III-2 STRESS-STRAIN-STRENGTH PROPERTIES

	Page No
1. Introduction	2
2. Three Most Common Tests	2
2.1 1-D Consolidation Test	2
2.2 Direct Shear Test	3
2.3 Triaxial Test	3
3. Strength of Cohesionless Soils (At "low" confinement)	5
3.1 Mohr-Coulomb Failure Criteria (+Table III 2-1)	5
3.2 Presentation of Triaxial Test Data	6
3.3 Interpretation of Direct Shear Test	6
3.4 Effect of Relative Density	7
3.5 Three Components of Strength (Rowe 1962)	7
4. Combined Effects of Density & Confining Pressure on Strength of Granular Soils	9
4.1 Overview of Data from Std. Triaxial Compression Tests (+Fig. III 2-1)	9
4.2 State Parameter ψ	11
4.3 Semi-Empirical Correlations (Bolton 1986)	12
4.4 Soil Model MIT-S1 (Pestana 1994)	13
5. Other Factors Affecting the Strength of Granular Soils	15
5.1 Intermediate Principal Stress	15
5.2 Method of Sample Preparation	16
5.3 Anisotropy	16
6. 1-D Behavior of Granular Soils	17
6.1 Data Presentation & Definition of Parameters	17
6.2 Factors Affecting Compressibility	18
6.3 Coef. of Earth Pressure at Rest (K_0)	19
7. Influence of Stress Path on Stress vs. Strain	20

Sheets

A: CIDC(L) data on loose & dense sand as $f(\sigma')$
B: Particle crushing
C1,2: State Parameter ψ
D: Bolton (1986)
E1-4: MIT-S1 model & shear data
F: Effect of b & sample preparation
G: Anisotropy data
H: K_0 data

PART III-2 STRESS-STRAIN-STRENGTH PROPERTIES (p2)

1. INTRODUCTION

(1) If soil were linear-elastic-isotropic with infinite strength, then one simple test → 2 elastic constants to completely define $\sigma-\varepsilon$ characteristics

(2) But soil is "particulate" system of finite strength wherein plastic strains result from:

a) Deformation of particles - elastic & crushing (granular at high σ')

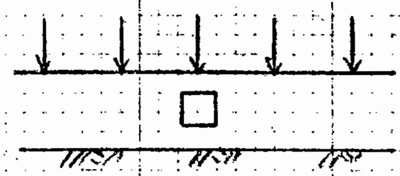
b) Sliding & rolling amongst particles

(3) Soil Mechanics therefore developed several types of tests that attempt to simulate typical conditions encountered in the field. (Long before existance of reliable SOIL MODELS)

2. THREE MOST COMMON TESTS

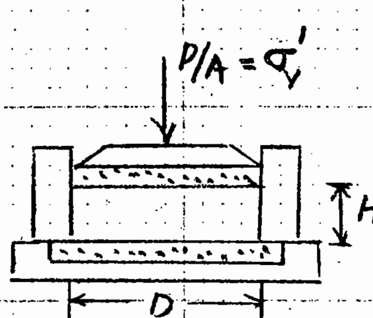
2.1 1-D Consolidation (Compression) = Oedometer Test (For Stress-strain)

(1) Field situation



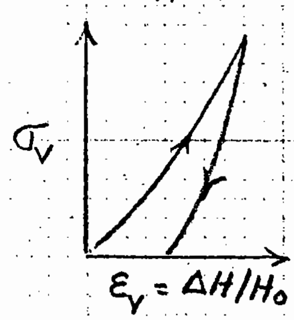
Loaded area is large wrt soil thickness; m & n → 5

(2) Lab test



$D \approx 2.5-3"$, $D/H = 3-4$
(60-75 mm)

(3) Typical $\sigma-\varepsilon$



- Strain hardening
- Plastic deformations
- Constrained modulus

$D = \Delta\sigma_v'/\Delta\varepsilon_v = 1/m_v$

(Coef. of volume change)

(4) "Elastic" relationships

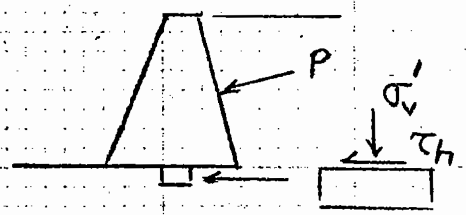
$$D = \frac{E'(1-\mu')}{(1+\mu')(1-2\mu')}$$

$$K_o = \mu'/(1-\mu') = 0.50 \text{ for } \mu' = 1/3$$

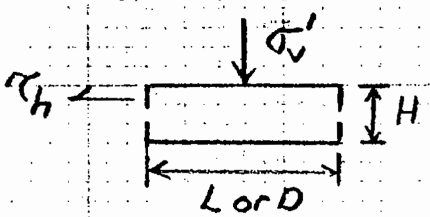
PART III-2 STRESS-STRAIN-STRENGTH PROPERTIES (p3)

2.2 Direct (Box) Shear Test (One of 1st strength tests)

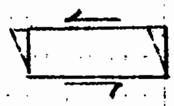
(1) Field situation



(2) Lab test

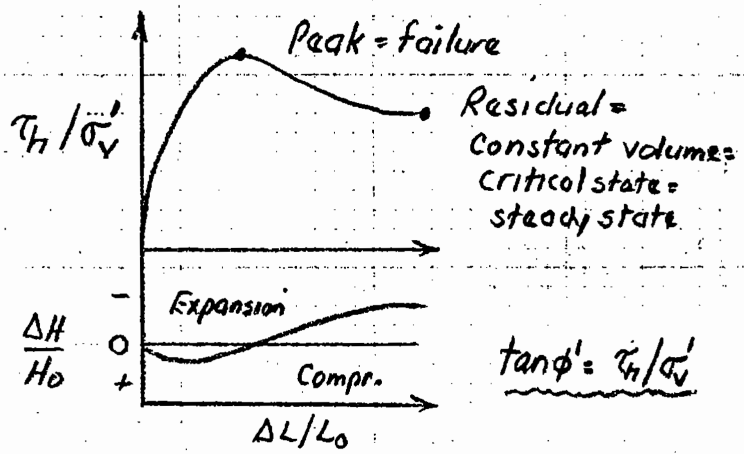


- Dimensions like oedometer
- Non-uniform strains
- Poor version of simple shear



(3) Typical τ_h - "Strain"

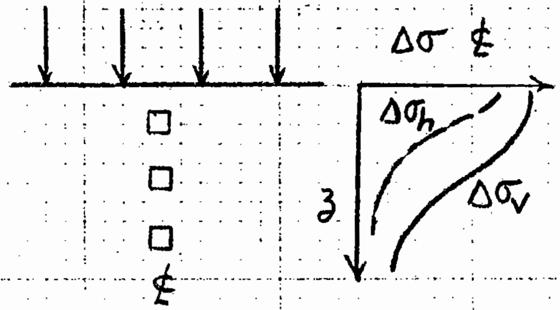
Medium-dense sand



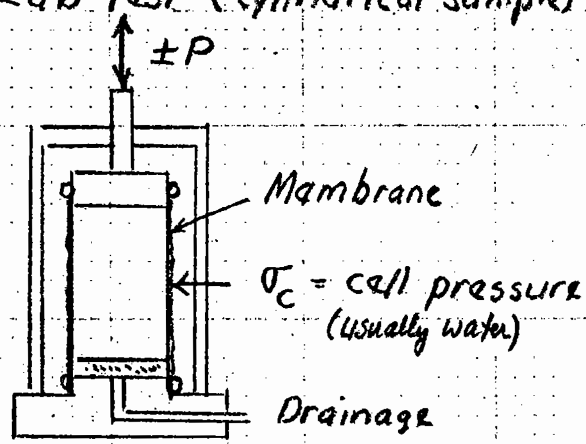
$\tan\phi' = \tau_h / \sigma_v'$

2.3 Triaxial Test (1st used soils 1930's; both σ-ε & strength)

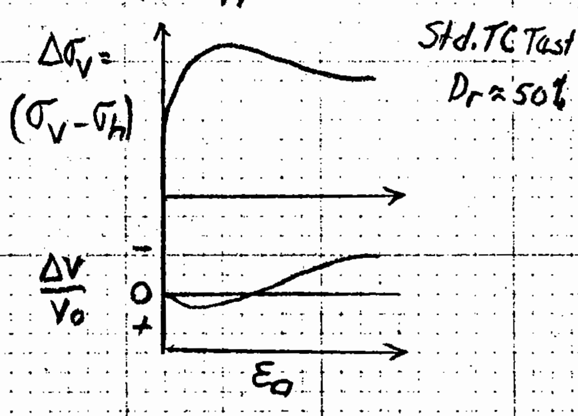
(1) Field situation



(2) Lab test (cylindrical sample)



