IIE STAGED CONSTRUCTION (Mostly abstracted from Ladd (1991) = Terzaghi Lecture)

(Mini-Questions - 1p)

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 analysis 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} (for the last two items)

1.2 Types of Stability Analysis & Definition of Factor of Safety (Sections 2 & 6)

1) $$FS = \frac{\text{Available shear strength of soil} = s}{\tau_m = \text{shear stress required for equilibrium} = \text{mobilized } \tau}$$

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

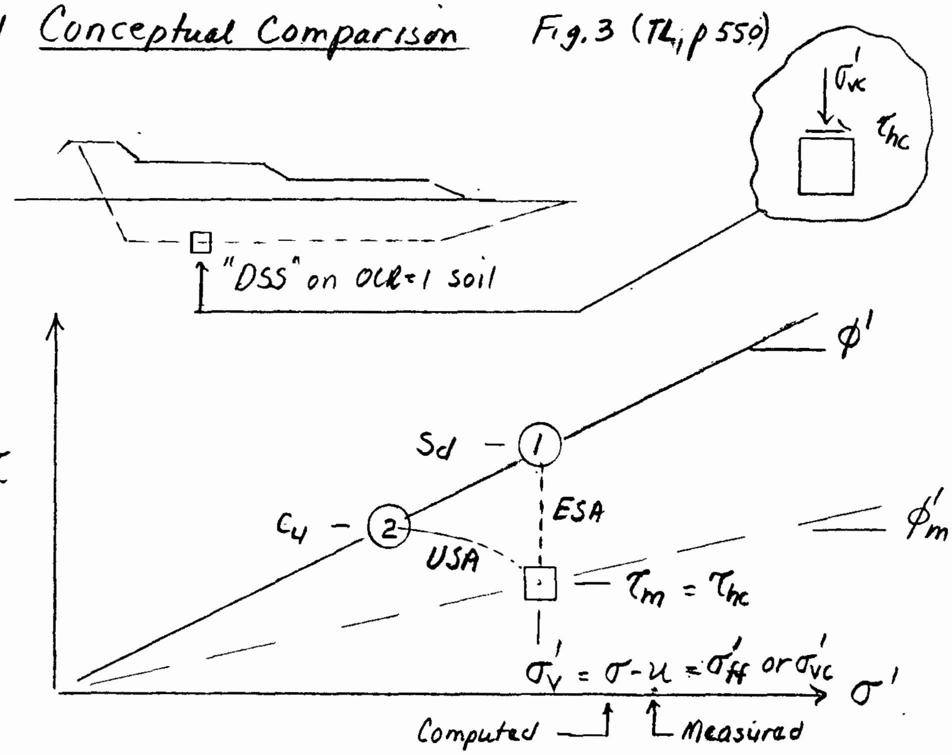
4) USA = Undrained Strength Analysis

$FS = c_u/\tau_m$

- 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

- $\frac{FS(ESA)}{FS(USA)} = \frac{\tan\phi' / \tan\phi'_m}{c_u/\tau_m} = \frac{\tan\phi'/(\tau_m/\sigma'_{vc})}{(c_u/\sigma'_{vc})/(\tau_m/\sigma'_{vc})} = \frac{\tan\phi'}{c_u/\sigma'_{vc}} = \frac{\tan\phi'}{S(DSS)}$

- $\phi' = 25°$ & $S = 0.20$; $\phi' = 30°$ & $S = 0.25$ → $\frac{FS(ESA)}{FS(USA)} \approx 2.3$!

2.2 Case Histories (Section 3)

Example	Condition	FS($\frac{ESA}{USA}$) Table 2 (p 561)	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 slow, drained failure (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
% incr. in c_u =	0	5±5%	30±10%	

   b) Should have used S from CK₀U tests with $t_c \approx t_p$ (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

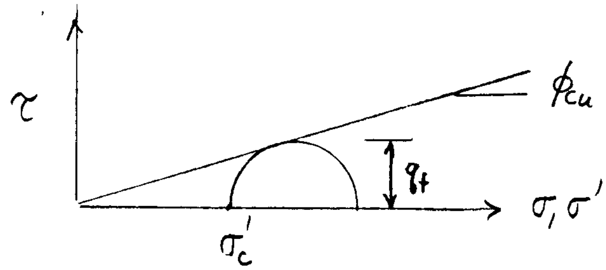
      } + horizontal surface with τ_d

   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)



$c_u = \sigma'_{fc} \tan\phi_{cu}$
$= \sigma'_n \tan\phi_{cu}$

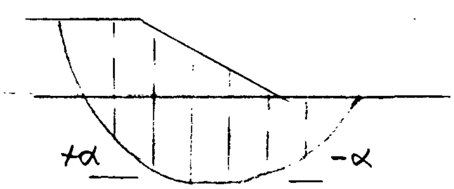
- What is physical significance of ϕ_{cu}? (Answer = NONE)

