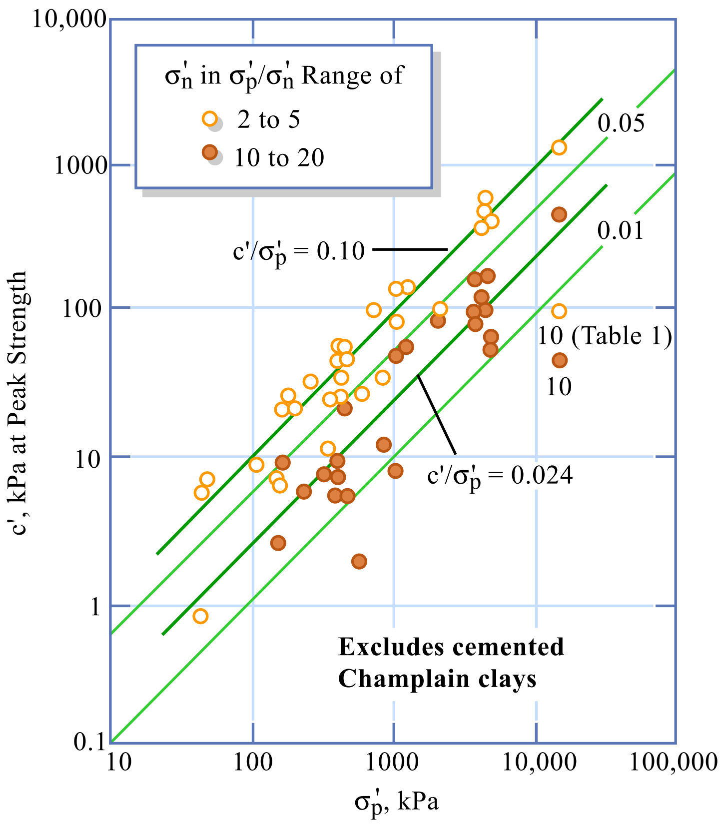
c) Magnitude of c'/σ'_p from Mesri & Abdel-Ghaffar (1993)



- 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 $I_p \approx 20-80\%$



Relationship between Cohesion Intercept and Preconsolidation Pressure

Figure by MIT OCW.

CCL Conclusion For Mechanically OC Clays:

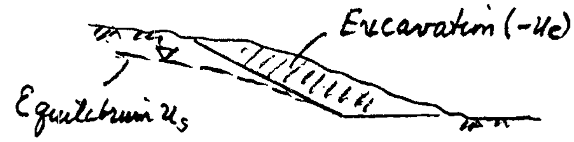
	σ'_n/σ'_p	c'/σ'_p
○	0.2-0.5	0.05-0.1
●	0.05-0.1	0.03±0.02

LONG TERM (CD) STABILITY : PROBLEM SOILS

C1 STIFF FISSURED & STRATIFIED CLAYS AND CLAY SHALES

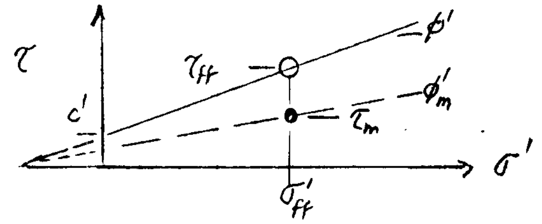
1.1 Introduction

1) Critical condition



$s_d = \tau_{ff} = c' + (\sigma - u_s)\tan\phi'$

$FS = \frac{s_d}{\tau_m} = \frac{\tan\phi'}{\tan\phi'_m}$



2) Values of c' & ϕ' to use in analysis depend on:

a) 1st time vs prior (reactivated) slide

b) Homogeneous (also probably 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

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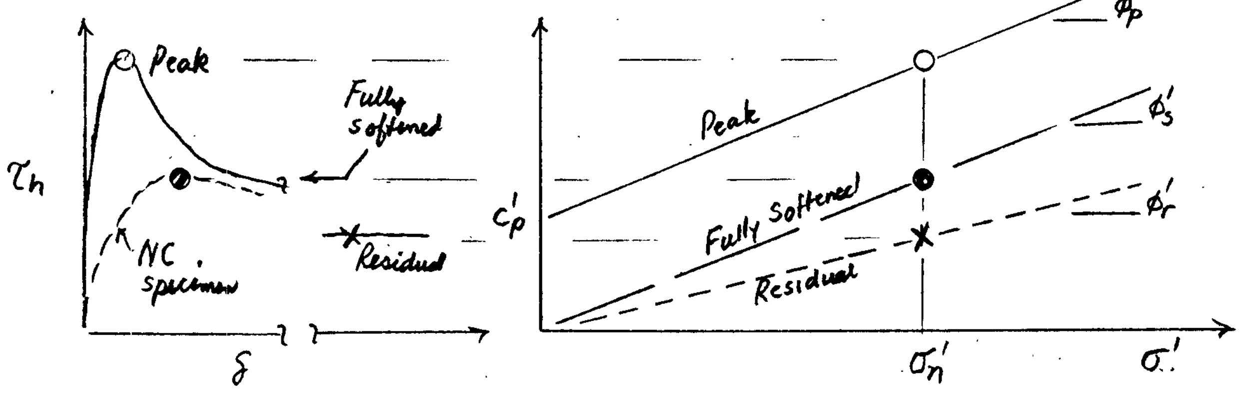
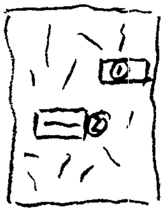
1.2 Definition of 3 Envelopes (e.g. Skempton 1964, Geot. 14(1), 77-101)

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 vs inclined 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. [illegible]
- 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 specimens
- Skempton also postulated that have "minor shears not yet linked into a continuous surface" at fully softened state

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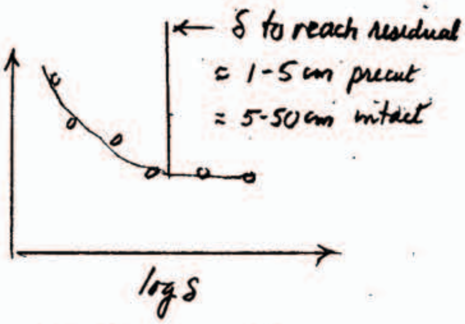
1.3 Measurement of Residual Envelope

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)

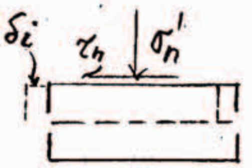


2) Testing methods

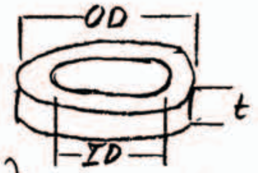
a) Repeated direct shear, i.e., shear to δ_i, push back, shear again, etc.

