# ESTIMATION OF DESIGN $s_u$ IN PRACTICE

PAGE

# 1. INTRODUCTION

NOTE: Will focus on medium-to-low OCR, saturated sedimentary cohesive soils since most critical for field loading conditions, e.g., "$\phi=0$", $c=s_u$ stability analysis for UU Case. Also "clay" = cohesive soils both above & below A-line

## 1.1 Conventional Practice

1) Basically assumes that the in situ $s_u$ is uniquely related to $w_f$ à la Principle II. Hence any shear test on soil with $w_f = w_N$ will give $s_u$ values appropriate for design.

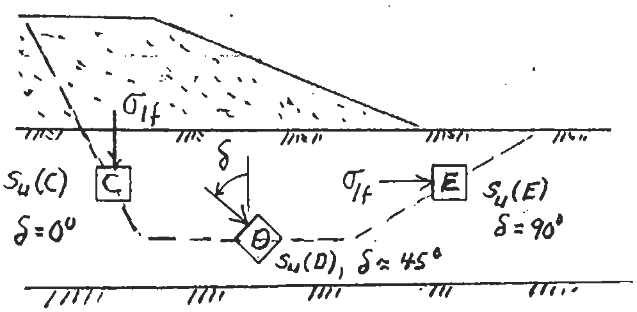
2) Common shear tests include:
   - Lab - Mostly UUC with supplemental Torvane (TV), fall cone (FC), pocket penetrometer (PP) & miniature lab vane (LV)
   - In situ - field vane test (FVT) & cone penetration test (CPT), with supplemental Standard Penetration Test (SPT).

3) However, this approach is highly empirical and often unreliable because it neglects to account for three principal factors that affect the measured $s_u$. These are: i) anisotropy ($\delta$ angle); ii) strain rate (or time to failure); and iii) sample disturbance

## 1.2 Three Principal Factors Affecting $s_u$ of Given Clay ($S=100\%$)

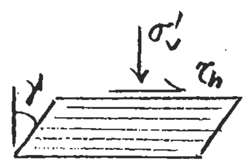
### 1.2.1 Anisotropy ($\delta$ angle = direction of $\sigma_{1f}$ wrt. vertical)

1) Problem definition illustrated for long embankment (plane strain b)




- Element C : Should be $CK_0UPSC$, but usually modeled via TC ($b=0 \rightarrow$ lower $s_u$ by $8 \pm 5\%$)
- Element E : Should be $CK_0UPSE$, but usually modeled via TE ($b=1 \rightarrow$ lower $s_u$ by $18 \pm 2\%$)

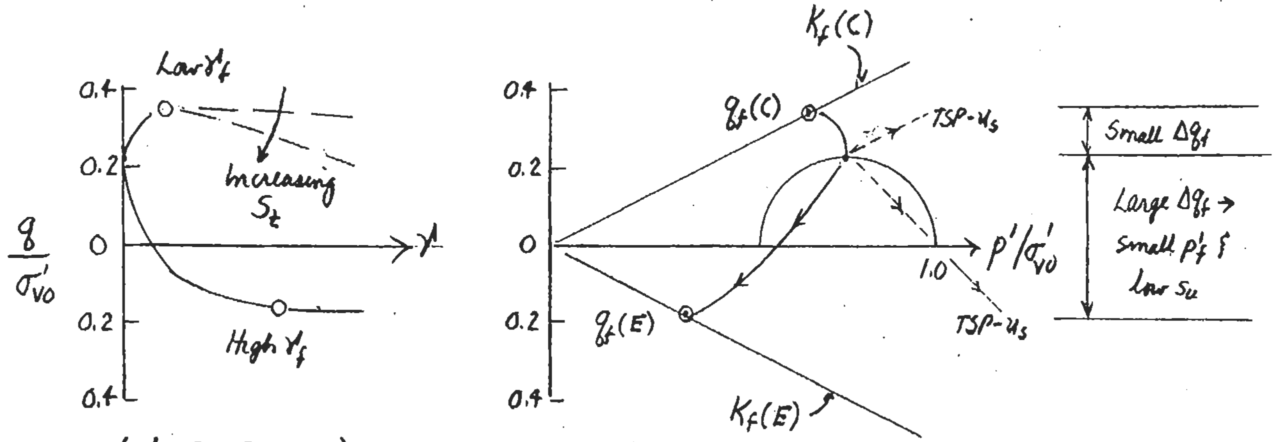
- Element D: Usually modeled via Direct Simple Shear (DSS)
  - Wire reinforced rubber membrane → $K_0$ consolidation & uniform $\gamma$




Vary $\sigma'_v$ to keep constant height & hence $\Delta V=0$

(1.2.1 Continued)

2) $CK_0UPSC/E$ behavior for OCR ≈ 1




$(\gamma = \varepsilon_1 - \varepsilon_3 = 2\varepsilon_1)$

3) Equations for $q_f/\sigma'_{vc} = f(K_c, A_f$ & $\phi')$: See Sheet A for derivation

$$\frac{q_f(C)}{\sigma'_{vc}} = \frac{[K_0 + (1-K_0)A_f]\sin\phi'}{1+(2A_f-1)\sin\phi'} \quad ; \quad A_f = \frac{\Delta u - \Delta\sigma_h}{\Delta\sigma_v - \Delta\sigma_h}, \text{ since } \Delta\sigma_h = \Delta\sigma_3$$

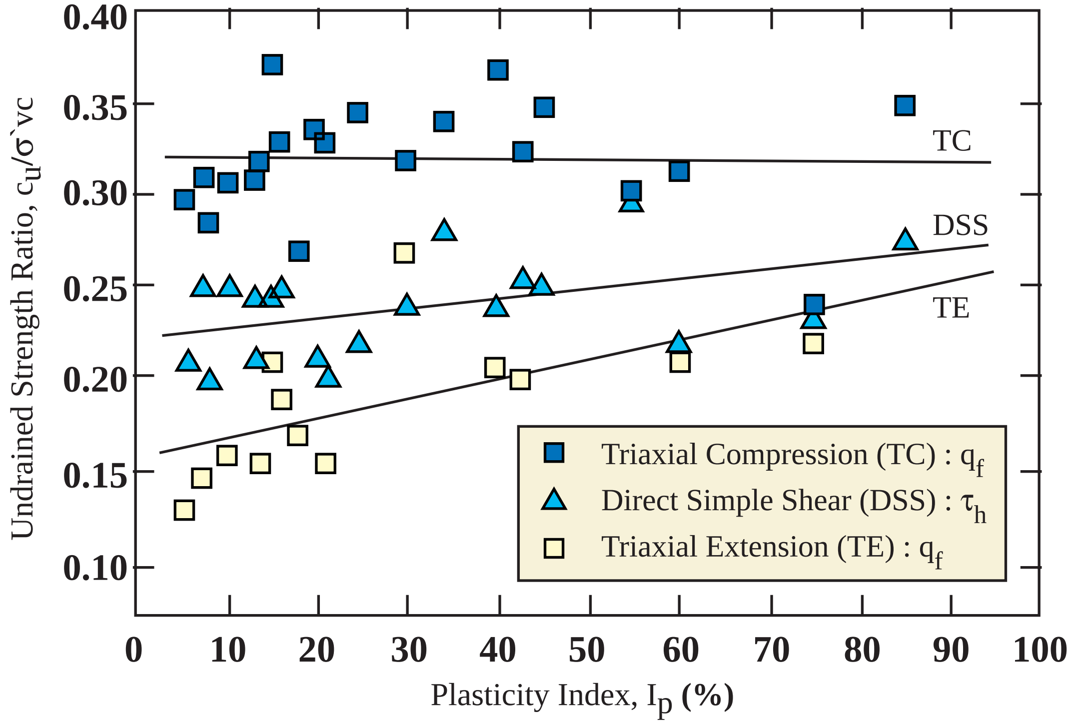
$$\frac{q_f(E)}{\sigma'_{vc}} = \frac{[1-(1-K_0)A_f]\sin\phi'}{1+(2A_f-1)\sin\phi'} \quad ; \quad A_f = \frac{\Delta u - \Delta\sigma_v}{\Delta\sigma_h - \Delta\sigma_v} \text{ since } \Delta\sigma_v = \Delta\sigma_3$$

- Eqn. valid for $K_c \neq K_0$ and both plane strain & triaxial testing
- For $K_0 = 0.50$ and constant $\sin\phi' = 0.50$ & $A_f = 1$

  $q_f(C)/\sigma'_{vc}$ = $\qquad$ $q_f(E)/\sigma'_{vc}$ = $\qquad$ $K_s = q_f(E)/q_f(C)$ =

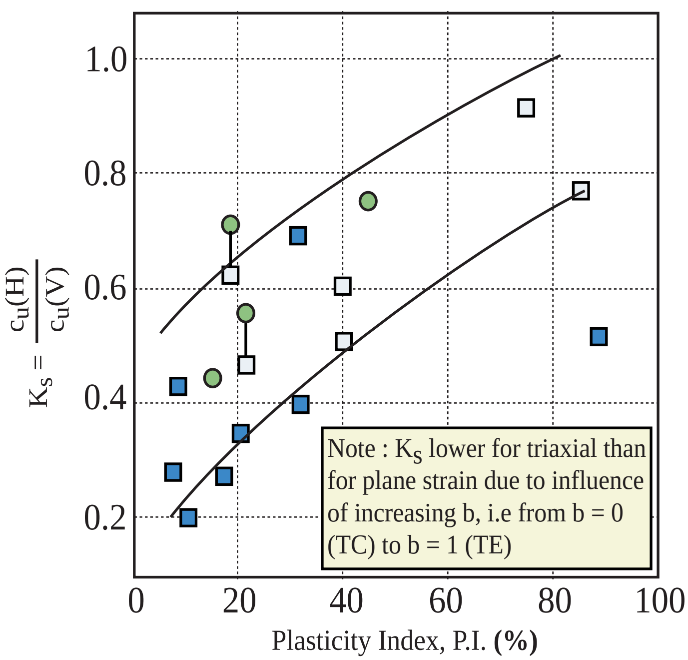
(Note: In general, both $\phi'$ & $A_f$ may vary with increasing δ and b)

4) See Fig. V 4-1 for experimental data on low OCR soils → anisotropy most important with low $I_p$ soils, esp. if high $S_t$




**Undrained Strength Anisotropy from $CK_oU$ Tests on Normally Consolidated Clays and Slits**

Adapted from Ladd (1991)



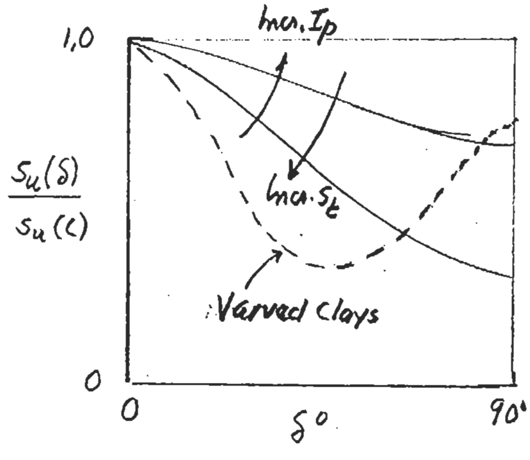

| TE / TC | PSE / PSC | Stress History | Reference |
|---|---|---|---|
| □ | ● | $\sigma'_{vc} \geq 1.5 - 2 \times \sigma'_{vm}$ | Table 1 Fig. 22, MIT and NGI |
| ■ | | $\sigma'_{vc} = \sigma'_{vo}$ and $\sigma'_{vm} / \sigma'_{vo} = 1.15 - 1.8$ | Berre and Bjerrum, (1973) |

**Data on Undrained Strength Ratio Anisotropic of Low OCR Cohesive Soils $C_u = S_u$**

Adapted from Ladd et al. (1977)

(1.2.1 Continued)

5) Average $s_u$ for stability analyses to account for anisotropy




—— Non-layered cohesive soils

- - - Varved clay (special soil)

$K_s \approx 0.9 \rightarrow 0.4$ (Fig. 23, p4)