- $c_u/\sigma'_{fc} = \tan\phi_{cu} = \dfrac{q_f/\sigma'_c}{\sqrt{1+2q_f/\sigma'_c}}$

q_f/σ'_c	$\tan\phi_{cu}$
0.25	0.204
0.30	0.237
0.35	0.268

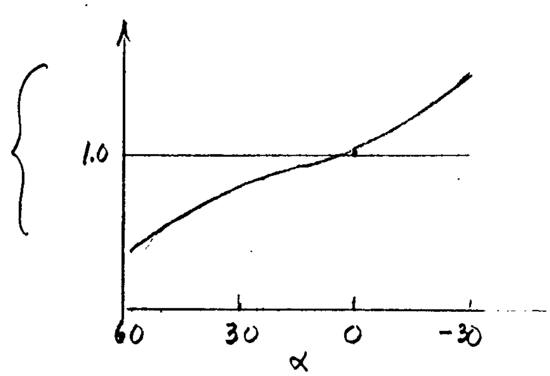
↑ Reasonable range

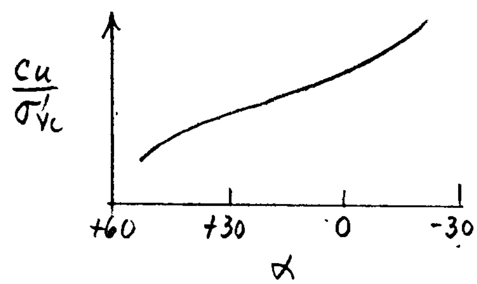
3) Computed c_u/σ'_{vc} Fig. 21 (Sheet B)



$$\frac{\sigma'_n}{\sigma'_v} = \frac{\sigma'_{fc}}{\sigma'_{vc}} = \frac{1}{1 + \frac{\tan\alpha \tan\phi_{cu}}{FS}}$$

Simplified Bishop (Remember 1.361 plot)



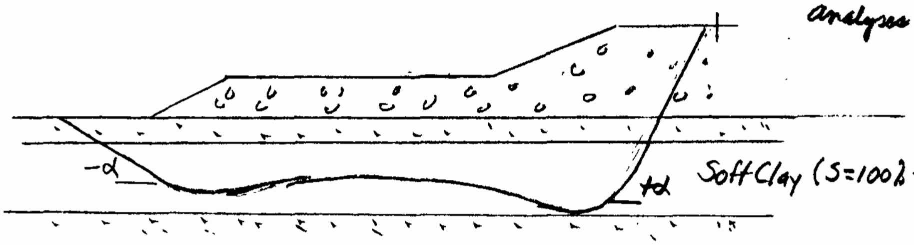


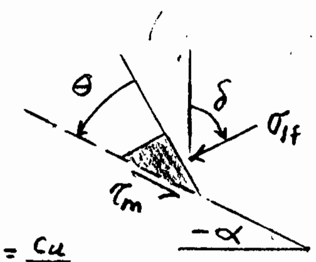
How compare with your understanding of c_u anisotropy?

4. NON-CIRCULAR STABILITY ANALYSES WITH ANISOTROPIC UNDRAINED SHEAR STRENGTHS

4.1 Problem Definition

Patterned after UTEXAS 3 analyses of TPS





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

Require input of $c_u = f(\alpha)$

Two major questions $[q_f = 0.5(\sigma_1 - \sigma_3)_f]$

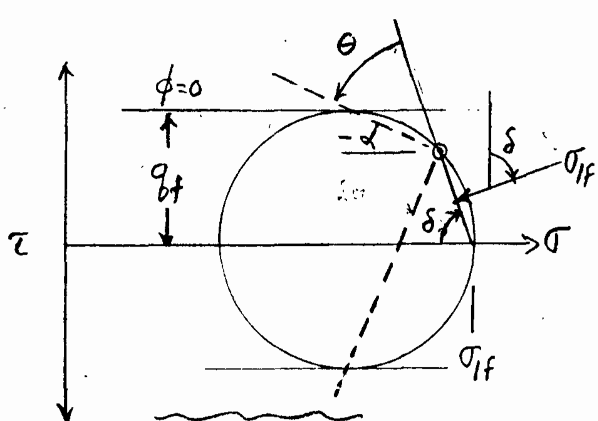
1) Definition of $c_u = q_f \cos\phi$
   Use $\phi = 0$ or $\phi = \phi'$?

