5/7/99 ≡ 4/99 ≈ 5/01

BEHAVIOR OF COMPACTED CLAYS

Page No

1. Introduction

2. Compaction Curves

3. Structure of Compacted clays

Sheet A1, 2 : Compaction : Harvard Miniature ; Sheepsfoot & rubber-tired rollers

B1, 2 : Compaction curves & $u_c = 5m$ (Olson & Langfelder 1965)

C : kaii, line of optimums & effect soil type.

D1, 2 Examples of effects of structure on drying/soaking & UUC

(Plus 2 pages of mini-problems)

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Page No

4. Hydraulic Conductivity

5. Compressibility Characteristics

Sheets E1-E5 : Data on Coef. of permeability
" F1,2 " " Compressibility

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6. Shear Characteristics

Sheets G1-G3 UUC & CBR data
" H1,2 CD DS data

CCL 5/7/98 5/3/01 1.322 | Compacted Clays - Mini Problems/Questions

1) What experimental techniques are used to measure $u_c = s_m$?

a) For high s_m, say > 20 bar

b) For low s_m, say < 1 bar

c) For moderate s_m $\begin{matrix} >1 \\ <20 \end{matrix}$

d) Which of these measure $s = s_m + s_s$ rather than s_m?

2) What occurs at $w_m = w_{opt}$ to explain why further increase in $w_m \to$ lower γ_d? (at constant compactive effort)

3) The simplest method to prepare a lab specimen with the same w_m & γ_d as in the field is via static compaction. For w_m < & > w_{opt} in field, what is the effect on behavior of static vs field compaction for:

Property	$w_m < w_{opt}$	$w_m > w_{opt}$
a) Shrinkage during drying		
b) Swelling after soaking		
c) q_f from UUC tests		
d) q at low strain from UUC tests		
e) As-molded $u_c = s_m$		

4) As a contractor that is required to obtain a certain % compaction = Relative Compaction, is the field w_m important with respect to the no. of passes that your equipment will have to make in order to meet the specification?

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5) As a smart contractor that is required to construct a clay liner with $k \leq 10^{-7}$ cm/sec, which of the following variables are most important?

a) Liner thickness (say 30cm vs 60cm)

b) Borrow-material + what method of lab compaction would you use to run lab tests; evaluate Δ borrow sites

c) w_m relative to w_{opt} for your equipment

d) γ_d per se

e) Uniformity of compacted soil wrt w_m & γ_d

f) What technique(s) would you use to prove that your liner meets $k \leq 10^{-7}$ cm/sec

6) The specs for a very thick fill in S. California states that you must obtain a RC ≥ 90% (Modified), but also states that you will be liable for "excessive" movements of homes & office bldg. that will be constructed at the site. Note: borrow = SC to CL soils.

a) Would you be willing to simply follow the specs → Answer = No, but why?

b) What lab testing program would you want to run to decide on what w_m-γ_d criteria you would use in the field?

c) In submitting your bid, what "qualifications" would you add to protect you against ongoing movements many years after construction

7) Does "more is best" apply to the short term performance of compacted cohesive fills? (refers to amt. of compactive effort)

8) Is w_m (relative to w_{opt}) important regarding changes in slope stability of compacted fills as go from UU Case to CU or CD Case?

9) For evaluating the short & long-term stability of a slope ($S \leq 70\%$), are the relative magnitudes of σ & u_w important? If yes, cite a specific example.

BEHAVIOR OF COMPACTED CLAYS

1. INTRODUCTION

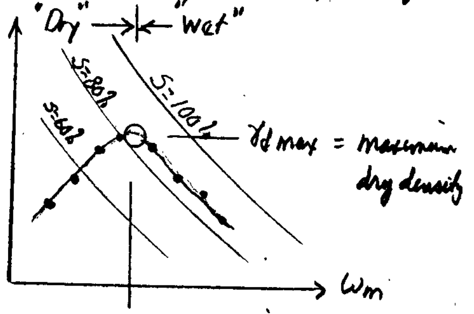
1.1 Definitions

1) Compaction = densification via mechanical energy at Constant water content (ie, decrease air voids).

2) Compaction test: mix soil at varying molded water content (w_m), apply constant compactive effort, & measure resultant dry density $= \gamma_t/(1+w_m)$

3) Compaction curve: plot of w_m vs γ_d

$$\frac{\gamma_t}{1+w_m} = \gamma_d$$



γ_d max = maximum dry density

Wopt = optimum water content

$w_m < w_{opt}$ = dry of optimum

$w_m > w_{opt}$ = wet of optimum

Always draw S=100% line

1.2 Types of Compaction

1) Laboratory (Most common for cohesive soils)

a) Impact = Dynamic = Proctor = AASHO (Most common)

	Name	Hammer	Fall	Mold Size	No. Layers	No Blows	Compactive Effort
ASTM D1698	Standard	5.5 lb ↑	12 in.	4"φ × 4.6"	3	25	12,400 lb-ft/CF
		φ = 2" ↓		6"φ × 5"	3	56	600 kN-m/m3
ASTM D1557	Modified	10.0 lb	18"	4"φ	5	25	56,000 lb-ft/CF
				6"φ	5	56	2,700 kN-m/m³

4" mold < 20% + No. 4 sieve

6" mold > 20% + 3/8"

Causes alot of shearing of soil during compaction

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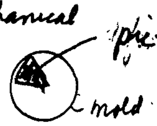
1.2 Cont.

b) Kneading

(1) Harvard miniature ; "Std" 40lb spring - 5 layers - 25 tamps/layer (Sheet A1)
$\phi = 1.3''$, $H = 2.8''$ "mod." 80lb " - 10 " - " " "

(2) Mechanical



} vary applied pressure, # applications/layer & # of layers

Also causes alot shearing

c) Static = 1-D compression with hydraulic ram; vary pressure; from one or both ends

Easier to do, but causes minimal shearing of soil $p \approx 50-2000$ psi?

2) Field Compaction (Sherard et al. 1963 book on dams, Wiley; CCL 1959, 1.39)

a) Sheepsfoot rollers : Min. $\phi = 5'$ with tamping feet { $l \geq 9''$, area ≈ 7±3 in^2 Typical $p = 200$ psi

- Min. wgt 4000 lb/LF

Causes alot of shearing

See Sheet A2 for photos

b) Rubber-tired rollers: Std → 50 Ton/wheel with tire pressure 90±10 psi (very important)

- Can include vibratory attachment

Less shearing during compaction

c) Steel drum rollers, usually with vibratory attachment

Note: "Weaving" during field compaction indicates that compaction effort is too large for field w_m, i.e., very wet of optimum

Undrained shear deformation

1.3 Soil Suction and Effective Stresses ($S \leq 100\%$)

1) Soil Suction (Summary from 1.322 Part A II-2); also 1.361 Part IV-1, Sect. 2.6)

a) Used as measure of "free energy" of soil water relative to pure H_2O at atm. pressure (at same T & elev.). → how much this free energy is less than zero. Can be expressed in terms of: • potential (J/kg)
• head (ℓ)
• pressure (kPa)

Often use $pF = \log_{10}$ (cm of H_2O)

b) Components: $\boxed{\text{Total suction}(s) = \text{matric}(s_m) + \text{solute}(s_s)\text{ suction}}$

- $s_m = u_c = (u_a - u_w)$, where water in measurement system has same salt conc. as bulk water in soil
- s_s = osmotic pressure due to salt conc. in bulk water in soil

[s_s (atm) ≈ 24 (molar conc. of cations + anions) at 20°C]

. Molar = No. moles of solute per liter of solution

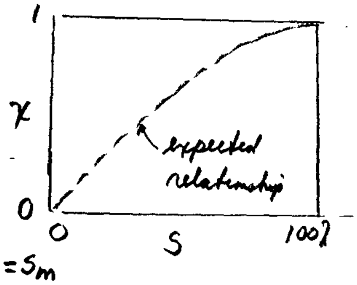
" 2.6 of 1.361 IV-1,

c) See 2.4 of A II for measurement techniques:

(1) Fine porous stone used for low-moderate suctions = direct method
- Pressure plate: $u_c = u_a - u_w$ (apply $u_a \rightarrow u_w \approx 0$) ⟶
- Pressure probes now can directly measure $u_c = s_m$ to 10-15 bar (e.s. K. Sjoblom)

(2) Filter paper & relative humidity used for high suctions = indirect method with lots of problems (CCL opinion)

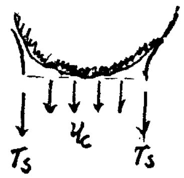
2) Effective Stress Equations for $S < 100\%$



a) Bishop (1960) Boulder Conf.

- $$\sigma' = \underbrace{(\sigma - u_a)}_{\text{Externally applied stress}} + \chi \underbrace{(u_a - u_w)}_{u_c = \text{matric suction} = s_m}$$

- Although χ was expected to vary between 0 and 1, will see that can get χ < 0
χ > 1



RSJ
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b) Preliminary discussion

- the 2 components of σ' will often affect compressibility and strength in different ways, e.g.

$$\tau_{ff} = c' + (\sigma - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b$$

$\uparrow$ ϕ' for S=100% $\uparrow$ Different ϕ'

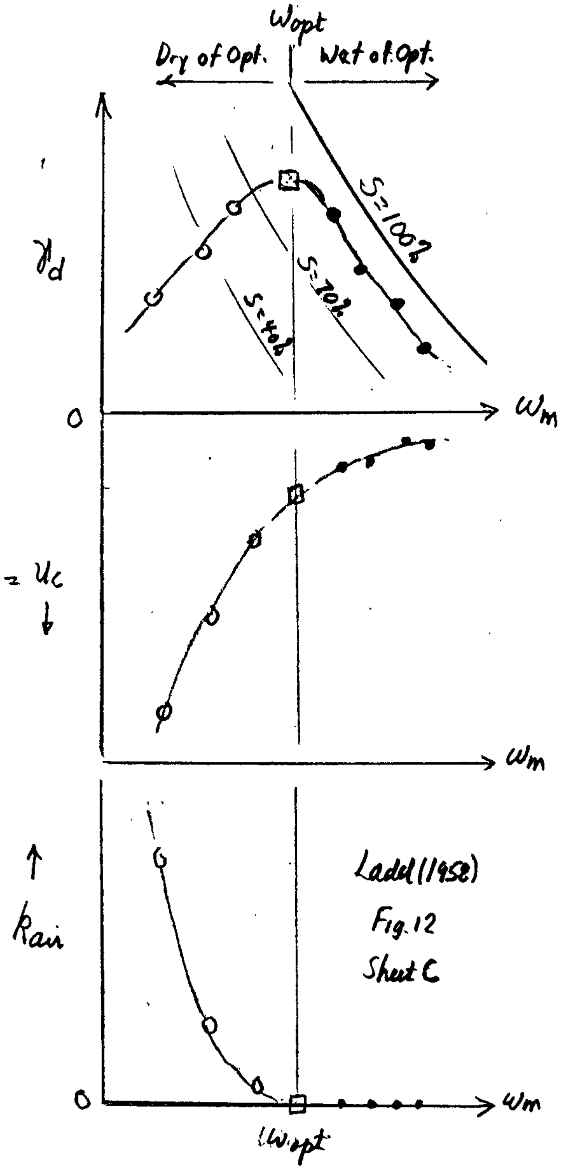
Fredlund & Rahardjo (1993) Soil Mechanics for Unsaturated Soils, Wiley
" "R&J (93)"

$$\tau_{ff} = c' + (\sigma - u_a)\tan\phi' + b(u_c)^a \quad , \; a \approx 0.5, \; b = 2\text{-}10$$

Mesri & Shahien (1994) 2nd Geot. Eng. Conf., Cairo, Egypt

2. COMPACTION CURVES

2.1 Explanation of Compaction Curve (Given clay, type & amt. of compaction)



Ladd (1958)
Fig. 12
Sheet C

1) Start with moist clay at lowest w_m
- Clay aggregates with large u_c → high strength of aggregates that resistance deformation during compaction

2) Increasing w_m dry of optimum
- Incr. w_m → lower u_c → less strength of aggregates

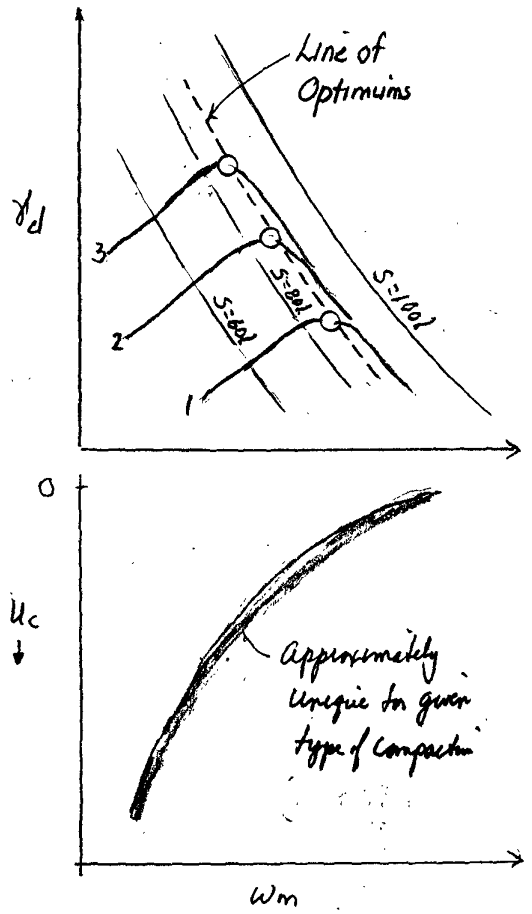
∴ Incr. γ_d under same compaction effort

3) at optimum water content
- Air voids become discontinuous

∴ Air cannot flow out to decrease air voids & hence increase γ_d

4) Increasing w_m wet of optimum
- Simply replacing solids with incr. amt. water → decr. γ_d.

2.2 Influence of Increasing Compaction Effort



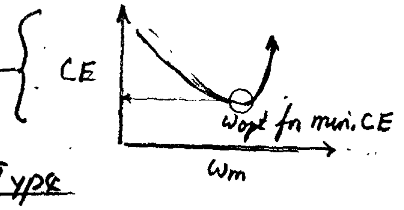
1) Compaction curves for 3 compaction efforts (CE1 < CE2 < CE3) of same type

2) Line of optimums = line connecting w_{opt} - $\gamma_{d\,max}$ for each compaction effort
   - Most typically occurs along line of approx. constant $S \approx 80$-90%

3) All compaction curves merge wet of optimum at S = constant

4) Olson & Langfelder (1965) show approx. unique $u_c = S_m$ vs w_m independent of compaction effort. See Sheets B1 & 2 for data on 5 soils (Unfortunately, mostly using static compaction)

5) Compaction effort to get a specified γ_d varies significantly with w_m



2.3 Influence of Type of Compaction and Soil Type

1) Difference types of compaction, both in lab & in field, may → different shape of compaction curves and will → different line of optimums. See Sheet C, Fig 11.3.12 for data on ML-CL soil (WES Compaction research in 1950s)

2) See Sheet C, Fig. 7.2, for idea of influence of different soil types (I need get better data!)

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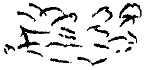
3. STRUCTURE OF COMPACTED CLAY

3.1 Conceptual Structures

1) Effect of w_m (kneading compaction)

- Dry of optimum "flocculated structure"
  - Aggregates with flocculated fabric
  - Large voids between aggregates
  - High u_c → high strength of aggregates
- Wet of optimum "dispersed structure"
  - Aggregates broken down → more uniform distribution of particles; also more parallel orientation
  - Low u_c → low strength.