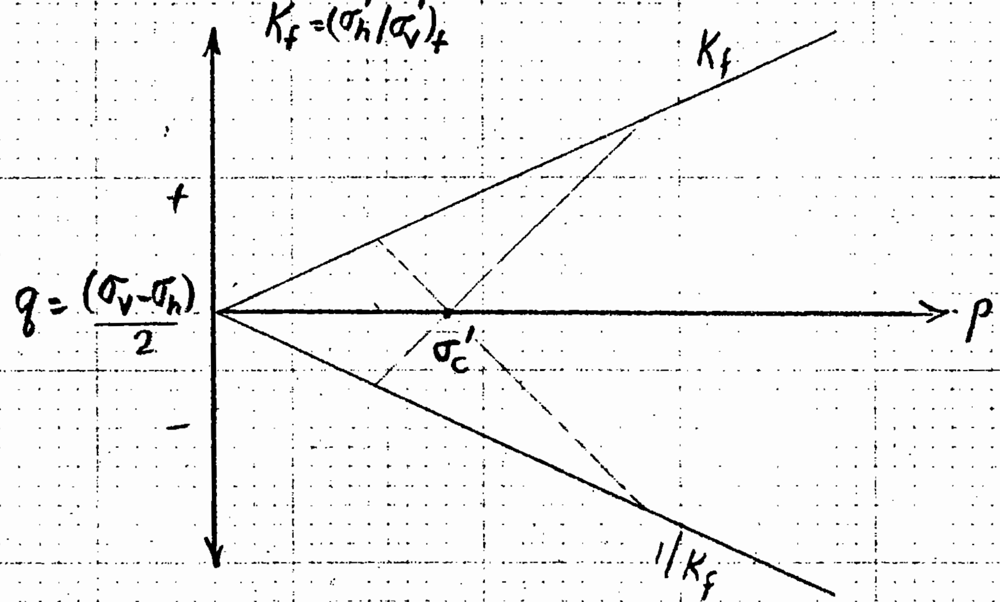
(3) Typical σ-ε



- $D = 1.5-3''$, $H/D \approx 2 - 2\frac{1}{2}$
- $\sigma_v = \sigma_c + P/A$ (P + or -)
  $\sigma_h = \sigma_c$
- Controlled stress or strain (Constant $\dot{\varepsilon} = d\varepsilon/dt$ most common)
- $\varepsilon_a = \Delta H / H_0$

2.3 <u>Triaxial Test</u> (Continued)

(4) Four basic types of tests (drained starting from isotropic σ_c')



<u>Triaxial Compression</u> ($\sigma_2 = \sigma_3 = \sigma_h$)

- $\sigma_v > \sigma_h$ ($K < 1$)
- Axial (vertical) compression of sample
- $b =$ ___

<u>Triaxial Extension</u> ($\sigma_2 = \sigma_1 = \sigma_h$)

- $\sigma_h > \sigma_v$ ($K > 1$)
- Axial (vertical) extension of sample
- $b =$ ___

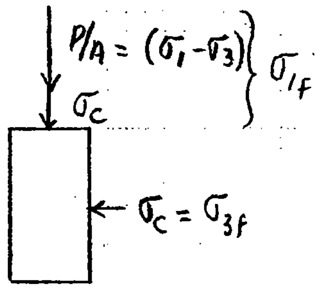
Letter	$\Delta\sigma$	Description	Field Case
A	$+\Delta\sigma_v$, $\Delta\sigma_h = 0$		
B	$\Delta\sigma_v = 0$, $-\Delta\sigma_h$		
C	$\Delta\sigma_v = 0$, $+\Delta\sigma_h$		
D	$-\Delta\sigma_v$, $\Delta\sigma_h = 0$		

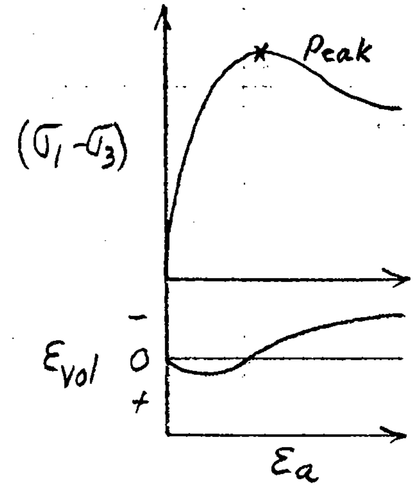
- Can, of course, vary both σ_v & σ_h during testing
- σ_2 condition defined by $b = \frac{(\sigma_2 - \sigma_3)}{(\sigma_1 - \sigma_3)}$ = ___ TC
  = ___ TE

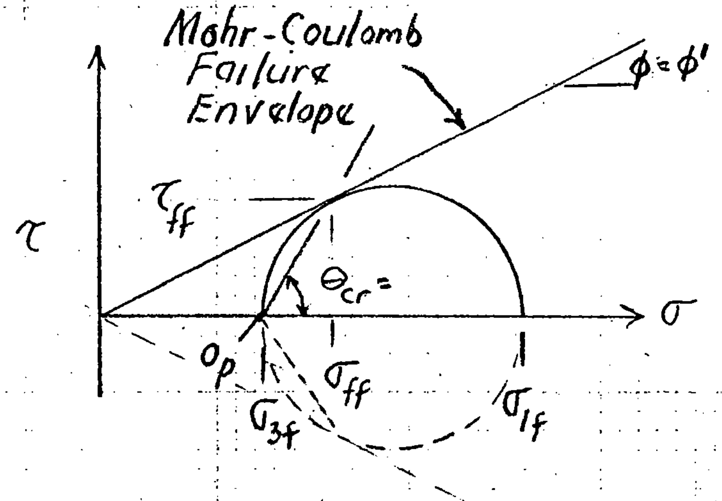
3. STRENGTH OF COHESIONLESS SOILS (At "Low" Confinement)

3.1 Mohr-Coulomb Failure Criteria

(1) Std. TC tests at varying σ_c' on med-dense sand (Drained, $\sigma = \sigma'$)





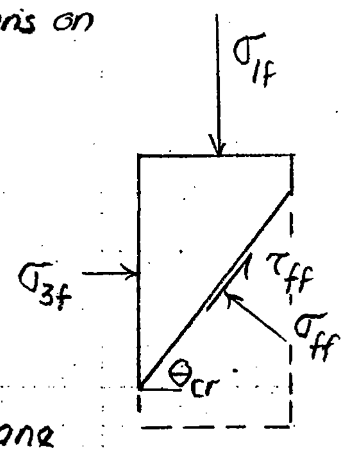


- ϕ' = friction angle for straight line thru origin
- Bottom half is symmetrical

(2) Mohr-Coulomb failure criteria state:

No. 1 Envelope represents limiting condition of state of stress, i.e. cannot have SOS for which Mohr circle lies above envelope

No. 2 When Mohr circle tangent to envelope, then point of tangency represents conditions on the failure plane = rupture surface where shear stress = shear strength; leading to large deformations

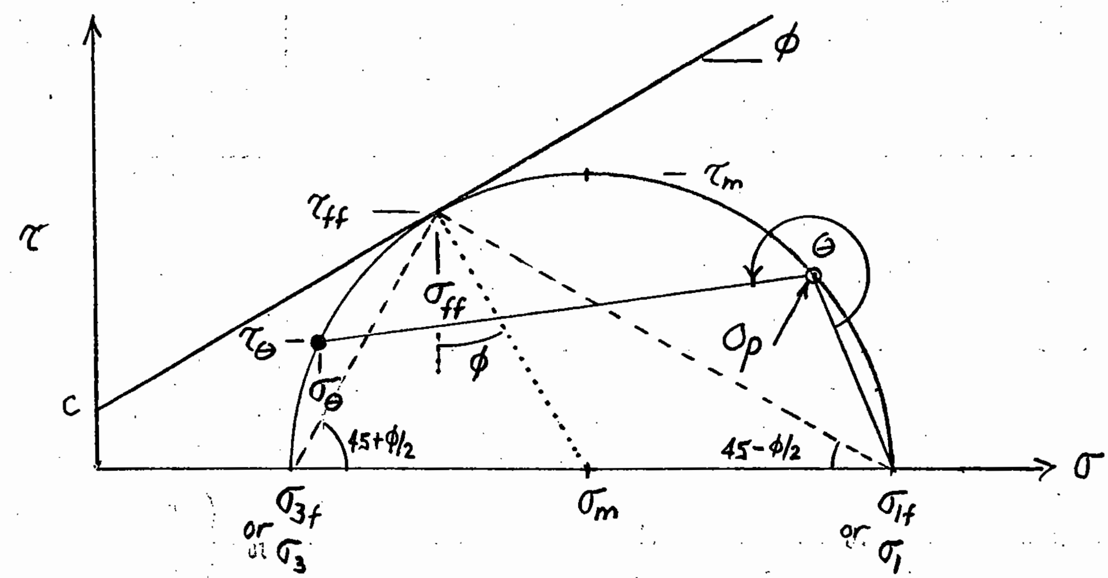


- $\tau_{ff} = \sigma_{ff}' \tan\phi'$
- ff = on failure plane at failure
- θ_{cr} = ∡ between failure plane & σ_{1f} plane

NOTE: See Table III 2-1 for some useful equations (p 5a)

Table III 2-1 Equations for Computing Stresses with Mohr Circle

Note: θ is angle between plane and σ_1 plane (+)



Definitions & Identities

⟨1⟩ $N_\phi = \dfrac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\phi/2)$ { For $c=0$, $R_f = (\sigma_1/\sigma_3)_f = N_\phi$; $\sin\phi = \dfrac{(R_f-1)}{(R_f+1)}$ ⟨2⟩

⟨3⟩ $\sqrt{N_\phi} = \tan(45+\phi/2) = \dfrac{\cos\phi}{1-\sin\phi}$; $\dfrac{1}{\sqrt{N_\phi}} = \tan(45-\phi/2) = \dfrac{\cos\phi}{1+\sin\phi}$ ⟨4⟩

For Any State of Stress

⟨5⟩ $\tau_{max} = \tau_m = 0.5(\sigma_1 - \sigma_3)$; $\tau_\theta = \tau_m \sin 2\theta = 2\tau_m \sin\theta \cos\theta$ ⟨6⟩

⟨7⟩ $\sigma_{mean} = \sigma_m = 0.5(\sigma_1 + \sigma_3)$; $\sigma_\theta = \sigma_m + \tau_m \cos 2\theta = \sigma_1 \cos^2\theta + \sigma_3 \sin^2\theta$ ⟨8⟩