2) Value of θ = angle between failure plane and σ_{1f} plane $= 45 + \phi/2$
   leading to $\alpha = \theta - \delta$

4.2 Theoretical Relationships

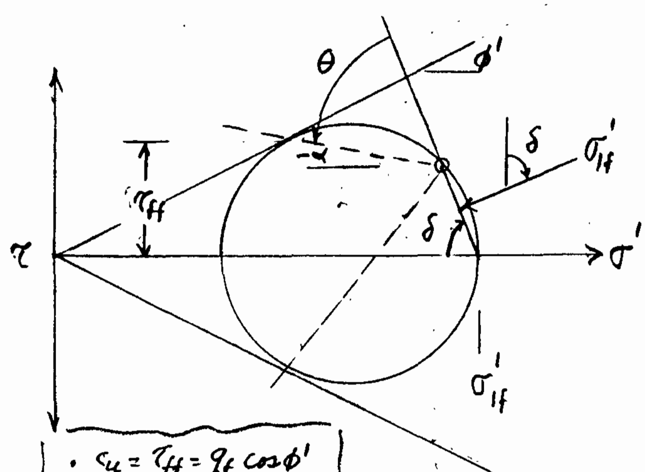
- From CK_oU test like DSC, have known q_f vs. δ ; $\alpha = \theta - \delta$

Using $\phi = 0$



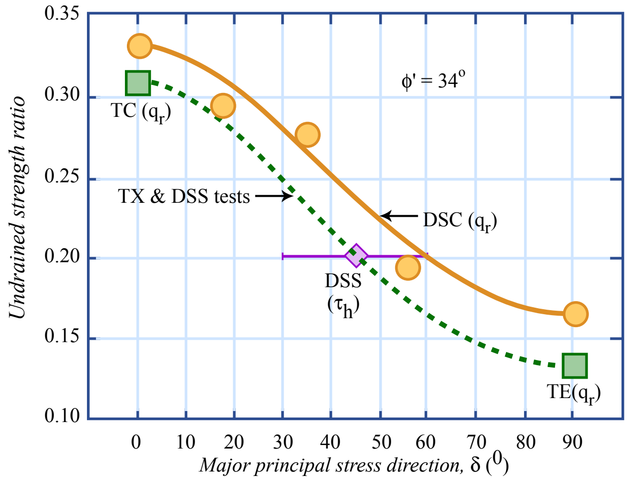
- $c_u = q_f$
- $\theta = 45°$
- $\alpha = 45° - \delta$

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_0U 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:

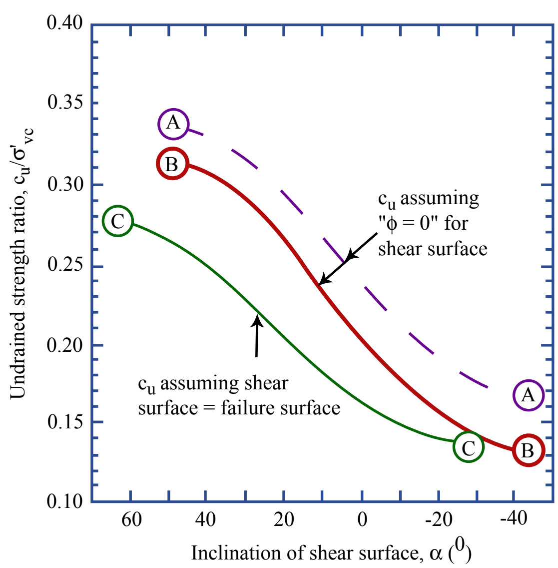
$\phi = 0 \rightarrow c_u = q_f$
$\alpha = 45° - \delta$