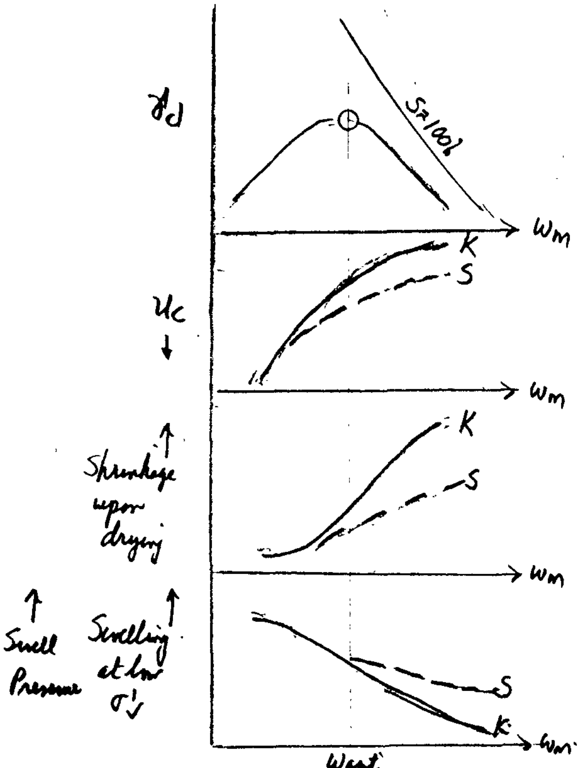
2) Effect of Static Compaction (vs kneading)

- Dry of optimum → little difference
- Wet of optimum → large difference, because low degree of shearing → more 'flocculated' fabric and higher u_c

3.2. General Trends: Drying & Soaking

K —— kneading compaction (large shearing)

S – – – static compaction (small shearing) to same γ_d

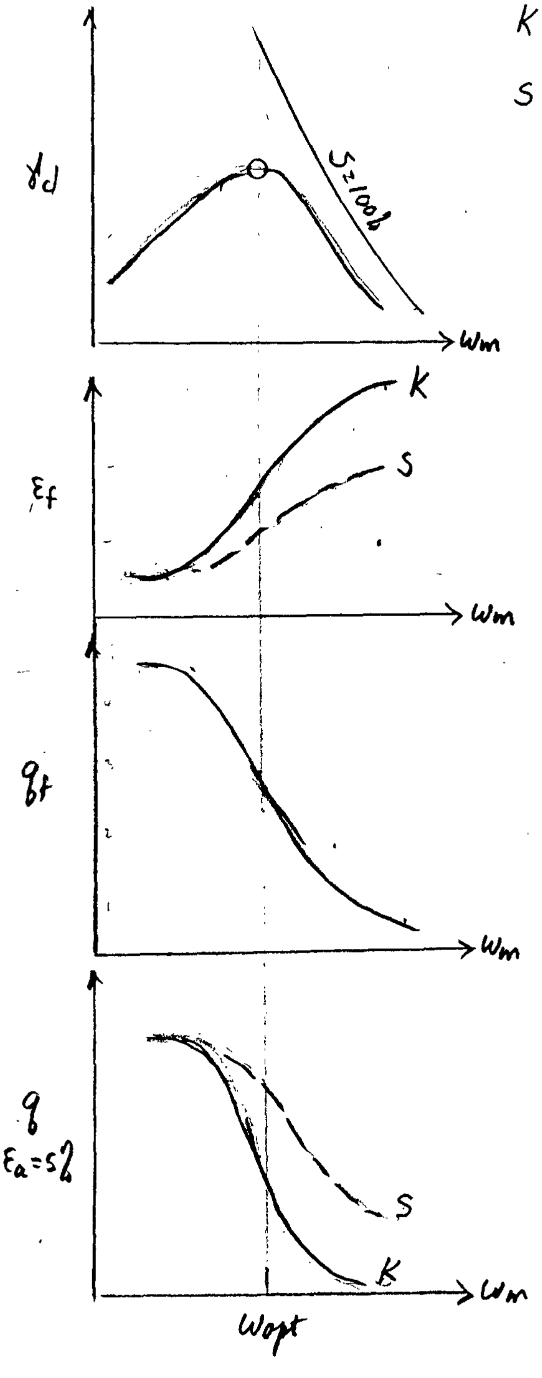


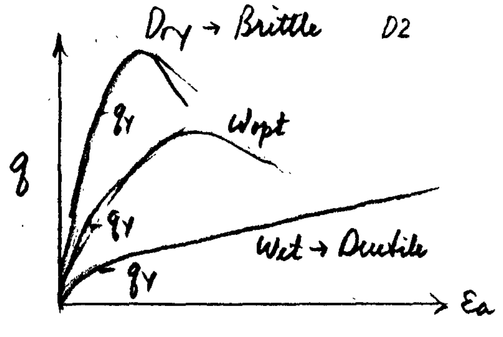
- S → higher u_c due to less shearing during compaction B2
- Incr. w_m → more shrinkage < Incr. $\Delta\sigma'$ during drying; more dispersed fabric
- S → less shrinkage < smaller $\Delta\sigma'$; less dispersed D1
- Incr. w_m → less swelling, lower swell pressure } lower $\Delta\sigma'$ when soaked
- S → more swelling, lower swell pres. } larger $\Delta\sigma'$ when soaked D1

3.3 General Trends : UUC Behavior (Relatively low σ_c)

K —— Kneading

S - - - Static to same γ_d





1) Dry of Optimum (K)

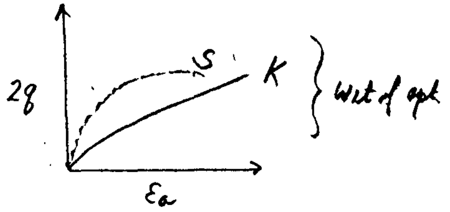
- High u_c → high σ' at aggregate contacts → high q_f
- Brittle behavior – low ε_f
  – also strain softening

2) Wet of Optimum (K)

- Low u_c → low σ' within matrix → low q_f
- Ductile behavior – high ε_f
  – also strain hardening after initial yield

3) Static vs Kneading (Wet of opt)

- Higher u_c → higher σ' → stiffer response D1,2



3.4 Examples of Effects of Structure (Wet vs Dry, Kneading vs Static)

1) Soil Suction and Drying / Wetting Sheet D1

- Fig 34.7 : K → higher u_c & less shrinkage (Kaolin)
- Fig. 2 Incr w_m → more parallel particle orientation & more shrinkage (Kaolin)
- Fig 14 S → higher swell pressure at same γ_d (high compaction % Sat.) (PSC & VSC)
- Fig 26 S → less shrinkage. (VCS)

2) UUC Shearing Sheet D2

- Fig 34.10 : Incr. w_m → lower q_f & incr. ε_f (Kaolin)
- Fig. 7 : Dry of opt soaked to wet of opt w/ γ_d → much stiffer initial response (VSC)
- Fig 15 : S & K → same behavior dry of opt (VSC)
  S → much stiffer initial response wet of opt

		w_L (%)	I_p (%)	<2μm (%)	Activity
VSC	Vicksburg Silty Clay :	37	14	24	0.58
VBC	Vicksburg Buckshot Clay	63	38	36	1.06
PSC	Pittsburg Sandy Clay (CA)	35	16	24	0.67

Compacted Clays

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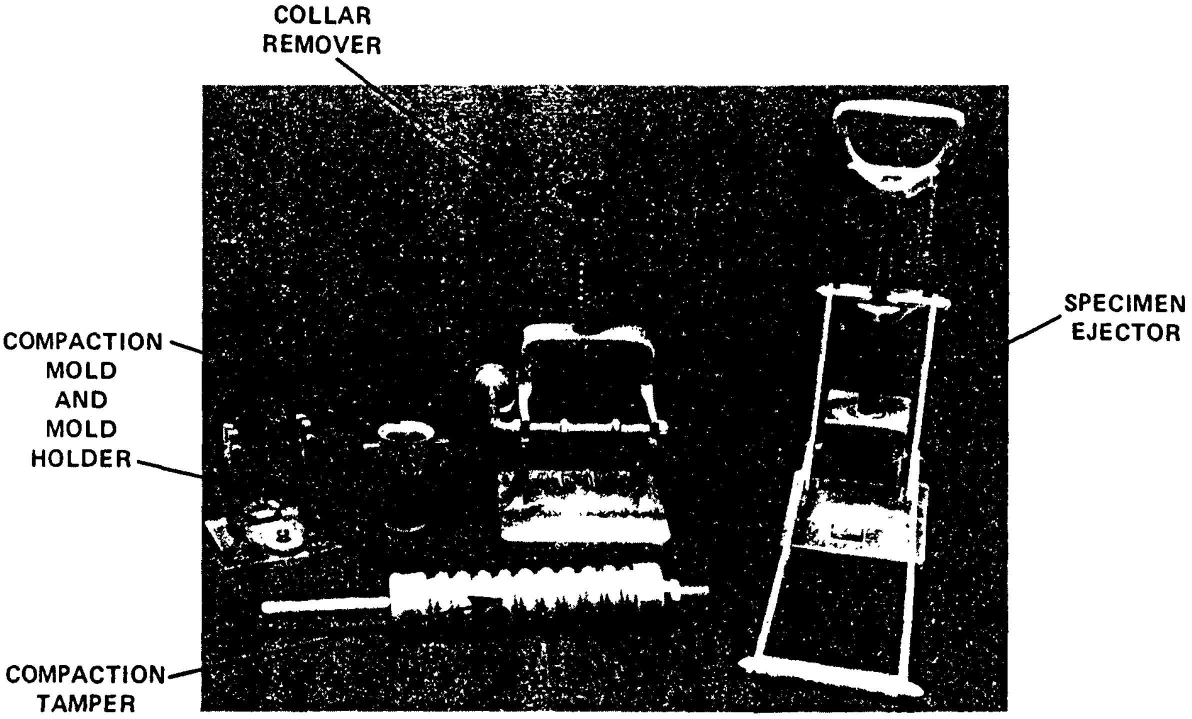

Figure 1. – Harvard miniature compaction test equipment.

A1

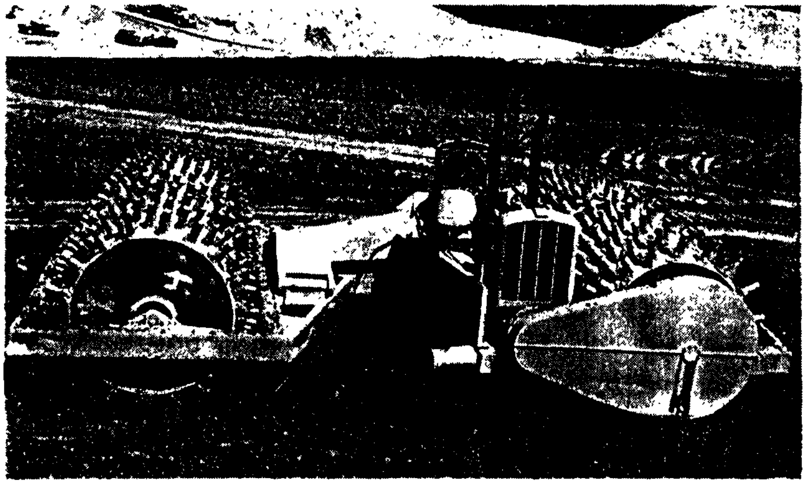

Fig. 11.3:6 Recently developed self-propelled sheepsfoot roller meeting USBR specifications. (Courtesy USBR)

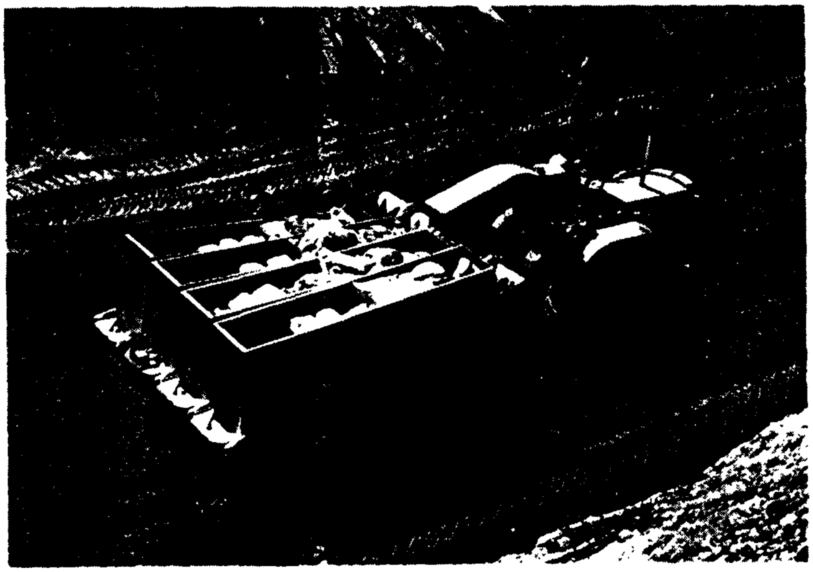

Fig. 11.2:8 Typical "standard" 50-ton rubber-tired roller.

From Sherard et al. (1963)

Sheepsfoot & Rubber-Tired Rollers

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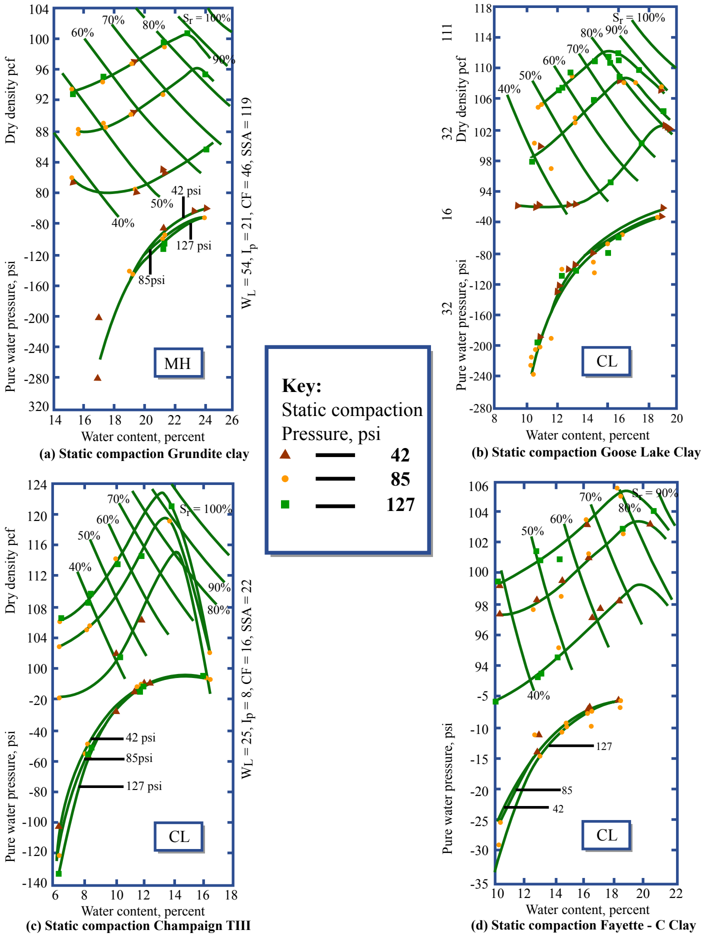
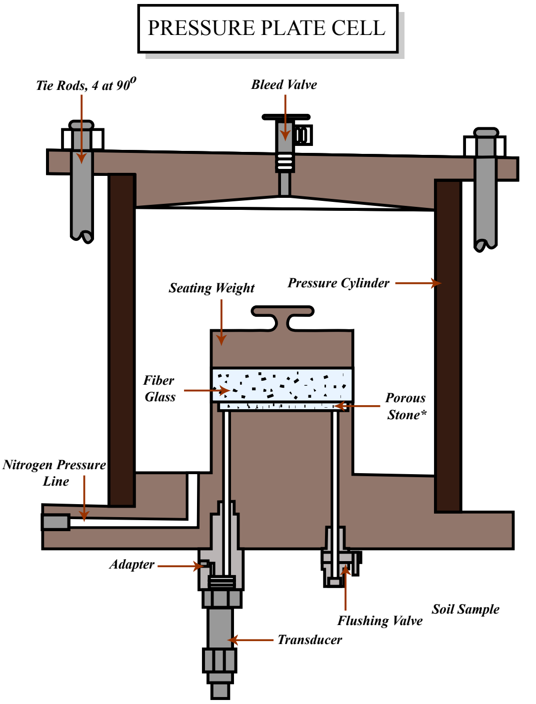
Adapted from: Olson, R.E. & Langfelder, L.J. (1965), "Pore Water Pressures in Unsaturated Soils", JSMFD, ASCE, 91(4), pp 127-150

July, 1965 SM



Experimental Data

Figure by MIT OCW.



*Fine stone with air entry pressure = 250 psi

Figure by MIT OCW.