- Either precut natural (to greatly reduce δ)
- Or remold & consolidate on plate ([illegible])

} Both used



b) Rotational shear = ring shear



OD	ID	t (cm)	
7.1	5.1	0.4 t	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)

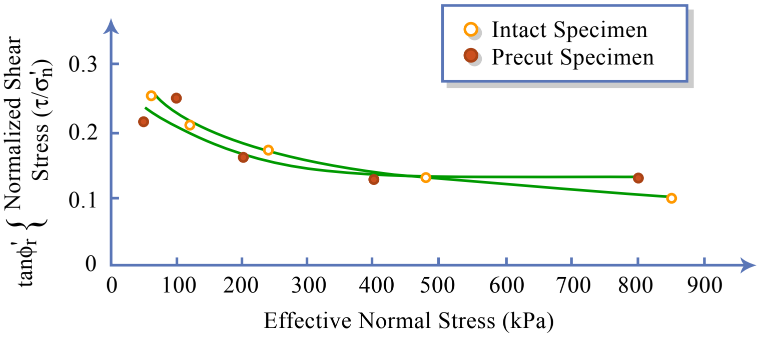
} Best procedure → minimum envelope.

Typical $d\delta/dt = \dot{\delta} \approx 1$ 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)

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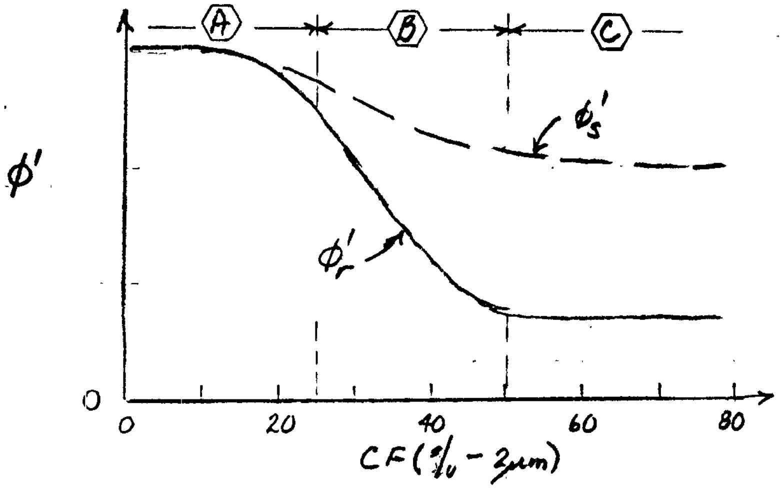
ALVIN

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 →

- 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(3)] & 1977 [9th ICSMFE, Vol 3]

1) Along PRIOR FAILURE SURFACE having movements of ≈ 1-2 m

<u>Must use ϕ_r' independent of age of prior failure</u>

- CD DS tests on block sample from 10k 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 → $\tau_m \approx \frac{1}{2}(\tau_s + \tau_r)$ for LL > 60%, i.e. half way between fully softened & residual)

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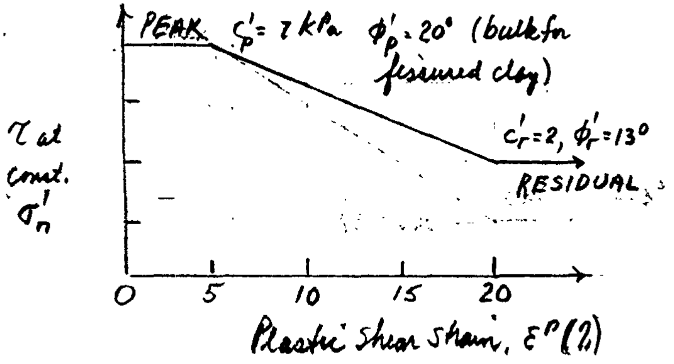
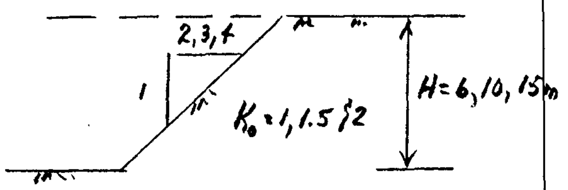
1.6 Results of Research by Potts et al. (1997) "Delayed collapse of cut slopes in stiff clay" Geot 47(5), 953-982

1) Conducted coupled FE analyses (i.e., included k) of cut slopes in a strain softening clay (patterned after London clay)