$\phi' = 34° \rightarrow c_u = 0.839 q_f$
$\alpha = 62° - \delta$

NOTE: 34° = measured θ in NC DSC tests

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

$\phi' = 34°$ $\theta = 62°$



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	$\phi = 34°$	0.185

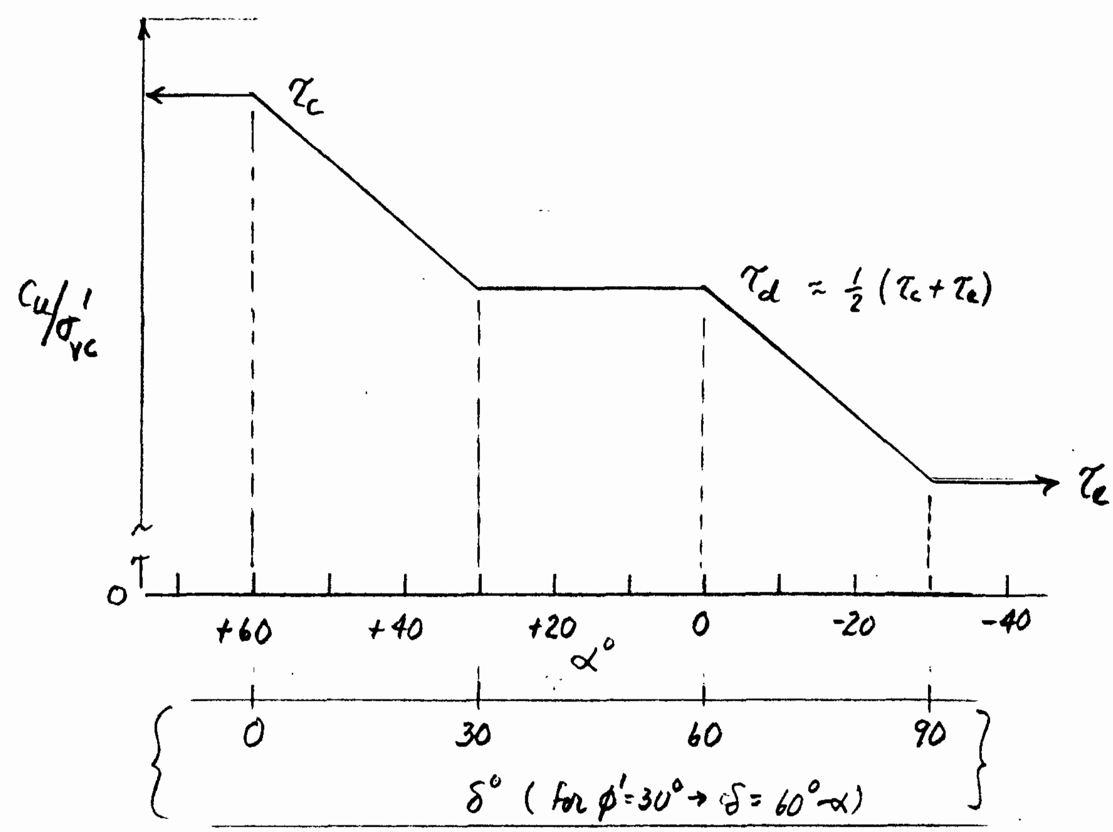
* For $\alpha = +45°$ to $-30°$

3) Conclusion: If searched failure surface close to actual potential failure surface, then "$\phi = 0$" assumption $\rightarrow c_u = q_f$ & $\alpha = 45° - \delta$ is very UNSAFE (by ≈ 40% for PS data & ≈ 20% for TX data)

No. 5505 Engineer's Computation Pad

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 Ⓒ 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 Ⓒ

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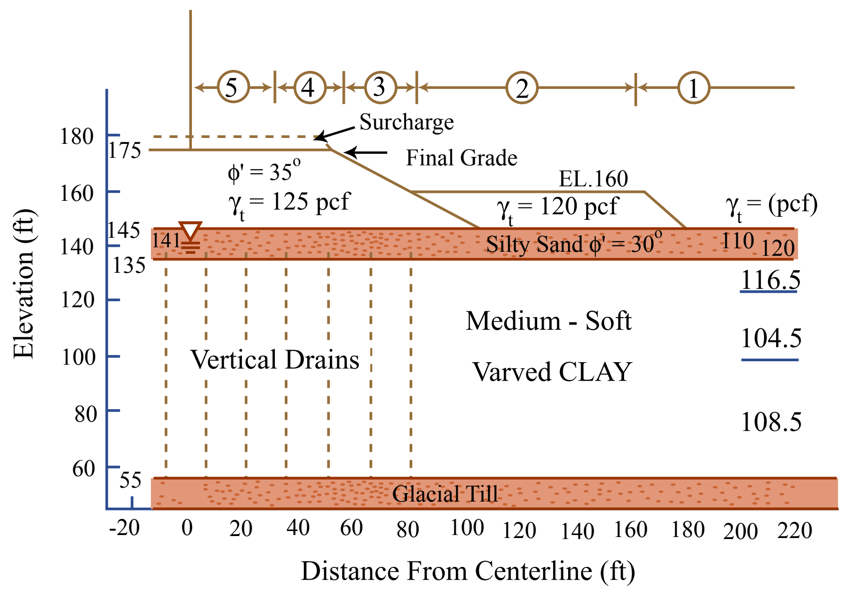
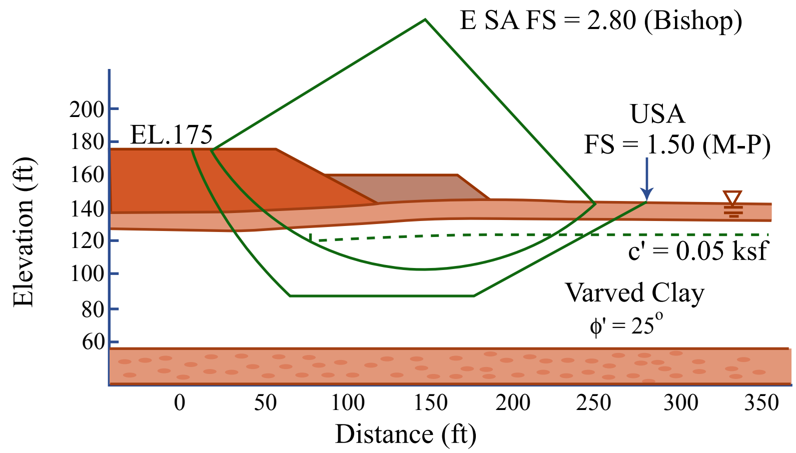


Design problem for highway embankment on connecticut valley varved (1 ft = 0.305 m; 1 pcf = 0.157 kN/m³)



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.