B1

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5/89 5/96

Experimental Data

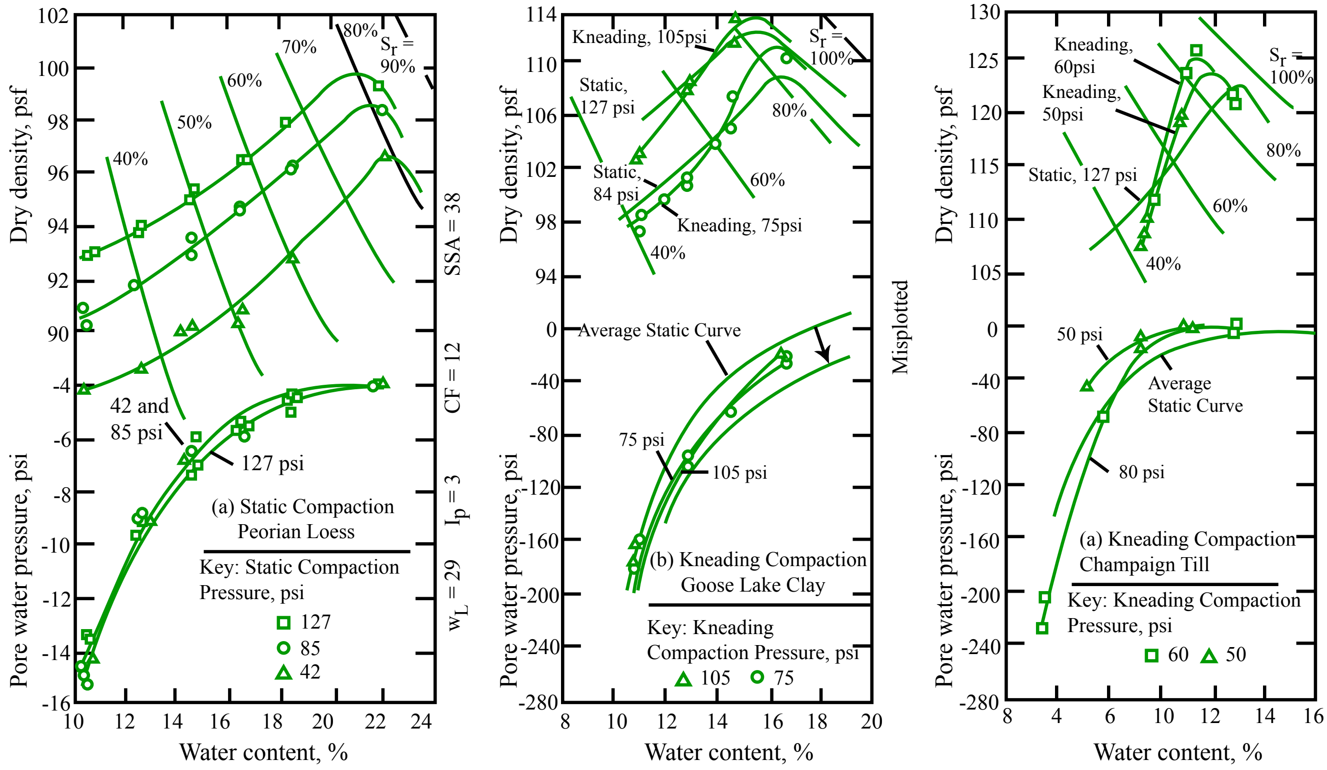
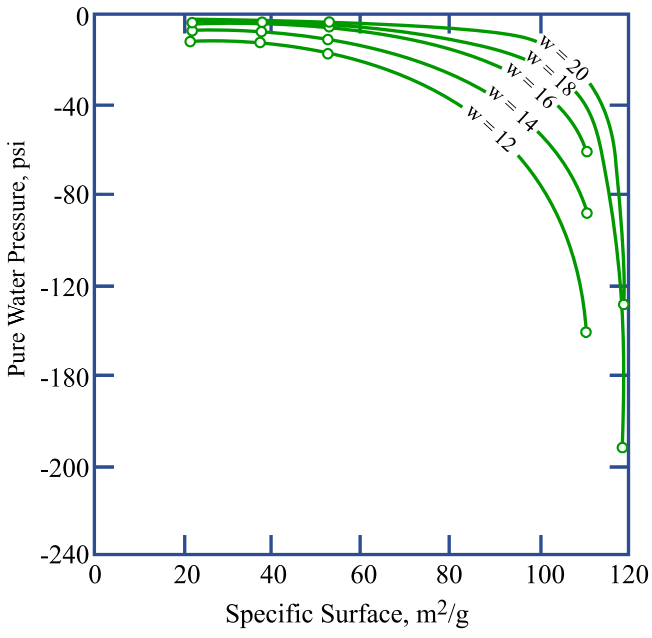


Figure by MIT OCW.



Influence of Specific Surface on the Negative Pore Water Pressure in Samples Subjected to Static Compaction at 127 psi

Figure by MIT OCW.

Adapted from: Olson & Langfelder (1965)

Compacted Clay

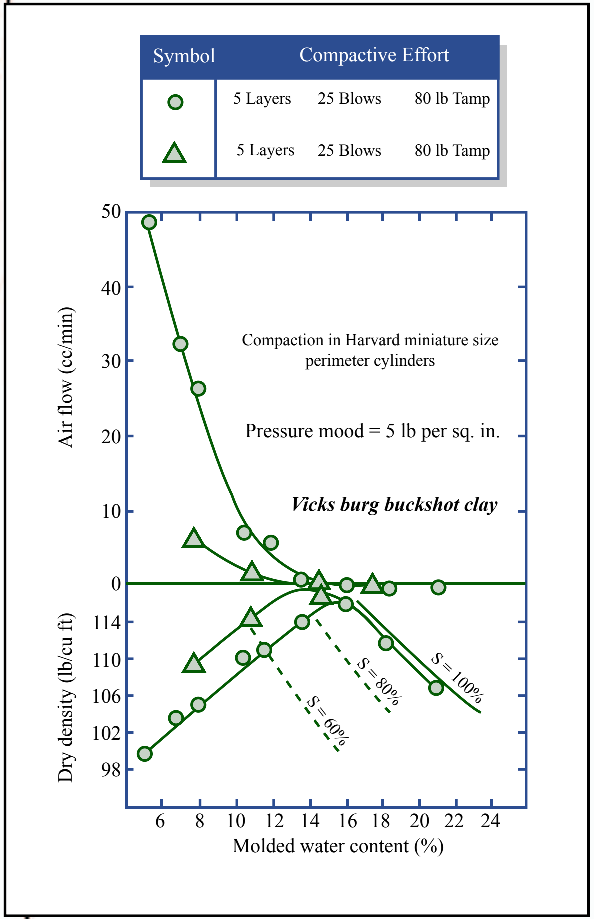
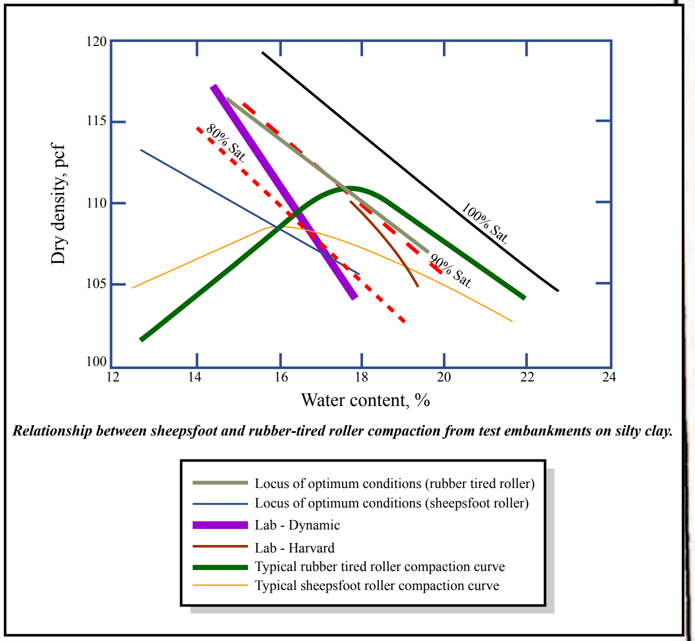


Figure by MIT OCW.



Relationship between sheepsfoot and rubber-tired roller compaction from test embankments on silty clay.

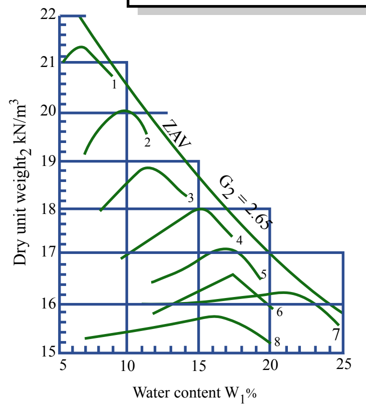
Figure by MIT OCW.

Adapted from:

Sherard et al. (1963)

Water content, % Vicksburg Silty Clay

PHYSICAL AND GEOTECHNICAL PROPERTIES OF SOILS



Standard compaction curves for several soils.

Soil No.	Description	w_L	I_P
1	Well-graded loamy sand	16	NP
2	Well-graded loamy loam	16	NP
3	Medium-graded sandy loam	22	**4**
4	Lean sandy silty clay	28	**9**
5	Lean silty clay	36	**15**
6	Loessial silt	26	**2**
7	Heavy clay	67	**40**
8	Poorly graded sand	~	NP

ote: That as the percent passing the No. 200 sieve (soils 4 through 7) increases the dry unit weight drops rapidly. The low nit weight for soil 8 indicates the difficulty in compacting sand by impact.

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Figure by MIT OCW.

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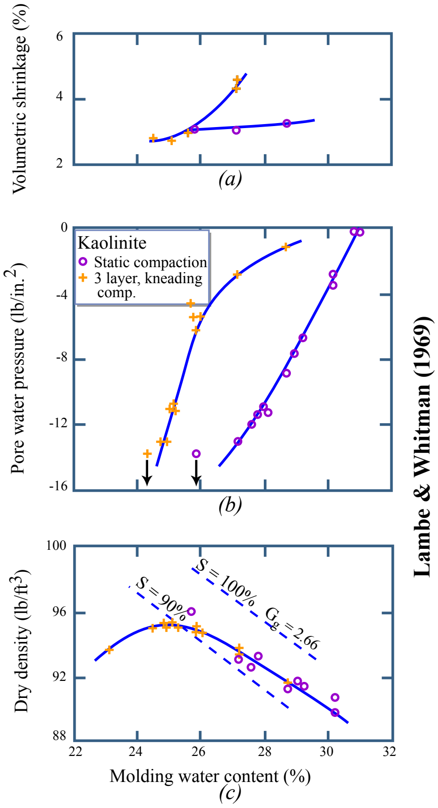
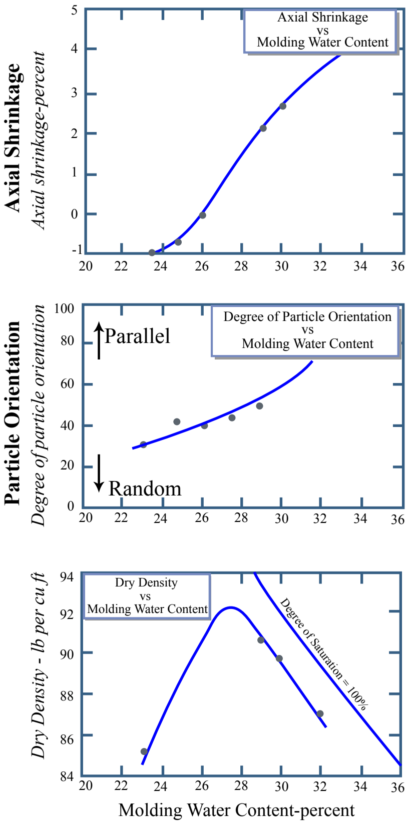
Adapted from: Bowles (1984) McGraw-Hill Book

Std. ASTM Compaction



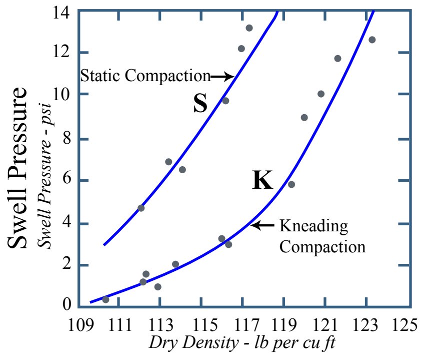
Pore pressure in compacted kaolinite. . .
(From Lambe, 1961).

Lambe & Whitman (1969)

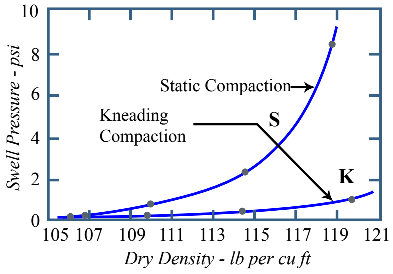


Influence of molding water content on particle orientation and axial shrinkage for compacted samples of Kaolinite.

Seed & Chan (1959)



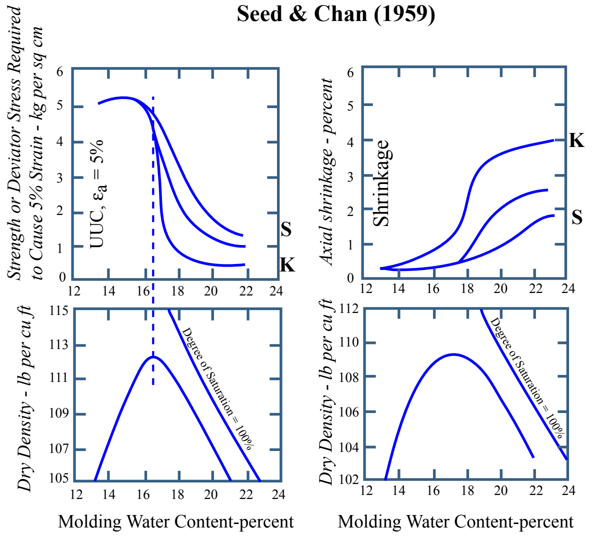
(a) Pittsburg Sandy Clay



(b) Vicksburg Silty Clay

Effect of method of compaction on swell pressure for samples compacted to high degree of saturation.

Seed & Chan (1959)



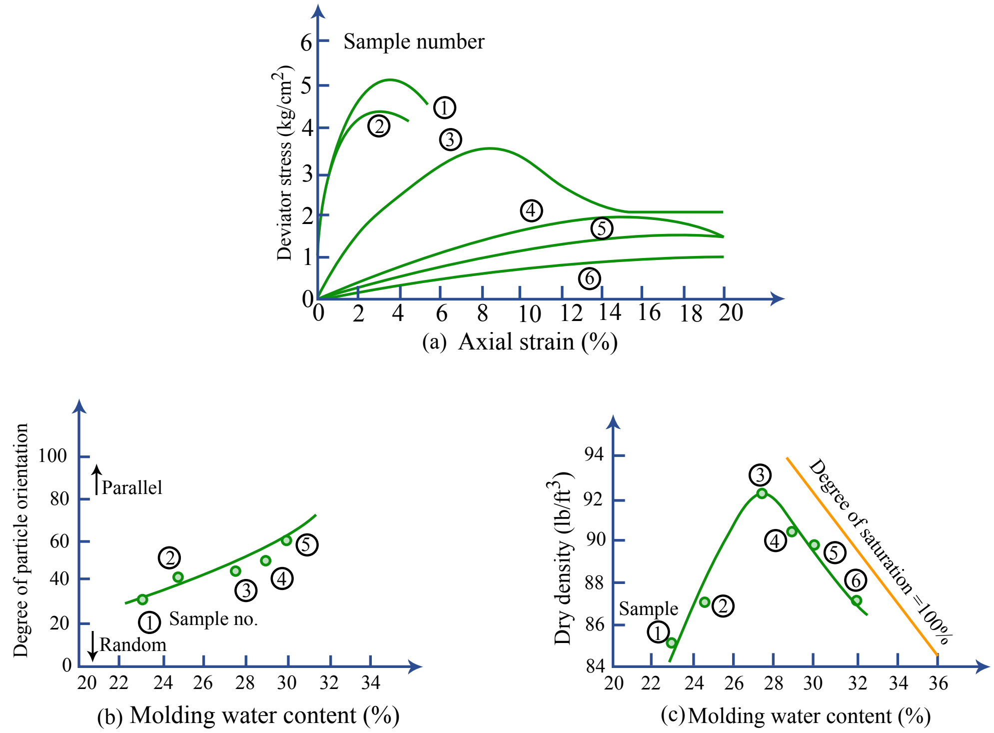
Infulence of method of compaction on strength and shrinkage of silty clay

Seed & Chan (1959)