For States of Stress at Failure

⟨Eq. No.⟩

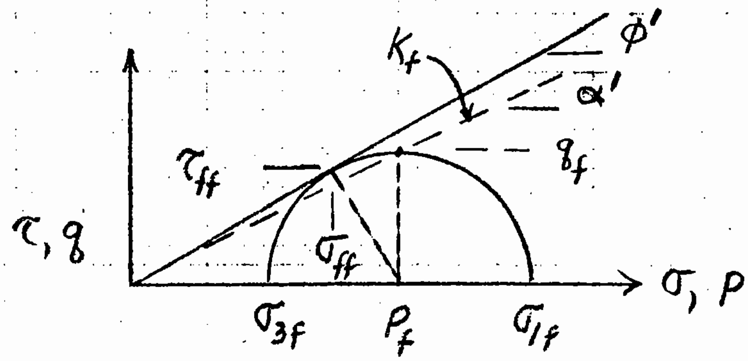
⟨9⟩ $\tau_{ff} = \tau_m \cos\phi = c + \sigma_{ff} \tan\phi$ ⟨10⟩

⟨11⟩ $\sigma_{ff} = \tau_{ff} / \tan\phi = \sigma_m - \tau_m \sin\phi$ ⟨12⟩

⟨13⟩ $\sigma_{1f} = \sigma_{ff} + \sqrt{N_\phi}\, \tau_{ff} = \sigma_{3f} N_\phi + 2c\sqrt{N_\phi}$ ⟨14⟩

⟨15⟩ $\sigma_{3f} = \sigma_{ff} - \dfrac{\tau_{ff}}{\sqrt{N_\phi}} = \dfrac{\sigma_{1f}}{N_\phi} - \dfrac{2c}{\sqrt{N_\phi}}$ ⟨16⟩

3.2 Presentation of Triaxial Test Data ($\sigma = \sigma'$)



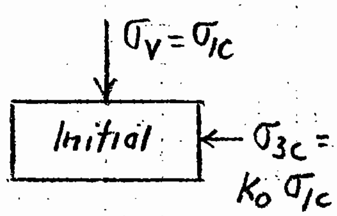
$$\frac{q_f}{p_f} = \tan\alpha' = \sin\phi'$$

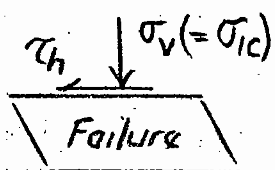
For ease of presentation, NOT different failure criteria

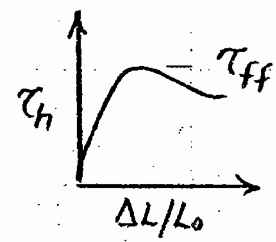
3.3 Interpretation of Direct Shear Test (Really indeterminate)

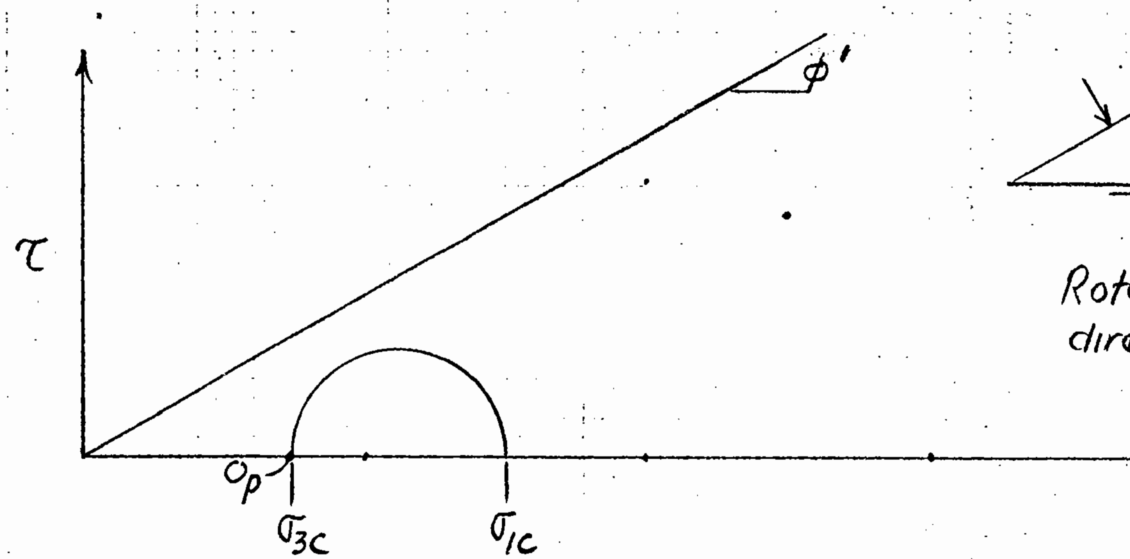
(1) Usual assumption of horizontal failure plane, i.e., $\tau_{h\,max} = \tau_{ff}$ ($\sigma = \sigma'$)

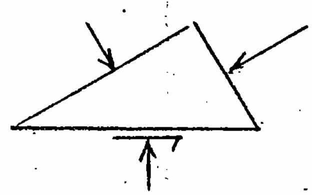
- For NC sand starting from K_o condition





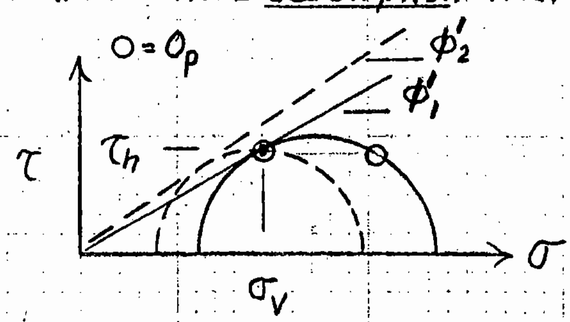






Rotation σ_1 direction =

(2) Alternative assumption that $\tau_h(max) = \tau_{max}$ ($\sigma = \sigma'$)



$\phi_1' = \arctan \tau_h/\sigma_v$ } Common, Conservative

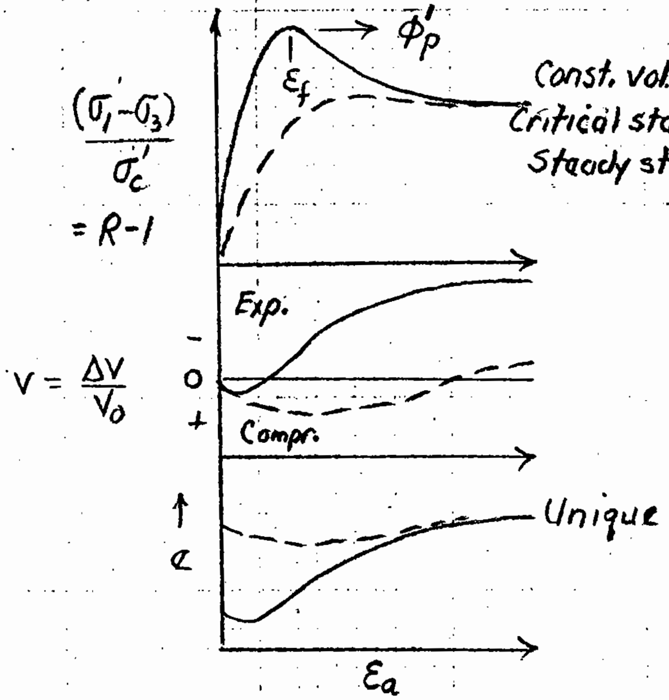
$\phi_2' = \arcsin \tau_h/\sigma_v$

($\tau_h/\sigma_v = 0.6 \rightarrow \phi' = 31^\circ$ vs. 37°)

3.4 Effect of Relative Density (Illustrated via Std. TC tests)

(1) Stress-strain data ($\sigma = \sigma'$)

——— Dense, - - - - Loose $\Big\}\ \sigma_c' = \sigma_{3f}' \approx 1\,atm$



Dense

- Small ε_f
- Significant strain softening
- Initial small contraction, then large expansion (dilation)

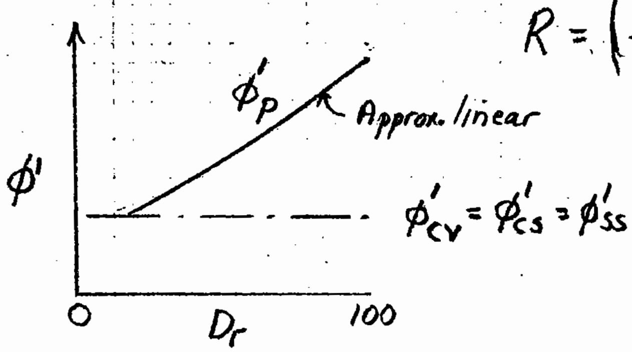
Loose

- Large ε_f
- Little strain softening

For Both at Critical = Steady State

* Unique e-q-p condition
* with continued shearing
[Called Critical State Line = Steady State Line]

(2) Variation in ϕ'

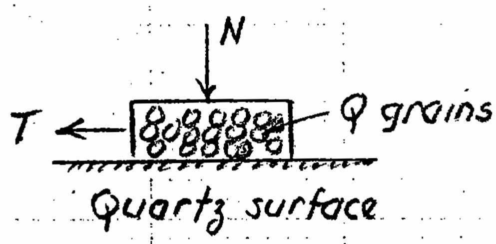


$$R = \left(\frac{\sigma_1}{\sigma_3}\right) = \tan^2(45 + \phi'/2) = \frac{(1 + \sin\phi')}{(1 - \sin\phi')} \quad \Big\}\ \text{L\&W Fig. 11.5}$$

- Also $\sin\phi' = (R-1)/(R+1)$

3.5 Three Components of Strength (Rowe, 1962; differs from L&W)

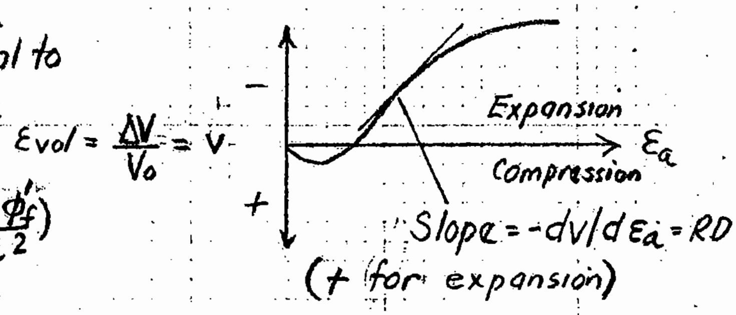
(1) Frictional resistance



- Coef. of friction $\mu = T/N = \tan\phi_\mu'$
- Rowe (1962) states that ϕ_μ' due to sliding only
- But more recent research indicates that also rolling at high ϕ_μ' (Skinner, 1969, Geot.)

