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

1. Introduction

2. Types of Anisotropy

3. Use of UV Type Tests to Measure Anisotropy

4. Test Variables for CU Testing

5. Experimental Capabilities
   (TX, PS, TTA, DSS, TSNC & DSC)

6. Influence of $K_c$ and $b$  
   * Replaced by Section 6.1 (SP)

7. Influence of Rotation of Principal Stresses  
   * Replaced by Section 7.3 (SP) & 7.4 (AP)

8. Progressive Failure  
   (Shear SC1-5)

9. Consideration of Anisotropy in Undr. Str. Analyses
   9.1 Bearing Capacity
   9.2 Circular Arc Analyses
   9.3 Interpretation Sc for UTREA3 Stability Analyses

CCL 4/12/87 4/88 1.322
4/13/88 4/89 = 4/99 = 4/01

CCL 4/19/81 Sorry that I did not have time to rewrite these notes.
STRESS SYSTEM: Experimental Techniques & Results
(For saturated clays; granular soils later)

1. INTRODUCTION

1.1 Definition

Stress system = Direction of $\sigma_1$ wrt vertical (SS angle)
\[ \rightarrow \text{anisotropiс behavior} \]
\[ + \text{Effect of } \sigma_2 \text{ à la } b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \]

1.2 Objectives and why
- How does SS affect behavior?
- How to measure experimentally - in situ, lab
- Magnitude of effects
  - When $\delta$ vs $b$ important?
  - Effect soil type & OCR
- $\sigma_u = \text{function of}\ 1) \text{initial } \sigma_1 (\overline{\sigma_K}, k_c)$
  2) $\Delta \sigma (\delta_1, A_1)$
  3) Envelope ($E, B$)

1.3 Overview of Experimental Capabilities

For $\sigma_2 = \sigma_1$

\[ \text{TE} \]
\[ \text{PSE} \]
\[ \text{TMA} \]
\[ \text{DSC} \]
\[ \text{PSC} \]

\[ \theta \]
\[ \text{s} \]
\[ 90^\circ \]

à la JTG (1982)  
Note: other test devices to be added

\[ \text{Doesn't include Cavity Expansion = SBPT (} \sigma_2, \sigma_1) \]
2. TYPES OF ANISOTROPY

2.1 Initial Anisotropy of Clay with 1-D History

1. Inherent (due to depositional & consolidation history)
2. Transversely Isotropic

(a) "Structural" due to preferred "soil structure"
(b) "Material"
   e.g. varved clay, fissures, bedding planes

2.1.1 Shear Stress (when $K_0 \neq 1$)

- Hansen & Gibson (1949)
- CKo UPS C/E

\[ q = \frac{(\sigma_v - \sigma_h)}{2} \]

\[ q_f(C) = \frac{[K_c + (1-K_c)A_f] \sin \phi}{1 + (2A_f - 1) \sin \phi} \]
\[ A = \frac{\Delta u - \Delta \sigma_h}{\Delta \sigma_v - \Delta \sigma_h} \]

\[ q_f(E) = \frac{[1 - (1-K_c)A_f] \sin \phi}{1 + (2A_f - 1) \sin \phi} \]
\[ A = \frac{\Delta u - \Delta \sigma_v}{\Delta \sigma_h - \Delta \sigma_v} \]

- Can produce su anisotropy
- Can have any inherent anisotropy (i.e., for same $K_c$, $A_f$ & $\sin \phi$)
- Combined = Inherent + $K_0 \neq 1$
2.2 Other Types of Anisotropy

1) Prestaining isotropic soil → subsequent anisotropy behaviour à la Arthur et al. tests on sand (INDUCED)

2) Evolving (TL Fig.12)

\[ \sigma_{1c} \rightarrow \sigma_{fc} \rightarrow \sigma_{vc} \rightarrow \tau_{hc} \]

Δ shape of yield surface (Treated in Section 7.4)

3. USE OF UU TYPE TESTS TO MEASURE ANISOTROPY

3.1 In Situ

1) FV with varying shape (Tokyo 4.2.4)
   - Disturbance + Progressive failure + Unknown stresses → unreliable results

2) NGI special in situ OS device (Table 11.2 of CCL, 1971)
   - Margclay
     - Quick clay
     - Margclay
     - Ip = 0.5, Ss = 100

3.2 Lab UUC Cut at Varying $\delta$

Tokyo F21

- Homogeneous sedimentary; ma. St + mae effect
- Varved clay, $S_u$(OSS) min
- Stiff fissured
Problems with UVC(s)

1) Neglects buckled shear component \( K_c \) and \( K_o \)

2) Sample disturbance

\[ K_s = \frac{u_c(H)}{u_c(V)} \]

<table>
<thead>
<tr>
<th>UVC(s)</th>
<th>CKUSPS</th>
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<td>0.15</td>
<td>0.44</td>
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<td>0.8</td>
<td>0.56</td>
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<tr>
<td>0.6</td>
<td>0.9</td>
</tr>
</tbody>
</table>


shag weak

Conclusion: Need CKoU Type testing

4. TEST VARIABLES FOR CU TESTING

4.1 Stress level \( \sigma_{VC} = \sigma_{VO}, \sigma_{VM} = \sigma_p \) SHANER vs RECOME

4.2 \( K_c + \) stress path \( \rightarrow K_c \) (Covered Part II B)

4.3 Sample orientation \( \nabla H \)

4.4 \( \sigma_f \) direction = S angle

4.5 \( \sigma_2 \) magn. = b value

(Note: Really need to specify \( \sigma_2 \) direction, e.g. PSE vs S8P7)
5. EXPERIMENTAL CAPABILITIES

5.1 Triaxial

- $C_k U/E \rightarrow S=0/90^\circ$ but $b=0 \rightarrow 1$

- Use of $T_1/T_2$ on "horizontal" sample
  - On box $S$ plot
  - Problems: Wrong $\Phi_u$

5.2 Plane Strain Campanella & Vardal (1974)

- $P S C / E \rightarrow S=0, 90^\circ$ with "constant" $b$
  - Correct but limited capability