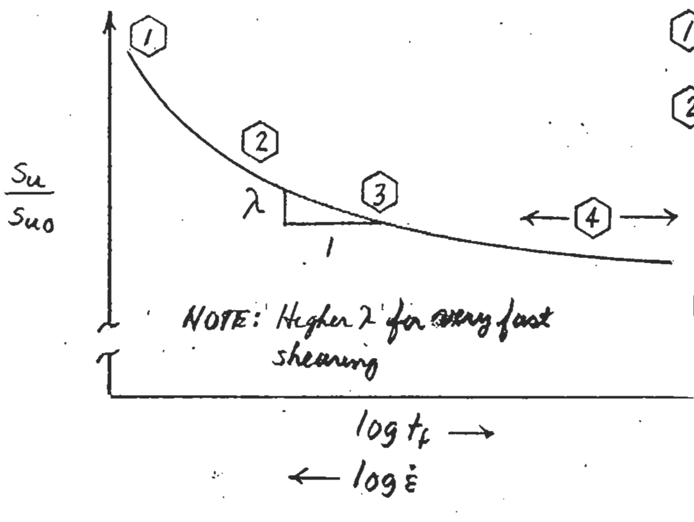
For linear $s_u(\delta)$ vs. $\delta$:

$$s_u(\text{Ave}) = \frac{1}{2}\left[s_u(C) + s_u(E)\right]$$

## 1.2.2 Strain Rate (Time to Failure)

1) General trends

• $\lambda = \dfrac{(\Delta s_u / s_{uo})}{\Delta \log t_f}$, where $s_{uo} = s_u$ at reference $t_f$ (or $\dot{\varepsilon}$)




① CPT & Lab TV, FC, PP : $t_f \approx$ seconds

② FVT & Lab UUC ($\dot{\varepsilon} = 1\%/\text{min}$) : $t_f \approx$ minutes

③ Lab CU ($\dot{\varepsilon} \approx 0.5\text{-}1\%/\text{hr}$) : $t_f \approx$ hours

④ Typical field loading : $t_f \approx$ days - weeks

2) Example of potential strain rate effects

| | | | | |
|---|---|---|---|---|
| Shearing Rate ($t_f$) | 5 sec. → ① | 5 mins → ② | 5 hr. → ③ | 2 weeks ④ |
| λ (%) : | 15-20 | 10 | 5 | |
| $\frac{\Delta s_u}{s_{uo}}$ (%) : | 27-36% | 18% | 9% | |
| Approx $\frac{\text{Measured } s_u}{\text{Design } s_u}$ | X1.6 | X1.3 | X1.1 | Reference |

∴ In situ CPT & FVT & lab "UU" type tests can overpredict appropriate design $s_u$ by 30-60% (based only on $\Delta t_f$)

(1.2.2 Continued)

3) Remarks on factors affecting strain rate effects:

- Most important with testing → very low $t_f$ (seconds → minutes)
- For $CK_oU$ testing, $\dot{\varepsilon}_a \approx 0.5-1\%/hr$ accepted practice
  $\dot{\gamma} \approx 5\%/hr$ " "
- $\lambda$ tends to be highest for soils with – high plasticity
  – high organic content
  – low OCR

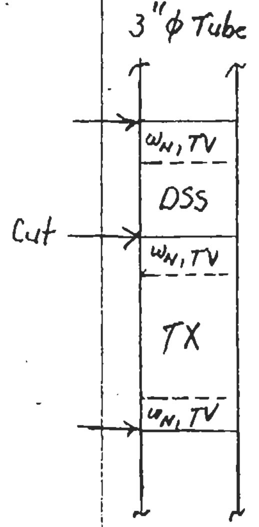
1.2.3 Sample Disturbance (Restricted to "undisturbed" tube samples)

1) Overview of influence of increasing disturbance → lower $\sigma_s'$

- For UU type testing, lower $\sigma_s'$ → lower $p_f'$ → lower $s_u$
- For CU type testing, lower $\sigma_s'$ → lower $w_c$ & $w_f$ (higher $w_N - w_f$) → higher $s_u$

2) Factors typically causing increased disturbance (lower $\sigma_s'/\sigma_{vo}'$)

- Use of small dia. (< 3") push samples rather than ≥ 3" dia. fixed piston samples
- Use of lightweight drilling fluid that can cause extension type failure of soil at bottom of hole prior to sampling (esp. in low OCR soils): USE HEAVY WEIGHT MUD → in situ $q \approx 0$ (see Part V-4, p8)
- Extrusion of bonded soil from tube; should cut tube and run piano wire around circumference prior to extrusion




3) Should radiograph tubes before testing:

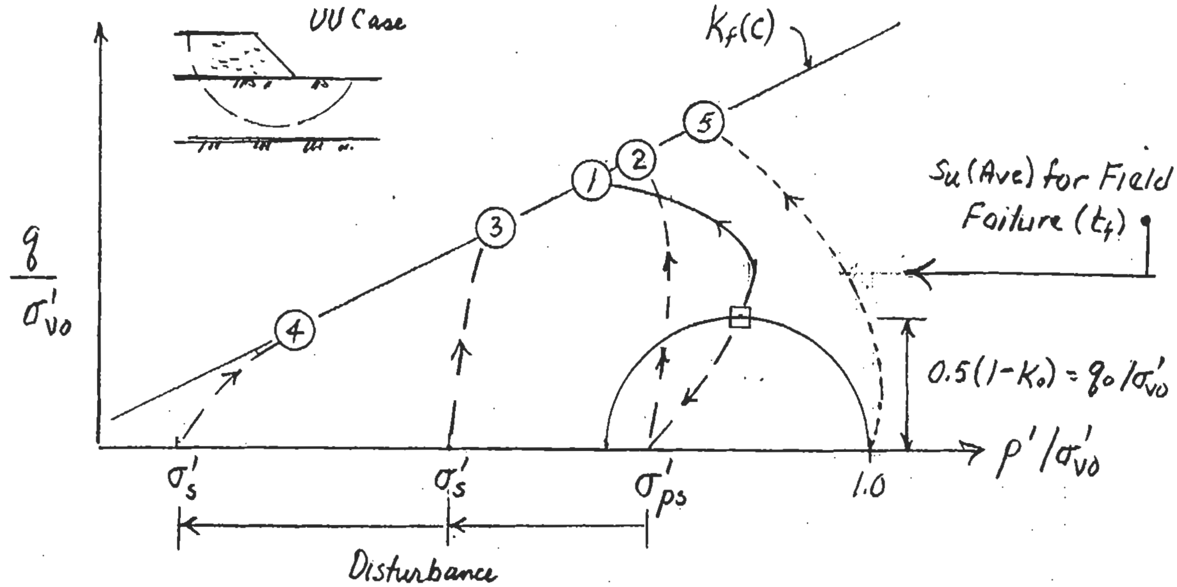
- Identify zones of excessive disturbance
- " changes in soil type
- Presence of shells, stones, etc.

} Then pick best quality, most representative sections for testing

# 2. EVALUATION OF COMMON METHODS OF ESTIMATING $s_u$

## 2.1 Laboratory UUC Tests (Illustrated for low OCR soil)

1) Trends




(1) In situ $s_u(C)$ at slow $\dot{\varepsilon}$ : Is this $s_u$ good for design? Discussion →

(2) Lab UUC at Std. $\dot{\varepsilon}$ on Perfect Sample (Undrained release of in situ $q_o \rightarrow \sigma'_s = \sigma'_{ps}$). Why is $s_u$② > $s_u$①?

(3) Lab UUC at Std. $\dot{\varepsilon}$ on sample with small disturbance. ($s_u$ too high)

(4) " " " " " " " " large " ($s_u$ too low)

2) Conclusions

Use of UUC testing to estimate design $s_u$ depends on uncontrolled compensating errors:

a) Increased $s_u$ due to
- neglecting anisotropy, since $s_u(C) > s_u(Ave)$
- too fast shearing since $t_f$ << field

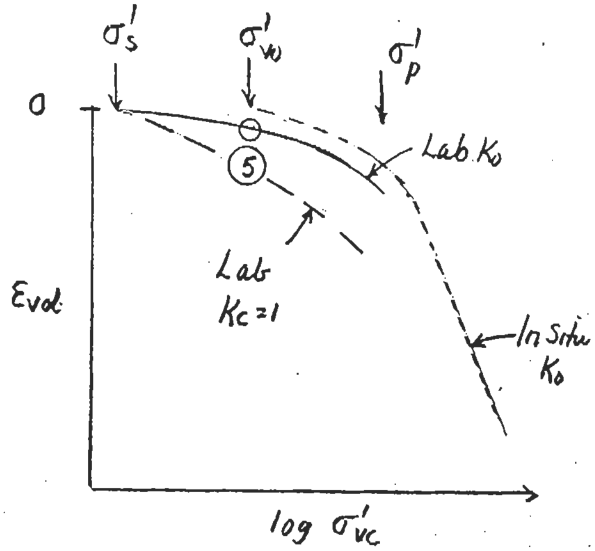
b) Decreased $s_u$ due to sample disturbance that reduces $\sigma'_s$

∴ Depending on luck for $+\Delta s_u(a) = -\Delta s_u(b)$

## 2.2 Laboratory Recompression CU Tests

### 2.2.1 Conventional CIUC Tests ($\sigma'_c \approx \sigma'_{v0}$)

Illustrated for low OCR soil




(5) CIUC reconsolidated to $\sigma'_c = \sigma'_{v0}$ → significant volume decrease (unless in situ $K_0 \gtrsim 1$).

- See p7 for resultant ESP (even with slow $\dot{\varepsilon}$)

**Measured $s_u$ is UNSAFE**

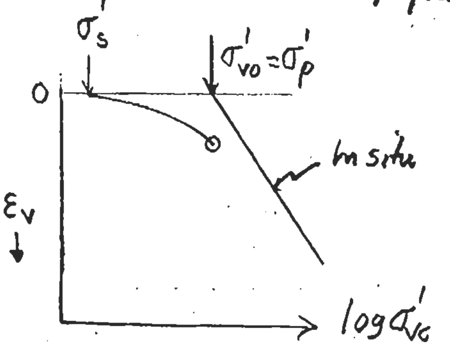
- Too high due to $w_f \ll w_N$

* Too high since $s_u(C) > s_u(Ave)$ due to anisotropy

### 2.2.2 Recompression $CK_0U$ Tests (TC, DSS & TE)

1) Accounts for 3 principal factors by:
   - Using reconsolidation to $\sigma'_{v0}$ & est. $\sigma'_{h0}$ (pt. ○) < Minimizes $-\Delta w$; Correct initial stresses
   - Shearing with different modes to measure $s_u$ anisotropy
   - Using slow strain rate ($\dot{\varepsilon}_a \approx 0.5-1\%/hr.$ for TX)

2) See Section 4.4 of Ladd (1991) for further details
   - Is recommended for "highly structured" clays (high $I_L$ & $S_t$) if have good quality samples
   - But should not be used when in situ OCR = 1 since will → significant decrease in volume → $w_f < w_N$ → $s_u$ too high



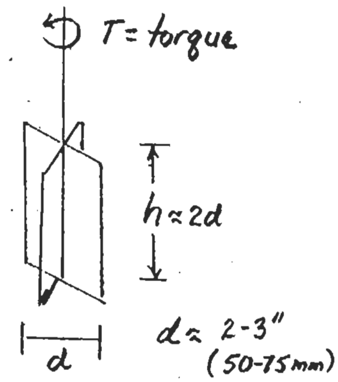

**Recompression UNSAFE for In situ OCR = 1**

## 2.3 Field Vane Test (FVT)

### 2.3.1 Procedures and Discussion

1) Test equipment & procedures (ASTM D 2573)

- Although ASTM allows tapered ends, CCL recommends square ends
- Standard $d\theta/dt = 6°/\text{min}$ which requires gear system (hand torque wrench → too low $t_f$)
- For rectangular blades




$$s_u = \frac{T}{\pi\left(\frac{d^2h}{2} + \frac{d^3}{6}\right)}$$

Assumes same $s_u$ acts on all surfaces

Geonor : casing
Nilcon : rod

2) Discussion

- Small $t_f$ (minutes) → $s_u$ too high
- Disturbance with thick blades → $s_u$ too low
- Very unusual mode of failure (i.e., cylindrical rotation on vertical surface) → $s_u$ that is difficult to interpret. (Some believe that $s_u$ is near $s_u$ for DSS mode)