(2) Resistance due to <u>Dilation</u>

- Component due to expansion of soil during shear against the confining stresses (expansion from "interlocking")
- Magnitude is proportional to rate of volume change



- $R_p = \left(\frac{\sigma_1'}{\sigma_3'}\right)_{max} = (1 + RD)\underbrace{\tan^2(45 + \frac{\phi_f'}{2})}_{R_f}$

MEASURED, BACKCALCULATED

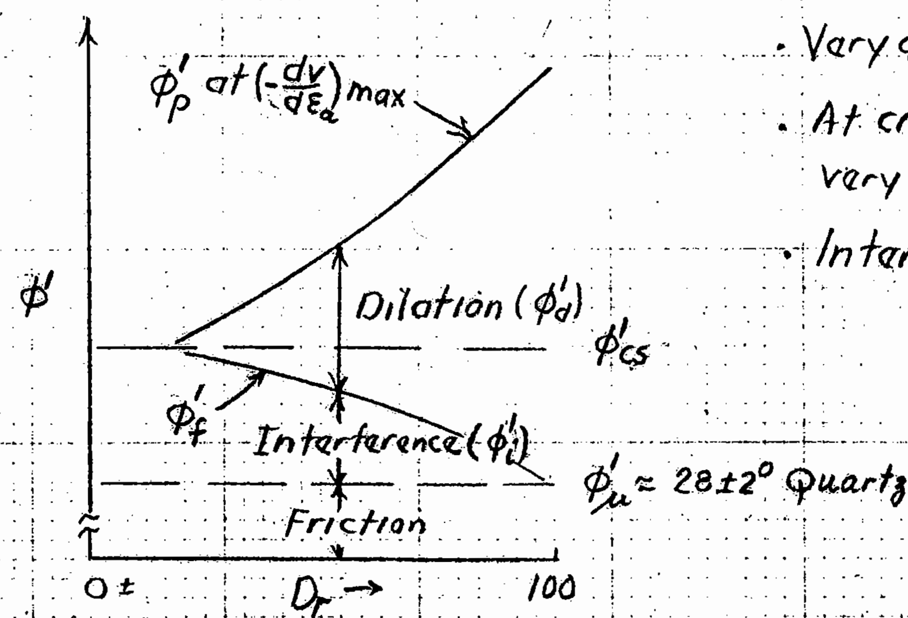
NOTE: ϕ_p' occurs at max. slope

(3) Resistance due to <u>Interference</u>

- "Interlocking" also results in fact that sand particles cannot move in a straight line, but must go around each other
- At const. vol. ($dv/d\varepsilon_a = 0$) → $\tan\phi_{cs}' \approx \frac{\pi}{2}\tan\phi_\mu'$

($\frac{\pi}{2} = \frac{1}{2}$ circumference / diameter) (really not that simple)

(4) Summary



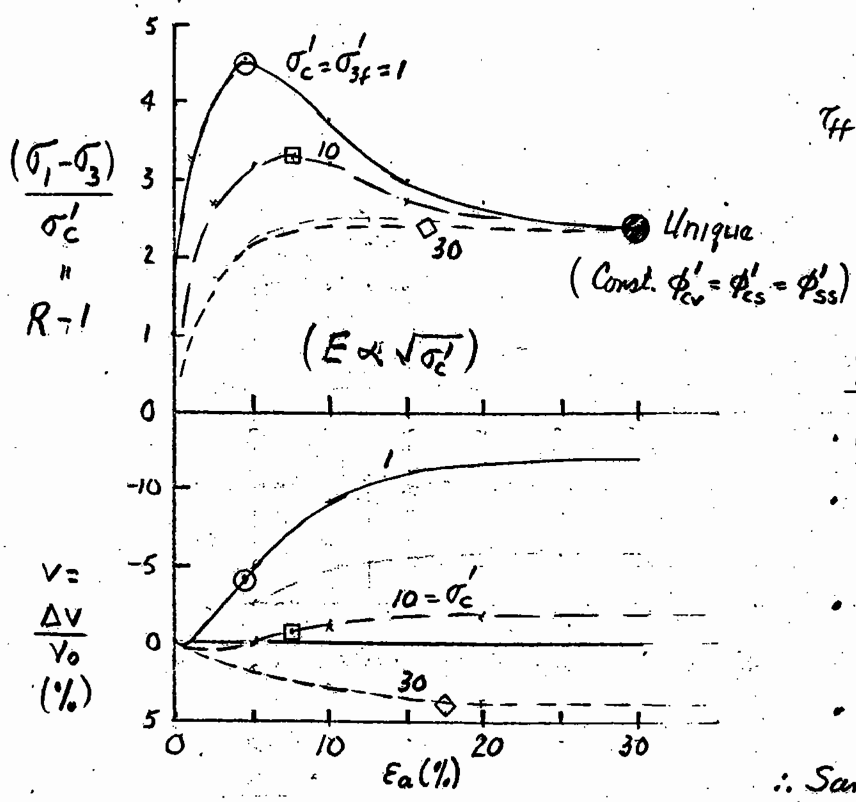
- Very dense: $\phi_p' = \phi_\mu' + \phi_d'$
- At critical state and very loose: $\phi_p' = \phi_{cs}' = \phi_\mu' + \phi_i'$
- Intermediate: $\phi_p' = \phi_\mu' + \underbrace{\phi_i' + \phi_d'}_{\text{L\&W "interlocking"}}$

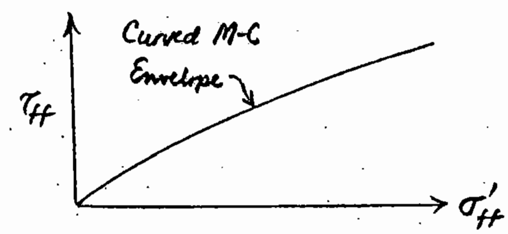
NOTE: ϕ_f' calculated from measured ϕ_p' (i.e. R_{max}) & max. $(-dv/d\varepsilon_a)$

4. COMBINED EFFECTS OF DENSITY AND CONFINING PRESSURE ON STRENGTH OF GRANULAR SOILS

4.1 Overview of Data From Standard Triaxial Compression Tests

1) Effect of confining stress level on stress-strain behavior of dense sand (idealization of data shown in Fig. 3 of Sheet A)





Trends for Increasing σ'_c

- Increasing ε_f (ε at peak)
- Less expansion (dilation) to only compression (contraction)
- Lower max rate of dilation ($RD = -dv/d\varepsilon_a$)
- Decreasing ϕ'_p

∴ Same basic trends as effect of decreasing D_r at low σ'_c

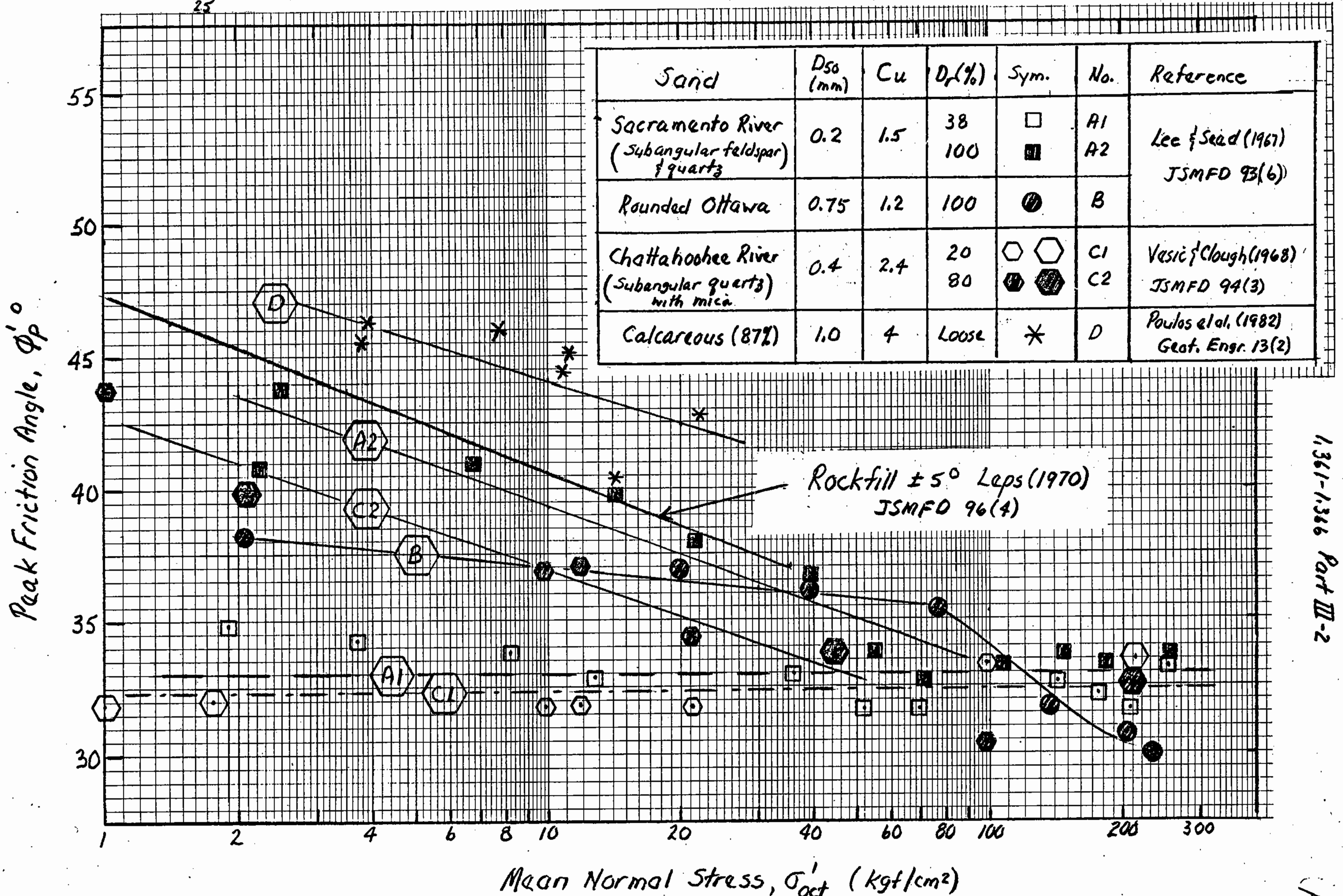
2) Peak friction angle (ϕ'_p) vs. confining stress at failure (see Fig III 2-1, p 9a)