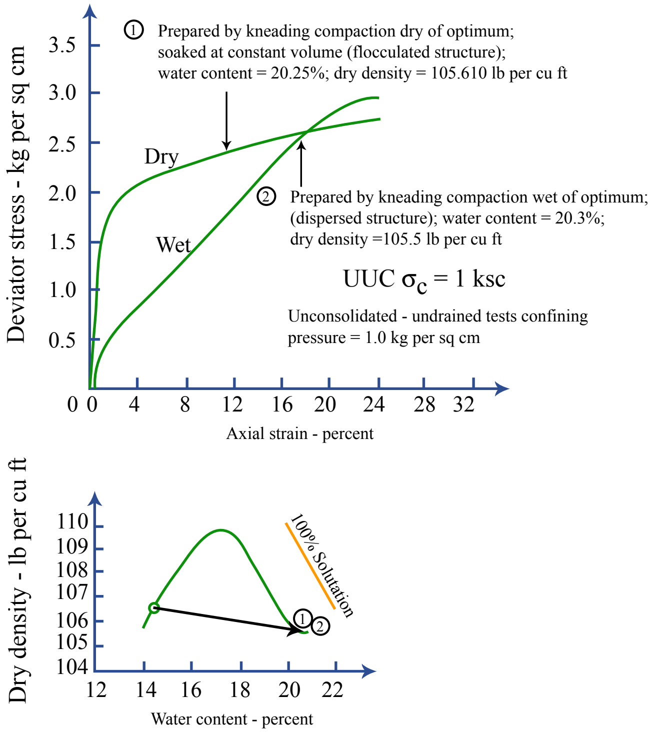
ASCE JSMFD : 5(SMS), 87-128

D1

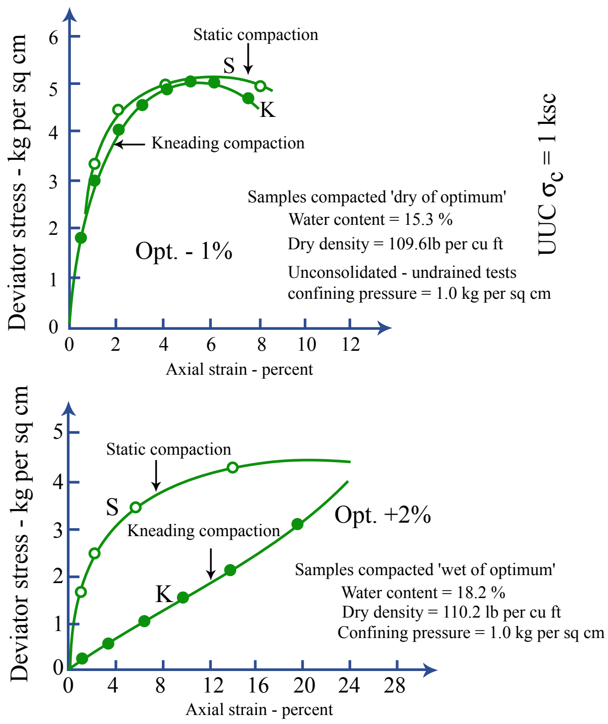
Figure by MIT OCW.



Influence of molding water content on structure and stress-strain relationship for compacted samples of kaolinite. (a) Stress versus strain relationship for compacted samples. (b) Degreee of particle orientation versus water content. (c) Dry density versus water content. (From Seed and Chan, 1959)



Influence of soil structure on stress vs deformation relationships for silty clay Seed & Chan (1959)



Stress vs deformation relationships for samples of silty clay prepared dry and wet of optimum by kneading and static compaction

Seed & Chan (1959)

Figure by MIT OCW.

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4. HYDRAULIC CONDUCTIVITY (Coef. of Permeability)

(Note: k to water at S ≈ 90-100% except as noted)

4.1 Measurement Techniques

1) Laboratory (Sheet E1)

a) Rigid wall = compaction mold (with or w/o u_b = backpressure → higher S)

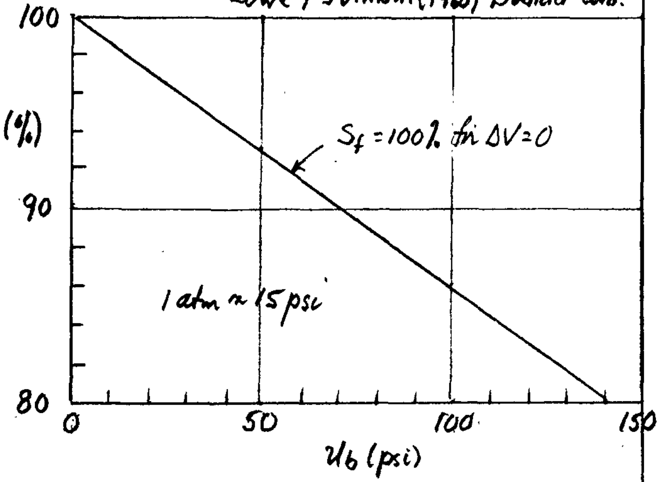
b) Consolidation permeameter: easy to study effect of $\Delta\sigma'_v$

c) Triaxial cell: best for field samples to minimize leakage along sample perimeter

Note on Effect of S

- See E2, Fig. 2 for k data on compacted sandy clay with u_b = 0 & 5 atm
- Also E2, Fig. 6 for data on compacted silty clay at S = 90 & 95%
- For high S, $k \propto (S_f)^3$

Lowe & Johnson (1960) Boulder Conf.



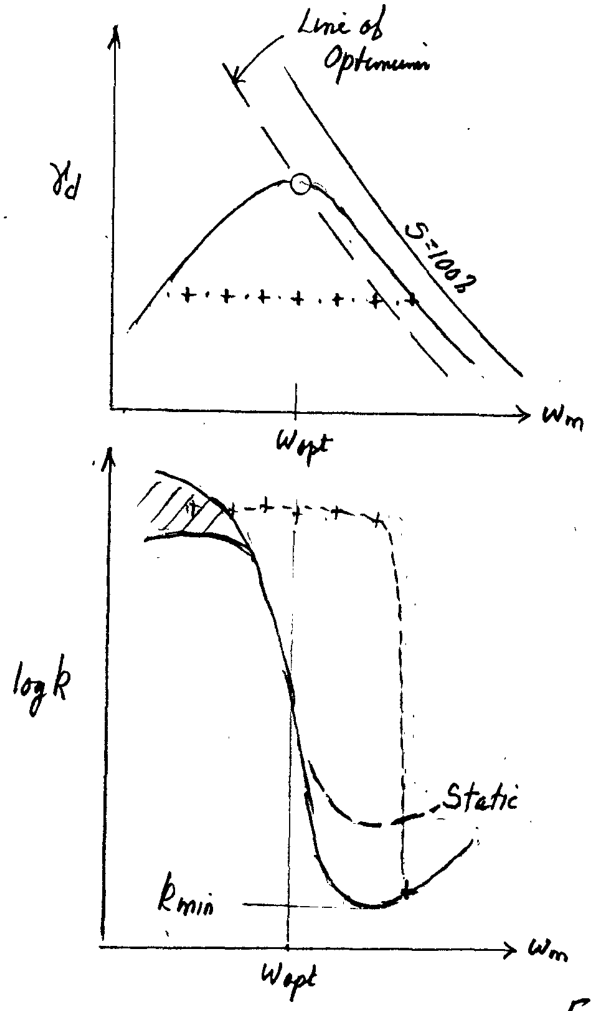
2) Field

a) Most common method is to take Shelby tube samples for lab testing with TX cell

b) ASTM D 3385 describes use of Double-Ring Infiltrometer for $k \approx 10^{-2}$ to 10^{-6} cm/s.

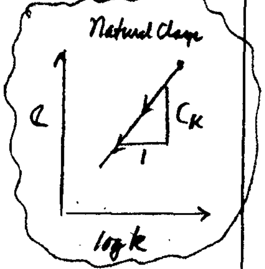
- CCL doesn't like for clays since does not account for soil suction.
- Plus well compacted clay should have $k < 10^{-7}$ cm/sec

4.2 Basic Trends [Kneading Compaction (Except as noted); $\Delta V \approx 0$ & $S \approx 100\%$]



1) Effect of Increasing w_m at Constant Compaction Effort

- Dry of Opt → very high k due to 'low γ_d and large voids between aggregates' (flocculated fabric)
- Incr. w_m → higher γ_d & less floc. fabric → lower k
- Minimum k occurs (usually) at w_m = wet of opt. Slightly lower γ_d more than offset by more dispersed fabric (uniform small voids)
- Then incr. k due to lower γ_d



Examples: E2, Figs. 2, 10 & 15
E3, Fig. 10 & 13

2) Effect of Increasing w_m at Constant γ_d

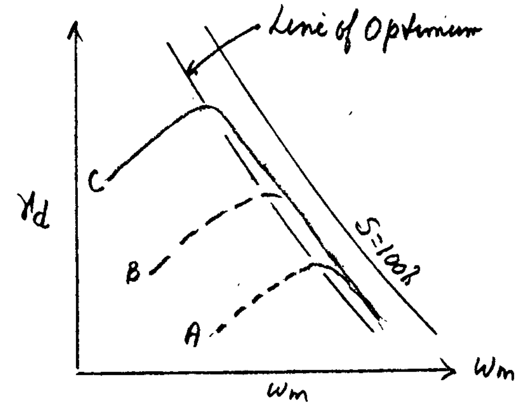
- Reported k essentially constant (E2, Fig. 6) until get near line of optimums; then dramatic decrease in k due to rapid change to dispersed fabric

3) Effect of Static Compaction

- Causes increased k wet of optimum since lack of shearing during compaction → less dispersed (more flocculated) fabric (E2, Fig15)

4.2 Cont.

4) Effect of Increasing Compaction Effort (Kneading compaction)

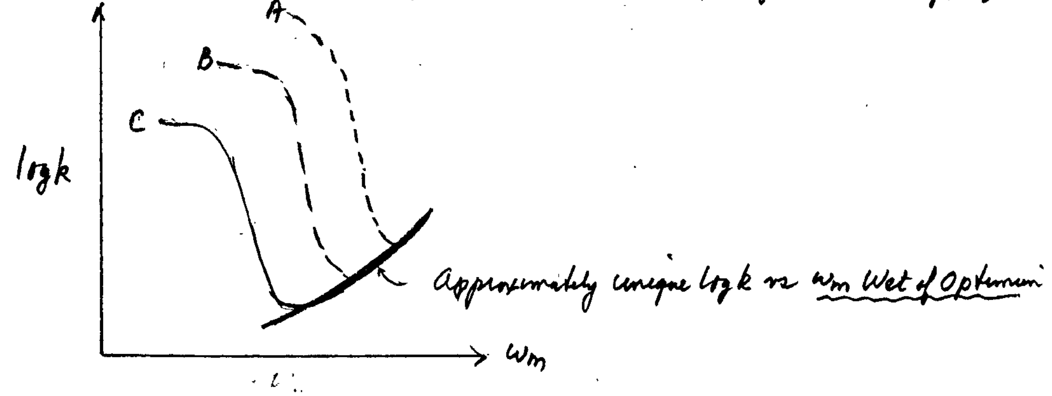


- Compaction effort C > B > A
- For actual data, see

E 2, Fig 10 ; Vicksburg silty clay, S=100%

E 3, Fig 9,10 : CH clay, u_b=0

E 3, Fig 18 : CL clay, u_b=0



4.3 Clay Liners

1) For hazardous waste containment, EPA requires $k \leq 1\times10^{-7}$ cm/sec

2) Good construction is far more important than liner thickness per se,
* [i.e., mix borrow, breakdown clay clods, compact wet of optimum,
scarify each lift.]
See Sheet E4.

3) See Sheet E5 for brief overview of environmental effects on k of compacted liners

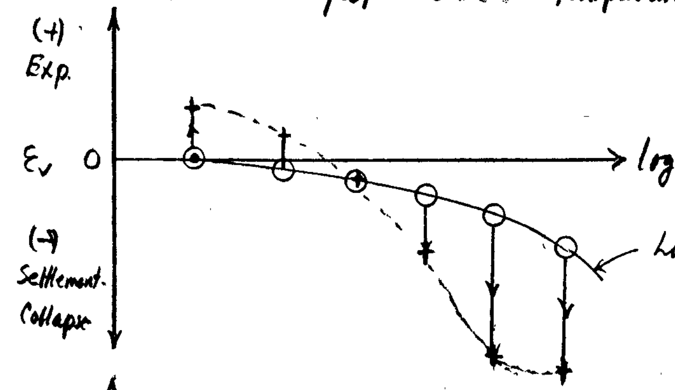
3/97, 5/7/98

5. COMPRESSIBILITY CHARACTERISTICS

5.1 Introduction

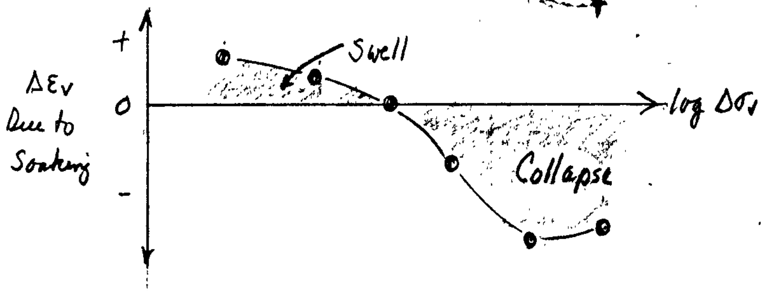
1) Will focus mainly on what happens when compacted clay is given access to water that causes a decrease in soil suction

2) 1-D volume change due to soaking as $f(\sigma_v)$.
(Called collapse/swell or swell/compression tests)



○ = Add $H_2O \rightarrow S_m = u_c = 0$
+ = Strain after soaking

Loaded at w_m: Occurs During Construction/Fill



● = $\Delta\varepsilon_v$ due to soaking
Occurs After Construction of Fill → Damage

NOTE: Important for construction on thick compacted fills, e.g. CA [p14, 2)]

$\pm \Delta(\sigma - u_a) \rightarrow \mp \Delta V$
$+ \Delta S_m \rightarrow$ compression
$- \Delta S_m \rightarrow$ Swelling at "low" $\sigma - u_a$
$\rightarrow$ collapse at "high" $\sigma - u_a$

3) Mechanisms Causing Swelling & collapse due to Soaking ($u_c = S_m \rightarrow 0$)

a) Swelling: (Most important @ low σ_v, high γ_d, low w_m (high u_c), high I_p)
From 1.322 Part A { 1.361 Part IV-4, Sec. 2.2
(1) particle unbending (stored elastic energy)
(2) increase in closest spacing due to - Osmotic pressure (DL expansion) ← Fig 8, Sheet F1
- adsorbed H_2O (for v. low w_m)
(3) maybe entrapped air (Ladd 1958)

b) Collapse (Most important @ high σ_v, low γ_d, low I_p → "metastable")
- From $\sigma' = \sigma - u$ principle, since u is increasing → $-\Delta\sigma'$, would expect swelling.
∴ Must have "metastable" structure.
e.g., decreasing S_m → dec. strength of aggregates
→ collapse of open fabric

("Softening & distortion of macropeds" Lawton et al. (1992) JGE 118(9)

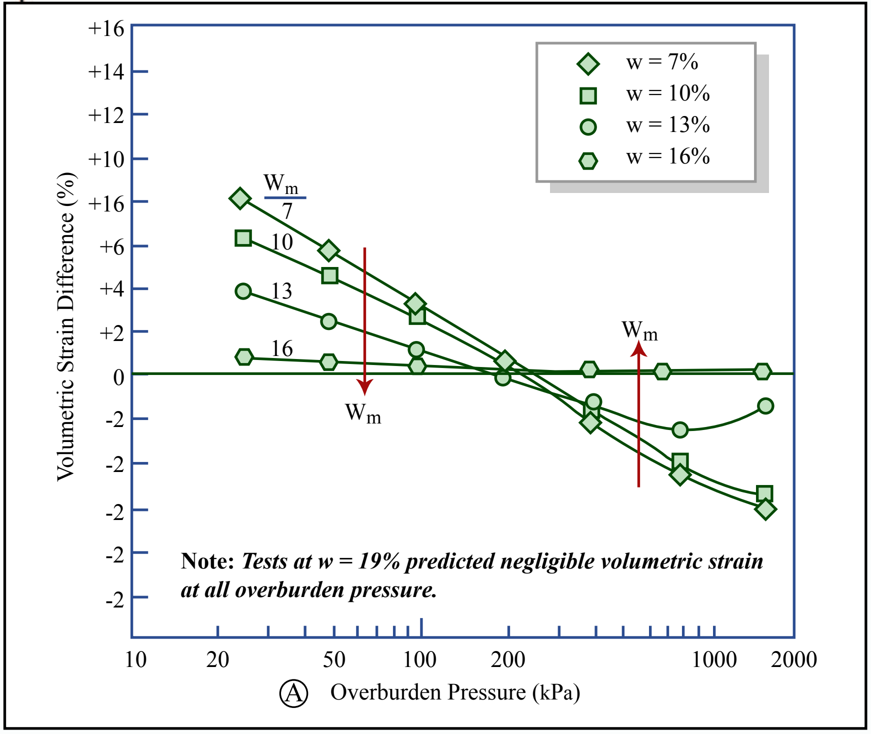
5.2 1-D Compressibility Due to Soaking

1) Note: Data from Lawton et al. (1989) JGE 115(9)

Compacted Clayey Sand: $D_{50} = 0.15$ mm, CF = 15%, $w_L = 34$, $I_p = 15$

Sheet F1, Fig. 1 Modified $w_{opt} = 10\%$, $\gamma_{d\,max} = 2.025$ g/cc ~ 126.4 pcf $RC = \frac{\gamma_d}{\gamma_{d\,max}}$, % = Relative Compaction

Used double oedometer method (involves some errors)



Note: *Tests at w = 19% predicted negligible volumetric strain at all overburden pressure.*

Figure by MIT OCW.