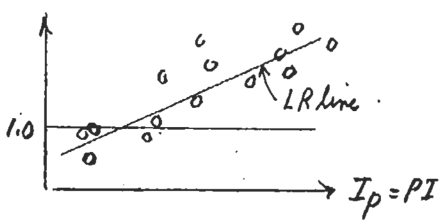
∴ Must view as "strength index" test (see 2.3.2)

∴ See Section 4.2 for measuring spatial variations in $s_u$ and stress history (OCR)

### 2.3.2 Bjerrum's FV Correction Factor

1) Bjerrum (1972) evaluated 14 case histories of embankment failures (F=1)

$$F = \frac{s_u(FV)}{\tau_m}$$




In situ $s_u = \tau_m = \frac{s_u(FV)}{F} = \mu\, s_u(FV)$

where $\mu$ = Bjerrum correction factor = 1/F

(2.3.2 Continued) $c_u \equiv s_u$

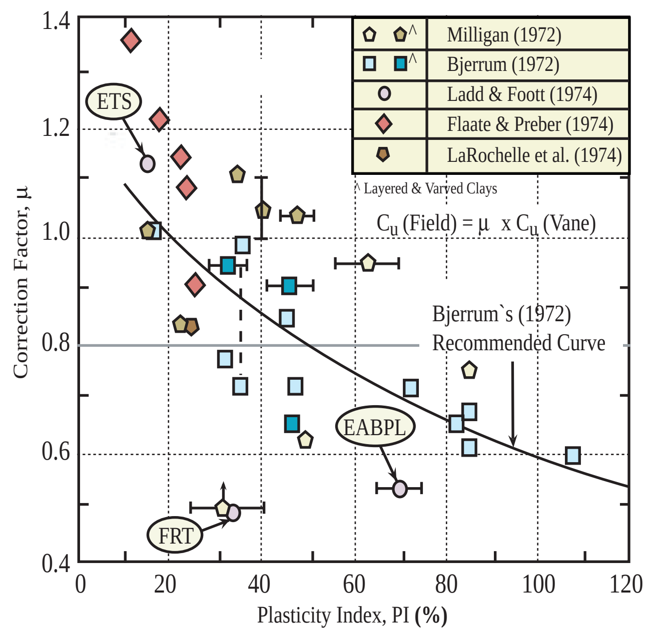
2) Discussion

- See Fig. 51 for plot of $\mu$ vs PI
- See Sheet © for more "precise" plot

NOTE: $\mu \approx 1 - 0.5 \log\left(\frac{PI}{20}\right)$ PHT (1974) (PI = 20–80%)

- Coef. of variation (COV) decreases w/ increasing PI from ≈ 20% to ≈ 10% (See Note)
- Use of design $s_u = \mu\, s_u(FV)$ most reliable of all in situ tests EXCEPT when soil contains – excessive shells or sand lenses } e.g. FRT – also fibrous peat

Note: Linear regression on $F[s_u(FV)]$ vs $I_p$ → SD = ±0.19 for n = 29 (excludes two cases).



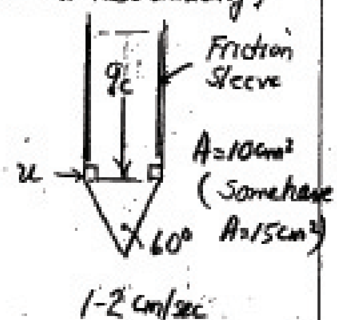

**Field vane correction factor vs. plasticity index derived from embankment failures (Ladd, 1975).**

{Low $I_p$: $\mu > 1$ due mostly to anisotropy}
{High $I_p$: $\mu < 1$ " " " fast shearing}

## 2.4 Cone Penetration Test (CPT)

1) Procedures (see Section 3.8 of Part III-4)

* Note that must measure u to get reliable $q_t = q_c + u(1-a)$ in low OCR clays with electric cones ($a \approx 0.7 \pm 0.05$)



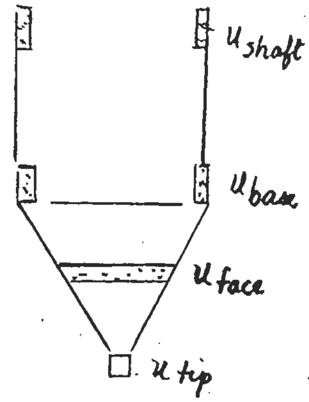

2) Interpretation

- $s_u = \frac{(q_t - \sigma_{v0})}{N_{kt}}$, where $N_{kt}$ is empirical cone factor derived from correlating $(q_t - \sigma_{v0})$ vs reference $s_u$
- $N_{kt} \approx 14 \pm 5$ for medium to low OCR cohesive soils (COV ≈ 35%) based mostly on using reference $s_u = \mu\, s_u(FV)$

( • For perspective, early correlations using $s_u$ from UUC and measured $s_u(FV)$ produced cone factors ranging from 5 to 70! )

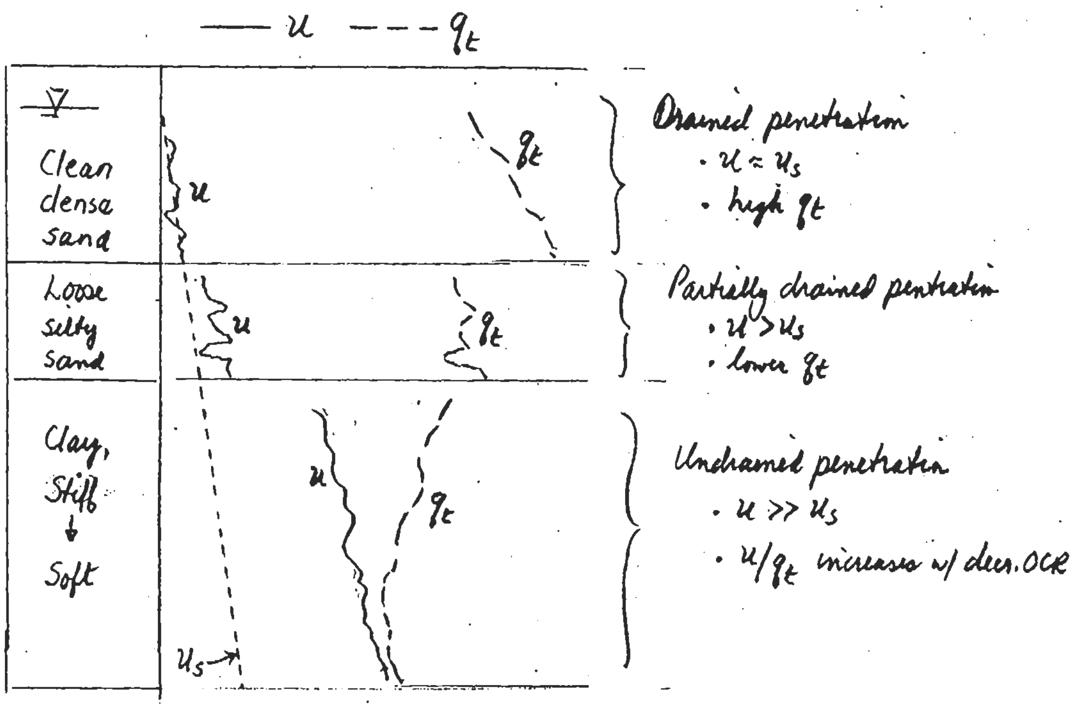
## 2.5 Piezo-Cone Penetration Test (CPTU)

1) Procedure




- Same as CPT, but with porous stone added to measure $u$ during penetration (also $du/dt$ when stop → $c_h$ plus est. of equil. $u$)
- Location of $u$ has not been standardized
  - $u_{shaft}$ = not recommended
  - $u_{base}$ = most common } → $q_t$, but large gradient in $u$ (AJW)
  - $u_{face}$ = 2nd most common
  - $u_{tip}$ = best for identifying soil type, but prone to damage.

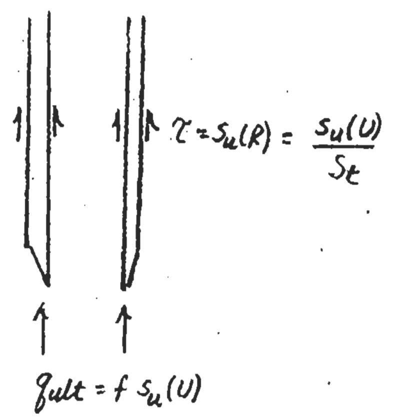
2) Use for soil profiling (soil stratigraphy): Conceptual




CPTU should be part of all major site characterization programs, but unfortunately available equipment and data reduction often not of high quality

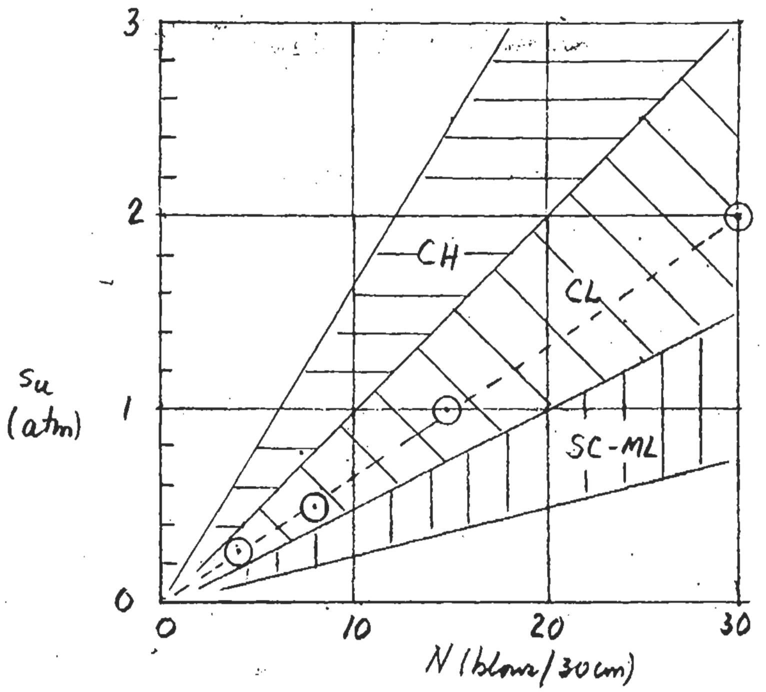
## 2.6 Standard Penetration Test (SPT N Values)

1) N during undrained penetration




- N mainly controlled by $s_u$(R = remolded) and hence very poor measure of $s_u$ in low OCR soils, especially those with moderate - high $I_L$ (→ incr. $S_t$)
- Can get N = 0 (WOR = weight of rod or WOH = weight of hammer) in low OCR clays even at large depths (e.g. z > 25m for Boston Blue Clay)

2) Example of correlation




⊙ T&P (1967) Table 45.2

$s_u(atm) \approx \frac{N}{15}$ ← Range = 6-40!