- See large variation in "pressure sensitivity" ($d\phi'_p / d\log \sigma'_{oct}$) for different densities and type of granular soil
- In general, larger pressure sensitivity
  - With increasing D_r, e.g. data of Lee & Seed (1967) and Vesic & Clough (1968)
  - With weak sand grains, e.g. Ottawa sand (Quartz) → calcareous sand

∴ Therefore related to compressibility of test material

Note: $d\phi'_p / d\log\sigma' \rightarrow$ 5-10° for rockfill, very dense sand & gravel and for calcareous soils (even when loose, see line D in Fig. III 2-1)

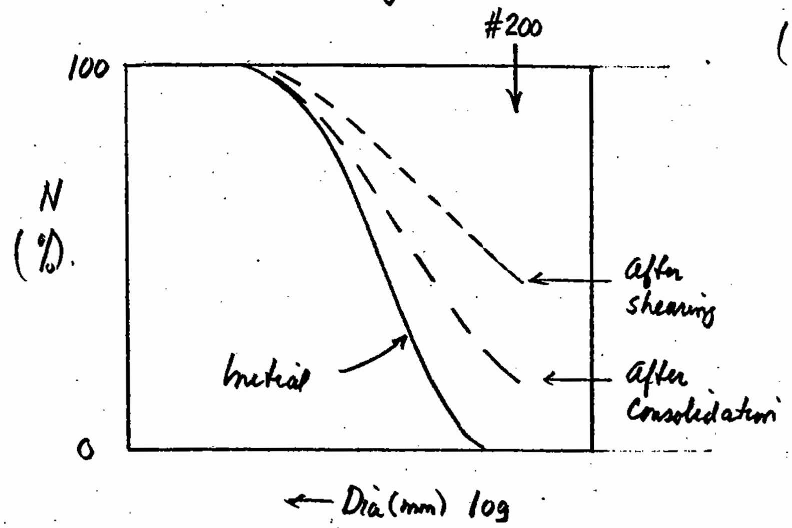
CCL 9/19/95 9/96
25



Sand	D_{50} (mm)	Cu	D_r(%)	Sym.	No.	Reference
Sacramento River (Subangular feldspar & quartz)	0.2	1.5	38 100	□ ■	A1 A2	Lee & Seed (1967) JSMFD 93(6)
Rounded Ottawa	0.75	1.2	100	●	B	
Chattahoochee River (Subangular quartz with mica)	0.4	2.4	20 80	⬡ ⬢	C1 C2	Vesic & Clough (1968) JSMFD 94(3)
Calcareous (87%)	1.0	4	Loose	*	D	Poulos et al. (1982) Geot. Engr. 13(2)

Fig. III 2-1 Effect of Stress Level on Peak Friction Angle of Loose and Dense Granular Materials

3) Particle crushing



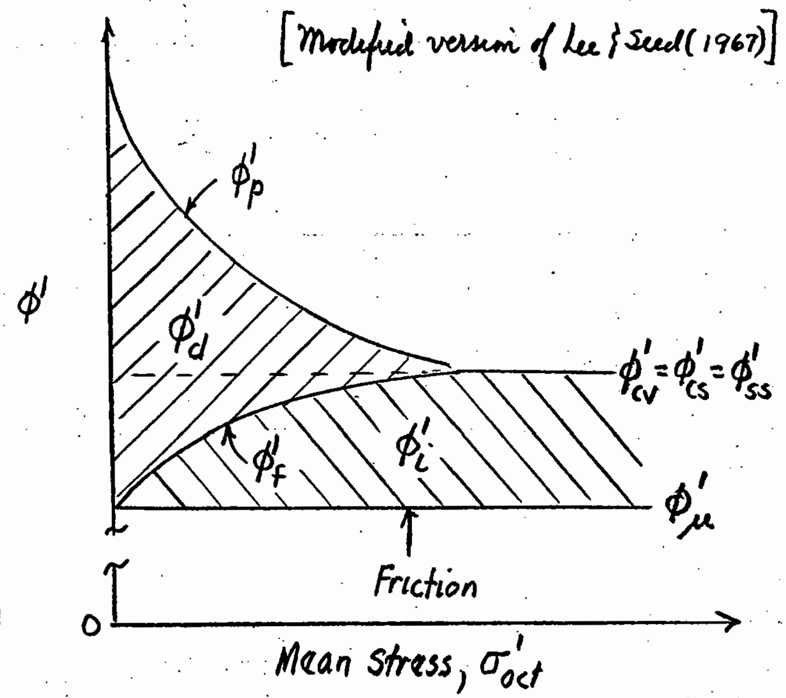
(See Sheet B for actual data)

Dense Ottawa: little crushing at $\sigma_c' = 40$ ksc and low pressure sensitivity

Dense Chattahoochee & Sacramento River Sands: alot of crushing and high pressure sensitivity

4) Strength components of dense sand with increasing confinement

[Modified version of Lee & Seed (1967)]



$R_p = \tan^2(45 + \phi_p'/2)$

$R_f = \tan^2(45 + \phi_f'/2)$

Rowe (1962)

$R_p = (1 + RD)\, R_f$

Bishop (1954) & Taylor (1948)

$R_p = RD + R_f$, i.e. predicts smaller effect of dilation

where $RD = \max(-dv/d\varepsilon_a)$

Notes: (1) ϕ_i' = interference + particle crushing

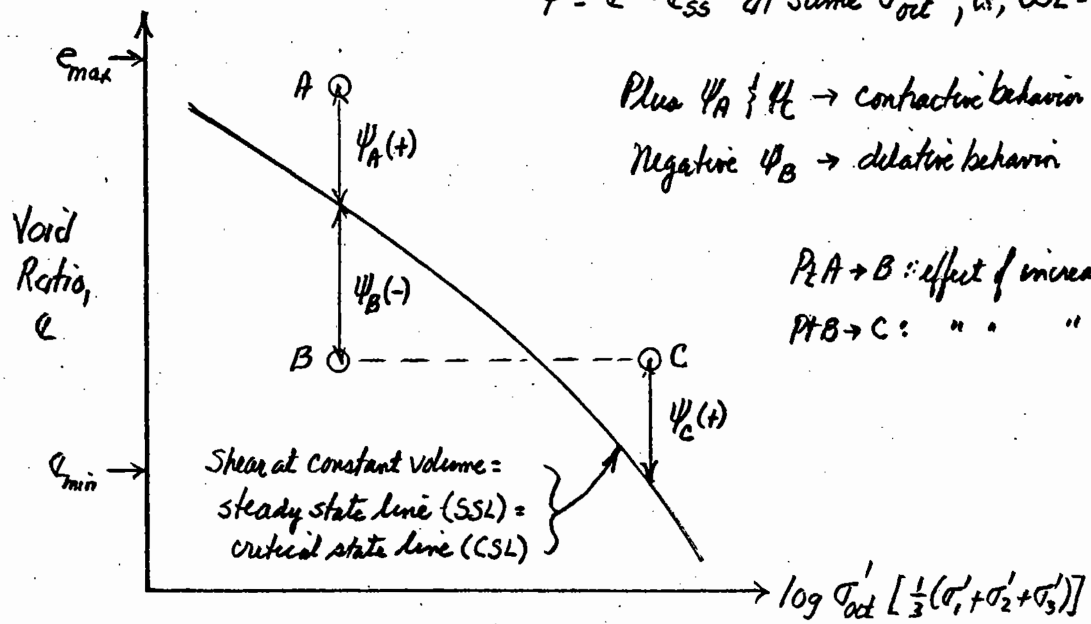
(2) At very high σ_{oct}', may require very large strains to reach CSL = SSL (e.g., see Fig. 5.27, Sheet E3)

Will next look at three different approaches for evaluating the combined effects of density and confinement on stress-strain-strength of sands

4.2 <u>State Parameter, ψ</u> (Been & Jefferies 1985; Been et al. 1991)

1) Definition

$\psi = e - e_{ss}$ at same σ'_{oct}, i.e., SSL = reference state



Plus ψ_A & ψ_C → contractive behavior } during shear
Negative ψ_B → dilative behavior }

Pt A → B : effect of increasing D_r
Pt B → C : " " " σ'

2) Examples of "unique" correlations between ψ and shear behavior

See Sheet C1 for actual correlations:

- Fig 14: Decreasing ψ → increasing rate of dilation at peak strength
- Fig 15: " " " " $\phi'_p - \phi'_{ss}$ ($\phi'_{ss} = \phi'_{cv} = \phi'_{cs}$) } ± 2-3°
- Fig 16: " " " " ϕ'_p

3) Problems with application in practice

a) Slight variations in particle size distribution can have a large effect on location of the SSL, e.g. Sheet C1, Fig. 7 for uniform medium sand with 0, 2, 5 & 10% fines (-#200)

b) Whether or not drained vs undrained shear from ± ψ will produce the same SSL = CSL is still controversial. Sheet C2, Fig. 12 indicates that drained shear with $-\psi$ does not reach SSL (maybe due to non-uniform conditions (shear planes) in test specimens)

c) Get marked curvature in CSL at stresses → significant crushing, e.g. Sheet C2, Figs. 8, 11 & 12

CCL Conclusion: Excellent concept (especially for teaching), but difficult to use quantitatively in practice

4.3 Semi-Empirical Correlations (Bolton 1986)

1) Approach: Evaluated drained TC and PS (plane strain) shear data from 17 test programs (Sheet D, Table 1) to determine effects of D_r and σ' level on max. rate of dilation and especially $\Delta\phi' = \phi'_p - \phi'_{cs}$ ($\phi'_{cs} = \phi'_{cv}$).