A1

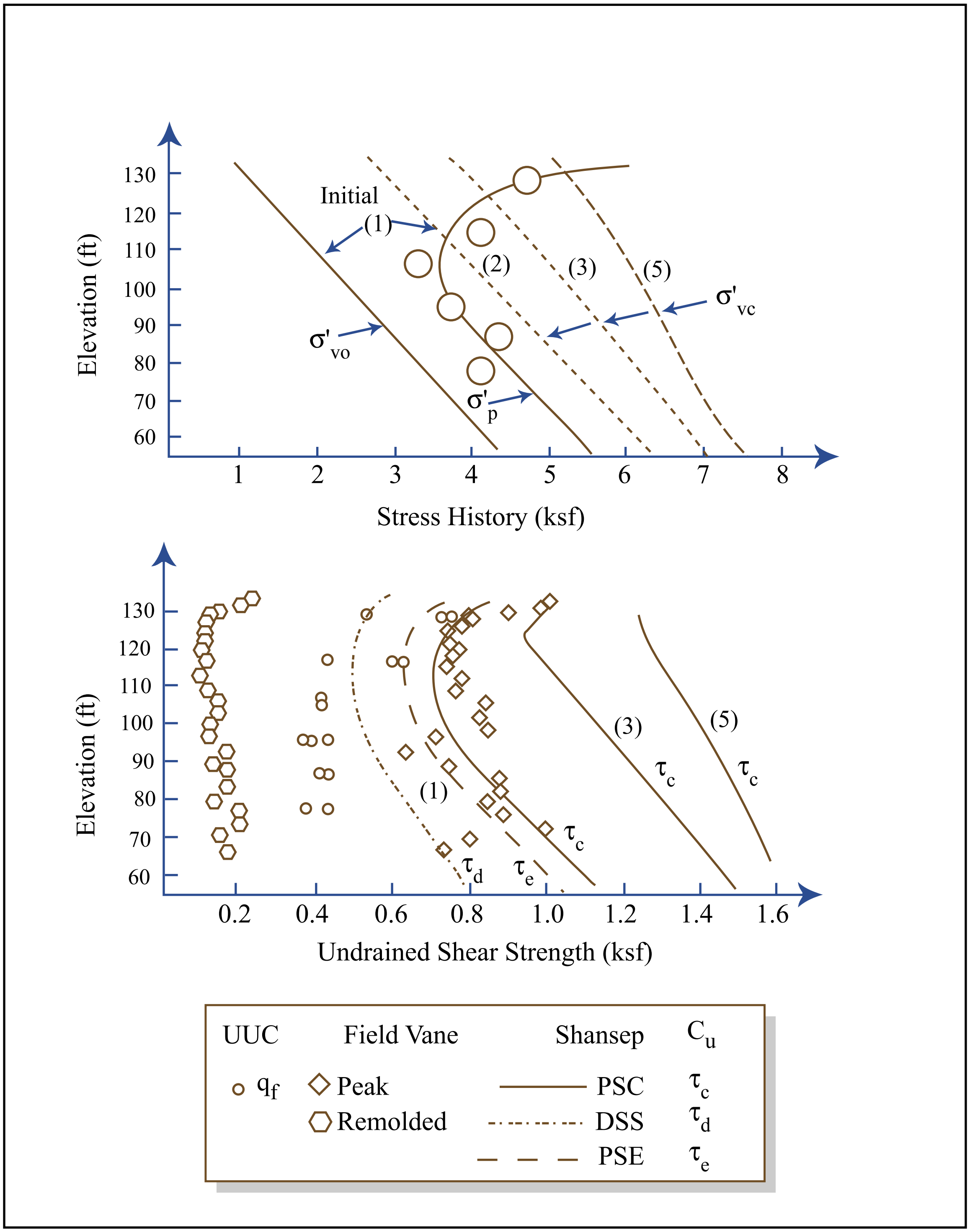


Figure by MIT OCW.

Embankment on Varved Clay: $\bar{U}$ = 100%
(Ladd 1991)