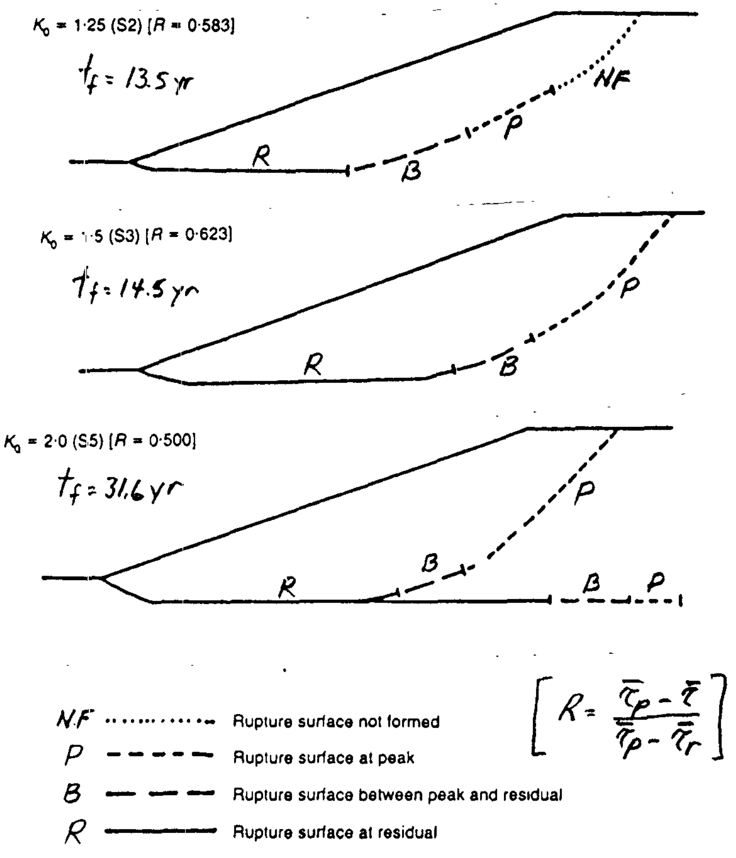
Boundary Conditions

* Surface suction = 10kPa



Note: Also did analyses without strain softening → longer t_f and higher $\bar{r}_u$ = average $r_u = u/\sigma_v$ on rupture surface. Incr. suction also → much longer t_f (i.e. vegetation on slope helps)

2) Results for H = 10m, 1V:3H as f(K_0) : Collapse = failure when analyses showed abrupt increase in δ_h at mid-slope



$$\left[R = \frac{\bar{\tau}_p - \bar{\tau}}{\bar{\tau}_p - \bar{\tau}_r} \right]$$

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_0).
- Varying slope height and angle also → varying percentages

∴ No single envelope at failure

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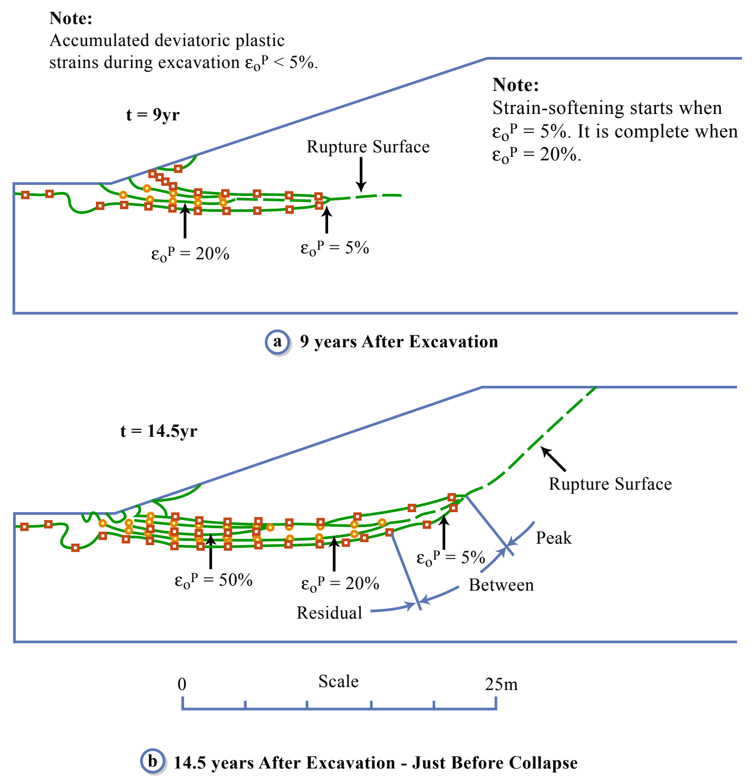
3) Results showing strain contours for $H=10m$, $1V:3H$ and $K_0=1.5$ at $t=9yr$ & $t=t_f=14.5yr$. $\varepsilon >$



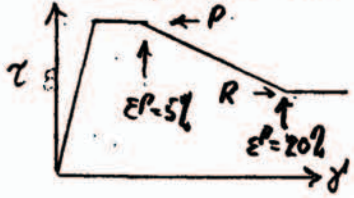
Note:
Accumulated deviatoric plastic strains during excavation $\varepsilon_o^P < 5\%$.

Note:
Strain-softening starts when $\varepsilon_o^P = 5\%$. It is complete when $\varepsilon_o^P = 20\%$.

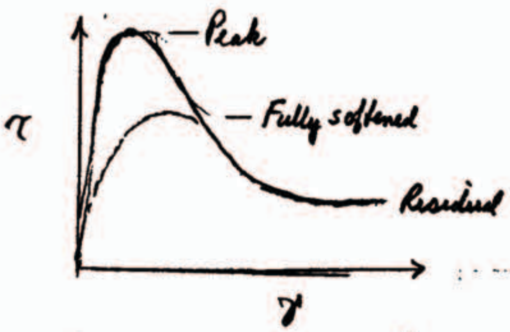
Typical Analysis (S3): 3:1 slope, 10 m high, $K_o = 1.5$, Surface Suction 10 kPa. Contours of Accumulated Deviatoric Plastic Strain, ε_o^P

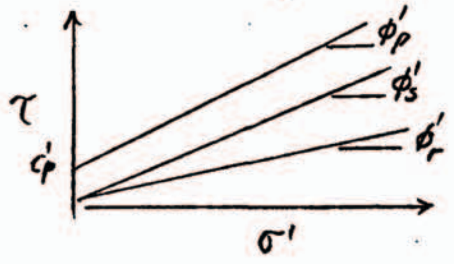
Figure by MIT OCW.

Simple Shear τ vs γ



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 → failure)
- Get progressive failure mechanism starting at toe and moving upslope
- No single envelope at failure.