2) Results of study (for tests that dilate during shear, i.e., start with $\psi < 0$) led to I_R = relative dilatancy index (Note: at $\sigma' \uparrow \rightarrow$ significant crushing)

$$I_R = D_r\,(10 - \ln \sigma'_f) - 1$$ where D_r = relative density (decimal), $\sigma'_f = \sigma'_{oct}$ at failure in kPa

3) Resulting correlations for Std. TC [CIDC(L)]

$$\left.\begin{array}{l}\Delta\phi' = \phi'_p - \phi'_{cs} = 3\cdot I_R^{\circ} \text{ and Max. } RD = 0.3\, I_R\end{array}\right\}\ 0 \le I_R < 4$$

(Note: For plane strain, $\Delta\phi' = 5\cdot I_R^{\circ}$)

4) Examples of predictions vs. measured data (See Sheet D)

- Fig. 7 Effect of increasing D_r on $\Delta\phi'$ and max RD at $\sigma'_{oct} \approx 300$ kPa (both TC & PS)
- Fig 9 Effect of increasing D_r on $\Delta\phi'$ (TC) at σ'_{oct} = 20, 50, 100 & 600 kPa (quite good)
- Fig. 10 Effect of increasing σ'_{oct} at varying D_r on $\Delta\phi'$ (TC)

5) Bolton (1986) also suggests that: typical error in $D_r = \pm 5\%$; for mainly quartz grains, $\phi'_{cs} = 33° \pm 1\text{-}2°$; for feldspar grains, $\phi'_{cs} \approx 40°$

6) Pestana (1994) concludes from results in Section 4.4 that $\Delta\phi'$ (TC) is reasonable at $\sigma'_{oct} > 100$ kPa, except when $D_r \rightarrow 100\%$. But $\Delta\phi'$ (PS) is too high.

4.4 Soil Model MIT-S1 (J. Pestana 1994 ScD Thesis)

1) Background on formulation for granular soils

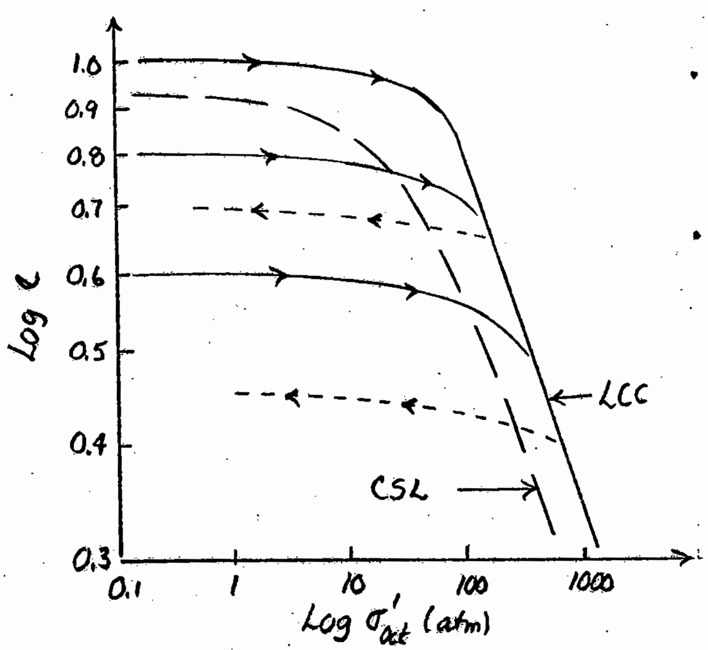
- Uses Limiting Compression Curve (LCC) = linear portion of $\log e$ vs. $\log \sigma'_{oct}$ compression curve at high stresses where particle crushing predominates as the reference state (like VCL for clays).
  See Sheet E1, Fig. 2.2 & 2.5
- For shear behavior, starts with "basic" elasto-plasticity theory (à la MCC), but adds alot of new features to incorporate hysteresis, anisotropy, strain softening, etc. See Sheet E1, Fig. 4.8

2) Input parameters for Toyoura Sand and some remarks

a) See Sheet E2, Table 5.2 for testing to obtain 14 parameters. Main tests are:

- Compression test to high σ'_c to define location of LCC. with U/R cycle (hysteresis) and values of K_0
- Undrained TC test from σ'_c on LCC } shape of bounding surface, ϕ_p & ϕ_{cs}, etc.
- Drained TC test at low σ'_c
- Resonant column to get small strain stiffness

b) Sketch of Sheet E2, Fig. 5.4 & 5.24



- Using input data from a test at one e_0, predicts compression & unloading at all values of e_0
- Also predicts location of the critical state line (CSL)!

3) Prediction of drained TC shear behavior of Toyoura Sand

a) Effect of varying D_r at low $\sigma_c' = 100$ kPa (≈ 1 atm)

Sheet E3, Fig. 5.26

b) Effect of varying σ_c' at $e_0 = 0.8$ ($D_r = 45\%$)

Sheet E3, Fig. 5.27

c) Combined effects of varying D_r and σ_c' on ϕ_p'

Sheet E4, Fig. 5.28 Note $\approx$ linear ϕ_p' vs e_0 at constant σ_c'

d) Comparisons with Bolton's (1986) empirical eqn. for ϕ_p' and RD

Sheet E4, Fig. 5.29 → rather remarkable agreement

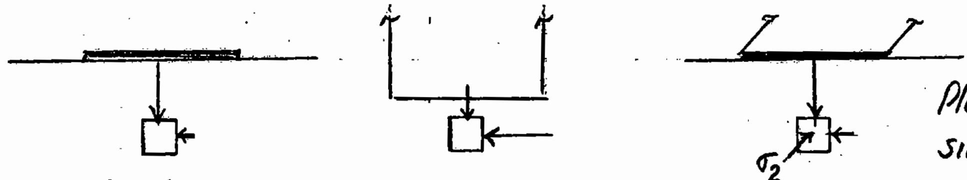
NOTE: Most of the measured triaxial compression shear data on Toyoura Sand are from undrained shear tests. Comparison of predicted vs. measured behavior is covered in 1.322 (Soil Behavior)

5. OTHER FACTORS AFFECTING THE STRENGTH OF GRANULAR SOILS

5.1 Intermediate Principal Stress (σ_2)

1) Field conditions leading to different values of $b = \dfrac{(\sigma_2 - \sigma_3)}{(\sigma_1 - \sigma_3)}$

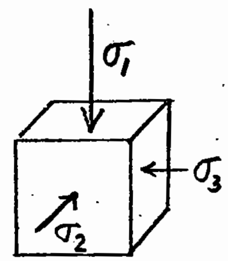
Circular Footing | Circular Excavation | Strip Footing



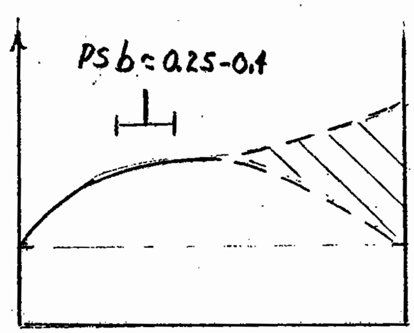
$\sigma_2 = \sigma_3 = \sigma_h$ | $\sigma_2 = \sigma_1 = \sigma_h$ | $\sigma_1 > \sigma_2 > \sigma_3$

TC, b = ___ | TE, b = ___ | PS, $b \approx 0.25 - 0.4$

2) Device to measure the effect of varying b = True Triaxial
(very complex apparatus due to "corner" conditions)



3) Some experimental results (Sheet F, Fig. 13)



a) Conflicting data as $b \to 1$, probably due to experimental problems.
Note: MIT-S1 uses $\phi'_{TC} = \phi'_{TE}$

b) All data show increase in ϕ'_p as b increases from zero (TC) to b for $\varepsilon_2 = 0$ (PS = plane strain)

c) Comparison of ϕ'_{PS} vs. ϕ'_{TC} (of greatest practical importance)

- Bolton (1986) recommends $\Delta\phi' = \phi'_p - \phi'_{cs} = 3 \cdot I_R^\circ$ for TC
  $= 5 \cdot I_R^\circ$ for PS
  Hence $\phi'_{PS} - \phi'_{TC}$ increases with increasing D_r & decreasing $\sigma'_{\alpha t}$ (decr. ψ) à la Sheet D, Fig. 7