ⓐ Increasing w_m at Constant γ_d

(1) Less swelling: as would expect due to less Δ Suction (less $\Delta\sigma'$)

(2) Less collapse: probably combination of increasing ε_v at w_m and less Δ Suction

(based on Section 4.2, same γ_d → ≃ same fabric).

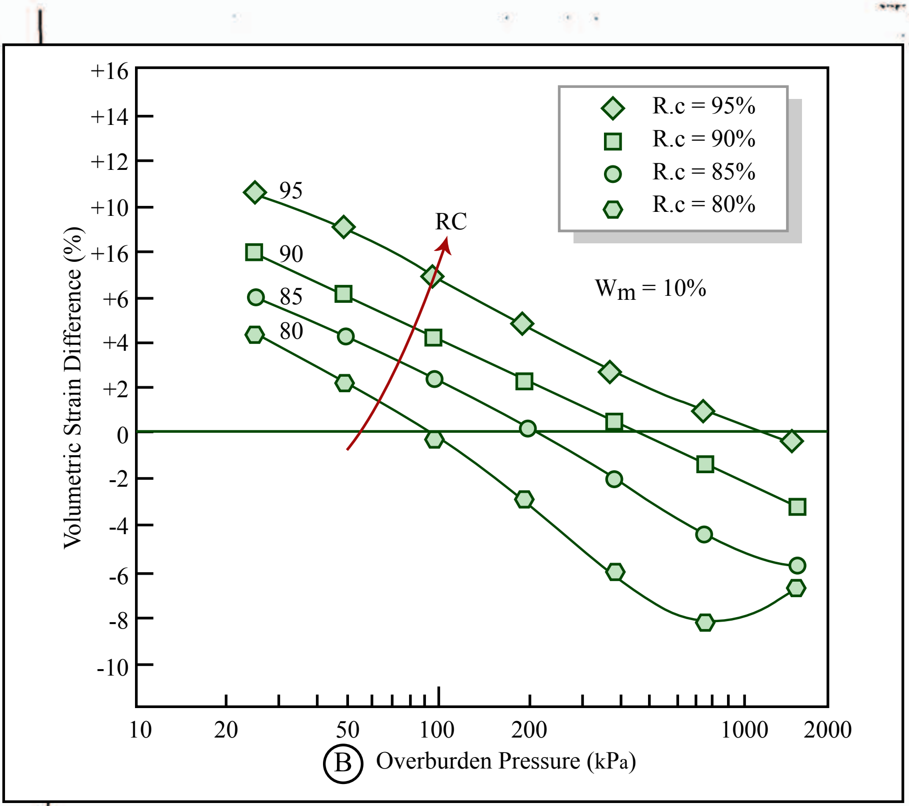
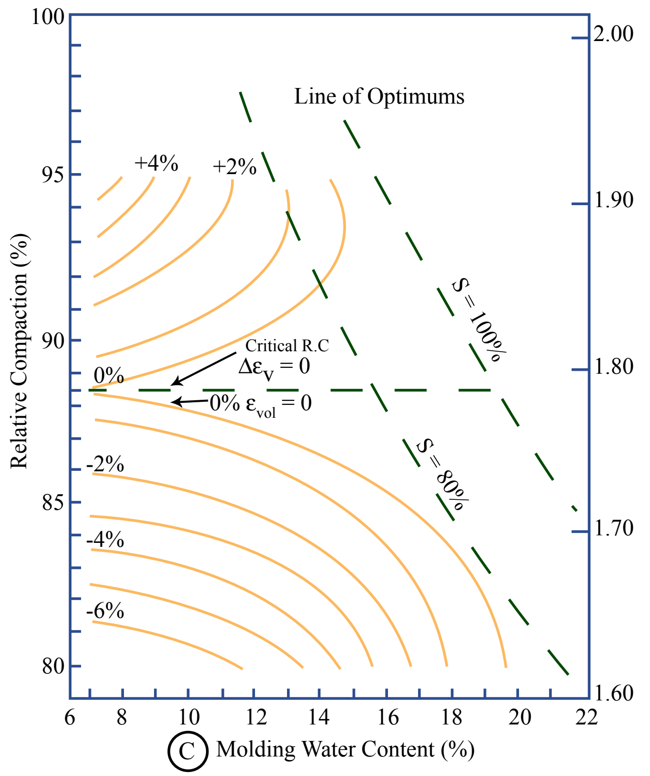


Figure by MIT OCW.

ⓑ Increasing γ_d at Constant w_m

(1) More swelling: denser packaging of aggregates for ≈ same Δ Suction (Section 2.2 → same u_c indep. of γ_d)

(2) Less collapse: denser packaging of aggregates → less metastable.



Isograms of volumetric strain difference as function of relative compaction and molding water content at overburden pressure = 400 kPa (Impact compaction)

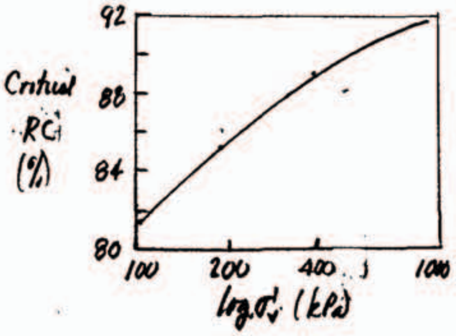
Figure by MIT OCW.

Ⓒ Combined Effect of w_m & γ_d

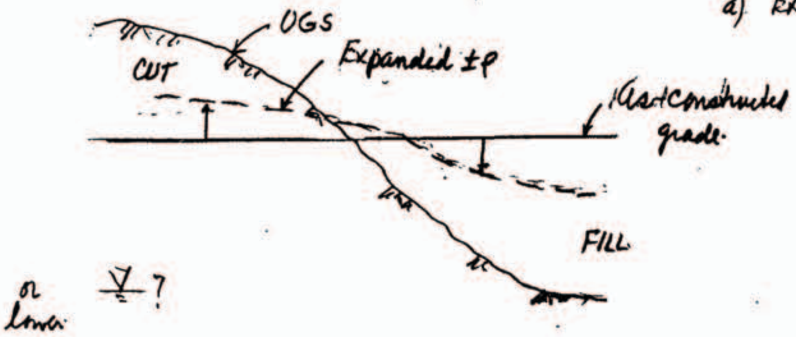
(1) For given σ_v, obtain triangular range of w_m-γ_d → $\Delta\varepsilon_v = 0$ due to soaking

- Critical Relative Compaction bisects triangle.

(2) Critical RC increases w/ σ_v



2) Discussion



a) Example of problems in CA, e.g.

- Brandon et al. (1990) JGE 116(10)
- Settlement → 18" after 10yr *
  $w_L = 40$, $I_p = 20$, RC ≈ 92% (Mod.)
  $w_m \approx w_{opt}$
- Development w/ start of irrigation
- Fill thickness → 80ft

b) Oversimplification

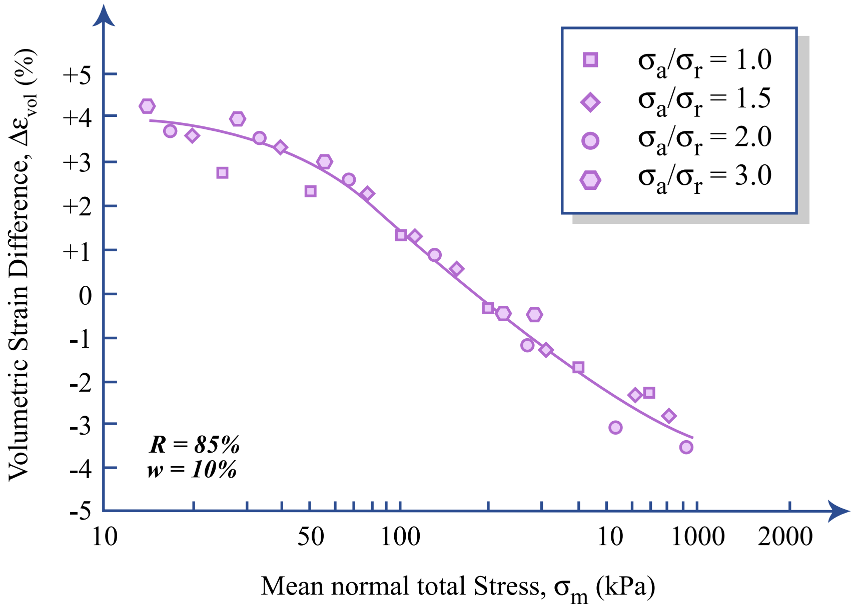
- Not strictly 1-D due to varying fill thickness
- Doubtful that Suction → zero as per testing (no in situ measurements of pore water pressure)
- Δρ → ground compression & stretching; latter especially important in causing cracking to walls, patios, etc.

* Called "hydrocompression"

5.3 3-D Compressibility Due To Soaking

1) Trends from Lawton et al. (1991) JGE 117(5)

- Same compacted sandy clay as in 5.2, but now "double triaxial tests"
- Testing at $w_m = 10$ (Mod. w_{opt}) and RC = 85% → $S_i = 50\%$ via Harvard Miniature compaction



Volumetric strain difference as function of normal total stress for triaxial tests

Figure by MIT OCW.

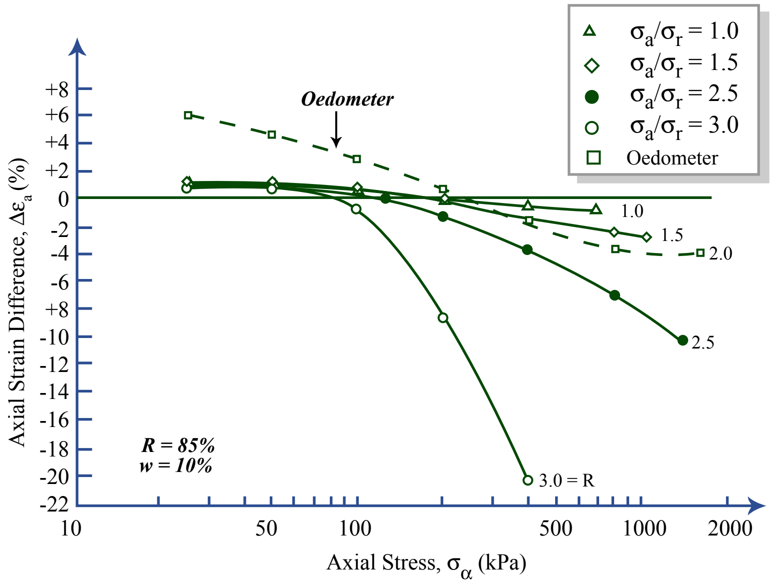
a) See Fig 2, Sheet F1 for log σ_a

- vs ε_{vol} at w_n
- vs ε_{vol} after soaking

} "double" TX testing

- vs $\Delta\varepsilon_{vol}$ = difference

b) Get unique $\Delta\varepsilon_{vol}$ vs log σ_m = log σ_{oct} indep. of $R = \sigma_a/\sigma_r$ of test (also found for another compacted clay)



Compression of axial strain versus axial stress curves for oedometer and triaxial tests

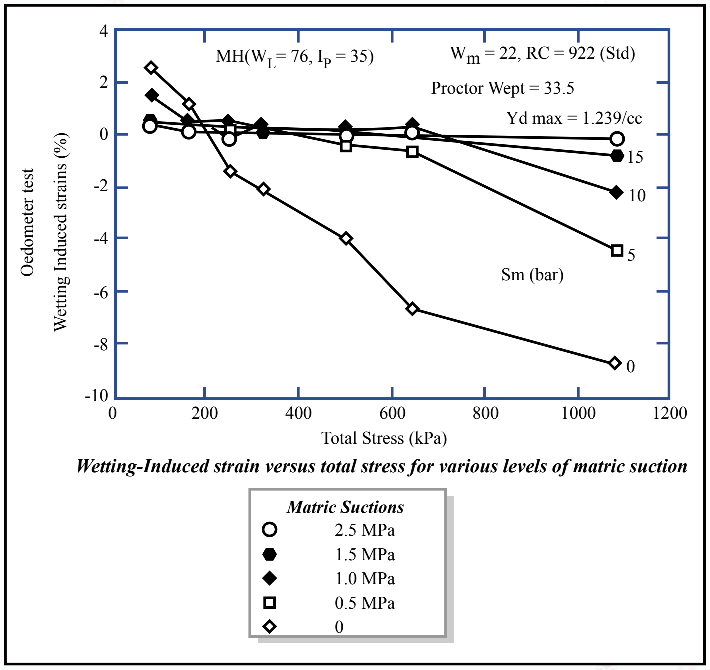
Figure by MIT OCW.

c) However, increasing $R = \sigma_a/\sigma_r$ → also more axial compression (collapse) as one would expect

5.4 Effects of Cyclic Wetting and Drying

1) There are very few data in literature that show influence of changing suction at constant σ_v for compacted clays (or for natural clays). This is topic of current MIT research (JTG & Kurt)

2) 3 examples



Wetting-Induced strain versus total stress for various levels of matric suction

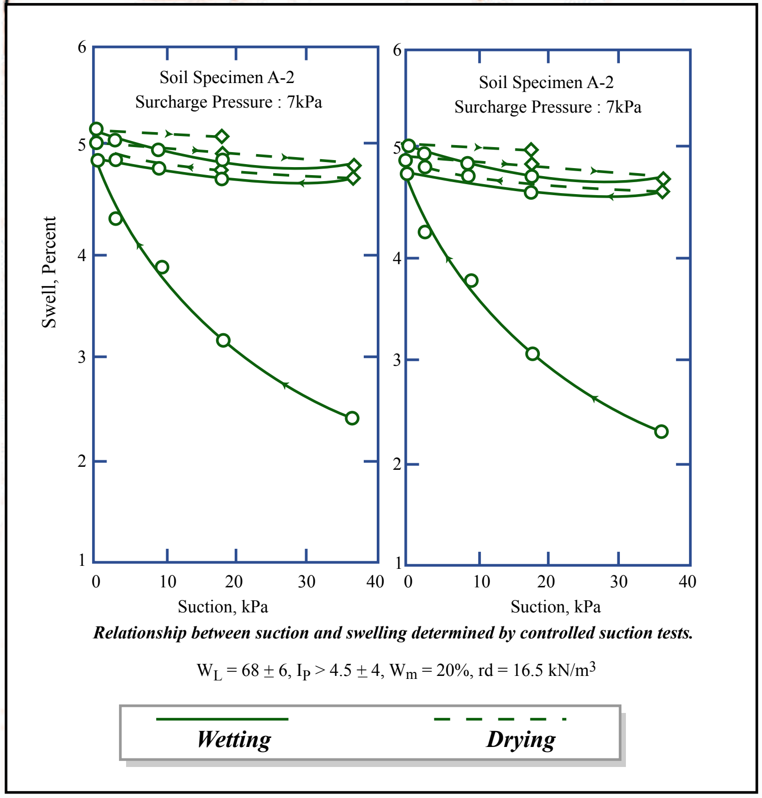
Figure by MIT OCW.

Adapted from: Vilar (1994) JGE 120(7)

Compacted to "close to Modified Proctor", S_i = 88%

a) Example of clay compacted very dry of optimum, with initial S_m = 25 bar.
- At varying σ_v, decrease S_m to 15, 10, 5 & 0 bar
- & "Wetting to $S_m > 0$ → much less collapse

Note: Get swelling at low $\sigma_v < \approx 200$ kPa



Relationship between suction and swelling determined by controlled suction tests.

$W_L = 68 \pm 6$, $I_P > 4.5 \pm 4$, $W_m = 20\%$, rd = 16.5 kN/m^3

Figure by MIT OCW. (after Chu and Mou 1973).