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1.7 Mesri et al. (SOA paper submitted to JGGE, 3/00)

1) Analysis of ≈ 100 case histories of failures (1st time & reactivated)

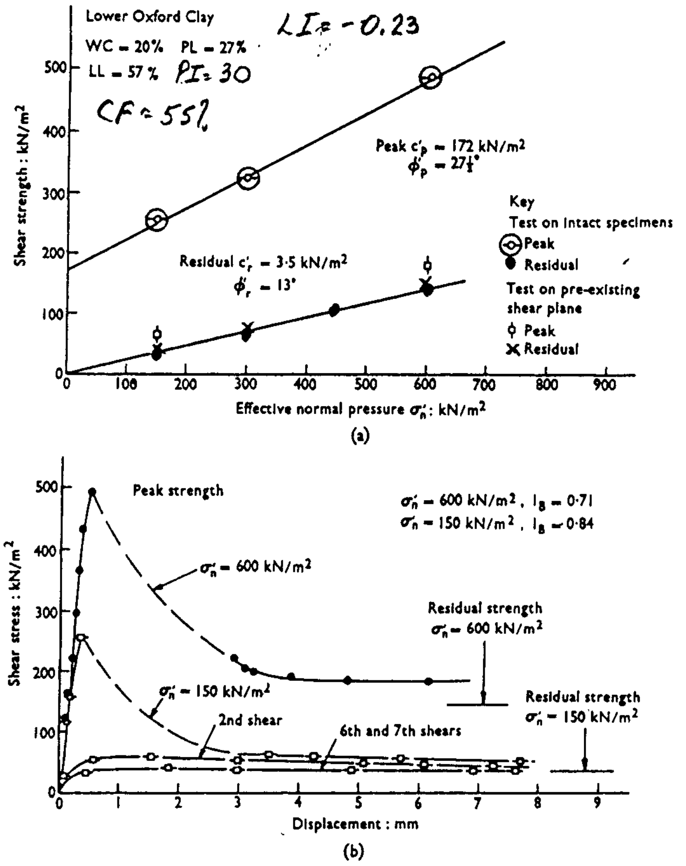
2) Principal conclusions

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 any case, 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% CaCO₃) and fissured clay shale

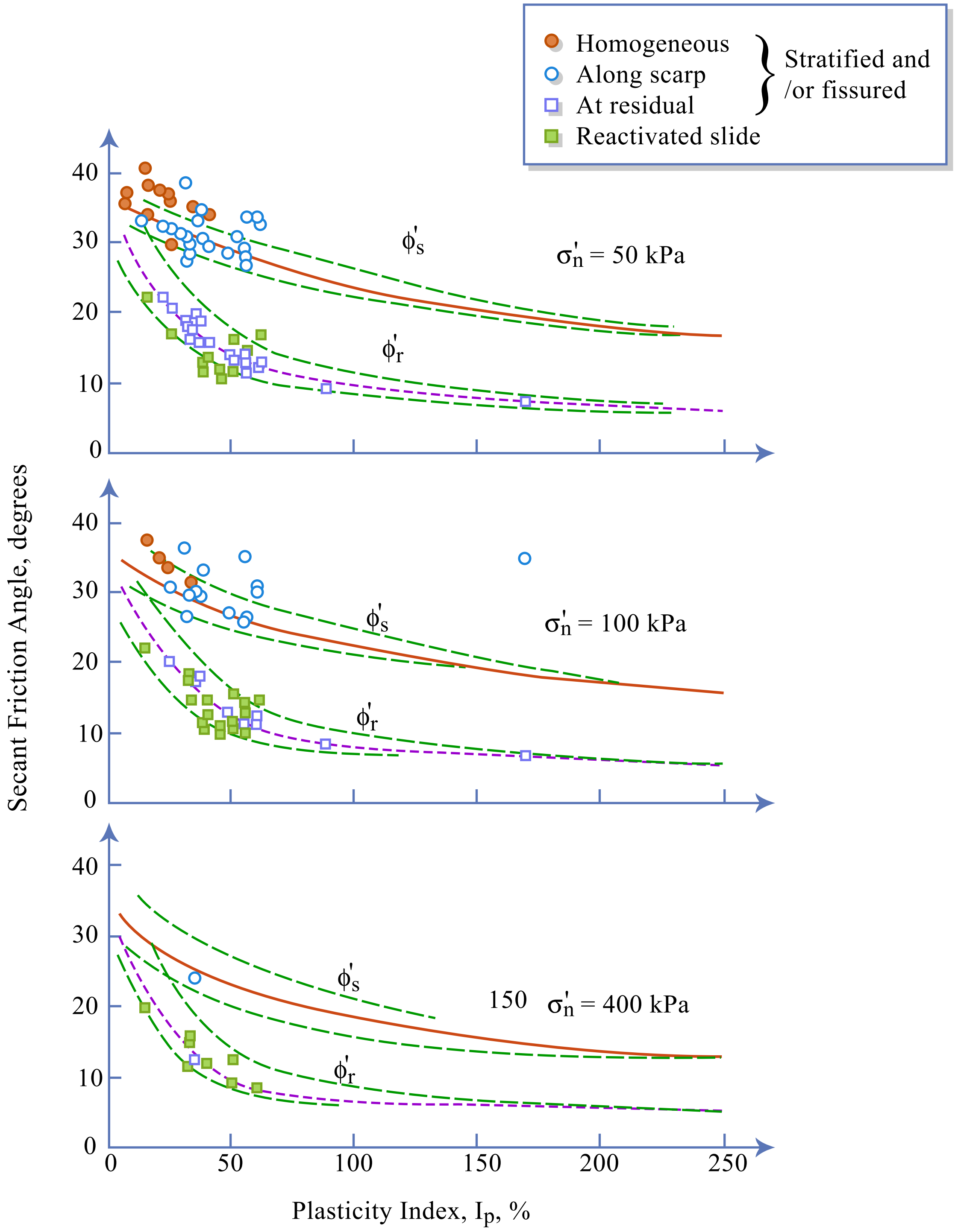
Example of large difference between peak & residual envelopes along bedding plane

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

No. 5505 Engineer's Computation Pad



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

4/90 4/96 3/98 4/3/01

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

CCL

CF <25% Low I_p : Relatively little particle reorientation Higher $\bar{\bar{\sigma}} \cdot a_m / \sigma$