Sowers (1979) 4th Edition Fig. 7.10(b) "Saturated soils"

CCL recommendation: use only in high OCR clays as last resort

# 3. SHANSEP DESIGN METHOD

Acronym for "Stress History And Normalized Soil Engineering Properties"
(Ladd & Foott, 1974, ASCE JGED, 100(7), 763-786)

## 3.1 Background

- Developed at MIT during the 1960s to provide a more rational & reliable method for estimating $s_u$ (and stress-strain data) that accounts for the effects of sample disturbance, anisotropy and (to lesser degree) strain rate effects.
- Based on the experimental observation (lab & field) that the normalized undrained stress-strain-strength behavior of most "ordinary" clays is controlled by the stress history (OCR) of the soil (for a given mode of shearing), e.g. $s_u/\sigma'_{vo} = S(OCR)^m$ That is, "ordinary" cohesive soils behave like the Technology Clay
- "Ordinary" = deposits with $I_L \lesssim 1$ and $\sigma'_p$ mainly caused by mechanical, desiccation and aging mechanisms, NOT cementation

## 3.2 Procedure

### 3.2.1 Overview (see Fig V4-2 for example design problem, p14)

1) Objectives are to develop profiles of $s_u$ for the initial in situ (virgin) Condition (UU Case) and as $f(\sigma'_{vc})$ for the CU Case

2) Three steps:

   i) Evaluate the stress history (both initial & during construction)

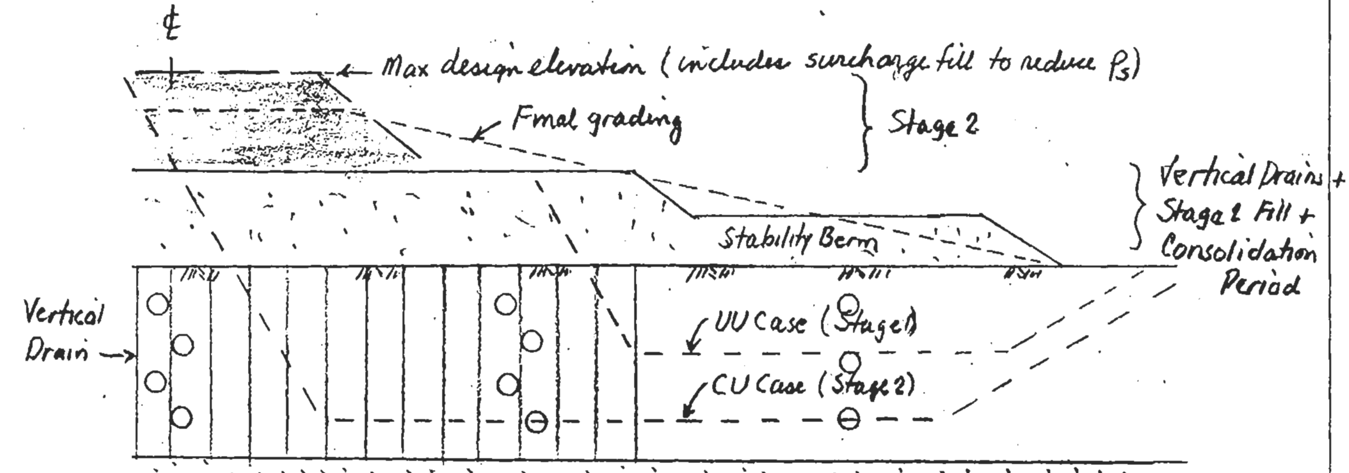
   ii) Conduct $CK_oU$ tests with varying failure modes and OCR on specimens reconsolidated beyond the in situ $\sigma'_p$ → values of S & m.

   iii) Apply the SHANSEP eqn, $s_u/\sigma'_{vc} = S(OCR)^m$, to compute profiles of $s_u$ for UU & CU Cases as follows [for initial $s_u(D)$]

| El. | $\sigma'_{vo}$ | $\sigma'_p$ | OCR | $s_u(D)/\sigma'_{vc}$ * | $s_u(D) = \sigma'_{vo} \times s_u(D)/\sigma'_{vc}$ |
|---|---|---|---|---|---|

* from selected $S_d$ & $m_d$

a) Design Problem




O = Piezometers; see Sect. 7 of Ladd ('91) for other field instrumentation

b) Soil Profile and Results of SHANSEP Analysis

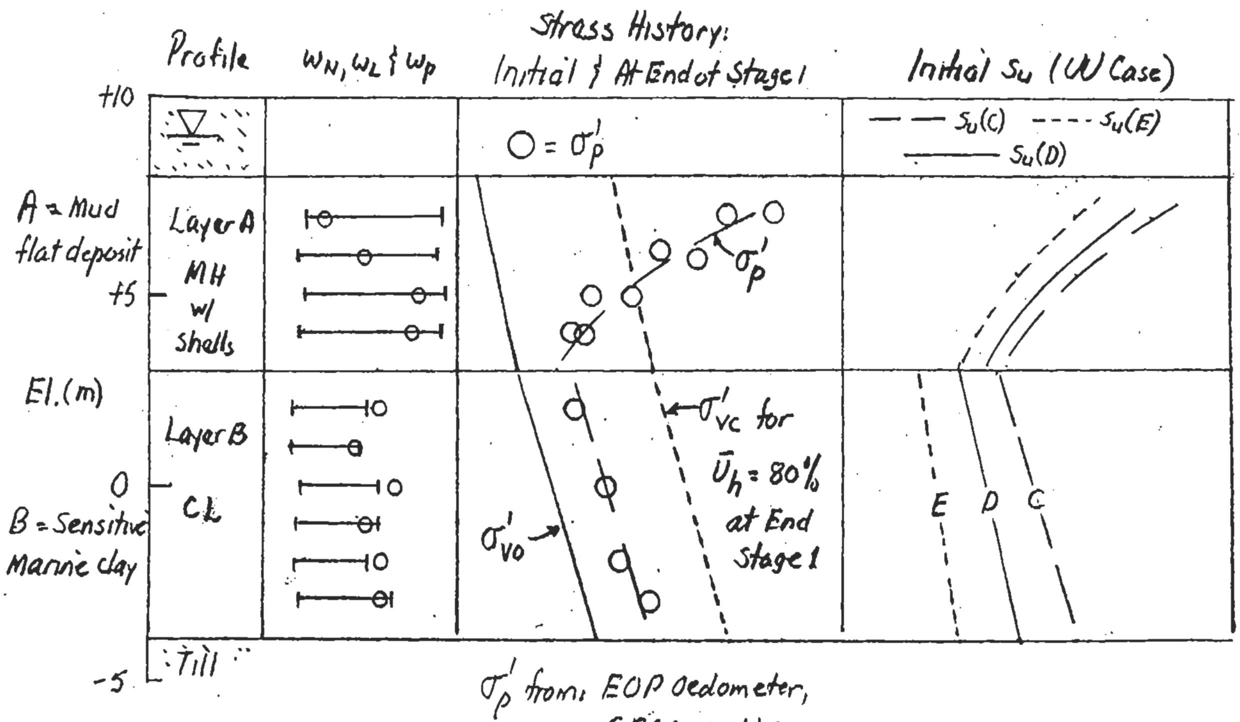



Fig. V4-2 Illustration of SHANSEP Design Method Applied to Staged Construction with Vertical Drains of Bridge Approach Embankment

NOTE: [ ] = sections in Ladd (1991), Terzaghi Lecture.

3.2.2 Evaluation of Stress History [4.2, 5.2]

1) Initial $\sigma'_{vo} = \sigma_{vo} - u_s$ : piezometers or CPTU with dissipation tests to check/measure $u_s$

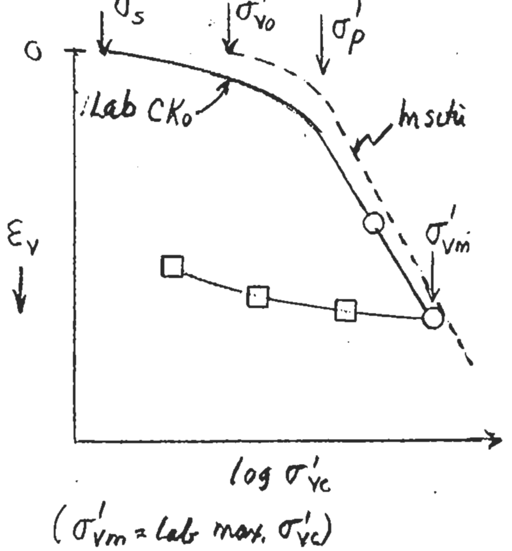
2) Initial stress history = profile of $\sigma'_p$ and OCR = $\sigma'_p/\sigma'_{vo}$
   - Incremental oedometer tests → EOP compression curves; may need to use lower LIR near $\sigma'_p$
   - Constant rate of strain consolidation (CRSC) tests have advantage of giving continuous compression curve (also $c_v$ and $k_v$)
   - From consolidation phase of $CK_oU$ SHANSEP test program
     - With automation, can get excellent $\sigma'_p$ data from both triaxial & DSS tests

   * See Section 4.2 for use of in situ testing to interpolate/extrapolate lab values of $\sigma'_p$

3) Profiles of $\sigma'_{vc}$ for partial/full consolidation
   - For design, use consolidation analyses [5.2]
   - During construction, use piezometers [7.2]

3.2.3 $CK_oU$ Test Program to Obtain NSP [4.3-4.8, esp. 4.4; 5.3]

1) $K_o$ consolidation to $\sigma'_{vm}/\sigma'_p \geq 1.5-2$




($\sigma'_{vm}$ = lab max. $\sigma'_{vc}$)

a) For OCR = 1 tests (minimum $\sigma'_{vm}/\sigma'_p \geq 1.5-2$) ← To obtain NC behavior; MIT now uses $\varepsilon_v \approx 10\%$ to ensure that on VCL
   - Can vary $\sigma'_{vm}/\sigma'_p$ to verify normalized behavior (e.g., constant $s_u/\sigma'_{vc}$)
   - For S of virgin OC soil, use $t_c \approx 10 t_p$, i.e., induce aging to "restore" initial structure
   - For S of strengthened NC soil, use $t_c \approx t_p$, i.e., no aging

b) For OCR > 1 tests
   - Use $t_c \approx 10 t_p$ at $\sigma'_{vm}$
   - Vary OCR over range of initial in situ OCR

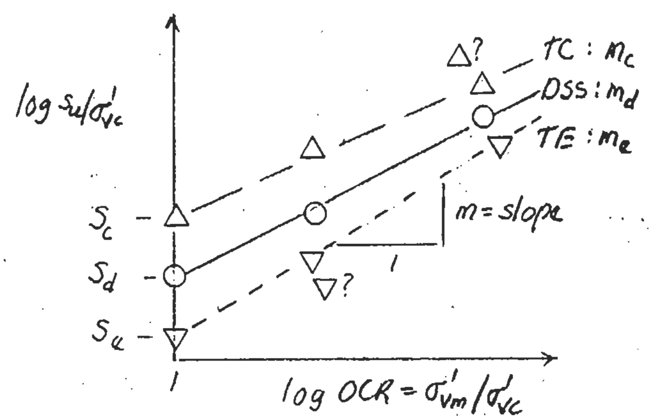
(3.2.3 Continued)

2) Selection of mode of shearing and strain rate

- For stability analyses using anisotropic $s_u$ profiles, run PSC/TC, DSS and PSE/TE
- For stability analyses using isotropic $s_u = s_u(\text{Ave})$ profiles, run DSS or TC & TE with $s_u(\text{Ave}) = \frac{1}{2}[s_u(C) + s_u(E)]$
- For strain rate, usual good practice → TX $\dot{\varepsilon}_a = 0.5-1\%/hr$ and DSS $\dot{\gamma} = 5\%/hr$. With highly rate sensitive soils (e.g. plastic, organic soils), can vary $\dot{\gamma}$ during DSS tests (covered in 1.322)

3) Evaluation of $s_u/\sigma'_{vc}$ vs OCR data → $s_u/\sigma'_{vc} = S(OCR)^m$ = SHANSEP equation

a) Plot log $s_u/\sigma'_{vc}$ vs log $OCR = \sigma'_{vm}/\sigma'_{vc}$ : CCL uses LR → values of S & m



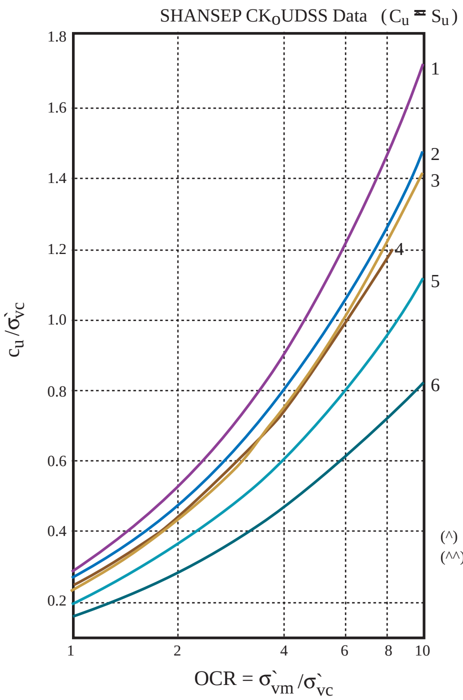

b) Compare results with data on other cohesive soils (see p17)