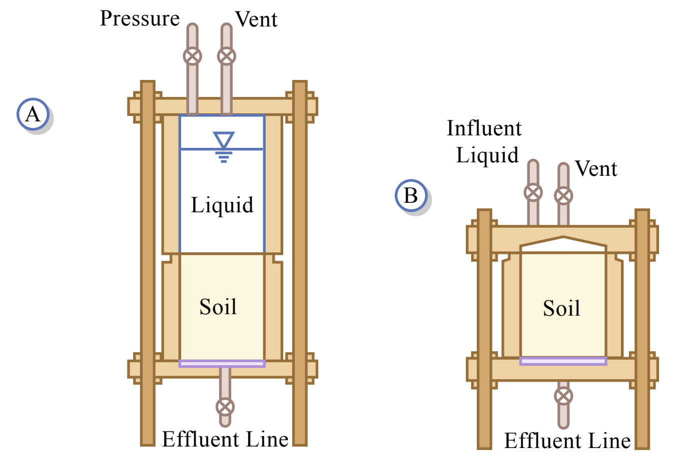
$W_L = 68 \pm 6$, $I_p > 41.5 \pm 4$, $W_m = 20\%$, $\gamma_d = 16.5$ kN/m^3

b) Example of clay compacted to near w_{opt} with S_m = 0.35 bar.
- Allowed to swell as $S_m \to 0$
- Then cycles of drying & wetting (ΔS_m = 0.35 bar)
- Note relatively small $\pm\Delta\varepsilon_v$ due to cycle drying & wetting after initial swelling (called "fatigue")

c) But see Sheet F2 for example where cyclic drying & wetting (after initial swelling) → very significant $\pm\Delta\varepsilon_v$

+ notes on "active zone" etc.

Permeameter Devices: From Daniel et al. (1985) ASTM STP 874



Compaction-mold permeameters with (A) a reservoir of permeant liquid contained within a collar located directly above the soil specimen and (B) a separate reservoir of permeant liquid.

Figure by MIT OCW.

- Best for lab compacted specimens with low σ'_v in field
- May get side wall leakage if permeant causes large depression of double layar (e.g, concentrated organic solvents)

- good for studying effect of $\Delta\sigma'_v$ on compressible soils

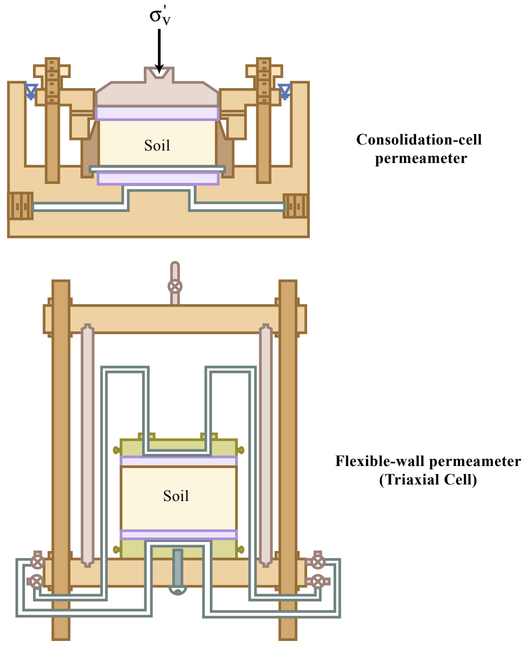
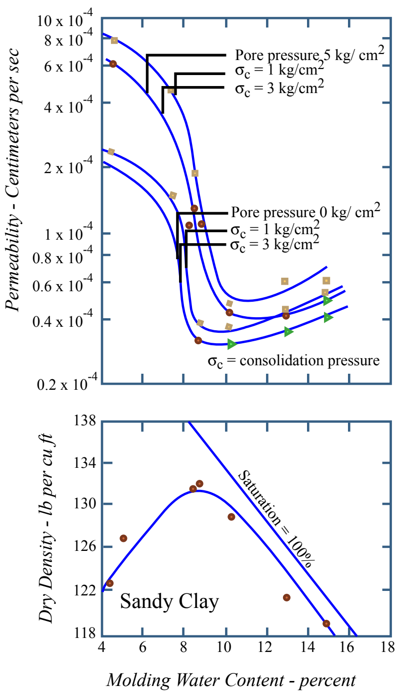


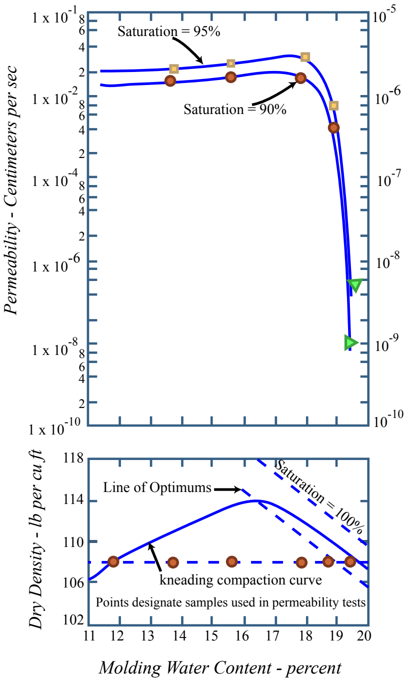
Figure by MIT OCW.

ASTM D 5084

- Best for field samples having irregular surface (problems with trimming, contained stones, etc.)
- Can easily backpressure to obtain $S \approx 100\%$



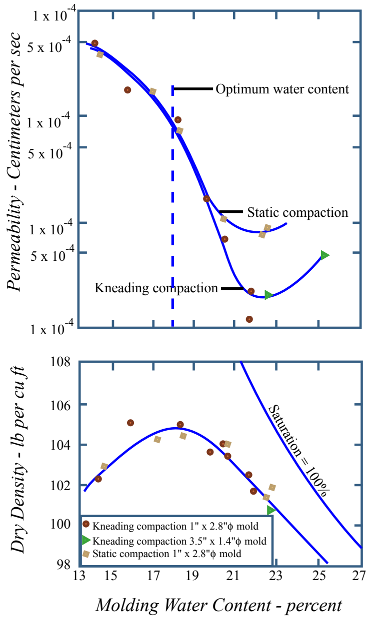
Effect of saturation using back pressure and consolidation pressure on permeability



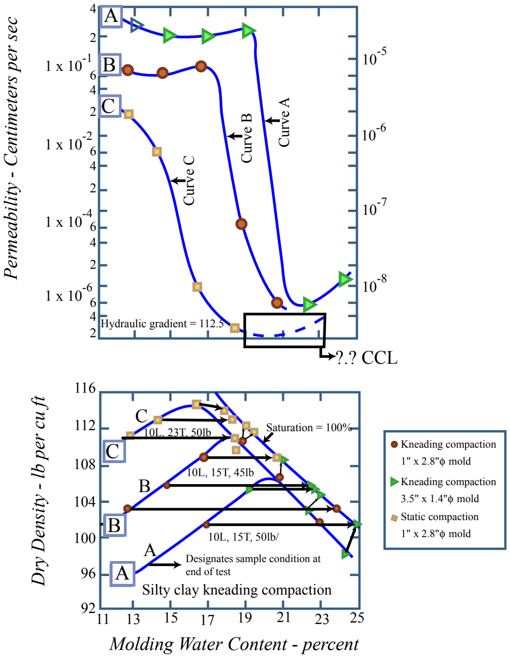
Permeability as a function of molding water content for samples of silty clay prepared to constant density by kneading compaction

Data on Std. Proctor, Sandy Clay from Bjerrum & Huder (1957) in Triaxial Cell

Data in Fig. 6, 10 & 15 for Vicksburg Silty Clay (Rigid wall, top Stone & ub)
wL = 37%, Ip = 14%, CF = 24%



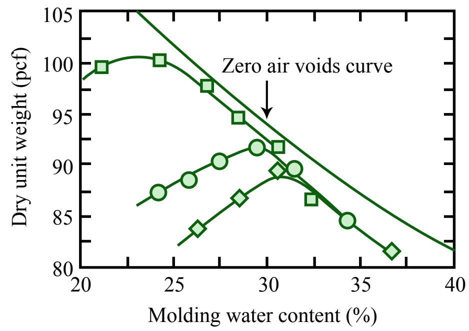
Influence of method of compaction on the permeability of silty clay



Permeability at 100% saturation for samples of silty clay prepared at three compactive efforts

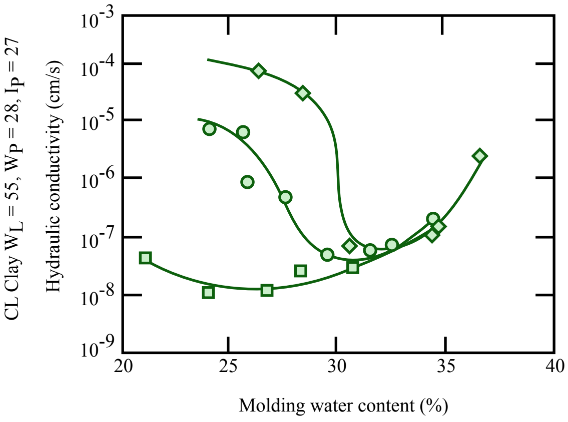
E2

Figures by MIT OCW. Adapted from: Mitchell et al. (1965) JSMFD 91(4)



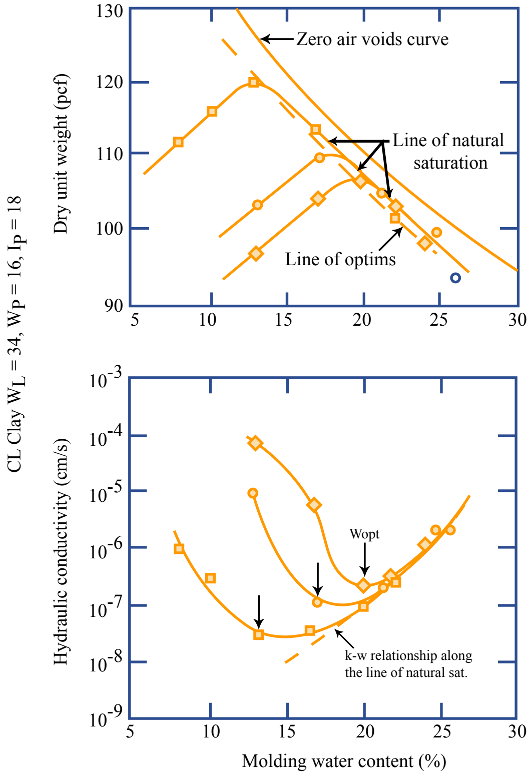
Compaction curves for type a soil from east borrow area (1 pcf = 0.157 kN/m³)

Figure by MIT OCW.



Hydraulic conductivity versus molding water content for type a soil from east borrow area.

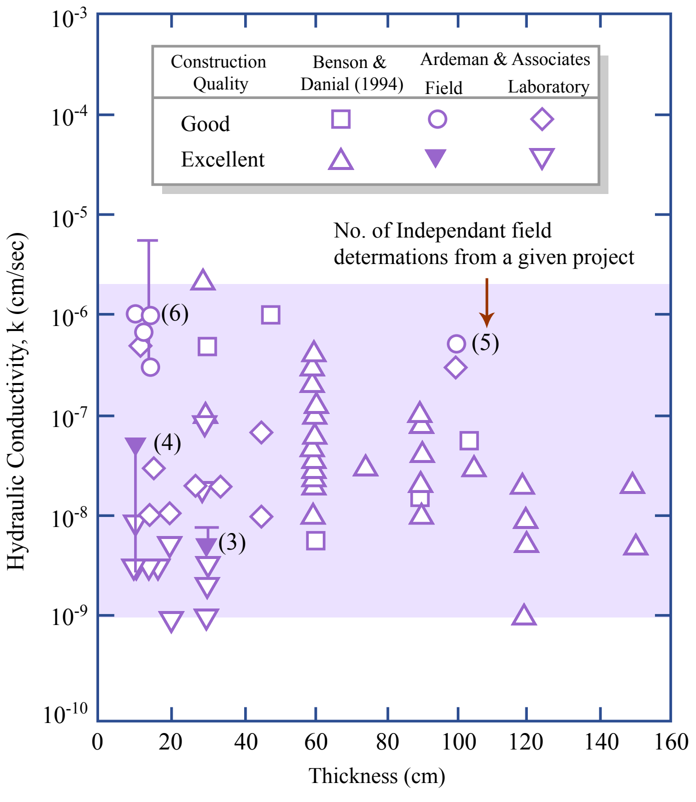
Figure by MIT OCW.



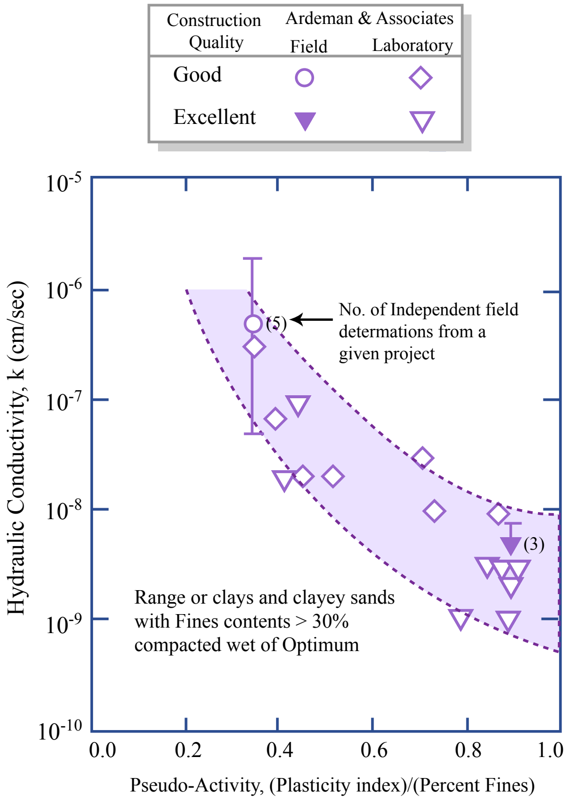
Compaction Curves: Hydraulic Conductivity versus molding water content relationships.

Figure by MIT OCW.

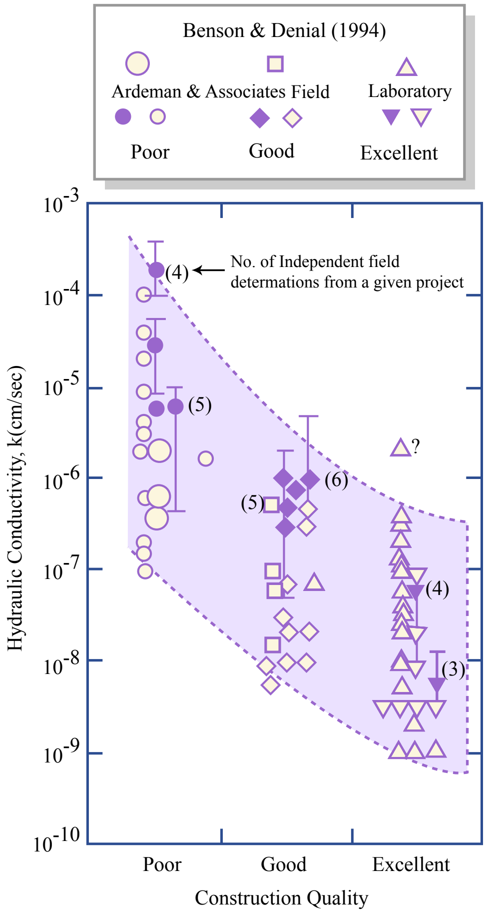
Rigid-Wall Permeameter without u_b : ∴ k dry of optimum probably too low



Hydraulic conductivity of Well-Built liners as function of thickness



Hydraulic conductivity of Well-Built liners as function of Pseudoactivity

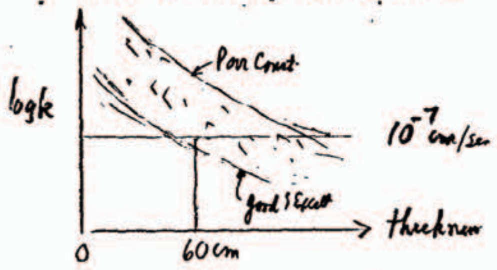


Hydraulic Conductivity as Function of Construction quality

Figures by MIT OCW.
Adapted from:

Fuleihan & Wissa (1995) JGE 121(6)
discussion of Benson & Daniel (1994)
JGE 120(1)

Note: Benson & Daniel (1994)
recommended minimum thickness of
clay liners = 60-90cm (2-3ft)
based on analysis of case histories
that included poor construction



5/96 5/98

App. D Environmental Effects on k of Compacted Liners
(Natural clays compacted with "pure" water)

REF: Daniel (1987) "Earthen Liners for Landfill Disposal Facilities" } ASCE Spec. Conf.
Mitchell & Madsen (1987) "Chemical Effects on Clay Hydraulic Conductivity" } GSP No. 13

Effects of Permeant

1) Strong Acids (low pH): can have large increase in k at low σ'_v if acid dissolves soil (but will take a long time) or if gross shrinkage & cracking.