CF >50% High I_p : Significant particle reorientation (Can get highly polished surface) Lower $\bar{\bar{\sigma}} \cdot a_m / \sigma$

Skempton (1985) Geot 35(1)

42-381 50 SHEETS 5 SQUARE
42-382 100 SHEETS 5 SQUARE
42-389 200 SHEETS 5 SQUARE
NATIONAL

CCL 4/2/87 1.322
4/89 4/96 4/a

ICA

Residual Friction Angle of Soils

Kenney (1977) "Residual Strength of Mineral Mixtures"
9th ICSMFE Vol. 1 pp. 155-160

Results from repeated Direct shear at $\sigma_n' = 1\ kg/cm^2$

$$R\phi_r' = \frac{\tan \phi_r'(\text{Mixture}) - \tan \phi_r'(\text{Clay})}{\tan \phi_r'(\text{Massive}) - \tan \phi_r'(\text{Clay})}$$

42.381 50 SHEETS 5 SQUARE
42.382 100 SHEETS 5 SQUARE
42.389 200 SHEETS 5 SQUARE
NATIONAL

TABLE II. RESULTS OF RESIDUAL STRENGTH TESTS ON MIXTURES.

Mixture	Salinity gm/l	* Mixture % dry weight	Mineral Content % dry wt. Total Massive	Total Clay	Residual State $\sigma_n' = 1.0$ kg/cm² w_{res} %	% Volume Clay and Water	tg ϕ_{res}
A. MIXTURES CONTAINING MONTMORILLONITE							
Montmorillonite – Na and Quartz	0	50/50	50	50	200	92	0.09
		25/75	25	75	96	80	0.11
		10/90	10	90	44	60	0.42
		5/95	5	95	39	54	0.61
Montmorillonite – Na and Quartz	30	50/50	50	50	72	84	0.24
		25/75	25	75	51	69	0.36
		10/90	10	90	34	54	0.56
Montmorillonite – Na and Amorphous SiO_2	30	50/50	50	50	150	90	0.21
		25/75	25	75	87	78	0.24
Bentonite and Quartz	0	75/25	44	56	61	84	0.12
		50/50	62	38	52	75	0.25
		25/75	81	19	42	63	0.40
		15/85	89	11	32	53	0.64
Bentonite and Amorphous SiO_2	0	91/9	32	68	70	89	0.10
		82/18	39	61	56	84	0.11
		68/32	49	51	68	81	0.13
		47/53	65	35	60	76	0.18
		25/75	81	19	47	66	0.27
		10/90	92	8	51	61	0.49
B. MIXTURES CONTAINING KAOLINITE AND GRUNDITE							
Kaolinite and Quartz	0	75/25	25	75	40	88	0.32
		50/50	50	50	30	73	0.49
		25/75	75	25	23	54	0.65
Kaolinite and Amorphous SiO_2	0	75/25	25	75	44	89	0.32
		50/50	50	50	31	74	0.41
Grundite – Na and Quartz	0	75/25	25	75	67	91	0.25
		50/50	50	50	49	79	0.40
		25/75	75	25	36	62	0.62
Grundite – Na and Quartz	30	75/25	25	75	63	91	0.22
		50/50	50	50	40	77	0.40
		25/75	75	25	36	62	0.62
C. MIXTURES CONTAINING HYDROUS MICA							
Hydrous mica I – Na and Quartz	0	75/25	48	52	30	74	0.35
		50/50	65	35	31	65	0.44
Hydrous mica I – Na and Quartz	30	75/25	48	52	32	75	0.38
		50/50	65	35	33	66	0.46
Hydrous mica I – K and Quartz	0	75/25	48	52	30	74	0.41
		50/50	65	35	30	65	0.47
Hydrous mica I – K and Quartz	30	75/25	48	52	41	78	0.49
		50/50	65	35	35	67	0.50
Hydrous mica II – Na	0	75/25	33	67	31	80	0.32
		50/50	55	45	22	66	0.46
Hydrous mica II – Na	30	75/25	33	67	41	83	0.45
		50/50	55	45	27	67	0.47
Hydrous mica III – Na	0	75/25	33	67	54	85	0.44
		50/50	55	45	40	74	0.48
Hydrous mica III – Na	30	75/25	33	67	52	85	0.44
		50/50	55	45	46	76	0.50

* think massive-clay % are reversed

"Low Clay"

"High Clay"