- Fig. 25 → very consistent pattern of data from $CK_0UDSS$ tests. Note very low $s_u/\sigma'_{vc}$ for CVVC, Soil ⑥
- Fig 26 → $m \approx 0.8 \pm 0.05$, except CVVC (should have used log-log plot)
- Values of m may vary with mode of shearing à la Fig. 16 (p17) & Fig V4-3 (p19a)

3.2.4 <u>Selection and Application of SHANSEP Design Parameters</u>

1) Refer to TL Sections 4.6 & 4.9 (and 1.322) for:

- Possible adjustment of TC/TE data to PSC/PSE conditions
- Use of strain compatibility method to account for effects of progressive failure caused by strain softening



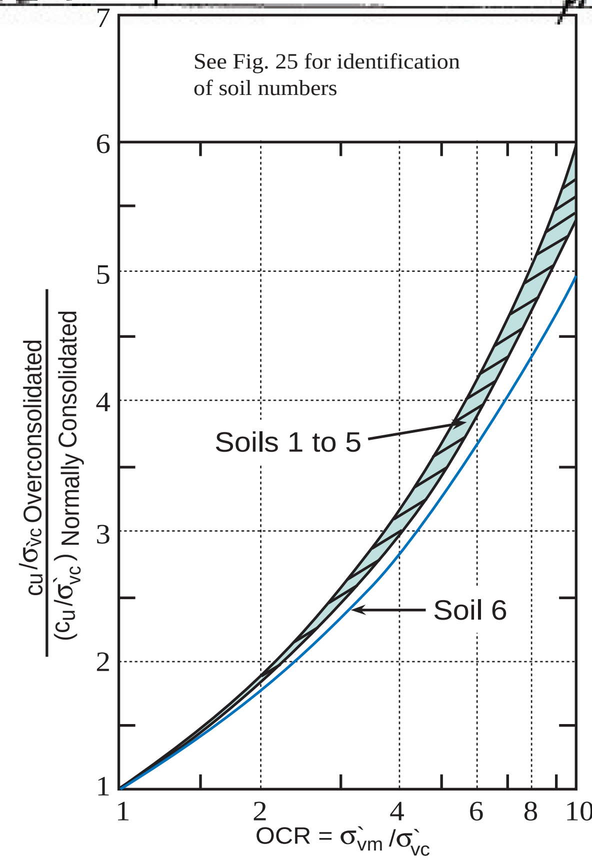

1. Maine Organic
2. Bangkok Clay
3. Atchafalaya Clay
4. AGS CH Clay
5. Boston Blue Clay
6. Conn. Valley Varved Clay

| Soil no. | PI(%) | LL(%) |
|---|---|---|
| 1. | 34 | 65 |
| 2. | 41 | 65 |
| 3. | 75 | 95 |
| 4.^ | 41 | 71 |
| 5. | 21 | 41 |
| 6.^^ | 39<br>12 | 65<br>35 |

(^) Unpublished data by MIT (1974)
(^^) "Clay" & "Silt" layers

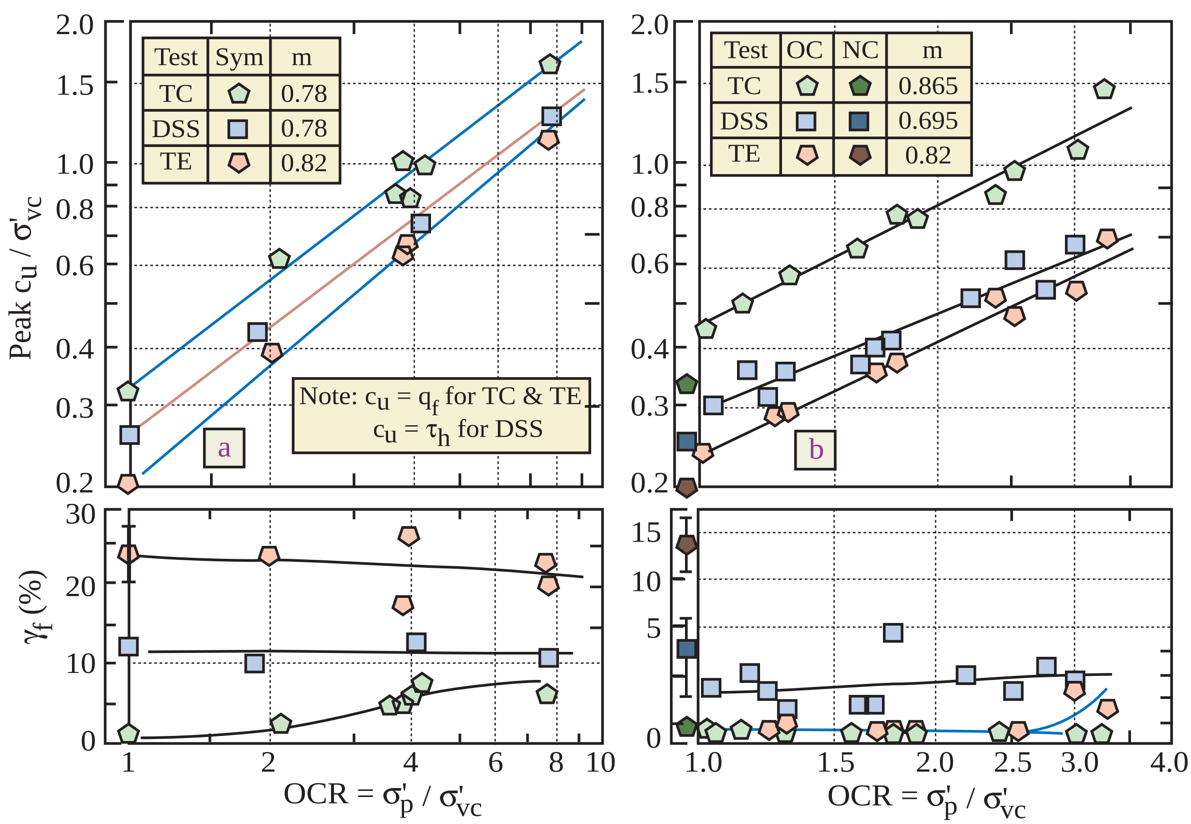
**Undrained strength ratio vs OCR from $CK_0U$ direct simple shear tests on six clays (Ladd and Edgers, 1972).**

Adapted from Ladd et al. (1977) son, 9th ICSMFE




Relative increase in undrained strength ratio with OCR from $CK_0U$ direct simple shear tests (replot of data in Fig. 25)

Adapted from Ladd et al. (1977) son, 9th ICSMFE




**OCR vs. Undrained Strength Ratio and Shear Strain at Failure from $CK_0U$ Tests: (a) AGS Plastic Marine Clay via SHANSEP and (b) James Bay Sensitive Marine Clay via Recompression [B-6 Data from Le-febvre et al. (1983)]**

Adapted from Ladd (1991)

(3.2.4 Continued)

2) Initial $s_u$ for virgin ground (illustrated in Fig V4-2)

$s_u = \sigma'_{vo} S (OCR)^m$, where $OCR = \sigma'_p / \sigma'_{vo}$ and different values of S & m for C, D, E or $s_u$(Ave).

3) Increased $s_u$ during staged construction

- For NC soil (i.e., $\sigma'_{vc} > \sigma'_p$ in most of zone with $\bar{U}_h \approx 80\%$), use values of S from $CK_oU$ tests with $t_c \approx t_p$ (EOP consolidation)
- For soil that still remains OC (i.e., top zone within vertical drains and for soil under stability berm), use values of S & m from $CK_oU$ tests with $t_c \approx 10 t_p$.

  Note: Values of S will be ≈ 10% higher than for EOP tests

## 3.3 Discussion

### 3.3.1 Disadvantages of SHANSEP

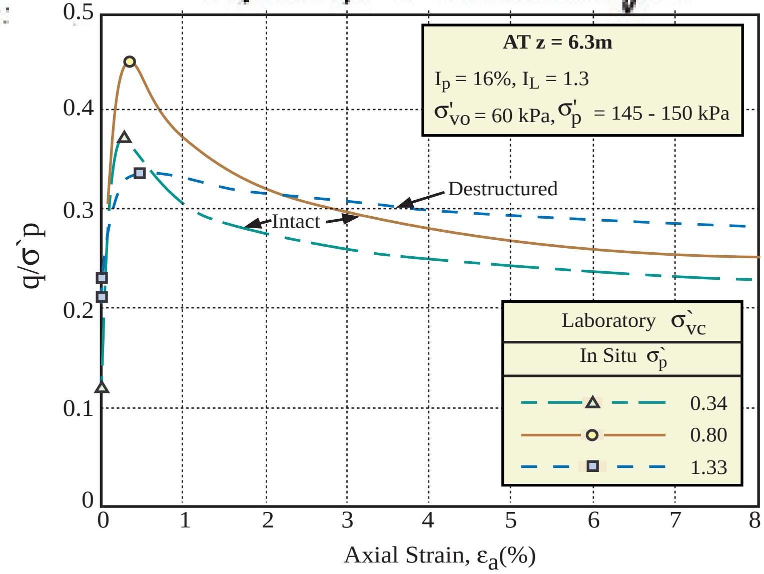
1) Requires reliable estimate of stress history profile(s) **
   - May require extensive lab testing and judgement
   - Always attempt to tie in with likely geologic history of deposit (1.38), e.g., glaciation, changes in sea level, migration of sand dunes, influence of artesian/pumping conditions, aging, erosion, etc. • Also see Section 4.2 (in situ testing for spatial variability)
2) Requires $CK_oU$ testing (often with varying modes of shearing, OCR & $t_c/t_p$)
   - More expensive & difficult than UUC, CIUC, etc.
   - However, can use trends on other soils to reduce scope of testing, e.g., restrict to OCR=1 tests and assume m=0.8 if virgin soil has low OCR

---

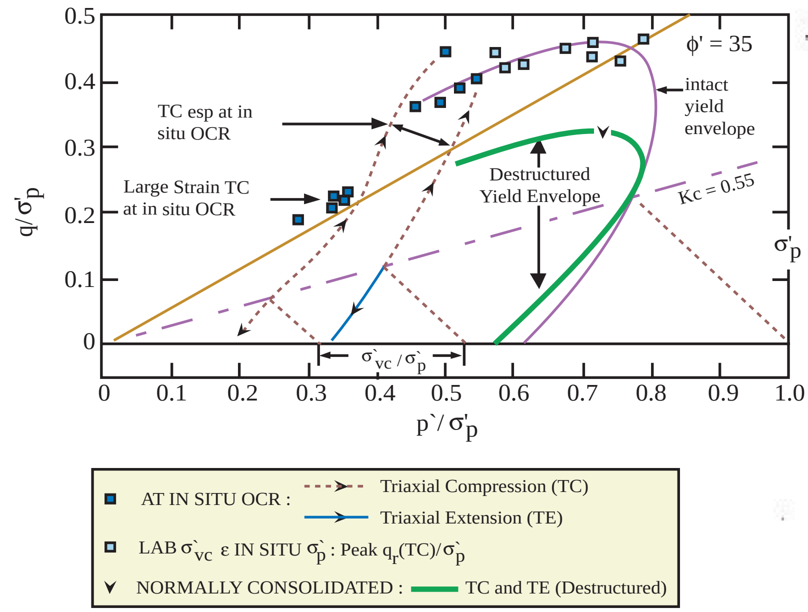
** With automated $CK_o$-DSS & TX testing, now obtain excellent 1-D compression curves → more reliable $\sigma'_p$ data → great advantage

(3.3.1 Continued)

3) Not applicable to "highly structured" clays ($I_L > 1$, $S_t > 10$ and usually cemented) since reconsolidation of such soils beyond $\sigma'_p$ → "destructuring".