5.3 True Triaxial Apparatus (TTA)

1) Boundary conditions
   - Flexible (Rubber Bungs) UCL (MIT)
   - Rigid: Cambridge Univ
     - Mixed Lade (UCLA)

2) What can do in $b-S$ plot

3) Conclusions: Mainly useful for studying $b$

NOTE: Very little $C_k U$ data available from TTA
5.4 Direct Simple Shear $(GeoNoor) = OSS$

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

\[
\frac{\tau_h}{\bar{\sigma}_{vc}} \quad \frac{\phi}{\bar{\sigma}_{vc}}
\]

\[
\tau_h = \frac{\tau_h}{\bar{\sigma}_{vc}} \quad \phi = \frac{\phi}{\bar{\sigma}_{vc}}
\]

Secant $G = \frac{\tau_h}{\phi}$

(2) Problems

a) Non-uniform stresses


"Worst than OS" Elastic vs plastic

De Groot et al. (1991) p 60 Tests on rubber

b) Indeterminate state of stress

\( S = ? \) \( \text{N.E. } \epsilon, \tilde{\epsilon}, A \)

\( \bar{\sigma}_{ss} \leq \tau_h \leq \bar{\sigma}_{T} \), \( S = 40 \times 10^6 \)

c) Randolph & Wroth (1981) interpretation

Failure on vertical plane!

\[
\tan \phi = \frac{S \sin \phi \cos \phi}{1 + S \sin^2 \phi}
\]

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

Figure by MIT OCW.
Vertical stress during first reversal stage (Normalized by $\sigma'_v$ at $\gamma = 0\%$) versus shear for undrained cyclic geonor CK$_o$UDSS test on BBC and SFBM.

Schematic of hypothesis showing influence of DSS apparatus on behavior of OCR = 1 specimen in CK$_o$UDSS test.

Figure by MIT OCW.
Normalized results of cyclic Geonor CKoUDSS test on SFBM with $\sigma_{\text{max}} = 501$ kPa: (a) Shear stress-strain curve; and (b) Vertical effective stress versus shear strain.

Figure by MIT OCW.
(3) DSS = \rho \sigma \text{ vs. } \sigma_{\text{fl}}(\text{DSS})/\sigma_{\text{vc}}

Soydemir (1974)(4) Special DSS on inclined samples - Add field case

Bjerrum Memorial Vd.

(5) Geonar vs Marshall Silva Devri

[Graph showing comparison]

(6) Cambridge SSA

(7) CCC opinion of DSS (SHANSEP testing)

- Reasonable su fn stability analyses / elas/charp. CTW/E
- Reasonable En, k hypothetic parametr. in FEECA

DSS-2 p76 - Excessive strain softening at large strains, p66

5.5 Torsional Shear Hollow Cylinder (TSHC)

5.5.1 Stress States

(a) Saada et al. \rho = \rho_0 = \sigma_{\text{fl}}

\beta = 5\text{m}^2g

\sigma_{\text{fl}} = \sigma_{3}

\sigma_{3} = 45^\circ

b = 0.5

\sigma_{\text{fl}} = \sigma_{2}

\beta = 90^\circ

b = 1

TE

\sigma_{\text{fl}} = \sigma_{3}

\beta = 0

b = 0

TC
<table>
<thead>
<tr>
<th>Sym.</th>
<th>Deposit</th>
<th>Tests By</th>
</tr>
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<tbody>
<tr>
<td>X</td>
<td>Oriente Clay</td>
<td>MIT</td>
</tr>
<tr>
<td>+</td>
<td>Venezuela</td>
<td>MIT</td>
</tr>
<tr>
<td>*</td>
<td>North Pari Clay/Van.</td>
<td>MIT</td>
</tr>
<tr>
<td>□</td>
<td>Kesed, BBC (Ave)</td>
<td>MIT</td>
</tr>
<tr>
<td>□</td>
<td>Portsmouth Sensitive</td>
<td>MIT</td>
</tr>
<tr>
<td>△</td>
<td>Atchafalaya</td>
<td>MIT</td>
</tr>
<tr>
<td>○</td>
<td>Bangkok</td>
<td>MIT</td>
</tr>
<tr>
<td>△</td>
<td>Maine Organic</td>
<td>MIT</td>
</tr>
<tr>
<td>△</td>
<td>Conn Valley Varved</td>
<td>MIT</td>
</tr>
<tr>
<td>△</td>
<td>AGS Plastic (Ave)</td>
<td>MIT</td>
</tr>
<tr>
<td>▼</td>
<td>B-2</td>
<td>UofS</td>
</tr>
<tr>
<td>▼</td>
<td>B-6</td>
<td>UofS</td>
</tr>
</tbody>
</table>

NOTE: MIT tests on Oli yielded
\[
\frac{\bar{\sigma}_v}{\sigma_{yc}} = 0.615 \pm 0.04
\]

Empirical Correlation for Venezuelan Clays

Randolph & Wroth (1981) Theoretical Relationship: Modified Cam-Clay
with \( C_3/C_1 = 0.2 \) and
\[
\tan \psi = \frac{\sin \phi \cos \phi}{1 + \sin^2 \phi}
\]

\( \psi = \arctan \frac{\tau_h}{\sigma_{vc}} \)

CK_0 VDSS Test Results on Normally Consolidated Clays
Comparison of field and laboratory undrained strength ratios for non-varved sedimentary soils (OCR = 1 laboratory CK₀U testing)

Note: Linear Regression lines for clay data

Figure by MIT OCW.
Undrained strength anisotropy from $CK_OU$ tests on normally consolidated clays and silts.