Sand particles inhibit reorientation of clay particles

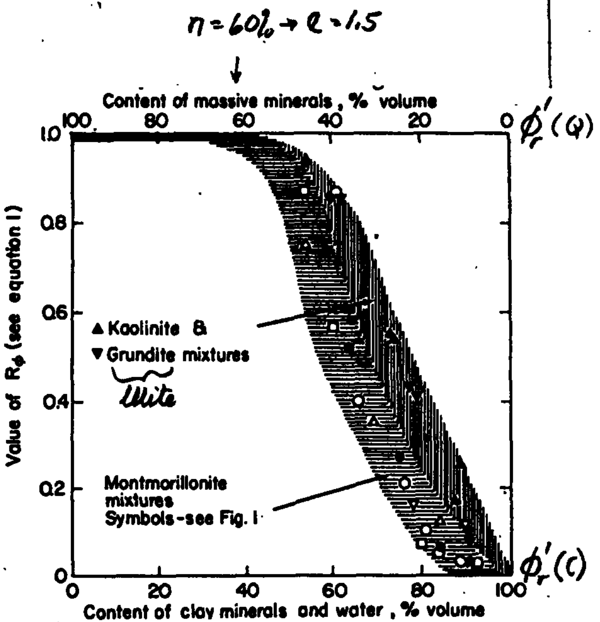


Fig. 4. Relative residual strength

CCL 4/2/87 1,322

4/89 4/3/01

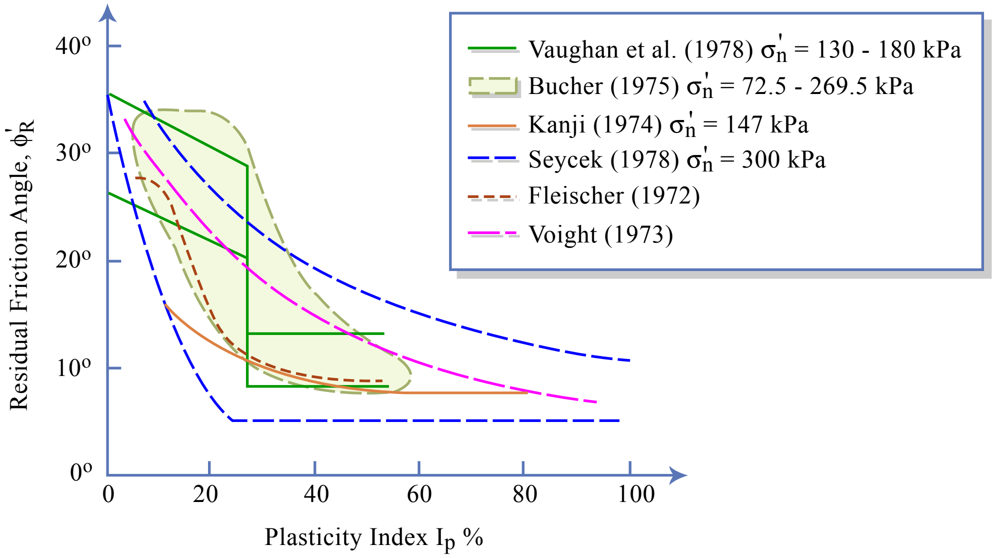
I010

1.9(Cont) Empirical Correlations : ϕ'_r (See IC7 for ϕ'_s)

Lupini, Skinner & Vaughan (1981) Geotechnique V31 No.2, pp 181-213

(Contains alot of fundamental research on topic)

DRAINED RESIDUAL STRENGTH OF COHESIVES SOILS



Residual Strength : Correlations with Plasticity Index

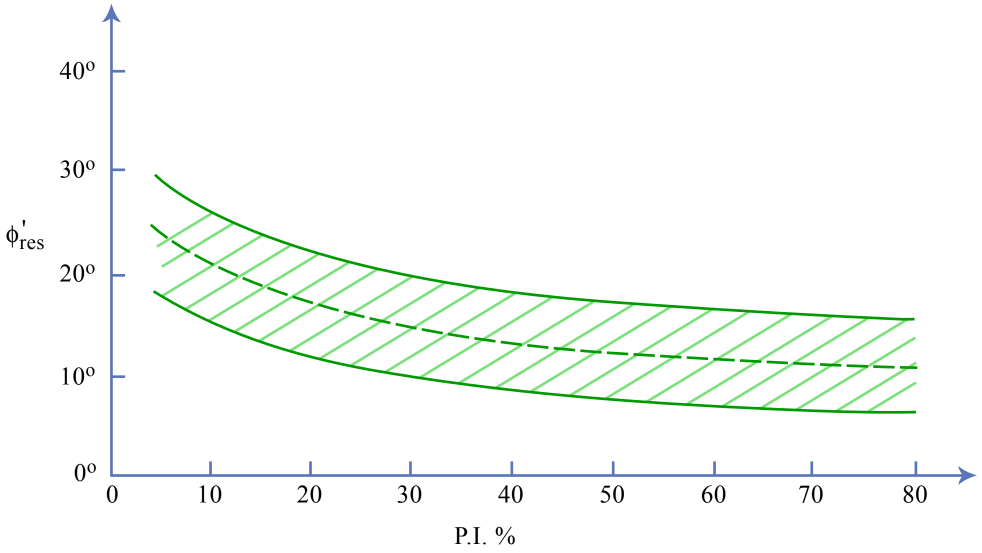


Figure by MIT OCW.

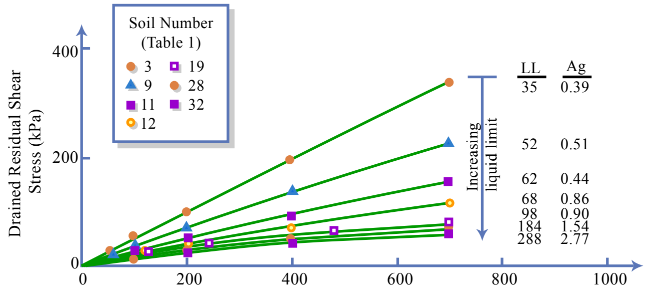
CC Ladd
4/74

CCL 4/2/96 1.322

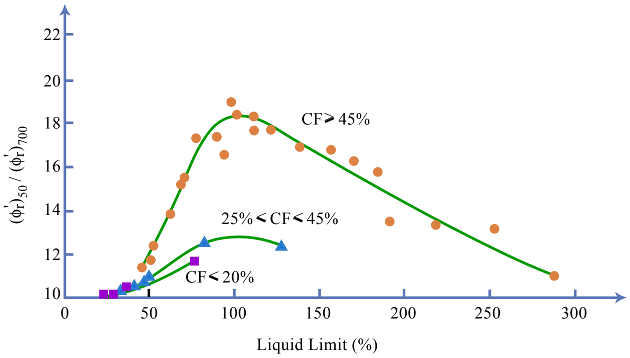
ICU

DRAINED RESIDUAL STRENGTH OF COHESIVE SOILS

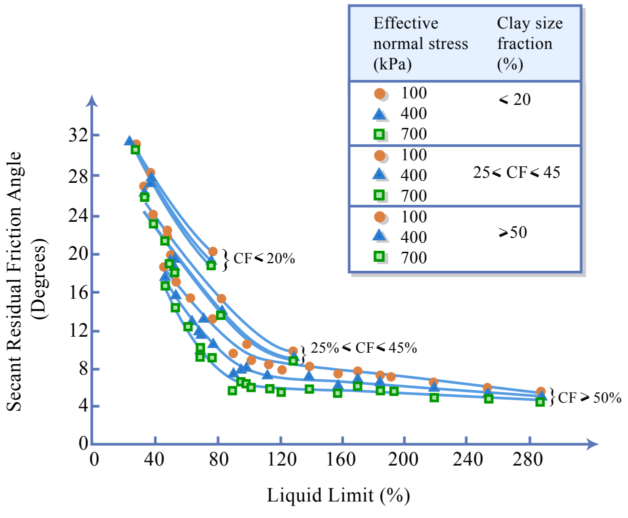
By Timothy D. Stark,[1] Associate Member, ASCE,
and Hisham T. Eid,[2] Student Member, ASCE