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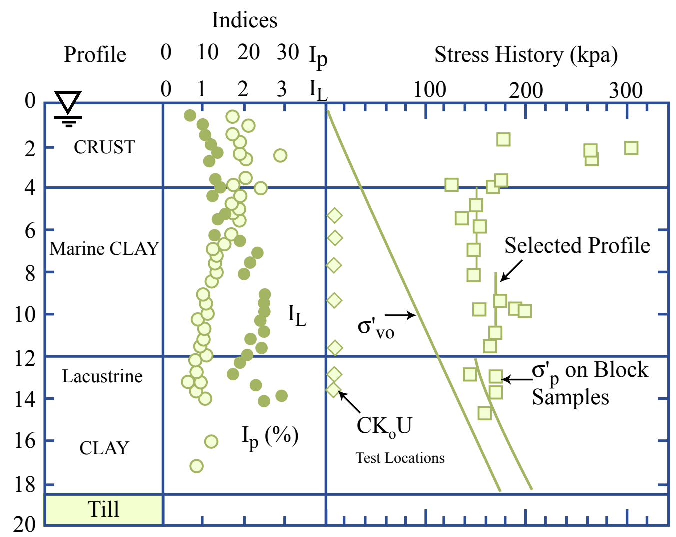
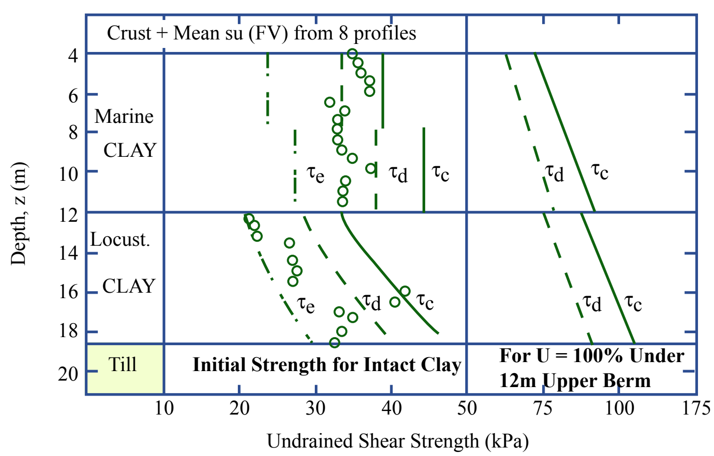
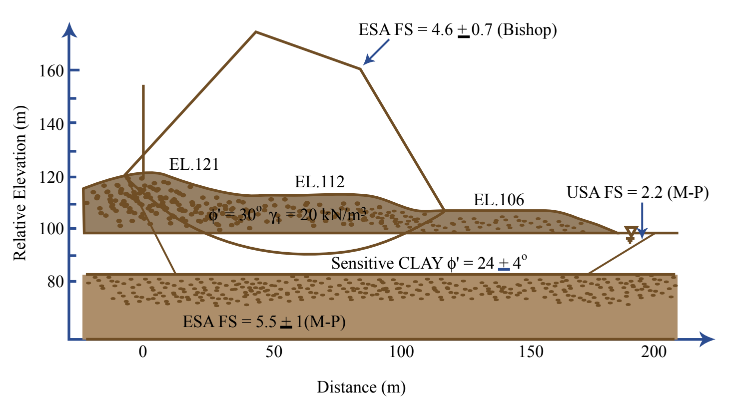


Soil profile, Index properties, and stress history at James Bay Site B-6



Field vane and anisotropic undrained strength profiles at James Bay Site B-6



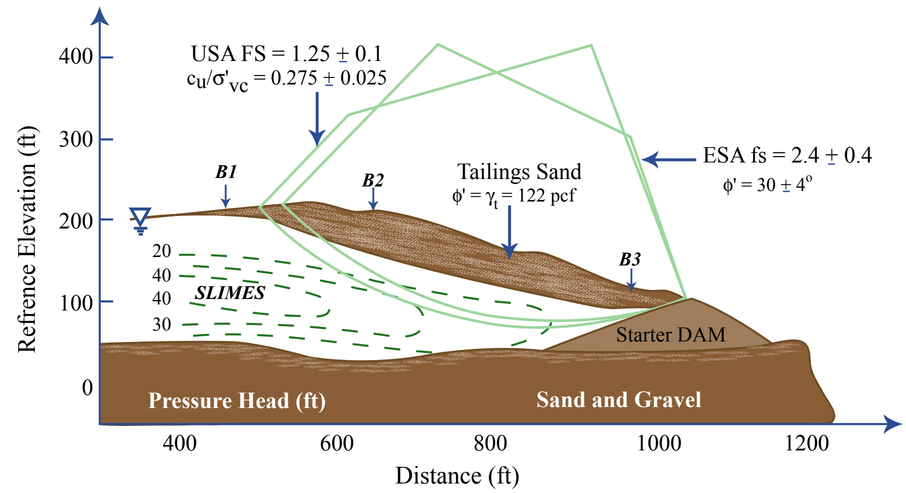
ESA and USA Factors of safety for Embankment Dam on James Bay sensitive clay at Ū= 100%

Note: *For Circular arc from crest to toe of berm, FS = 5.2 ± 0.7*

Figures by MIT OCW.