Figure by MIT OCW.
5.5.1 (a) Continued

- Where test plots on $b = \sin^2 \theta$  ($b = \sin^2 \theta$)
- Comments on SF = Fig. 19 (p 8a)
  - Variation in $\phi'$ : Expected for $b = 0 \rightarrow 1$
  - """ $\phi/0_c'$ : Diffuse from normal heads
  - Scatter: alot

(b) Imperial College High et al. (1983 geol. #4)

- $H = 25 cm$  $OD = 25 cm$  $t = 2.5 cm$  Measure strains in central portion
- $C_U$ & $C_D$ tests on sat. sand
- Apparently limited to $P_0/P_c = 1.2 - 0.9$ (with $S \leq 45^\circ$)
- $P_c > P_0$ to left of $b = \sin^2 \theta$ line
  $P_c < P_0$ """ right """
  $$\tau_0 - \tau_r = \frac{r c l \sigma_r}{d r}$$
  $$\tau_r = \frac{(P_0 b + P_a)}{(b+a)}$$
  $$\tau_0 = \frac{(P_0 b - P_a)}{(b-a)}$$
  $$\tau = \frac{3 M}{2(2b^2 - a^2)}$$

Advantages

- Most versatile of any device
- Data from $C_U$ tests on sand look excellent
- (Fig. 20 SF - cover later: under sand anisotropy)

Disadvantages

- Very complex & costly
- Non-uniform stresses with $P_0$ & $P_a$
- End effects
- Problems of testing clamps
- Need to measure strains internally
NOTE: Samples rebounded to $\sigma_c'$ after consolidation to $\sigma_h = \sigma_c$ and $\sigma_v = \sigma_c' / K_o$ with $K_o = 0.47 \pm 0.01$.
5.6 Directional Shear Cell (DSC) - Only plane strain

5.6.1 Principle (Developed by Arthur et al e UCL)

- Pressure bars + shear sheets → any \( \sigma \), angle

5.6.2 Sample Orientation

- \( Z = \) Vertical (depositin)
- (Can plot on b-S diagram)

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

- b) Shear in x-z plane (Inherent \( \psi \))
  - Measure inherent + initial shear
  - Shear anisotropy
  - Where falls b-S plot

5.6.3 Misc

- Radiography/photography → strain distribution
- UCL sand testing
- MIT clay testing (ITG'82;90)
- Limited to low stress (\( \tau < 50 \text{kPa} \); MIT version)

* Optical Comparator → displacements ± 2 μm
6. INFLUENCE OF $K_c$ AND $b$

6.1 Influence of $K_c$

4/1488 Replaced by 6.1-1 → 6.1-5 (qhm.p10)

6.2 Influence of $b$

(1) General considerations of increasing $b$
   • Effect on $\Delta u$
   • $\ldots \phi$
     Mohr-Coulomb →
     MCC →

   (Matsuoka, 1974)
   $I_1 \cdot I_2 / I_3$ = constant

   (Lade & Duncan, 1975)
   $I_1, I_3 = constant$

(2) CIU TTA N.C. Grundy
     Lade & Mura (1971) (1978)
   • Handout (p108)

   As $b$ increases $0 \rightarrow 1$
   $\Delta u$: increasing \ then decreasing
   $E_f$: decreasing \ then constant
   $\phi$: increasing \ then decreasing
   $A_f$: constant \ then increasing

   $PS \rightarrow TC \rightarrow$
   $\mu_s \rightarrow \psi$
   $\mu_s, \phi$
   $Pcs, E_f$
6. **INFLUENCE OF \( K_c \) AND \( b \)**

\[ K_c = \frac{\sigma'_c}{\sigma'_c}; \quad b = \frac{\sigma'_2 - \sigma'_3}{\sigma'_1 - \sigma'_3} \]

6.1 **Influence of \( K_c \) (OCR=1)**

6.1.1 **CAU vs CIU: General Trends**  
(Ladd 1965; Ladd Varallappp 1965)

![Graph showing influence of \( K_c \)]

- **Going from CIU to CAU**
  - Approx. same \( \sigma_c \pm 10-15\%
  - Large decrease \( \phi_c \)
  - Often increase in shear modulus
  - Always expect decrease in \( \phi_c \) from lower \( p' / \sigma'_c \), plus larger \( \sigma'_f \)

6.1.2 **Influence of \( K_o \) on \( K_{c/UC} \) Behavior**

1) Before \( \approx 1990 \), I had expected little effect from trends in 6.1.1,  
   plus \( \beta'(c)/\sigma'_c \) as \( I_p \approx 0.33 \pm 0.02 \) from \( K_{c/UC} \) testing (Fig.15, CCL '91)  
   But NOT TRUE

2) **Data on natural BBC** (See \( K_o \) 1 \& 2 for actual data)

![Graph showing influence of \( K_o \)]

Increasing \( K_o \) → decreasing \( \sigma_c \) due to:  
- Lower \( \phi'_c \)  
- Higher \( \sigma'_f \)  
- Also decreasing \( \phi'_f \)

\[ * \text{Rel } K_o = 0.51 \rightarrow 0.61 \text{ leads } \sigma_c = 0.30 \rightarrow 0.26 \]

\[ (+ 20\%) \text{ \( (-13\%) \)} \]
Natural BBC

RBBC

Approx. collective

Mexico city clay this investigation
- Boston blue clay (BBC)
- Barman (1993)
- Residemlced BBC
- Unpublished data
- Brazil
- Talped

Lateral stress ratio, \( K_o \) (Nc)

Undrained strength ratio, \( q_f / s'_{vc} (OCR=1) \)

Se = 0.049 + 0.178 Ko (r² = 0.39)

Figure by MIT OCW.

6.1.3 Influence of \( K_o \) on CK₀UE behavior

1) Should expect increasing \( K_o \) - increase in \( S_c \) since:
   - starting from higher \( p'_{100c} \)
   - smaller OCR [\( c_s, 80-85(\%) \)]

2) Only available data (below) supports this expectation

\[ S_c = q_f / s'_{vc} \]

In situ OCR > 2
In situ OCR < 2 (crust)
Leak → \( K_o \) too low

SAANSEP CK₀UE Data natural BBC

\[ S_c = 0.049 + 0.178 K_o \ (r^2 = 0.39) \]

Figure by MIT OCW.
6.1.4 Conclusions of Influence of $K_o$ on CKU Behavior

1) In TC, increasing $K_o$ → significant reduction in $S_c = \frac{g_E(c)}{g_E(c)}$
due to lower $\phi_t$ and higher $\theta_t$. Was not expected, but all data

2) In TE, increasing $K_o$ → significant increase in $S_c = \frac{g_E(c)}{g_E(c)}$.
   To be repeated, but limited data to support.