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)

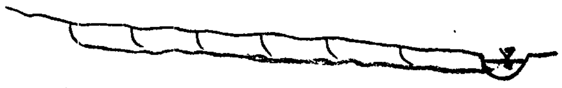
Results of torsional ring shear tests on 32 clays and shales → empirical correlation of ϕ'_r as function w_L, CF (%-2μm) and σ'_n. Tests on remolded specimens = mixture distilled H₂O with airdried soil after ball-milling to obtain 100% -#200 sieve (to break down aggregates).

NOTE: Ball-milling essential to breakdown airdried aggregates → correct AL; CF

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

C2. HIGHLY STRUCTURED, SENSITIVE CLAYS (Quick Clays)

2.1 Background



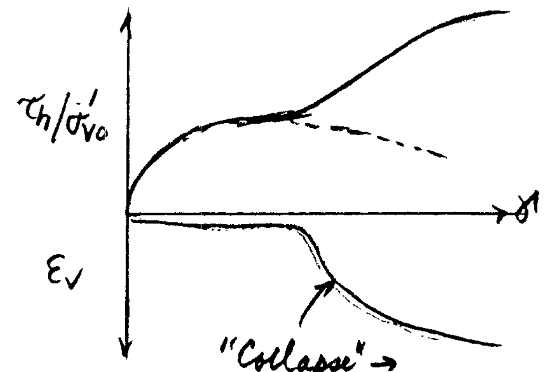
Flake and/or retrogressive

- Almost flat slope (Norway)
- "Minn" Δ geometry → massive flow slide
  - Pictures 3rd floor
  - Rissa film

- Approach = f(location)

2.2 Norway Aas (1981) ICSMFE

- Analysis 5 "flake" type slides : Treats as CU Case via USA



"Collapse" → Undrained condition ∴ $u_{sh} > 0$

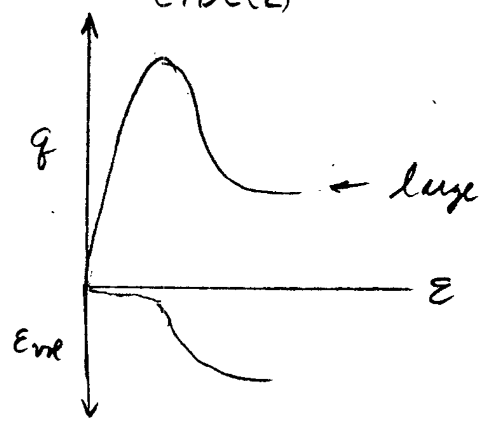
In Situ $\tau/\sigma'_{vo} = 0.18 \pm 0.035$

Lab CK₀UDSS $c_u/\sigma'_{vo} = 0.195 \pm 0.025$

2.3 Quebec Lefebvre (1981) CGJ p420

- Treats as CD Case using equilibrium u, but "large strain" values of c' & φ'

CIDC(L)



← large strain (ε ≈ 10%) used to select c' & φ'

See IC13

CCL: Seems more empirical than 2.2, but applied to circular arc type failures

NATIONAL 42-381 50 SHEETS 5 SQUARE 42-382 100 SHEETS 5 SQUARE 42-389 200 SHEETS 5 SQUARE

CCL 1.322 4/84 4/88

SLOPE STABILITY - QUICK CLAYS (Canada)