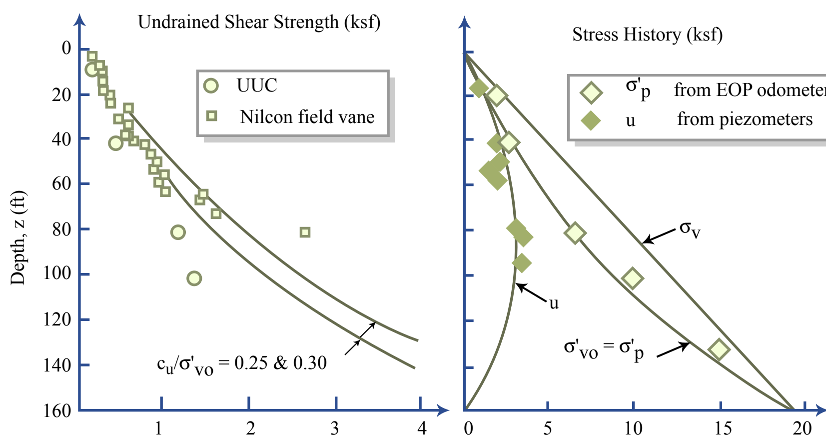
A2

Adapted from: Embankment on James Bay Quick Clay: Ū=100% (Ladd 1991)



ESA and USA Factors of safety upstream tailings Dam (1 ft = 0.305 m; 1 pct = 0.157 kN/m²)

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

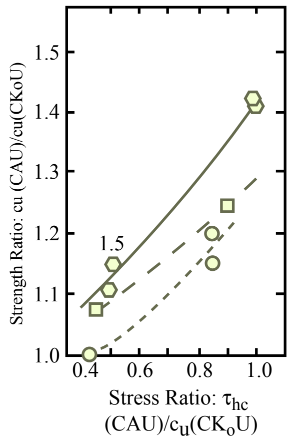
Ladd (1991)

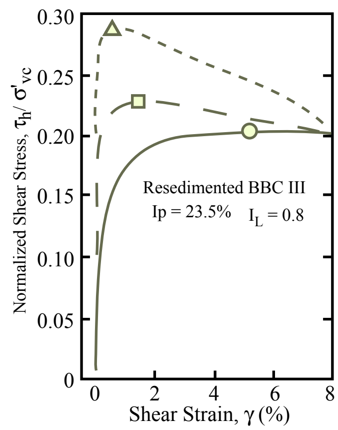
TABLE 2. Undrained Strength Parameters for Connecticut Valley Varved Clay and James Bay Sensitive Clay

Clay deposit (1)	MODE OF FAILURE: Compression S (2)	Compression m (3)	Direct Simple Shear S (4)	Direct Simple Shear m (5)	Extension S (6)	Extension 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.

CAUDSS Data
NC Clays





Soil		CK_oU c_u/σ'_{vc}	Sym
BBC	I	0.201	⬡
	III	0.203	⬡
Toilings		0.223	□
EABPL		0.235	○

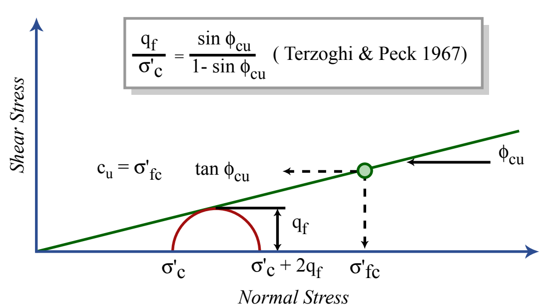
τ_{hc}/σ'_{vc}	Symbol
0	–○–
0.10	–□
0.20	-△-

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

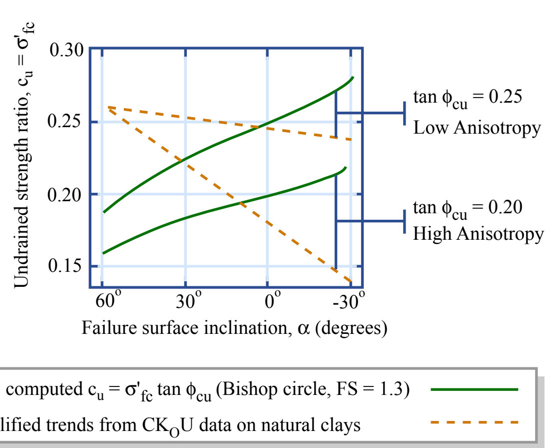
QRS Methodology

Definition of ϕ_{cu}



Angle of shearing resistance ϕ'_{fc} from isotropically consolidated - undrained triaxial compression (CIUC) tests as defined by A. Casagrande

Computed c_u/σ'_{vc} and
· via QRS
· Actual trends



Undrained strength ratios from QRS methodology compared to trends from CK_OU *testing for normally consolidated clay.*

(Ladd 1991)

B

Figure by MIT OCW.