- CCL recommends Bolton (1986). Therefore
  $\phi'_{PS} - \phi'_{TC} \approx 0$ for low D_r - high $\sigma'_{\alpha t}$ ($\psi \geq 0$)
  $\approx 5^\circ$ for high D_r - low $\sigma'_{\alpha t}$ ($\psi \ll 0$)

4) Comments • Bolton assumes same ϕ'_{cs} for TC & PS, whereas Pestana (1994) Concludes (correctly) that PS → higher ϕ'_{cs}

• Part III-4 will show that $\Delta\phi' = +5°$ → doubling of q_{ult} (bearing capacity)

5.2 Method of Sample Preparation

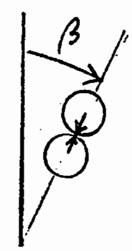
1) Most lab shear tests on sand are run on reconstituted samples. The two most common methods are:

- Pluviation, with or without vibration → more like natural deposits
- Tamping (compaction) of moist soil → non-uniform density

2) Sheet F, Fig. 4 show example of very different stress-strain behavior (even though $\phi'_p \approx$ constant)

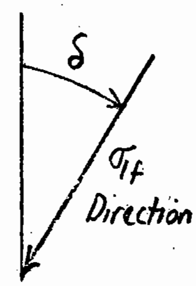
5.3 Anisotropy

1) 1-D deposition leads to a sand structure with



- Preferred orientation of elongated grains ⊥ to σ'_v (fabric)
- " " " particle contacts (even with perfect spheres). See Sheet G, Fig. 8.15

2) Hence natural sand (deposit) has an inherent anisotropy wherein shearing at different δ angles leads to different stress-strain-strength properties



- Shearing at $\delta = 0°$ → highest modulus & ϕ'_p
- Increasing δ → lower modulus & ϕ'_p

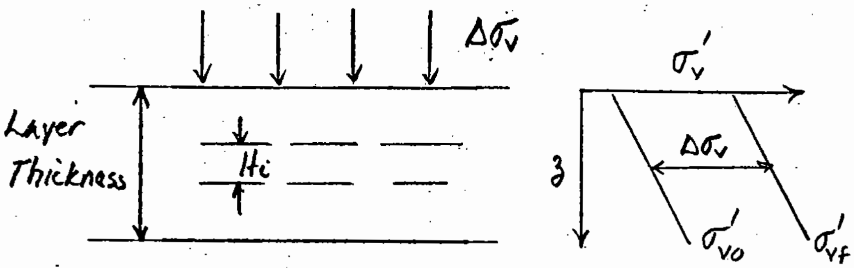
3) Examples of trends (Sheet G)

- Fig. 2 ϕ'_p vs. δ for several sands ($\Delta\phi' \approx 3 \pm 1°$)
- Fig 8 Effect of δ on ε_a vs $(\sigma_1 - \sigma_3)$ & ε_{vol} for dense sand

6. 1-D BEHAVIOR OF GRANULAR SOILS

6.1 Data Presentation and Definition of Parameters

1) Introduction: Estimate settlement for 1-D loading

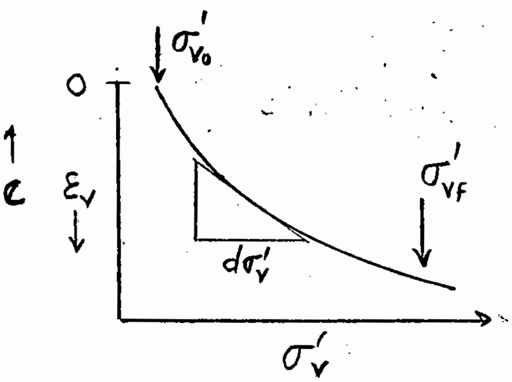


Settlement

$\rho = \Sigma\,(H_i \cdot \varepsilon_{vi})$

2) Conventional Methods of Plotting Oedometer Test Data

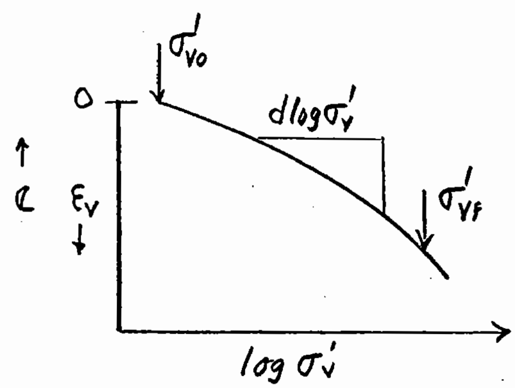
a) Linear plot



Strain hardening → decreasing slope with incr. σ'_v

- Coef. of compressibility, $a_v = -de/d\sigma'_v$
- Coef of volume change, $m_v = d\varepsilon_v/d\sigma'_v$ - (more common)
- $\varepsilon_v = \dfrac{\Delta e}{(1+e_0)} = \dfrac{a_v\,\Delta\sigma'_v}{(1+e_0)} = m_v\,\Delta\sigma'_v$

b) Semi-log plot (Note: $d\log_{10}\sigma = \log e\,\dfrac{d\sigma}{\sigma} = 0.434\,\dfrac{d\sigma}{\sigma}$)



- Virgin compression index, $C_c = -de/d\log\sigma'_v$
- Virgin compression ratio, $CR = \dfrac{C_c}{(1+e_0)}$

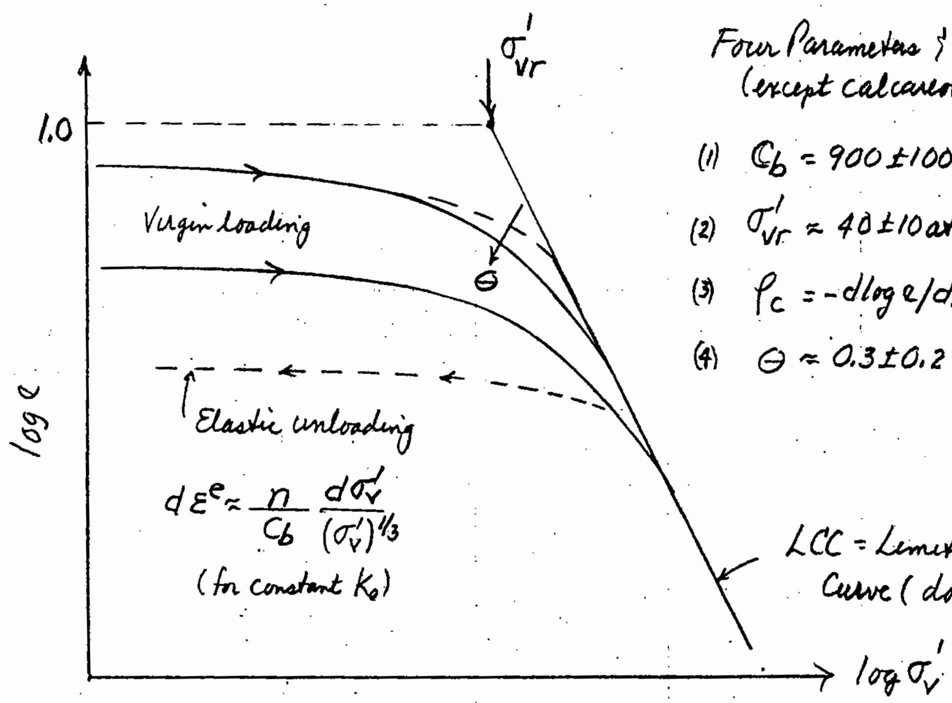
$$\doteq d\varepsilon_v/d\log\sigma'_v$$

- $\varepsilon_v = \dfrac{\Delta e}{(1+e_0)} = \dfrac{C_c}{(1+e_0)}\log\left(\dfrac{\sigma'_{vf}}{\sigma'_{v0}}\right) = CR\,\log\left(\dfrac{\sigma'_{vf}}{\sigma'_{v0}}\right)$

$$\approx 0.434\,CR\,\frac{\Delta\sigma'_v}{\text{Ave.}\,\sigma'_v}$$

NOTE: Because of difficulty of obtaining sand samples for lab testing, usually predict ρ from in situ penetration tests (Part III-4 & 5)

3) Pestana & Whittle [1995, Geot. 35(4)] MIT-S1 Model of Compressibility



Four Parameters & Typical Values (except calcareous sands)

(1) $C_b = 900 \pm 100$ (elastic)

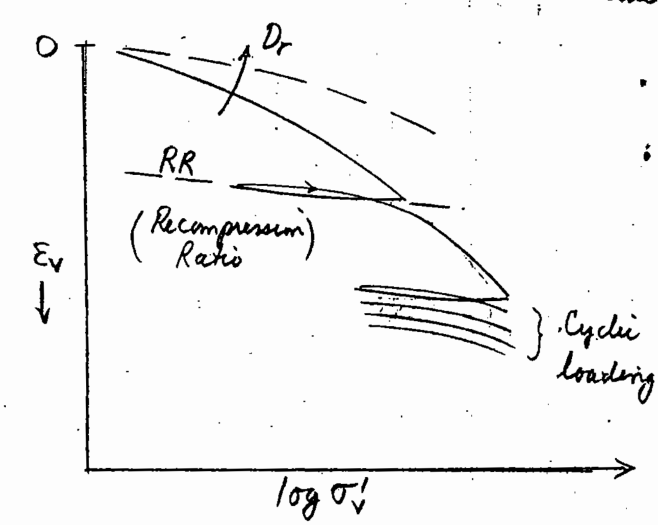
(2) $\sigma'_{vr} \approx 40 \pm 10$ atm (LCC at $e = 1.00$)

(3) $\rho_c = -d\log e / d\log \sigma'_v \approx 0.38 \pm 0.02$

(4) $\Theta \approx 0.3 \pm 0.2$ (curvature as $\sigma'_v \to$ LCC)