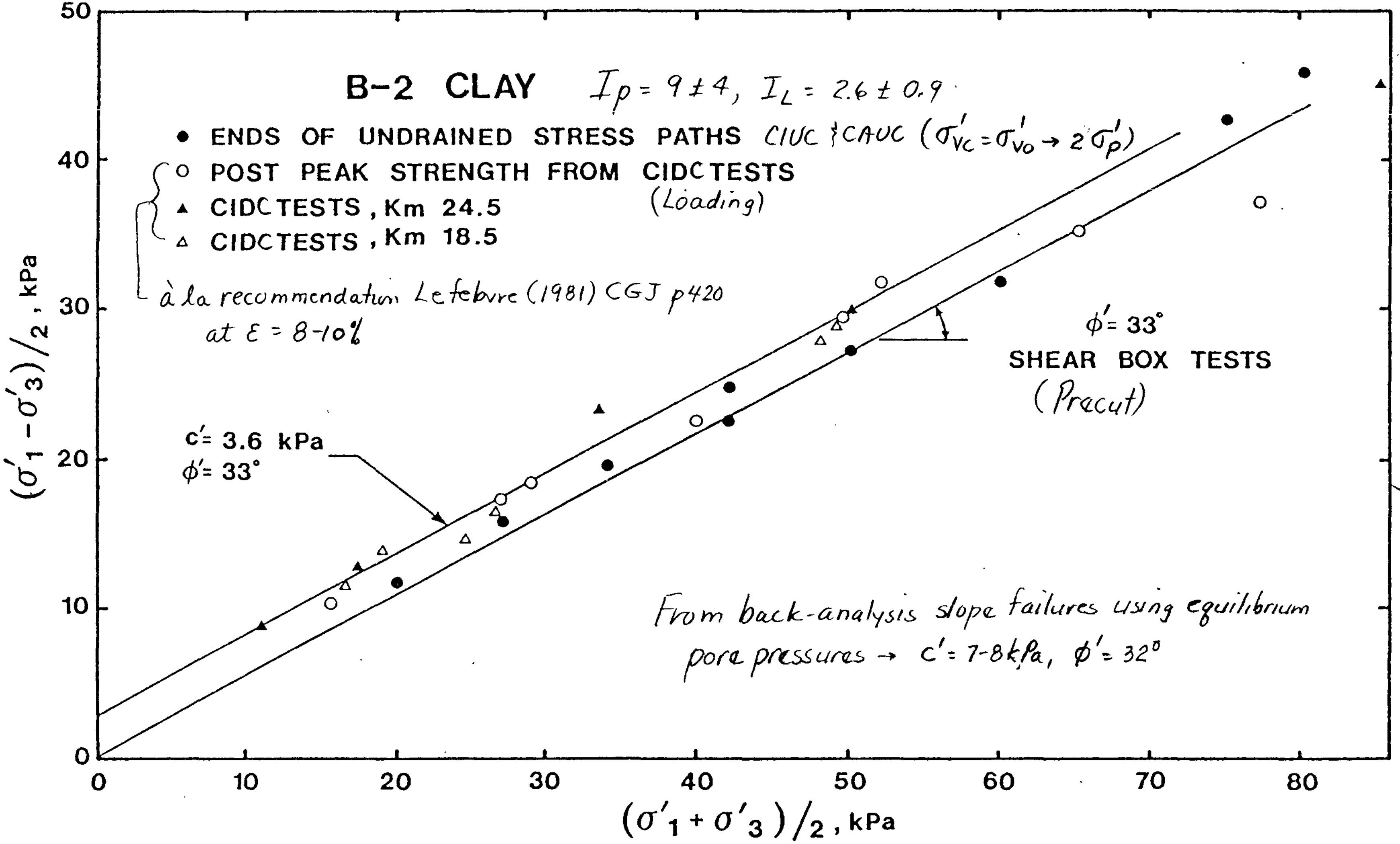


FIG. 5.5-9 TYPICAL EFFECTIVE STRENGTH ENVELOPES B-2

From SEBJ (not for publication)

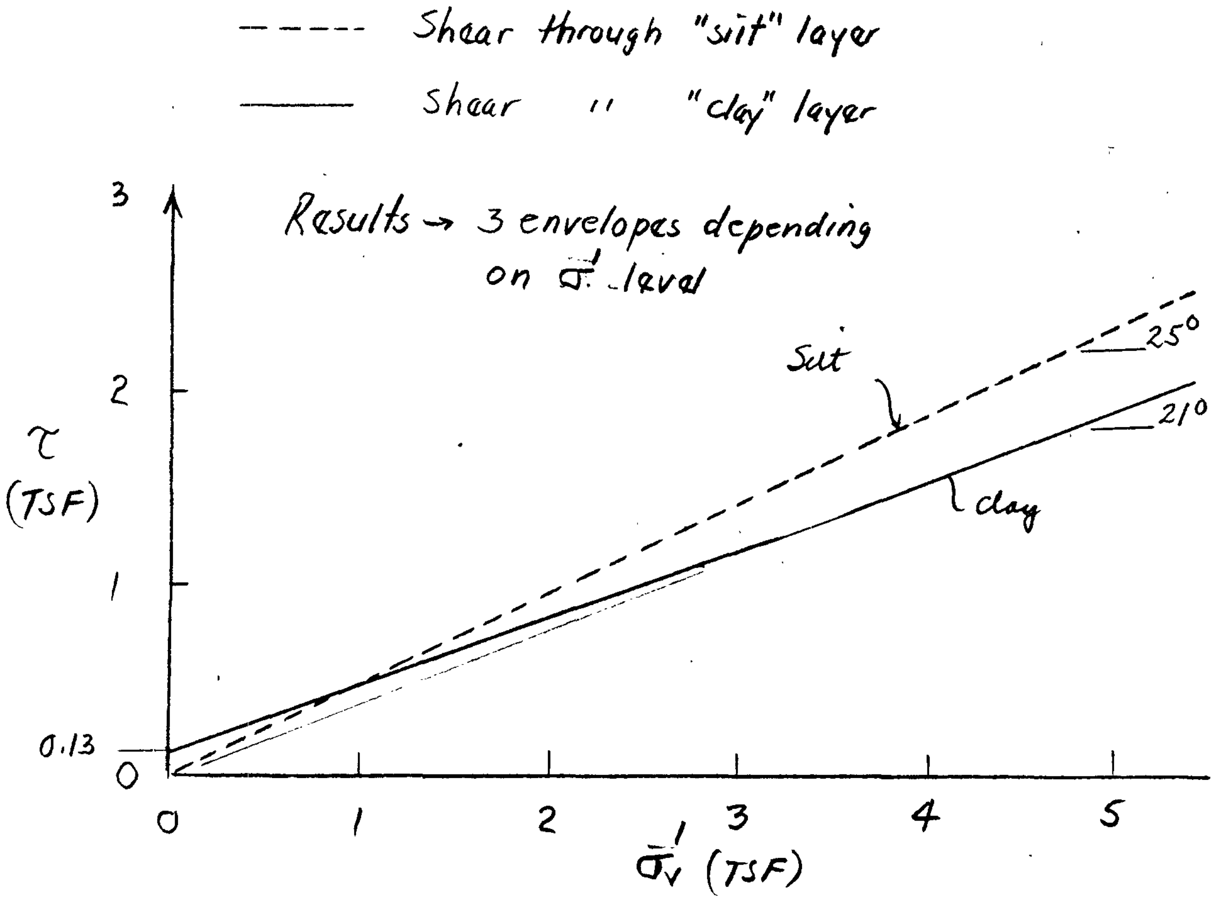
IC13

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/ $\bar{\sigma}_p' \approx 3.5$ TSF)



b) Summary of ESE Data Bulk I_p = 15-30% Ladd & Foott (1977) FHWA

CU Shear Across Varves
Compression & Extension SHANSEP Tests $c'/\bar{\sigma}_{vm}' = 0.012$ $\phi' = 30°$

CD Shear In Clay Varves $c'/\bar{\sigma}_p' = 0.025$ $\phi' = 20°$