3) Therefore using $K_o = \infty$ in the CKU for Recompression CKU test
   may be important for reliable values of $g_E(c)$.
USR and $\Phi_r$ at Peak vs. $K_c$ from NC SHANSEP $K_o$UC Triaxial tests

* During "Ko" consolidation $\rightarrow$ too low values of $K_c$

Figure by MIT OCW.

Adapted from de la Beaumelle (1991) SM thesis: NASA STP CAIR Project

- 1st $K_u$ data from MIT's automated triaxial system developed for CAIR STP on natural BBC
- One of 9x cells had a small leak (→ increased "measured" $\Delta \epsilon_{ext}$) $\rightarrow$ reduced $\sigma_{1c}'$ values of $K_o$ that were too low ($K_o$ $\approx$ 7, $10^5$Pa $\approx$ 10 MPa)
- But leakage rate too small to affect undrained shearing
- $S_c = 0.475 - 0.350 K_o$ ($r^2 = 0.85$) where $S_c = \Phi_r(0) / \sigma_{1c}'$
NC Undrained Strength Ratio vs. Lateral Stress Ratio for SHANSEP $\text{CK}_0$ UC Triaxial Tests: Collective Data

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

Figures by MIT OCW.
CIU True Triaxial on Remolded Grundite ($w_s=54\%, P_i=31\%$) 
$\sigma_c = 1.5 \text{ kg/cm}^2$  (Data from Lade & Musante, 1978 JGEO G72)

- Undrained Strength
- Strain at Failure
- Friction Angle
  (Agrees with a lot of data on sands)
- Horizontal Failure

**NOTE:** Authors concluded that $\sigma_f$ decreases slightly with increasing $b$ (including tests at $\sigma_0 = 1$ and 2)
(3) Comparison PS vs Triangular CKoU Data (Table 1 Tokyo)
   a) PSC vs TC 10clayp mostly NC
      • \( q_f \rightarrow 8 \pm 5\% 
      • \( q_w \rightarrow 2 \pm 2 \%
      • Maybe increased strain softening

   NOTE: TC \( y = 1.5\%
   PS \( y = 2\%

   b) PSE vs TE 4 NC clay
      \( q_f \rightarrow +20-25\%

   Conclusion: TX \( \rightarrow \) conservative \( su \) for PS problems, but need more data.

7. INFLUENCE OF ROTATION OF PRINCIPAL STRESSES
   (CKoU on low OCR clays mostly)

7.1 General Expectations \( (K_o < 1)\)
   With increasing \( \theta \)
   - Increasing \( \Delta q \rightarrow \text{max.} \Delta u \) & hence reduced \( p_f \)
   - \( \text{à la Hanai's Gibson} \)
   - Inherent anisotropy \& structure more resistant in radial direction

7.2 Available Test Data
   1) DSC BBC & OCR=4 \( \frac{11}{11} \)
   2) PSC/TE \( \theta = 0 \)
      DSS \( \theta = ? \)
      \( K_S = \frac{\sigma_u(H)}{\sigma_u(V)} \)
      \( PSE/TE \theta = 90\% \)

   Problem w/ TE/TE \( \theta \? \)
7.3 Results from DSC Tests on R BBC

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

Clay was thixotropic; therefore normalize to $\sigma'_p$

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

![Graph showing the effect of thixotropy on preconsolidation pressure of resedimented BBC.](image)

Figure by MIT OCW.

2) Proof Testing, 0.5 to pressure bags and shear sheets — same results?


![Graph showing normalized stress-strain and effective stress paths.](image)

Figure by MIT OCW.
3) Effect of $\delta$: Since $K_0=1$, all inherent anisotropy

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

  - Range from Isotropic tests
  - Note: 1) Data normalized to $\sigma'_p = 1.00$ ksc
  - 2) Solid symbols indicate failure

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

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

Figures by MIT OCW.
7.3.2 Collective DSC Data at OCR = 1 & 4 (OCR = 1 data from Seck 1990)

1) Trends on $S_1(S)$

- Similar shapes at OCR = 1 → max anisotropy since
  includes both inherent and initial observations ($g > 0$)

2) Both show similar increase in $A_f$ with increase $S$.

3) For OCR = 1, decrease in $S_f$ mainly due to increasing $O_f$
   with increasing $S$, i.e., approximately constant $O_f / S_f$.
7.3.2 Continued

Approximately constant $\phi' = 34^\circ$ for OCR=1 tests, but decreasing ESE with increasing $S$ for OC tests.

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

1. Experimental procedures very complex
   1.1 had to $K_s$ consolidate to OCR=1
   1.2 using rigid between rubber membrane
   1.3 shear sheets to reduce side friction
   1.4 then had to remove rigid cover that shear sheets could apply $T_a + T_b = \Delta T$

   $\sin c = 0 : +\Delta T_a \neq \Delta T = 0$
   $\sin c < 45^\circ : +\Delta T_a + \Delta T$
   $\sin c = 45^\circ : \Delta T = 0 + \Delta T$
   $\sin c > 45^\circ : +\Delta T_b + \Delta T$
   $\sin c = 90^\circ : +\Delta T_b / \Delta T = 0$. 

\[ G = \kappa (\varepsilon - \varepsilon_f), \rho' = \phi (\varepsilon / \varepsilon_f) \]
2) Prediction by Whittle (1987 Ph.D. Thesis) made before tests were run (Type B)

(a) Comparison of ESP

(b) Comparison of shear strain $\gamma$ vs $q$ and applied $q$

$[q = \frac{1}{3}(\sigma_1 - \sigma_2)]$
Evaluation of MIT-E3 Predictions of Peak Strength Conditions in DSC Tests: (a) Undrained Shear Strength; and (b) Pore Pressure Parameter, $A_f$

Figure by MIT OCW.
7.4 General Trends in Undrained Strength Anisotropy from CK0U RS, TS and DSS Testing

7.6.1 $s_u$ Anisotropy for NC Clays/Silt (Non-Varved)

**Bishop (1966) Equation:**

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

**Trends with $\delta$ (Fig. 3):**
- CCL would expect trends similar to DSC data on RBC
- Bishop (1966)
  $$q(\delta) = q_0 (1 - a \sin^2 \delta) (1 - b \sin^2 2\delta)$$
  $$\alpha = 1 - K_s$$
  $$b = \text{need to assume value for } q(\delta) \text{ to back calculate } b$$
7.4.2 **Soil Anisotropy of Varved Clay**

Varved clays are unusual since $C_{k0}$ is lowest, i.e., below compressive strength.