**(a) Normalized Stress-strain Data From CkoUC Tests**




**(b) Normalized Effective Stress Paths and Yield Envelops**

Adapted from Jamiolkowski et al. (1985)

a) Discussion of Fig. 4 $CK_0UC$ data
- Intact, OC clay has much higher $q_f(C)/\sigma'_p$

b) Also see Fig. 16b (p17) for $CK_0U$ TC, DSS & TE data on same clay → NC $s_u/\sigma'_{vc}$ (solid symbols) much lower than S from Recompression tests ($\sigma'_{vc} = \sigma'_{vo}$) on OC, intact clay

c) Conclusions
- SHANSEP reconsolidation → values of S & m that are too low for intact clay (modulus even lower)

∴ Should use Recompression technique to get values of S & m

d) See TL Section 4.4 & 1.322 for further comparison of SHANSEP vs. Recompression $CK_0U$ parameters, especially regarding stress-strain behavior.

4) Not applicable to deposits with highly variable stress history, e.g. as often occurs within highly desiccated crusts.

5) See Fig V 4-3 for detailed comparison of SHANSEP and Recompression $CK_0U$ tests on natural BBC (clay with moderate structure below crust)

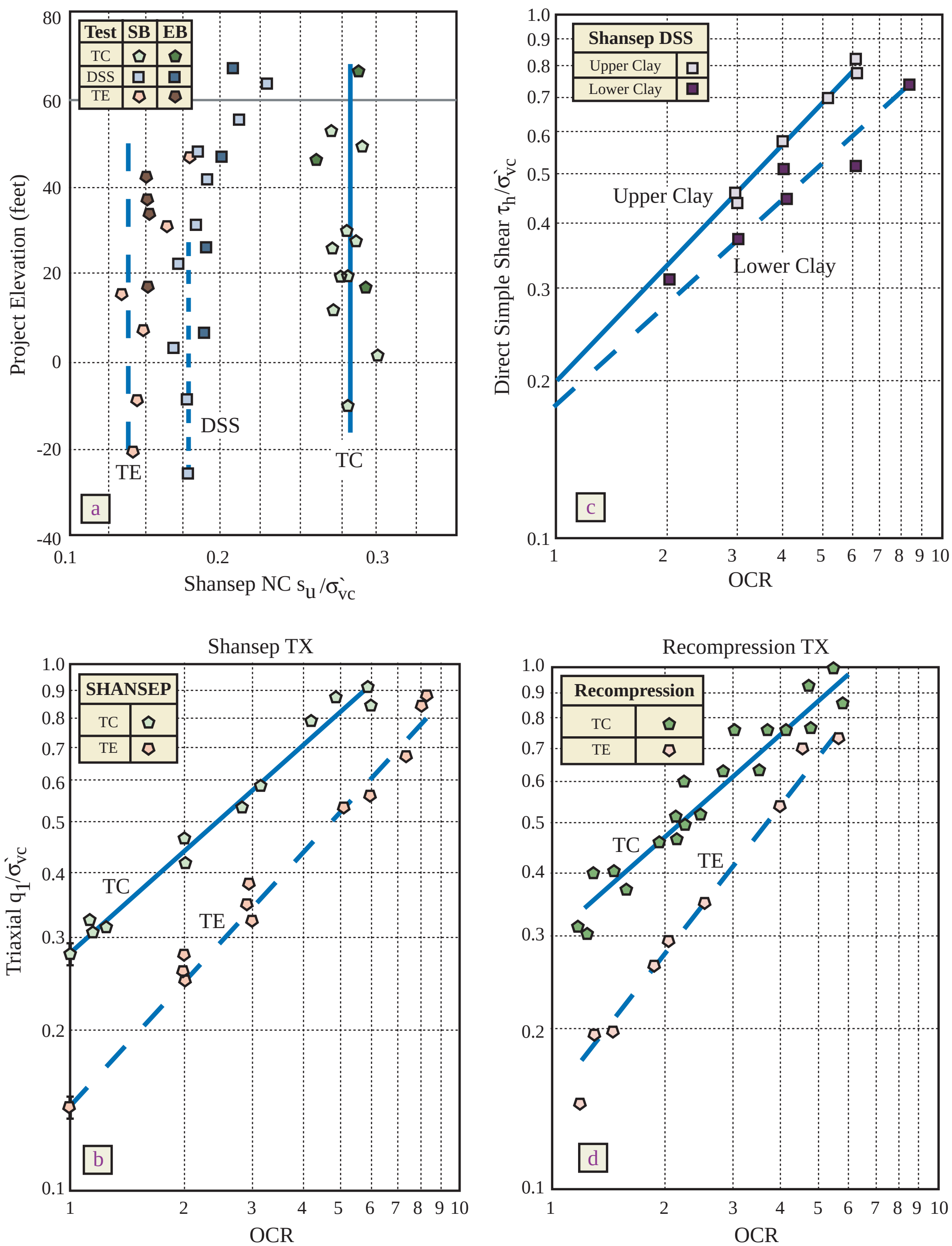
$CK_0U$ Testing Program

| CKoU Test (1) | Reconsolidation Technique | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Shansep | | | | Recompression | | | |
| | n (2) | S (3) | m (4) | COV (5) | n (6) | S (7) | m (8) | COV (9) |
| TC | 13 | 0.280 | 0.681 | 4.5% | 23 | 0.298 | 0.676 | 11.0% |
| TE | 17^ | 0.142 | 0.830 | 7.1% | 9 | 0.144 | 0.978 | 6.9% |
| DSS | | | | | | | | |
| Crust | 14^^ | 0.200 | 0.775 | 6.5% | — | — | — | — |
| Deep | 13^ | 0.180 | 0.660 | 7.4% | — | — | — | — |

^ For in situ OCR < 1.5
^^ For in situ OCR > 1.5

n = no. of tests
COV = Coef. of variation (%)

**Table 1. Normalized Undrained Strength Parameters from**




For Stress - Strain Behavior, Recompression has a "stiffer" response, i.e.
> Lower $\varepsilon_f$, especially for TE & at higher OCR
> Higher $E_{u50}/\sigma'_{vc}$ at OCR > 2, especially for TE

For Values of S & m, Recompression (compared to SHANSEP) Leads to:
> TC - Slightly higher $S_c$ & same $m_c$
> TE - Same $S_e$ & much higher $m_c$

**Normalized Undrained Strength Data from SHANSEP and Recompression $CK_0U$ Tests**

## Comparison of SHANSEP and Recompression CKoU Tests on Natural Boston Blue Clay (Ladd et al. 1998, ASCE GSP 91, 1-24)

### 3.3.2 Advantages of SHANSEP

1) Forces engineer to evaluate the stress history at the site, which is needed to "understand" the nature of the deposit and to tie in with its geologic history

2) With automation, $CK_0$-TX & DSS testing gives continuous 1-D compression curves during consolidation phase → superior values of $\sigma'_p$. (Also get $K_{ONC}$ and estimate of in situ $K_0$).

3) Should provide less scattered and more reliable estimates of $s_u$ for design than obtained via conventional practice based on lab UUC & CIUC tests; much more reliable than $s_u$ from CPTU; usually more reliable than $\mu\, s_u$(FV). Sheet B →

4) Can quantify uncertainty in estimated $s_u$ due to scatter and bias in stress history and values of S & m

- Eq. 3.3 $COV^2[s_u] = COV^2[S] + m^2 \cdot COV^2[OCR] + \ln^2 OCR \cdot SD^2[m]$
  where COV = coef. of variation = Std. Dev. (SD) / mean

- This allows engr. to use a reliability analysis to more rationally select an appropriate F of S for design: Computed $\beta$ → predicted nominal probability of failure ($P_f$); $\beta$ = reliability index

$$\beta = \frac{E[F]-1}{SD[F]}$$ ; E[F] = best estimate of F of S; SD[F] = uncertainty in F of S

Normal Distribution

| $\beta$ | $P_f$ (%) |
|---|---|
| 1 | ≈15 |
| 2 | ≈2 |
| 3 | ≈0.15 |

- References: Christian, Ladd & Baecher (1994) ASCE, JGE, 120(2), 2180-2207
  Baecher & Ladd (1997), Trans. Res. Record 1582, 49-52

5) Can predict changes in $s_u$ due to changes in $\sigma'_{vc}$
   - Consolidation for loadings
   - Swelling for excavations

6) Obtain values of $K_0$, stress-strain data, $c'$ & $\phi'$, etc. for finite element analyses using effective stress soil models such as MIT-E3 & MIT-S1

7) Can reuse NSP data on other projects involving the same basic soil deposit (after estimating the stress history)
   - Offshore - Gulf of Mexico
     - Arctic sites
     - Venezuela
   - BBC - CA/T project
     - MIT campus
   - Atchafalaya Flood Control levees
   - Salt Lake City I-15 project

# 4. RECOMMENDED PRACTICE

## 4.1 Objectives

1) Evaluate spatial variability in initial stress history & hence $s_u$

2) Estimate initial in situ $s_u$ for UU Case

3) Predict changes in $s_u$ with consolidation for CU Case
(or effect of swelling on $s_u$ for excavations)

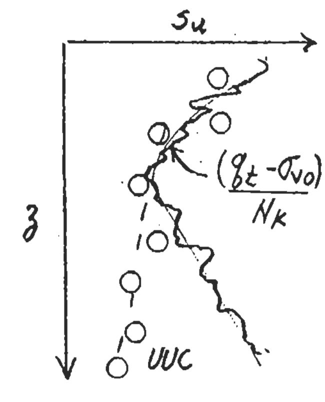


## 4.2 Spatial Variability

1) Should use in situ testing since

- Little or no influence from disturbance, whereas lab UU testing may → erroneous trends due to varying effects of sample disturbance (often poorer quality (lower $\sigma_s'/\sigma_{ps}'$) with increasing depth)

- More data at lower cost, especially via continuous penetration tests (CPT or CPTU)



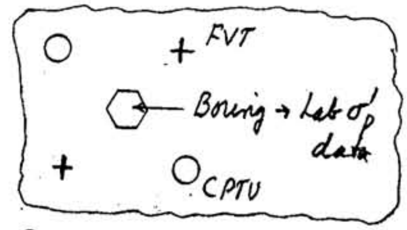

2) Recommendations

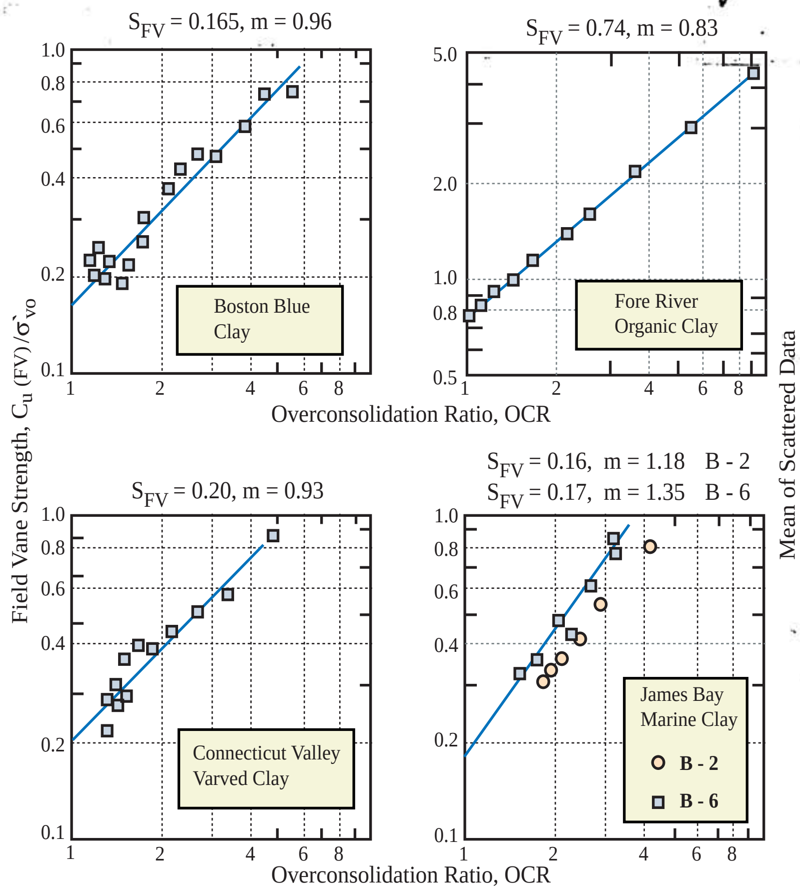
1st CPTU for all soil profiles (but check reliability of equipment)

2nd FVT for homogeneous soft clays, i.e. without excessive shells, sand lenses, etc.