2) Strong Bases (high pH): not much data (CCL - if kaolin, will weaken edge to face electrostatic attraction → potential piping)

3) Increased salts (inorganic, pH≈7): increasing salt conc. & cation valence → depression of double layer → theoretical increase in k (but probably minimal effect for well compacted clay - CCL opinion)

4) Neutral organic liquids

a) See M & M ('87) for detailed description of different organics and their properties. Important factors are:
(1) water solubility
(2) dielectric constant (decr. D_0 → decr. DL)
(3) polarity
(4) pure vs dilute solution

b) Pure organics usually affect k; may increase alot if formation of cracks, so that increase in σ'_v important. One example with methanol ($D_0 \sim 33.6$)

∴ Tests with rigid wall & $\sigma'_v = 0$ may overestimate increase in k

$\sigma'_v = 10$ kPa, $k \approx 4 \times 10^{-5}$ cm/sec
$= 50$ " $= 4 \times 10^{-6}$ "
$= 100$ " $= 2 \times 10^{-9}$ "

c) Overall, generally not a problem if ≥ 50% water and no separation of phases

Effects of Weather

1) Desiccation → cracking → incr. k. ∴ protect from drying

2) Freezing/thawing → cracking → incr. k ∴ " " freezing

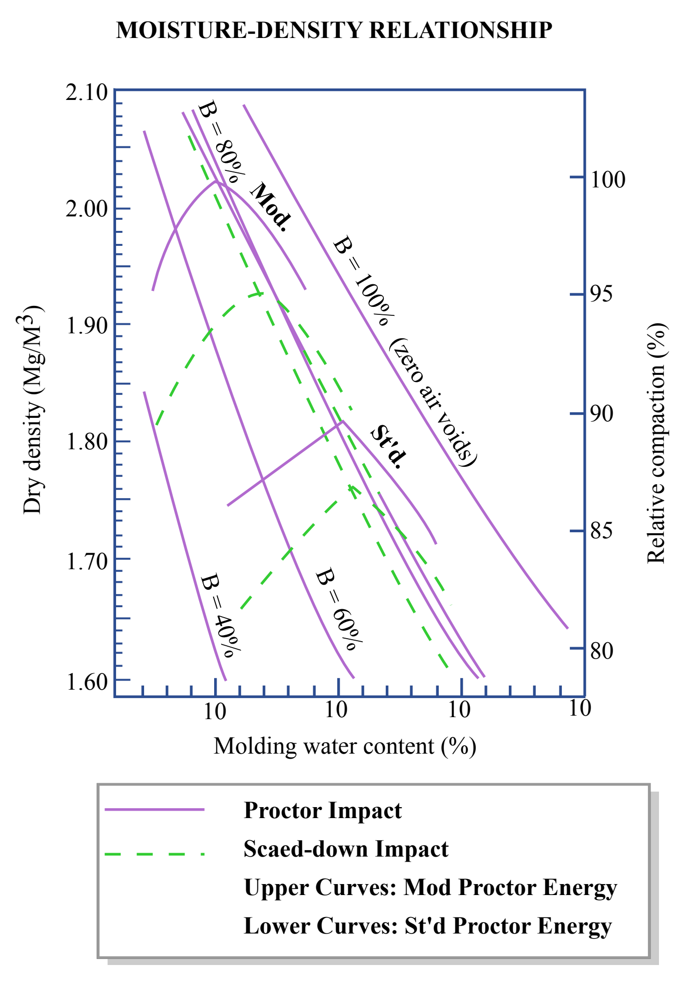
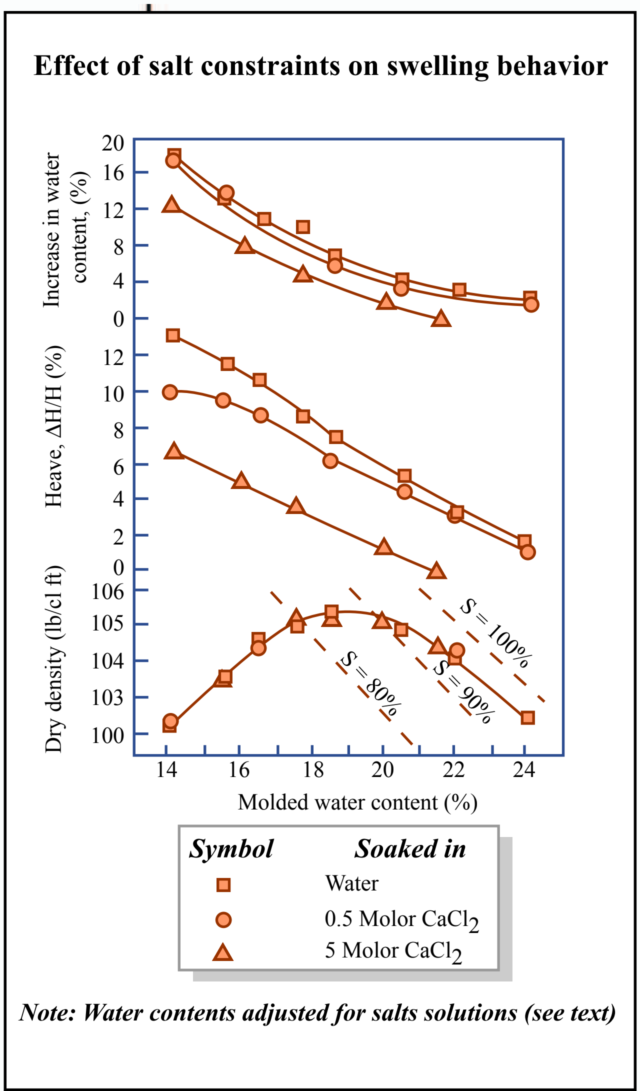


Figure by MIT OCW.
Adapted from: Lawton et al. (1989)

Vicksburg Buckshot Clay, CE = 2x Std. Proctor



Note: Water contents adjusted for salts solutions (see text)

Figure by MIT OCW.
Adapted from: Ladd (1958)

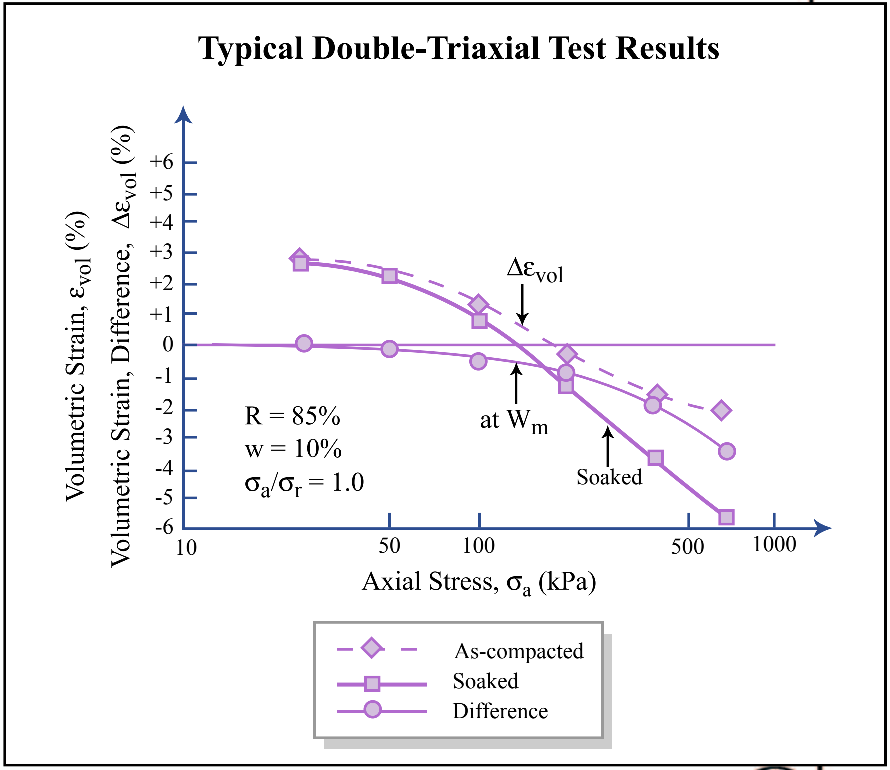
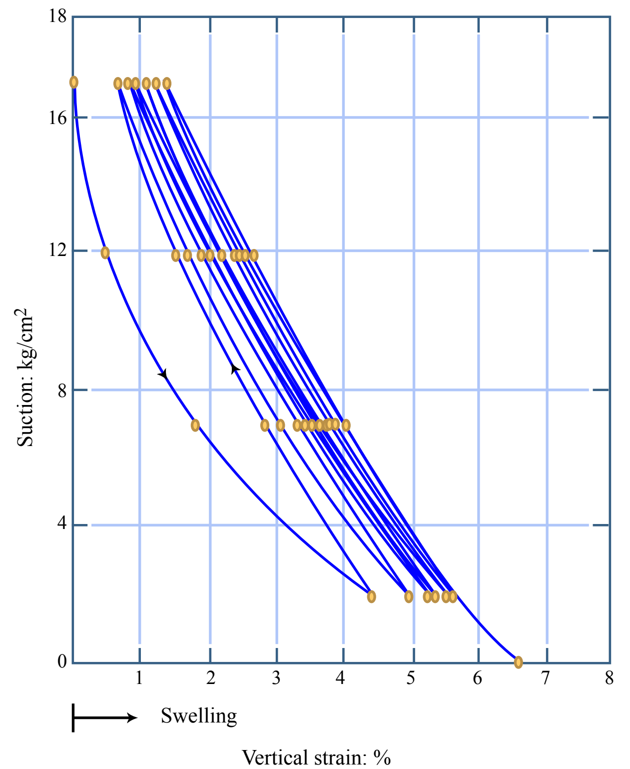


Figure by MIT OCW.

Adapted from: Lawton et al. (1991) Compacted Sandy Clay

F1



Swelling and shrinkage cycles under controlled suction changes, for a remoulded sample of Madrid grey clay ('Penuela') with the properties shown in TABLE 1 (initial conditions: $\omega = 24\%$; $\gamma = 1.34$ g/cm^3; surcharge = 0.1 kg/cm^2) (after Pousadas, 1982)

Figure by MIT OCW.

Effect of cyclic wetting & drying

Escario / Saez (1986) Geot. 36(3)

See Table 1, Sheet H1 for clay properties

MH : $w_L = 71$ $I_p = 35$

Std. Proctor : $w_{opt} = 33.7$, $\gamma_d = 1.33$ g/cc

Remoulded = static compaction dry of opt, but dense

$w_m = 24\%$, $\gamma_d \approx 1.34$ g/cc

$e_i \approx 1.015$ {$S_i \approx 65\%$} for assumed $G_s = 2.7$

5/7/98

Note on Active Zone : cyclic wetting / drying (swelling / shrinkage) due to seasonal variations in climate.

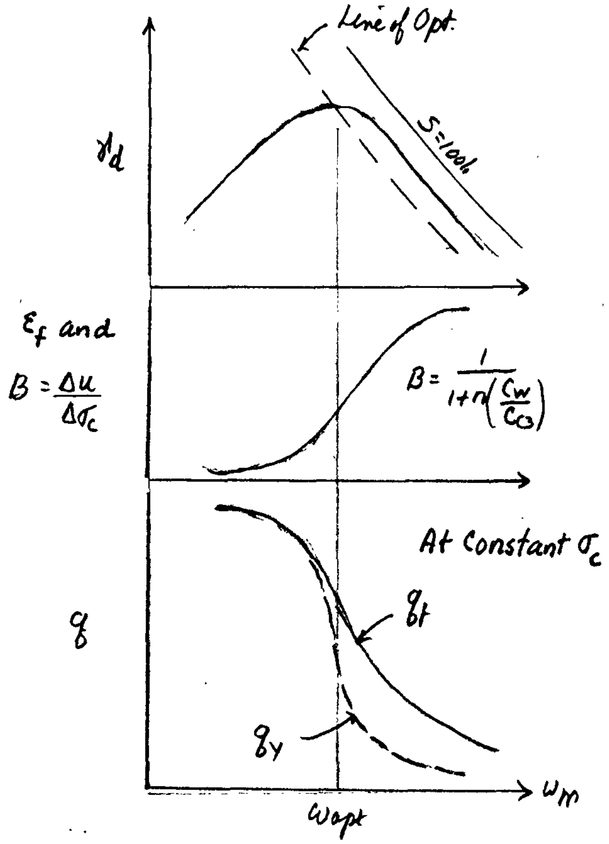
See Consolidation Notes : Problem Soils : Expansive Clays (ES-1 → 5)
Part VI

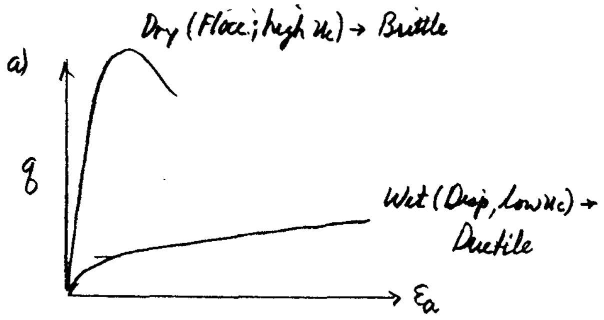
- Examples of water surplus / deficit during year
- Definition & depths of active zone, including very important influence of vegetation
- Examples of $\Delta\rho(t)$ & ΔWT elevation with time

6. SHEAR CHARACTERISTICS

6.1 UUC Data on As-Compacted Samples (Impact/Kneading Compaction)

1) General

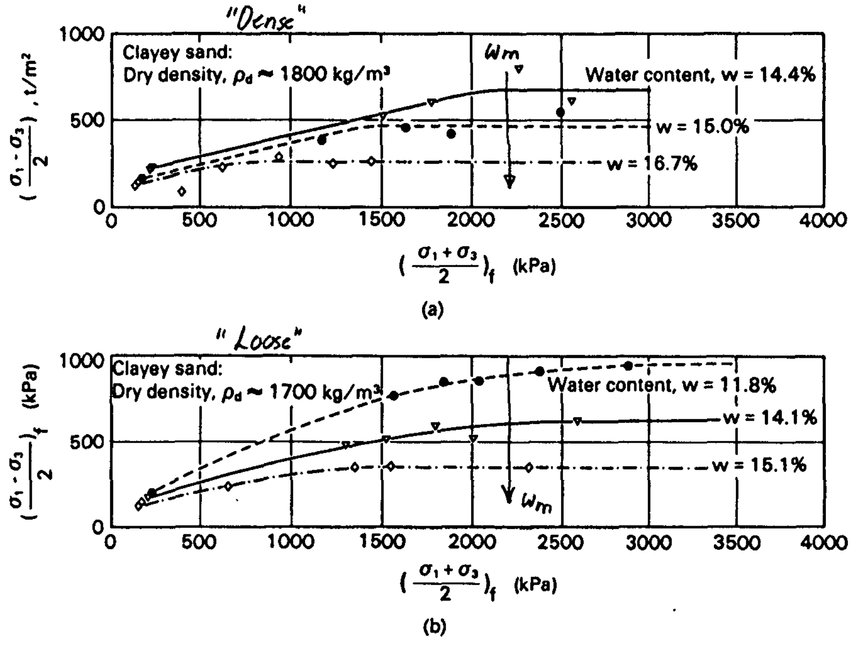




b) Dry: V. low B due to V. high C_W & "Low" C_{CS}
∴ $\Delta\sigma_c$ → large inc. q.
Wet: higher B due to lower C_W & higher C_{CS}
∴ $\Delta\sigma_c$ → less inc. q.
(See Fig. 10.44 below)

c) Wet: small yield stress followed by shear hardening

Sand = 52%
Silt = 18%
Clay = 30%
w_L = 30%
I_p = 11%
HM Compaction
UUC
$\dot{\varepsilon}$ = 6.1%/hr