In addition, $S_d$ is NC $T_d/T_N DSS$ is extremely low.

Fig. at left from Task 2 (CCL 91)
where $T = g_{soil}(n_{soil})$ needed for shear compatibility.

---

7.4.3 **Soil Anisotropy of OC Clay**

1) See p 7.4-3 for $C_{k0} T_d, DSS$ for data in OCR

   Fig. 6: SHANSEP testing on CH clay

   Fig. 7: Recompression testing on semicrystalline clay

   Note difference in $T_d$ trends

2) See p 7.4-4 for $log K_s$ vs log OCR on several clays: $K_s = \frac{S_c}{S_{oc}} (OCR) (m_e - m_c)$

   Increasing OCR → less anisotropy (except for BC clay).

   Should expect since more OCR = more $K_0$ → smaller $\beta_0$ →
   less effect of "bulk shear shear" anisotropy.

   Note that SHANSEP = less $S_u$ anisotropy (higher $K_s$) than SHANSEP
   for natural OCR. CCL thinks this may be generally true.
Note:

\( C_u = q_f \) for TC and TE
\( C_u = \tau_{\text{max}} \) for DSS

(a) Undrained Strength Ratio

<table>
<thead>
<tr>
<th>Test</th>
<th>Symbol</th>
<th>S</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC</td>
<td>△</td>
<td>0.325</td>
<td>0.78</td>
</tr>
<tr>
<td>DSS</td>
<td>○</td>
<td>0.255</td>
<td>0.78</td>
</tr>
<tr>
<td>TE</td>
<td>▽</td>
<td>0.200</td>
<td>0.86</td>
</tr>
</tbody>
</table>

(b) Strain and Pore Pressure Parameter at Failure

(a) Undrained Strength Ratio and (b) Strain and Pore Pressure Parameter \( A_f \) at Failure vs. OCR from SHANSEP CK\(_0\)U Tests on AGS CH Marine Clay.

Figure by MIT OCW.
Recompression $I_p = 13\%$, $I_L = 1.9$

(a) Undrained Strength Ratio

<table>
<thead>
<tr>
<th>Test</th>
<th>Intact</th>
<th>Destructured</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC</td>
<td>0.45</td>
<td>0.335</td>
</tr>
<tr>
<td>DSS</td>
<td>0.29</td>
<td>0.25</td>
</tr>
<tr>
<td>TE</td>
<td>0.235</td>
<td>0.20</td>
</tr>
</tbody>
</table>

(b) Strain and Pore Pressure Parameter at Failure

(a) Undrained Strength Ratio and (b) Strain and Pore Pressure Parameter $A$ at Failure vs. OCR from CK$_0$U Tests Run on Intact and Destructured James Bay B-6 Marine Clay.

Adapted from Jamiolkowski et al. (1985)
<table>
<thead>
<tr>
<th>Label</th>
<th>Clay</th>
<th>Program</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>B6 (R)</td>
<td>B-6 James Boy</td>
<td>CK0-TX R</td>
<td>Fig. 16 (Ladd, 1990)</td>
</tr>
<tr>
<td>AGS (S)</td>
<td>AGS CH</td>
<td>CK0-TX S</td>
<td></td>
</tr>
<tr>
<td>BBC (R)</td>
<td>Natural BBC</td>
<td>CK0-TX R</td>
<td>II B, BBC-3,4</td>
</tr>
<tr>
<td>BBC (S)</td>
<td>&quot;</td>
<td>CK0-TX S</td>
<td></td>
</tr>
<tr>
<td>Resed. BBC</td>
<td></td>
<td>CK0-DSC R</td>
<td>I Section 7.3</td>
</tr>
</tbody>
</table>

R = Recompression  S = SHANSEP

![Graph](image)

**Variation in Undrained Strength Anisotropy with OCR**
7.6 Example of Evolving Anisotropy (Insert bottom 3)

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

2) Results
   - MDSS-2 Schematic of problem
   - **-3 MDSS
   - **-4 Peak strength vs direction of shear
   - **-5 Typical stress-strain data vs direction of shear
   - **-6 Comparison with MIT-E3 predictions

De Groot, Léa & Germaine (1996) "Undrained multidirectional direct simple shear behavior of cohesive soils" JSCE, ASC
122(2), 99-108
Shear stresses on soil at structure Interface (Top of foundation soil)

$T_1$: Weight structure $\rightarrow$ Consolidation shear stress
$T_2$: Ice load $\rightarrow$ Undrained shear stress
$T_f$: Final $= f(r, \theta)$

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

Figure by MIT OCW.
a) Plan View Below Top Cap

b) Cross Section A-A
Maximum normalized shear stress $\tau_x/\sigma_{vc}'$ versus test angle $q$ for CAUMDSS and Geonor CAUDSS tests on BBC. (Degroot, 1989)

Figure by MIT OCW.