2nd Marchetti dilatometer (DMT) for non-homogeneous soil profiles, e.g. containing layers of sandy soils

3) For major project covering large area, conduct special test program to develop site specific correlations between in situ tests and stress history




**Undrained Strength Ratio vs. OCR from Field Vane Tests [Lacasse et al. (1978) ] ;**

(a) Boston Blue Clay, I-95 Saugus ; MA
(b) Connecticut Valley Varved Clay, Amherst, MA ;
(c) Organic Clay with Shells, Fore River, ME;
(d) James Bay B-2 and B-6 Marine Clays
[Ladd et al. (1983)].

Adpated from Jamiolkowski et al. (1985) SOA 11th ICSMFE

Evaluate in situ test data using SHANSEP eqn.

$$\frac{s_u(FV)}{\sigma'_{vo}} = S_{FV}(OCR)^{m_{FV}}$$

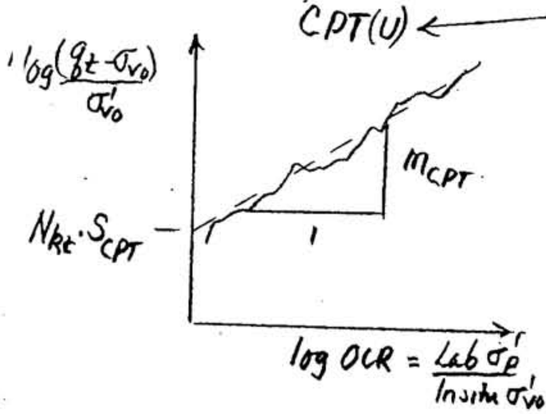
↓

$$OCR = \frac{\sigma'_p}{\sigma'_{vo}} = \left(\frac{s_u(FV)/\sigma'_{vo}}{S_{FV}}\right)^{\frac{1}{m_{FV}}}$$

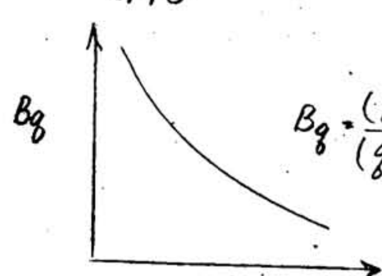
(See Sheet © for correlations recommended by Chandler 1988)

CPT(U) →

$$OCR = \left\{\frac{(q_t - \sigma_{vo})/\sigma'_{vo}}{N_{kt} \cdot S_{CPT}}\right\}^{\frac{1}{m_{CPT}}}$$




CPTU




$$B_q = \frac{(u - u_s)}{(q_t - \sigma_{vo})}$$

## 4.3. $s_u$ For UU Case

### 4.3.1 "Small Project" Where ±25% Error in F of S Acceptable (Site has "small" area)

1) Design $s_u = \mu\, s_u(FV)$ probably best, unless soil has shells & sand

2) " " $= (q_t - \sigma_{v0}) / N_k = 14 \pm 5$ less reliable

3) UUC (plus simple lab $s_u$ index = TV, PP, FC, etc.) can be used for comparison with FVT or CPT, or might be used alone

Note: If high OCR deposit with $K_0 \approx 1$, CIUC acceptable, but MUST reduce $q_f(C)$ to account for anisotropy

4) FOR ALL OF ABOVE, run some consolidation tests to check $s_u$ via SHANSEP equation $s_u/\sigma'_{v0} = S(OCR)^m$

- So-called Level C approach in Section 5.3 of Ladd (1991) based on results in Fig. 18 & Table 4 (p24) for sedimentary deposits

- Sensitive marine clays: $S = 0.20 \pm 0.015$, $m = 1$

- CL & CH clay of low-moderate $S_t$: $S = 0.20 + 0.05 I_p$ or $S = 0.22$
  $m = 0.88(1 - C_s/C_c)$ or $m = 0.8$

- Soils below A-line: $S = 0.25 \pm 0.05$ SD
  $m = 0.88(1 - C_s/C_c)$ or $m = 0.8$

- Northeastern varved clays: $S = 0.16$; $m = 0.75$ (for DSS mode of failure)

---

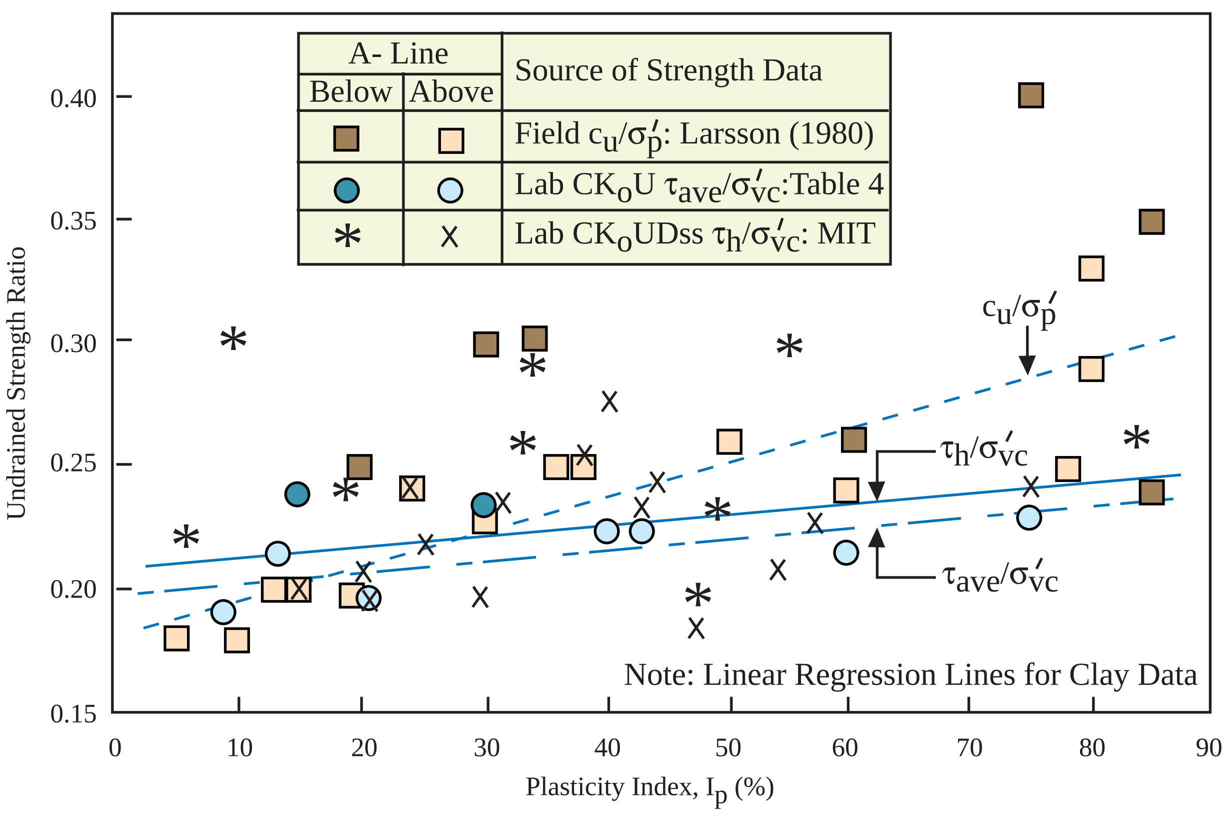
Eqn 4.3 If don't even know soil type $S = 0.22 \pm 0.03$ (SD) & $m = 0.8 \pm 0.1$ (SD)

Uncertainty in applying Eqn. 4.3 assuming COV[OCR] = 15% using Eqn 3.3

| OCR | $s_u/\sigma'_{v0}$ | SD[$s_u$] | COV[$s_u$] |
|---|---|---|---|
| 1 | 0.22 | 0.04 | 18% |
| 5 | 0.80 | 0.195 | 24% |
| 10 | 1.39 | 0.40[5] | 29% |

Image removed due to copyright reasons. Please see: Ladd (1991).

For ⬡ CL & CH clays, $\tau_{ave}/\sigma'_{vc} \approx 0.215 \pm 0.015$ SD




**Comparison of field and laboratory undrained strength ratios for nonvarved sedimentary soils (OCR = 1 for laboratory $CK_oU$ testing)**

Adapted from Ladd (1991)

4.3.2 "Large Projects" Where F of S More Critical (Site has "large" area)

1) Testing for 4.3.1 with most suitable in situ device for spatial variability

2) Alot more lab measurements of $\sigma'_p$ (provided by SHANSEP automated $CK_o$-TX & DSS tests)

3) $CK_oU$ test program (Level A or B) → values of S and m

a) Reconsolidation technique

- SHANSEP for "ordinary" clays → must use if OCR=1
- Recompression if highly structured

b) For isotropic $s_u$ analyses: DSS or ave. of TC & TE

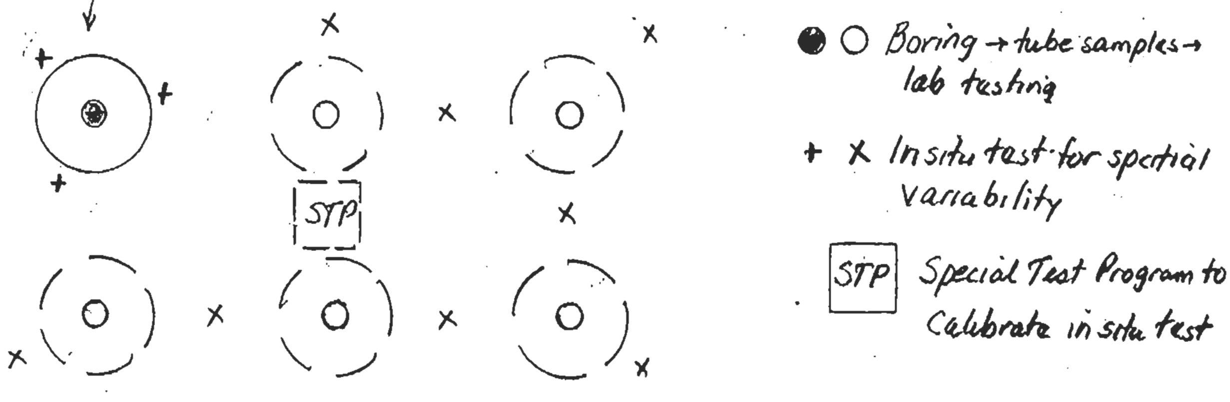
c) For anisotropic $s_u$ analyses: TC, DSS & TE

d) Compare with prior data on similar soils

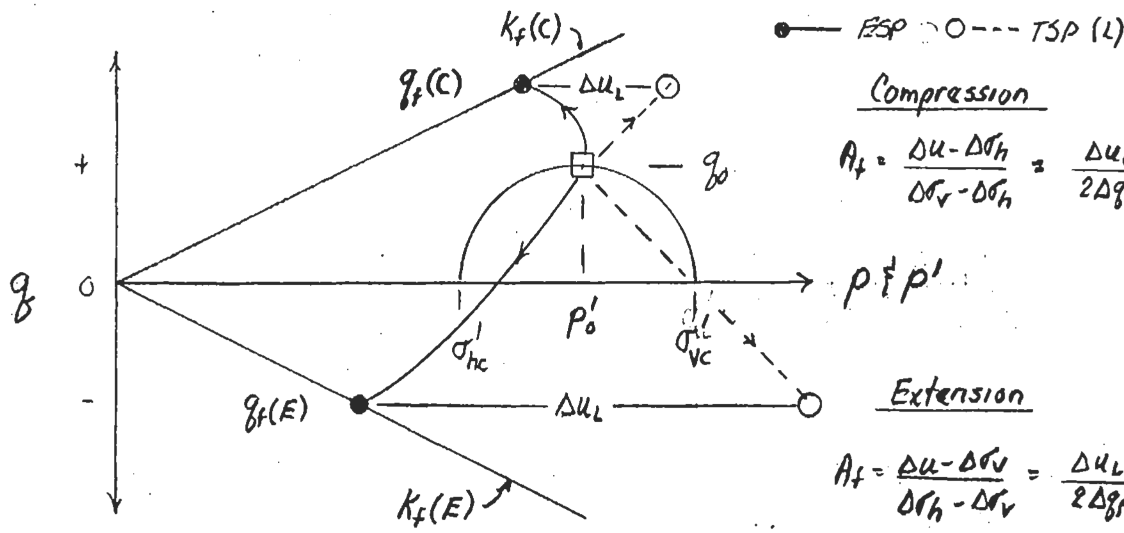
4.4 $s_u$ For Staged Construction (CU Case)