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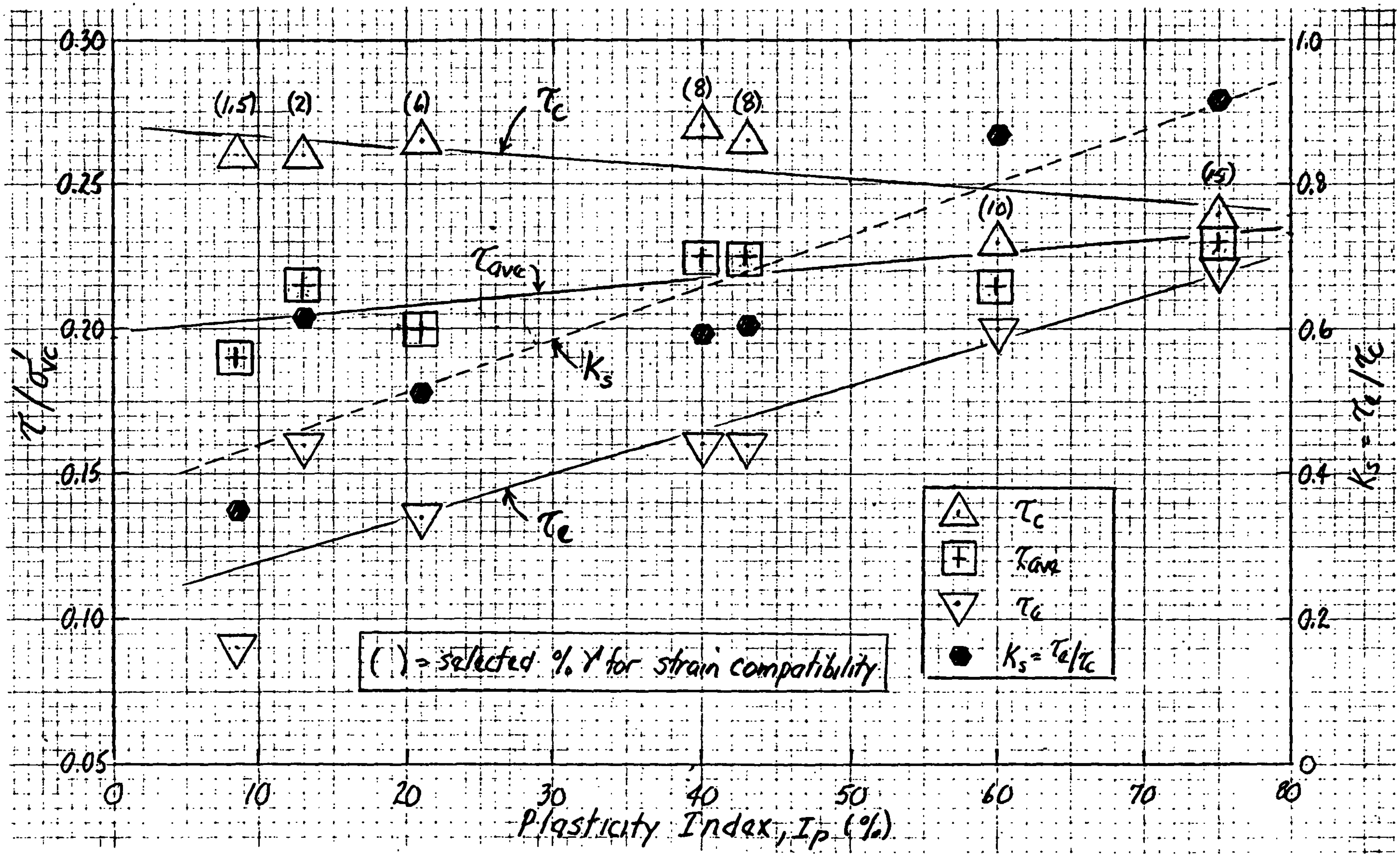


Undrained Shear Strength Ratios vs. Plasticity Index for CL and CH Clays Treated for Strain Compatibility (Data from Table 4, Ladd 1991)

(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:

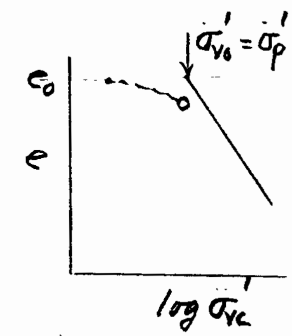
- Incr. s_u from δ = 0 & fast $\dot{\varepsilon}$ vs decr. s_u due disturbance
- Net error can easily be ± 25-50%

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 ≈ 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) BOTH Recompression & SHANSEP

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) → $s_u = \frac{1}{2}[q_f(C) + q_f(E)]$ → ok 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 : TC/PSC ≈ 0.92 ± 0.05 ; TE/PSE = 0.82 ± 0.02 } ≈ 0.87

(2) Strain compatibility : $\frac{q_f(\text{Ave}) \text{ at design } \gamma}{q_f(\text{Ave}) \text{ of peaks}} \approx 0.90$ 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" : $\frac{FS(3D)}{FS(2D)} = 1.11 \pm 0.06$ 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) <u>Plane strain failure</u> $L \to \infty$ (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 ($I_p \approx 20-80\%$) — $S = 0.215 \pm 0.015$ ($S = 0.20 + 0.05 I_p$)

NOTE: <u>Varved</u> $S \approx 0.16$ (N.E. US)

Below A-line
- Sedimentary silts & organic soils + clays w/ shells — $S = 0.25 \pm 0.05$

3) Typical "<u>end effects</u>" 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