Adapted from:

(Degroot, 1989)
Shear Stress-Strain Curves for CAUMDSS Tests on BBC.
OCR = 1 BBC  \( \tau_{c}/\sigma_{Vc} = 0.20 \)

**Peak Strength Comparison**

![Graph showing measured and predicted maximum shear resistance](image)

**Figure 6.13:** Measured and Predicted Maximum Shear Resistance \( \tau_{c}/\sigma_{Vc} \) Versus Test Angle \( \theta \) for CAUMDSS Behavior of BBC With \( \tau_{c}/\sigma_{Vc} = 0.2 \). (De Groot, 1989)
8. PROGRESSIVE FAILURE

8.1 Definition of Problem

Potential failure surface w/ representative elements that can be modeled in the lab

![Diagram of potential failure surface]

Typical CKw Data - low QCE

- Anisotropy → varying peak \( \tau \) at different strains
- PSC loses resistance before mobilizes peak \( \tau \) w/ DSS & especially PSE for strain softening clays

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

8.2 Strain Compatibility Technique

Koutsoufas & Ladd (1985)

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

2) Basic assumptions:
   a) Define \( \tau_e = \tau \) on shear plane at failure:
      \[
      \tau = 2 \cos \theta \quad \text{Triaxial} \quad \tau = \tau_h \quad \text{in OSB}
      \]
      For circular arcs
      \[
      \text{Wedge analysis (Not Conventional)}
      \]
   b) Uniform shear strain (\( \dot{\gamma} \)) along failure surface at moment before gross displacements → failure
3) Application - See SC-2 for AGS OCR = 1.4 (SHANYER) or Fig. 1, p.15 of Leed (1990) SC-3 for B2 OCR = 1.21 (RECOUP)

a) Plot $\tau = (\gamma \tau / \sigma_{vc})$ vs $\gamma$ ($= 1.5$ ft for triangle

$= 2.0$ ft for PS

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

- At given OCR, max resistance at max $\tau_{max}$
- If fan. clay has variable OCR, need
  judgment to select design $\gamma_{Tmax}$
- Also want $\gamma$ leading reasonable anisotropy
  strengths, i.e. values of $\tau_c$, $\tau_d$, $\tau_e$

c) For circular arc with "isotropy" strengths, use $\tau_{max}$
  wedge analyses, can use $\tau_c$, $\tau_d$, $\tau_e$

8.3 AGS Case History (KSL, 1985) - Handout

1) Background

- Breakwater for floating nuclear power plant with
- 3 stage construction (Fig.1,2)
- Initial m subs OCR = 4.2 ± 0.9 (Fig.2)

2) Application strain compatibility technique (Fig.7) = SC-2
at OCR = 1.4 + $\tau / \sigma_{vc}' = S (\sigma_{p}' / \sigma_{vc}')^m$ at $\gamma = 8$

<table>
<thead>
<tr>
<th>Mode of Failure</th>
<th>$S$</th>
<th>$m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSC $\tau_c$</td>
<td>0.265</td>
<td>0.79</td>
</tr>
<tr>
<td>DSS $\tau_d$</td>
<td>0.25</td>
<td>0.77</td>
</tr>
<tr>
<td>PSE $\tau_e$</td>
<td>0.16</td>
<td>0.88</td>
</tr>
<tr>
<td>Ave. $\tau_{ave}$</td>
<td>0.225</td>
<td>0.81</td>
</tr>
</tbody>
</table>
3) Resultant Cu profiles for initial inside condition
   (Fig.8 = SC-4)
   - $\tan / Cu(FV) = 0.725 \pm 0.015$ SD vs Bjerrum (1973)
   - $\mu = 0.84$ for $\phi_p = 43^\circ$, $\mu_p = 0.76$ after
     consideration of end effects à la Azzaoui et al. (1983)
   - Conclusions-wrt Bjerrum: Unsafe for PS failure ($\times 1.16$)
     OK for typical 3-D failure ($\times 1.05$)
   - Comments on Cu(UVC) data: ($\bar{\varepsilon} = 10\% / h$)
     - Increased scatter vs Cu(FV) & $\tan \pm 1$SD: Expected
     - Mean vs Cu(FV): more rapidly of depth-effect opposite
       to $\tau_{sc}$: 30% unsafe
       to $\tau_{sc}$: Larger - probably due to higher $\bar{\varepsilon}$
   - Conclusions:

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

<table>
<thead>
<tr>
<th>Method of Analysis</th>
<th>Cu Profile</th>
<th>$F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Wedge via M-P</td>
<td>SHANSEP</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>$\tau_c$, $\tau_d$, $\tau_e$</td>
<td></td>
</tr>
<tr>
<td>b) Same</td>
<td>$q_f$ from UVC</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>$C$ from $UVC$ ($q_f'/\sigma_c = 0.33$)</td>
<td></td>
</tr>
<tr>
<td>c) Wedge via USCE</td>
<td>Same</td>
<td>1.29</td>
</tr>
<tr>
<td></td>
<td>(in upper CL clay)</td>
<td></td>
</tr>
</tbody>
</table>

Conclusions - Wrong Cu + wrong analysis -> correct $F$
do to compensating errors

* Would get lower FS if used QRS envelope -> lower $\gamma_{so}$ = 0.26

1) $F(3-D)/F(2-D) = 1.11 \pm 0.06$ SD for 18 case histories (circular arc analyses
of embankment failures) = \[1 + 0.1(F)\]
8.4 Application to Several Clays

1) See SC-1, 1.2: for results that apply to PS failures for OCR=1 (SC-1 plots normalized Take 3Ks, plus Te ? Te nEp)

2) Based on these and some other data, typical effect of progressive failure on design cu is:
   Take, as above $\frac{\tau_p}{\tau_c}$ = ave. of peak $\tau$ values

Design 4% = 5 - 10%

- N.C. - $\frac{\tau_c}{\tau_p}$ = 0.9 ± 0.03
- OC - $= 0.95$ for low $s_u$ (e.g. BCC, IMGS)
- Design 2% OC - $= 0.85$ for very high $s_u$ like James Bay

Note: $\frac{q_c(TC)}{\tau_c} = 1.46 ± 0.18$

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

9.1 Bearing Capacity (PS)

1) D'Aniè & Christian (1971)

$$\Delta q_{4H} = \frac{1}{2} \left[ s_u(V) + s_u(H) \right] N_c' = f \left( \frac{b}{a} = \frac{s_u(45)}{\sqrt{s_u(V) \cdot s_u(H)}} \right)$$