- See Part 5 of Ladd (1991)
- Requires better definition of initial stress history and more extensive $CK_oU$ testing than for UU Case
- 1.322 treats in detail

One Tank vs Tank Farm (say 100 × 200 m with variable stress history)



# Derivation of $q_f/\sigma'_{vc} = f(K_o, A_f \,\&\, \phi')$ for $CK_oU$ PSC/E & TC/TE $(c'=0)$




**Compression**

$$A_f = \frac{\Delta u - \Delta\sigma_h}{\Delta\sigma_v - \Delta\sigma_h} = \frac{\Delta u_L}{2\Delta q_f}$$

**Extension**

$$A_f = \frac{\Delta u - \Delta\sigma_v}{\Delta\sigma_h - \Delta\sigma_v} = \frac{\Delta u_L}{2\Delta q_f}$$

NOTE: Equations apply to $K_c \neq K_o$ and $K_c \; {<}\,1,\; =1,\; >1$

## Both

$$q_o = \frac{\sigma'_{vc}}{2}(1-K_o) \qquad q_f = p'_f \sin\phi' \qquad \Delta u_L = A_f(2\Delta q_f)$$

$$p'_o = \frac{\sigma'_{vc}}{2}(1+K_o) \qquad \Delta p_f = \Delta q_f$$

## Compression

$$\Delta q_f = q_f - q_o = q_f - \frac{\sigma'_{vc}}{2}(1-K_o)$$

$$p'_f = p'_o + \Delta p_f - \Delta u_L = \frac{\sigma'_{vc}}{2}(1+K_o) + q_f - \frac{\sigma'_{vc}}{2}(1-K_o) - 2A_f\left[q_f - \frac{\sigma'_{vc}}{2}(1-K_o)\right]$$

$$\frac{p'_f}{\sigma'_{vc}} = \Big[\underbrace{\tfrac{1}{2}(1+K_o) - \tfrac{1}{2}(1-K_o)}_{K_o} + A_f(1-K_o) - \frac{q_f}{\sigma'_{vc}}(2A_f-1)\Big] \;;\; \frac{q_f}{\sigma'_{vc}} = \frac{p'_f}{\sigma'_{vc}}\sin\phi'$$

$$\frac{q_f}{\sigma'_{vc}}\left[1 + (2A_f-1)\sin\phi'\right] = \left[K_o + A_f(1-K_o)\right]\sin\phi' \rightarrow \boxed{\frac{q_f(C)}{\sigma'_{vc}} = \frac{[K_o + A_f(1-K_o)]\sin\phi'}{1+(2A_f-1)\sin\phi'}}$$

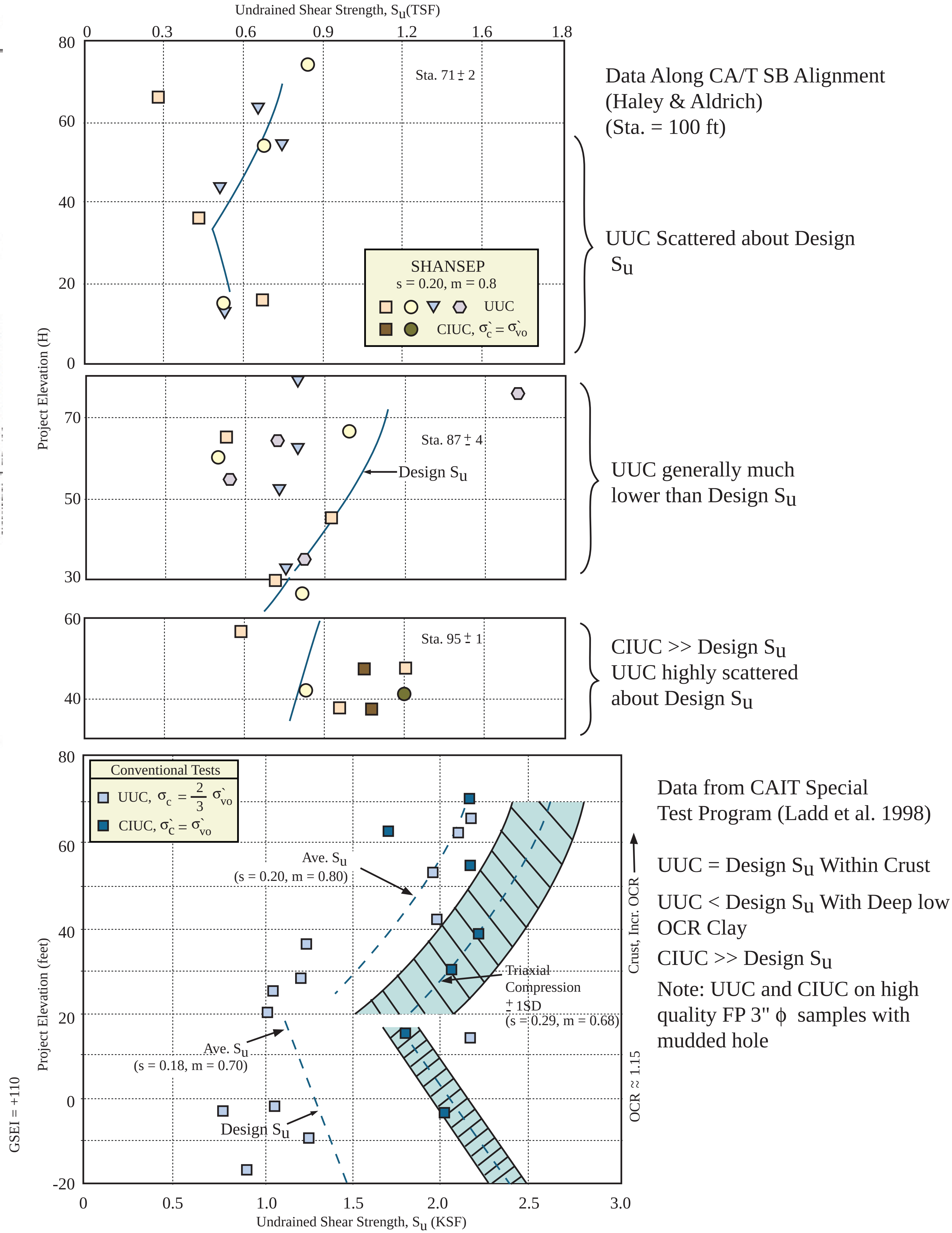
## Extension

$$\Delta q_f = q_f + q_o = q_f + \frac{\sigma'_{vc}}{2}(1-K_o) \;;\; p'_f = p'_o + \Delta p_f - \Delta u_L = \frac{\sigma'_{vc}}{2}(1+K_o) + q_f + \frac{\sigma'_{vc}}{2}(1-K_o)$$

$$-2A_f\left[q_f + \frac{\sigma'_{vc}}{2}(1-K_o)\right] \;;\; \frac{q_f}{\sigma'_{vc}} = \Big[\underbrace{\tfrac{1}{2}(1+K_o) + \tfrac{1}{2}(1-K_o)}_{1} - A_f(1-K_o) - \frac{q_f}{\sigma'_{vc}}(2A_f-1)\Big]\sin\phi'$$

$$\boxed{\frac{q_f(E)}{\sigma'_{vc}} = \frac{[1 - A_f(1-K_o)]\sin\phi'}{1+(2A_f-1)\sin\phi'}}$$

A

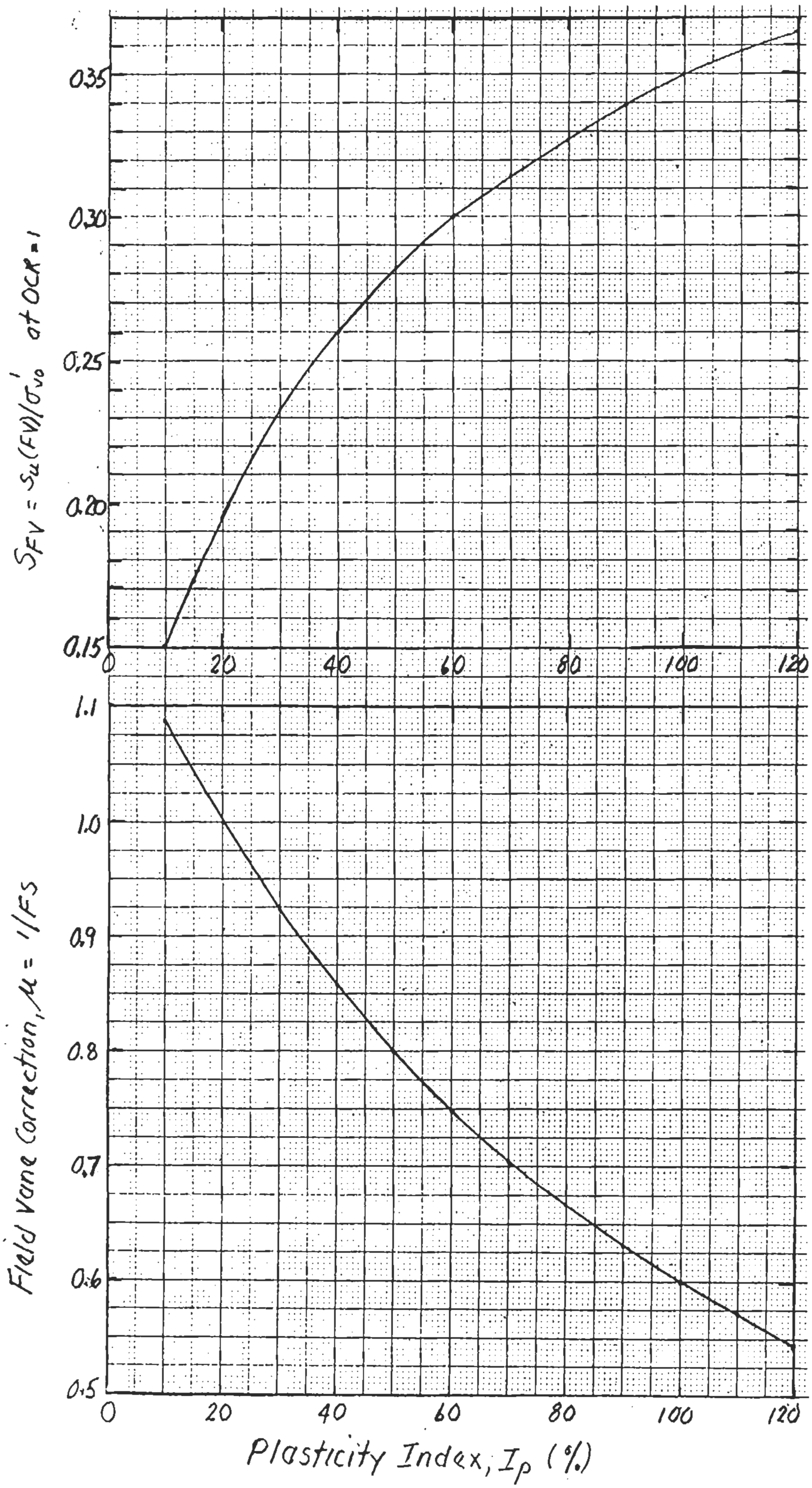



**Comparison of Undrained Strengths from Conventional Triaxial Tests with SHANSEP $s_u$ Profiles at SB Test Site**

## Comparison of Conventional Vs. SHANSEP $s_u$ Data: BBC

CCL 11/23/99 1.361-1.366 Part V-4

CCL 5/3/99 1.322




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 $I_p$

(C)