6.2 Factors Affecting 1-D Compressibility

1) At stresses significantly $< \sigma'_{vr}$: virgin loading $\to$ mostly plastic strains due to particle sliding & rolling



- Initial density, D_r
- Stress history, OCR
  (RR << CR, especially lower D_r)
- Unload-reload cycles
  (storage tanks, offshore platforms)

Note: Pestana & Whittle (1995) conclude approx. linear log e vs log σ'_{vc} for virgin loading

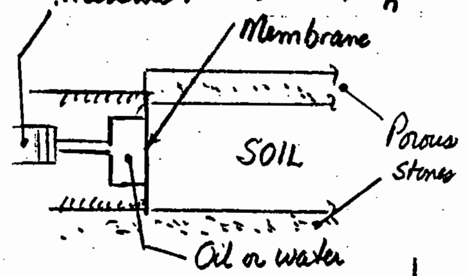
2) At high stresses $\to$ significant grain crushing (Pestana & Whittle 1995)

- Increasing D_{50} $\to$ higher contact forces $\to$ lower σ'_{vr}
- Increasing C_u (better gradation) $\to$ more rounded curve $\to$ higher Θ
- Increasing angularity $\to$ lower σ'_{vr} & higher Θ

6.3 Coef. of Earth Pressure at Rest (K_0) — BOTH Granular & Cohesive Soils

1) Lab Measurement Techniques

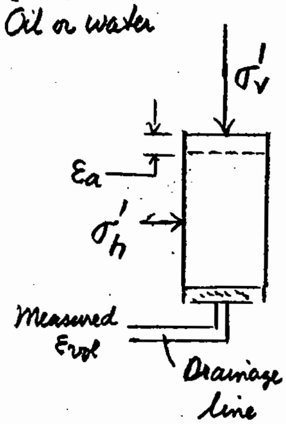
a) Lateral stress oedometer



- Closed system → problems with leakage & need $\Delta T \approx 0°C$
- Results affected by side friction

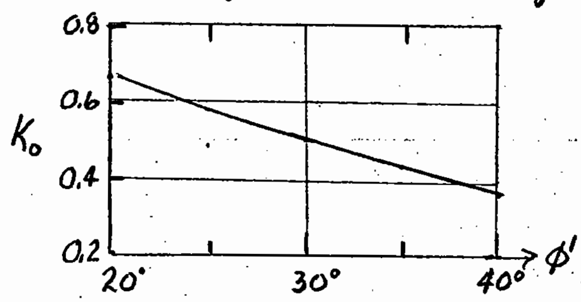
b) Automated stress path triaxial (MIT lab)

- Load at constant $\dot{\varepsilon}_a$ (incr. σ_v') and vary σ_h' to maintain $\varepsilon_{vol} = \varepsilon_a$



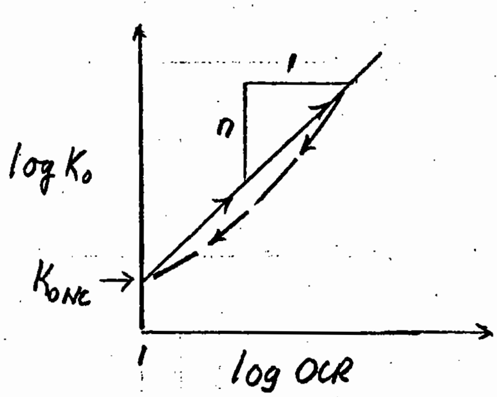
2) K_0 of NC (OCR=1) Soils

Jaky (1944) semi-empr. eqn. $K_0 = 1 - \sin\phi'$



- For clays, $K_0 \approx 0.45 - 0.7$
  Eqn. has SD ≈ ±0.05, ∴ quite good
  (See Sheet H, Fig. 30)
- For sands, $K_0 \approx 0.4 \pm 0.1$
  Eqn. doesn't data as well
  (See Sheet H, Fig. 14)

3) K_0 of OC Soils



a) Unloading → higher K_0 (locked in σ_h')

- $K_0 = K_{0NC}(OCR)^n$, where $n \approx 1 - K_{0NC} \approx \sin\phi'$
- For clays, works quite well; for sands, less well
  (Sheet H, Figs. 15, 31, 32)
- Max K_0 = failure in triaxial extension (stiff fissured clays)

b) Reloading → significant hysterisis
  (Sheet H, Fig 15 & 31)

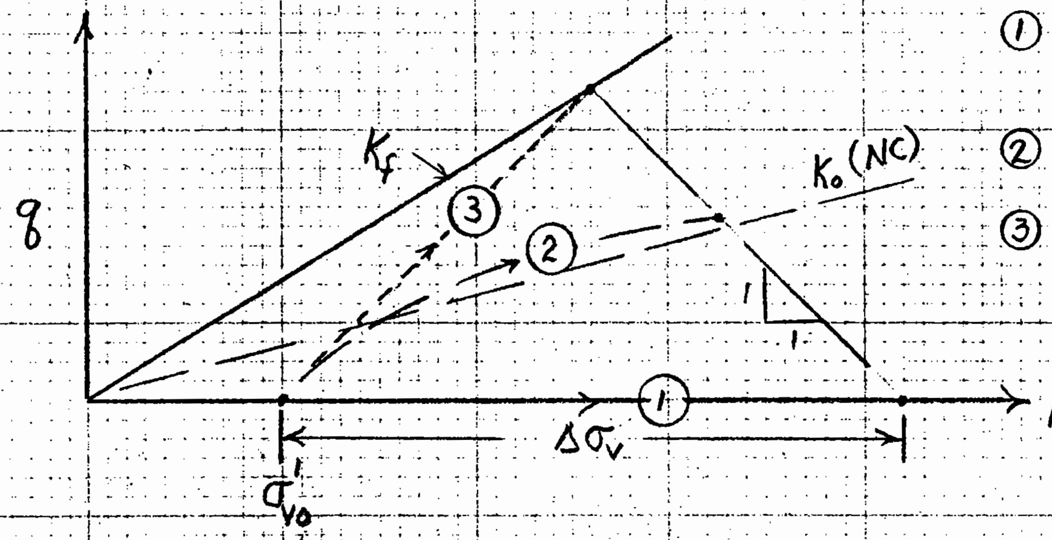
4) References

- Mayne & Kulhawy (1982), JGED, 108(6): summary of "all" data in literature
- Mesri & Hayat (1993) CGJ, 30 (4): new data on clays → effects of aging, recompression, preshearing, etc.

PART III-2 STRESS-STRAIN-STR. PROP. (p.20)

7. INFLUENCE OF STRESS PATH ON STRESS vs. STRAIN

(1) Stress paths for O.C. dense sand (initial $K_o = 1$)

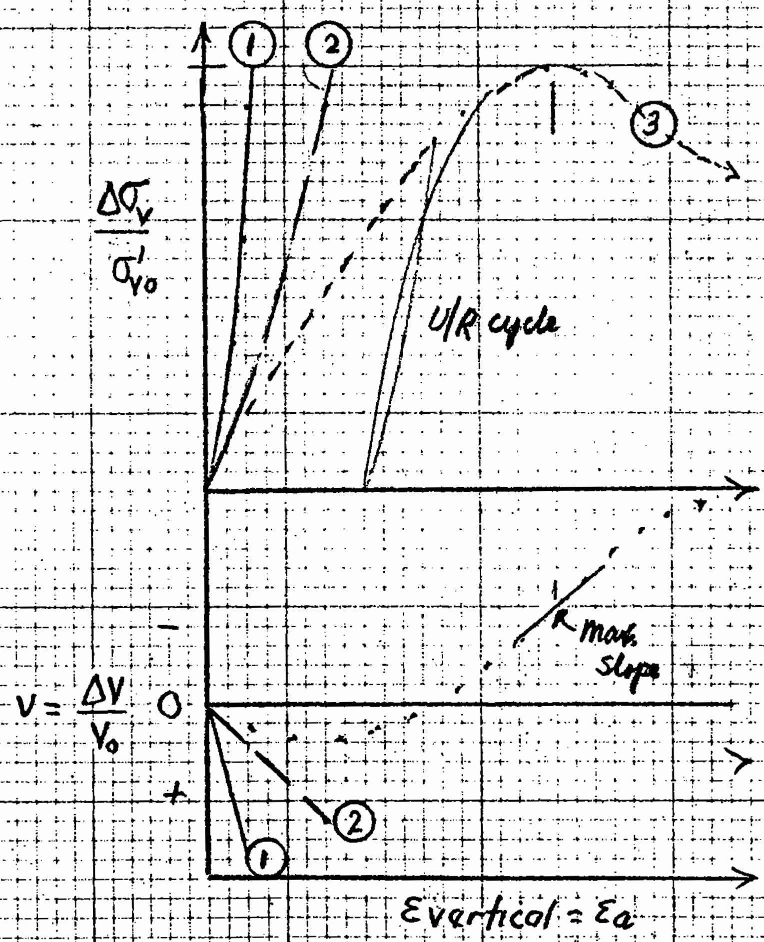


① Isotropic compression $\Delta K = 1.0$

② 1-D compression

③ Std. triaxial compr. $\Delta K = 0$

(2) Resulting stress-strain curves



Comments

① & ②

- Concave upward
- $\varepsilon_{a②} > \varepsilon_{a①}$ since higher q
- $V_① > V_②$ due to anisotropy & higher p'

③

- approx hyperbolic up to peak
$$\frac{\Delta\sigma_v}{\sigma'_{vo}} = \frac{\varepsilon_a}{a + b\cdot\varepsilon_a}$$
- Hence E decreases with incr. $\Delta\sigma_v$
- Max. $(-dV/d\varepsilon_a)$ at peak
- U/R → much stiffer response (Called evolving anisotropy or kinematic hardening)