- $= 5.14$ for $b/a = 10$
- $= 4.00$ for $b/a = 0$
- $= 5.0 ± 0.14$ for typical $b/a = 0.9 ± 0.1$

2) Definition $s_u = \circ$

3) Should apply strain compatibility to PS $C_{K_0}$ for PS problems

4) If use $C_{K_0} = C/E \rightarrow s_u(V) \cdot K_s$ for PS problem

Peak
- $TE/PSC = 0.92 ± 0.05 \rightarrow X \approx 0.87 \approx$ effects strain compatibility
- $TE/PSE = 0.82 ± 0.02$
5) Kinner & Hadd (1973) model footing tests on 
BBC at OCR = 1.2 & 4 (Table 11.4 of strength Nv_{UC}=SC-5) 
- Using peak \( q_f \) from CK, UPSCIE 
  predicted/measured \( q_f \) = 1.0 
- Explanation: Compensating errors: increased \( q_f \) due 
  to faster \( e \) offset shear compatibility

6) Other procedures to get \( s_u = C \) for \( q_f \) 
- \( q_f (UVC) \) DEPENDS ON COMPENSATING ERRORS (\( \varepsilon \) vs disturbance) 
- \( q_f (CVVC) \) ALWAYS UNSAFE 
- \( m s_u (FV) \)
  - For circular arc: neglecting end effects \( \rightarrow \) unsafe \( (x:11) \)
  - \( \varepsilon \) vs \( q_f \) \( = \) too low \( (x: \varepsilon CF = 0.87) \)
  - Compensating errors

9.2 Circular Arc Stability Analyses using "isotropic" strengths

1) Above comments/conclusions apply but now 
   presumably want \( \varepsilon \) vs \( q_f \) + end effects

2) Comparison of \( Cu(OS5) \) vs Table SC 
   From SC-1 \( Cu(OS5)/Ta = 1.07 \pm 0.07 \) (W: CVVC) 
   Slightly unsafe for plane strain failures 
   But for typical failure with 3-D effects, 
   on average is slightly conservative since \( F(3-D) = 1.11 \pm 0.0650 \)
   \( F(2-D) \)

3) Level C analysis using empirical correlation to estimate \( S_{Sm} \)
   as \( f(s) \) (soil type) \& 0.8 S_{Net} 5.4 of CCL (1991)
   - e.g. CL-CH \( S = 0.22 \) \( m = 0.8 \) 
     \( OH-NH = 0.25 \) \( m = 0.8 \) 
     \( CVCC = 0.16 \) \( m = 0.75 \)
STABILITY ANALYSIS OF EMBANKMENT

"Total stress" analysis → \( \phi = 0, c = c_u \)

- Critical shear surface from UTEXAS3 search (Spencer)
- Required input: \( c_u = f(\alpha) \)

Rockfill

\( \alpha \)

Sand

Clay (\( S = 100\% \))

TWO MAJOR QUESTIONS (Mohr-Coulomb Failure Criterion)

1) How to define \( c_u ? \)

\[
c_u = \frac{q_f \cos \phi}{2} \left[ q_f = 0.5 (\sigma_1 - \sigma_3) \right]
\]

2) Relationship between \( \alpha \) and \( S \)?

\[
\alpha = \theta - S
\]

\( \theta = 45 + \phi / 2 = \) angle between failure plane and \( \sigma_f \) plane.

SHOULD ONE USE: total stress \( \phi = 0 \) OR effective stress \( \phi' ? \)
**APPLICATION OF TWO HYPOTHESES (For 5 = 0°)**

**Using \( \phi = 0 \)**

- \( c_u = q_f \)
- \( \theta = 45° \)
- \( \alpha = 45° - \delta \)

**Using \( \phi = \phi' \) (Critical = Actual)**

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

---

**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.
ANISOTROPIC $c_u/\sigma_{vc}'$ RATIOS FOR STABILITY ANALYSES

<table>
<thead>
<tr>
<th>Line</th>
<th>Fig. 3 Data</th>
<th>$c_u = \frac{q_f}{\sigma_{vc}'} = \alpha \cdot \cos \phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>DSC</td>
<td>$q_f$ $45 - \delta$</td>
</tr>
<tr>
<td>B</td>
<td>TX &amp; DSS</td>
<td>$q_f$ $45 - \delta$</td>
</tr>
<tr>
<td>C</td>
<td>DSC</td>
<td>$q_f \cdot \cos \phi$ $\theta - \delta$</td>
</tr>
</tbody>
</table>

$\theta = 62^\circ$ $\phi = 34^\circ$

Figure by MIT OCW.

CONCLUSIONS

1) Run lab $C_k \sigma_U$ tests with varying $S$ to measure anisotropy
   - Apply corrections to TC/TE data · Assume $S=45\pm15^\circ$ for DSS

2) If the PREDICTED critical shear surface from a sophisticated search routine is close to the most likely actual failure surface, then:
   - Assuming $\phi = 0$ $\rightarrow$ $c_u = \frac{q_f}{\sigma_{vc}'} = 0.5(\sigma_1 - \sigma_3)$ $\downarrow$ Probably
     and $\alpha = 45^\circ - \delta$ $\downarrow$ UNSAFE
   - Assuming $\phi = \phi'$ $\rightarrow$ $c_u = \frac{q_f}{\sigma_{vc}'} = q_f \cdot \cos \phi'$ $\downarrow$ Recommended
     and $\alpha = (45 + \phi'/2) - \delta$ $\downarrow$
Simplified Approach Given Uncertainty in \( \alpha \) data for DST tests

Note: Drawn for \( \alpha = 60^\circ - 8 \) (\( \phi = 30^\circ \))

Replacing actual variation with stepped lines.
<table>
<thead>
<tr>
<th>No.</th>
<th>Soil</th>
<th>Index Properties</th>
<th>Peak $c_u / \sigma'_{vc}$</th>
<th>Strain Compatibility $c_u / \sigma'_{vc}$</th>
<th>C/E Testingb</th>
<th>Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>USC</td>
<td>Ip</td>
<td>Il</td>
<td>$q_f(TC)$</td>
<td>$r_{h(DSS)}$</td>
</tr>
<tr>
<td>(1)</td>
<td></td>
<td>(3)</td>
<td>(4)</td>
<td>(5)</td>
<td>(6)</td>
<td>(7)</td>
</tr>
<tr>
<td>1</td>
<td>B2 Marine Clay</td>
<td>CL</td>
<td>8.5</td>
<td>2.6</td>
<td>0.31</td>
<td>0.23</td>
</tr>
<tr>
<td>2</td>
<td>B6 Marine Clay</td>
<td>CL</td>
<td>13%</td>
<td>1.9</td>
<td>0.33</td>
<td>0.24</td>
</tr>
<tr>
<td>3</td>
<td>Resedimented BBC</td>
<td>CL</td>
<td>21%</td>
<td>1.0</td>
<td>0.33</td>
<td>0.20</td>
</tr>
<tr>
<td>4</td>
<td>Conn. Valley Varved Clay</td>
<td>CL/CH</td>
<td>12%</td>
<td>39%</td>
<td>0.25</td>
<td>0.16</td>
</tr>
<tr>
<td>5</td>
<td>Great Salt Lake Clay</td>
<td>CH</td>
<td>40%</td>
<td>1.1</td>
<td>0.37</td>
<td>0.24</td>
</tr>
<tr>
<td>6</td>
<td>AGS Marine Clay</td>
<td>CH</td>
<td>43%</td>
<td>0.6</td>
<td>0.325</td>
<td>0.255</td>
</tr>
<tr>
<td>7</td>
<td>Omaha, NE Clay</td>
<td>CH</td>
<td>60%</td>
<td>0.7</td>
<td>0.315</td>
<td>0.22</td>
</tr>
<tr>
<td>8</td>
<td>Arctic Silt A</td>
<td>ML</td>
<td>15%</td>
<td>0.3</td>
<td>0.37</td>
<td>0.245</td>
</tr>
<tr>
<td>9</td>
<td>Arctic Silt B</td>
<td>MH</td>
<td>30%</td>
<td>0.7</td>
<td>0.32</td>
<td>0.24</td>
</tr>
<tr>
<td>10</td>
<td>EABPL Clay</td>
<td>CH</td>
<td>75%</td>
<td>0.85</td>
<td>0.24</td>
<td>0.235</td>
</tr>
</tbody>
</table>

a Design shear strain selected for strain compatibility.
b $\text{TX} = \text{triaxial and PS} = \text{plane strain}$
c Triaxial $\tau_c$ increased by 5%.
d Approximate mean of plane strain and triaxial data.
Undrained Shear Strength Ratios vs. Plasticity Index for CL and CH Clays Treated for Strain Compatibility (Data from Table 4, Ladd 1991)
Normalized Stress-Strain Data used for the Strain Compatibility Technique.

Note:
Open symbol = Peak $\tau$
Solid symbol = Design $\tau$

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

Figure by MIT OCW.

Adapted from Köttöf and Ladd (1985)
Comparison of Initial Undrained Shear Strength Profiles
(1 ft = 0.305 m; 1 tsf = 95.8 kPa)

Figure by MIT OCW.

Adapted from Koutroulis & Ladd (1985)
## Predicted vs Measured Ultimate Bearing Capacity of Strip Footing on Boston Blue Clay

(from Kinnar & Lockl, 1970; Lockl, et al., 1971; Lockl and Edges, 1971)

<table>
<thead>
<tr>
<th>Undrained Shear Strength Determined From</th>
<th>OCR</th>
<th>Undrained Strength Ratio</th>
<th>Ultimate Bearing Capacity $q_{ult}/\overline{c}_{vc}$</th>
<th>Predicted</th>
<th>Predicted % Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\bar{C}K_o \bar{U}$ (3) Plane Strain Active &amp; Passive</td>
<td>1</td>
<td>0.265</td>
<td>0.34</td>
<td>0.19</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.47</td>
<td>0.57</td>
<td>0.37</td>
<td>2.41</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.81</td>
<td>0.95</td>
<td>0.67</td>
<td>4.15</td>
</tr>
<tr>
<td>$\bar{C}K_o \bar{U}$ (3) Plane Strain Active</td>
<td>1</td>
<td>0.34</td>
<td>0.34</td>
<td>–</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.57</td>
<td>0.57</td>
<td>–</td>
<td>2.93</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.95</td>
<td>0.95</td>
<td>–</td>
<td>4.88</td>
</tr>
<tr>
<td>$\bar{C}T\bar{U}$ (3) Triaxial Compression</td>
<td>1</td>
<td>0.325</td>
<td>0.325</td>
<td>–</td>
<td>1.67</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.555</td>
<td>0.555</td>
<td>–</td>
<td>2.85</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.90</td>
<td>0.90</td>
<td>–</td>
<td>4.62</td>
</tr>
<tr>
<td>$\bar{C}K_o \bar{U}$ (4) Direct-Simple Shear</td>
<td>1</td>
<td>0.20</td>
<td>–</td>
<td>–</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.37</td>
<td>–</td>
<td>–</td>
<td>1.90</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.61</td>
<td>–</td>
<td>–</td>
<td>3.14</td>
</tr>
<tr>
<td>$\bar{U} \bar{U}$ (3) Triaxial Compression (D'Appoloni, 1968)</td>
<td>1</td>
<td>0.18</td>
<td>0.18</td>
<td>–</td>
<td>0.925</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.36</td>
<td>0.36</td>
<td>–</td>
<td>1.85</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.60</td>
<td>0.60</td>
<td>–</td>
<td>3.08</td>
</tr>
</tbody>
</table>

1. Predicted $q_{ult} = N_c \bar{s}_u(ovl)$ with $N_c = 5.14$ (Davis & Christon, 1971)
2. Measured at $L/B = 0.1$ with $\overline{c}_{vm} = 3.4$ kN/cm²
3. $\bar{s}_u = \bar{q}_f = \frac{1}{2} (G - G_f)$
4. $\bar{s}_u = \bar{c}_m$ maximum

Ladd (1971) Table II-4