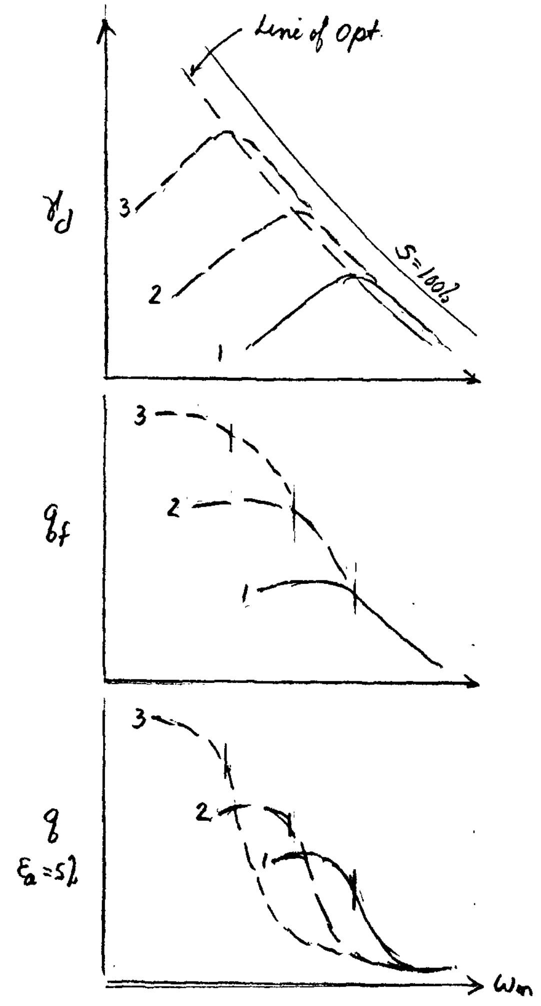


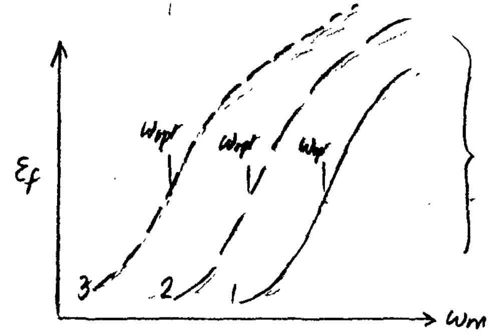
Fredlund & Rahardjo (1993)

Figure 10.44 Total stress point envelopes obtained from undrained triaxial and unconfined compression tests. (a) Total stress point envelopes for high density specimens; (b) total stress point envelopes for low density specimens (from Chantawarangul, 1983). AIT SM Thesis

6.1 Cont.

2) Effect of Varying Compaction Effort and "Overcompaction"

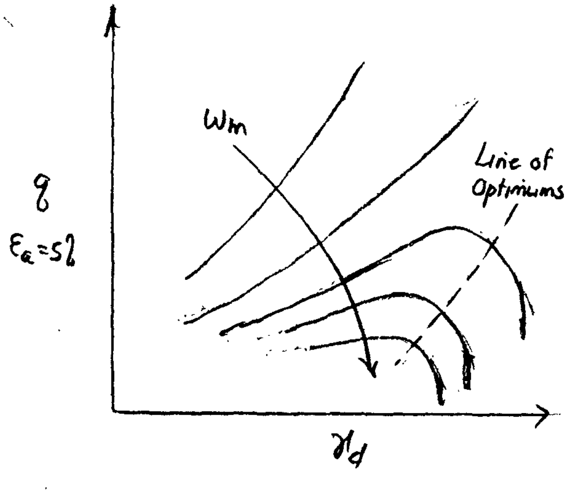




Not sure of trends w CE

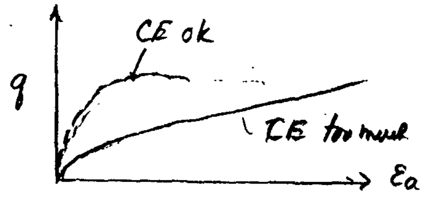
- q at small strain most applicable to pavement design

See Sheet G1 for data on 3 clays



- Low w_m : incr. γ_d → incr. q_5
- Mod. w_m : incr. γ_d → incr. & then decr. q_5
- High w_m : incr. γ_d → const. & then decr. q_5

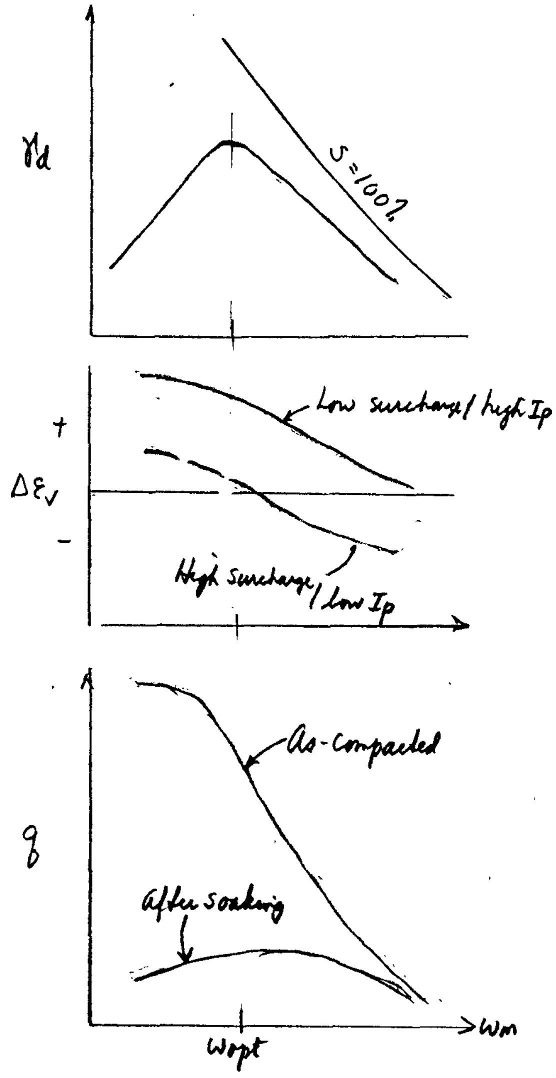
Overcompaction = too large CE → too much shearing ("beating") of soil



WE Story : Clay liner litigation

6.2 UUC Data on Compacted Samples After Soaking

1) General Trends

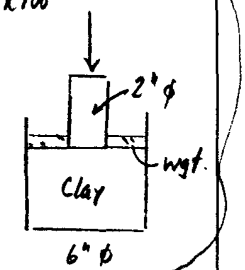


a) wm vs Δu_c soaking
- High vs low plasticity clay

b) Effect of surcharge (overburden) on ± $\Delta\varepsilon_v$
+ soil type

c) Δq due to soaking
- Surcharge level
- Soil type

CBR = Calif. Bearing Ratio
$= \frac{P \text{ at } 0.1''}{1000 psi} \times 100$

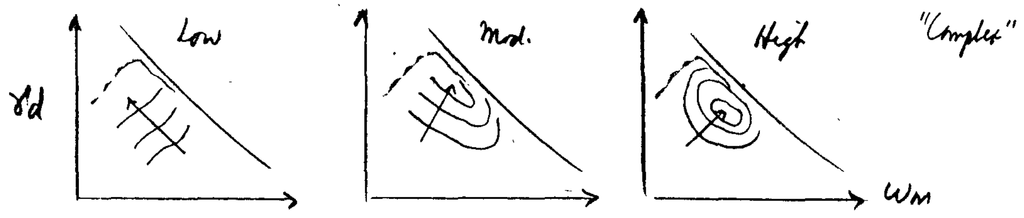


2) Data on Several Clays

Sheet G2, Fig 2 : VBC $\Delta V=0$ q_5 contours on wm vs γ_d

Fig 5,6 : Sandy Clay, σ_v > 35 vs 1 psi, q_{25} contours

Sheet G3 Soaked CBR data on low, moderate & high Ip clay



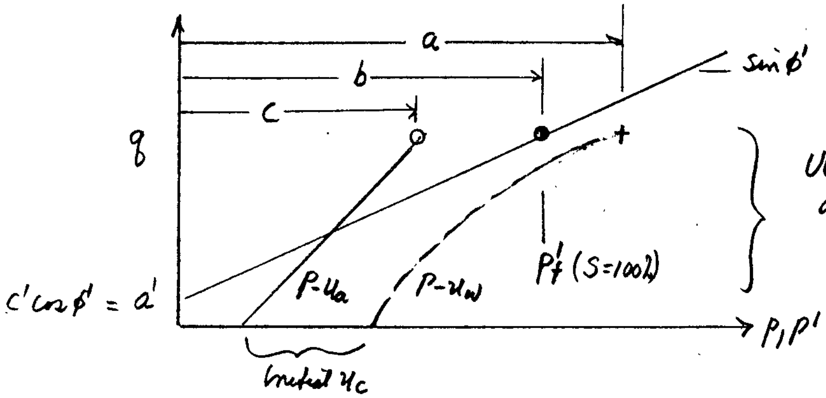
5/98

6.3 Effective Stress Eqn/Envelopes

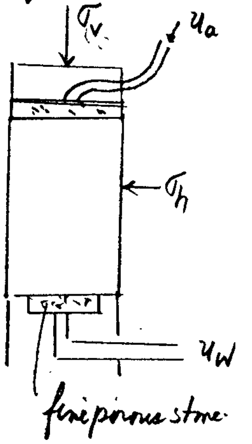
6.3.1 Bishop et al. (1960) Boulder Conf. ASCE

Eq. $\sigma' = (\sigma - u_a) + \chi\,(\underbrace{u_a - u_w}_{u_c = S_m})$

$\tau = c' + [(\sigma - u_a) + \chi u_c] \tan\phi'$; c' & ϕ' from testing at $S = 100\%$

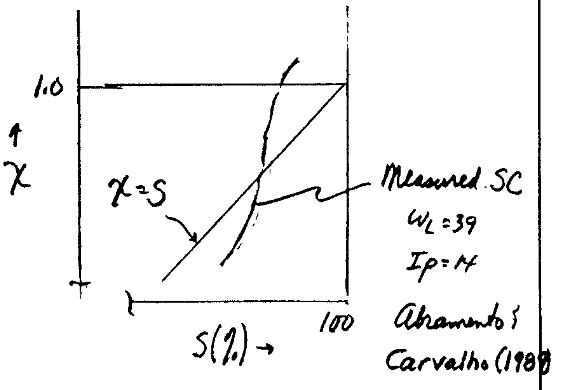


UUC at const. w and u_a, measure Δu_w



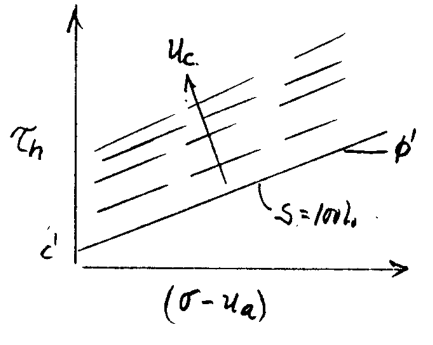
Need continuous air

$$\chi = \frac{p'_f - (p_f - u_a)}{[\underbrace{(p_f - u_w) - (p_f - u_a)}_{u_{cf}}]} = \frac{b - c}{a - c}$$

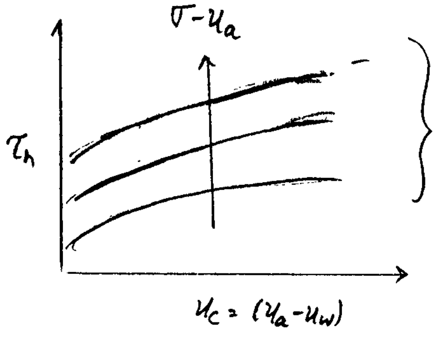


6.3.2 CD Direct Shear Tests on 3 Compacted Soils

(Sheets H1 & H2)



approx. linear



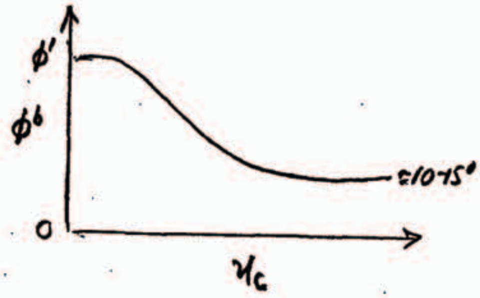
definite curvature

6.3.3 Eqn. for Failure Envelope

a) Fredlund et al +1993 Book

$$\tau = c' + (\sigma - u_a)\underset{\uparrow \atop S=100\%}{\tan\phi'} + u_c \tan\phi^b$$

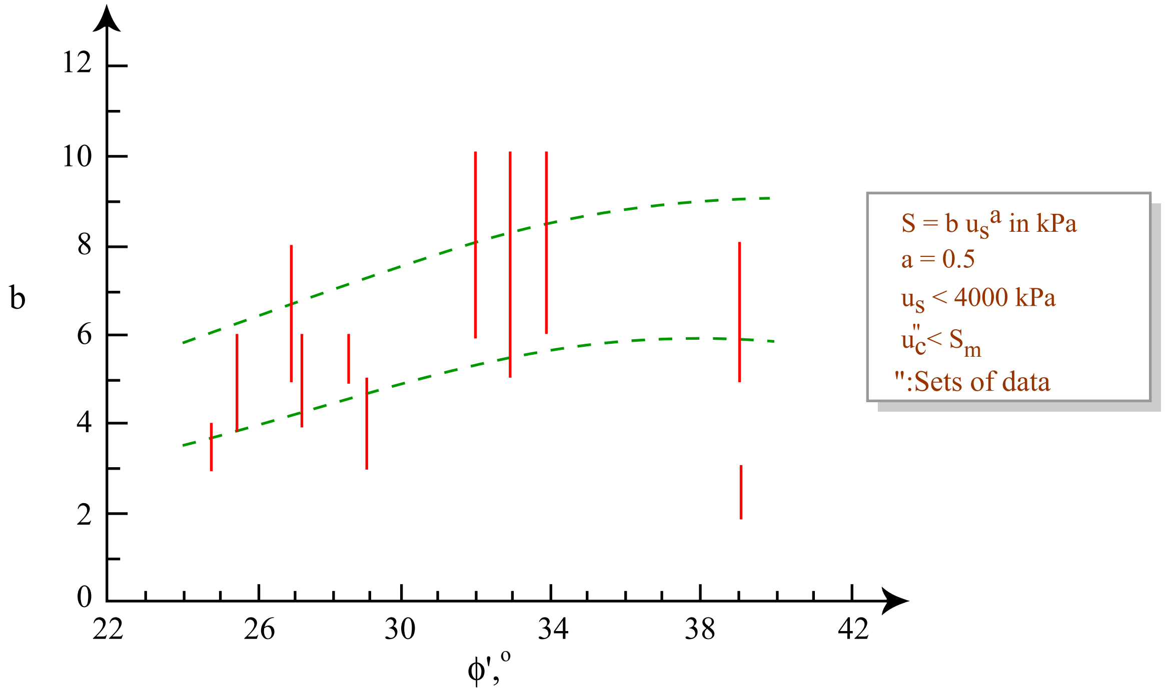
i.e., $\phi^b \neq$ constant



b) Abramento & Carvalho (1989) 12th ICSMFE

$$\tau = c' + (\sigma - u_a)\underbrace{\tan\phi'}_{\text{for } S=100\%} + b(u_c)^a$$

$a = 1/2$ $b = 6 \pm 4$

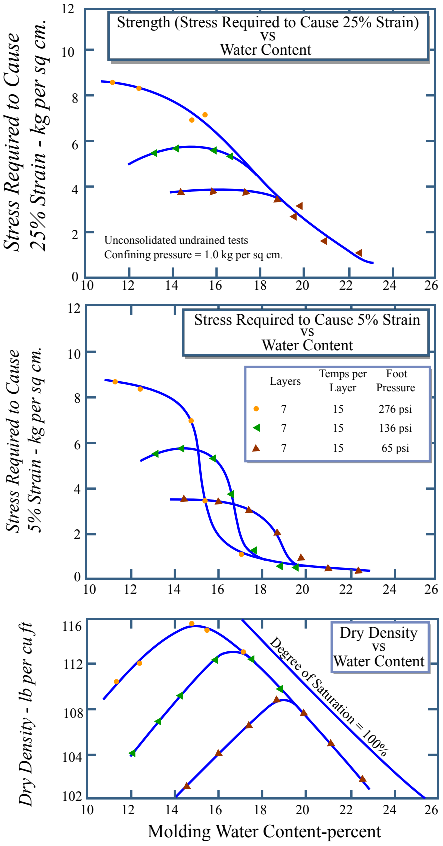


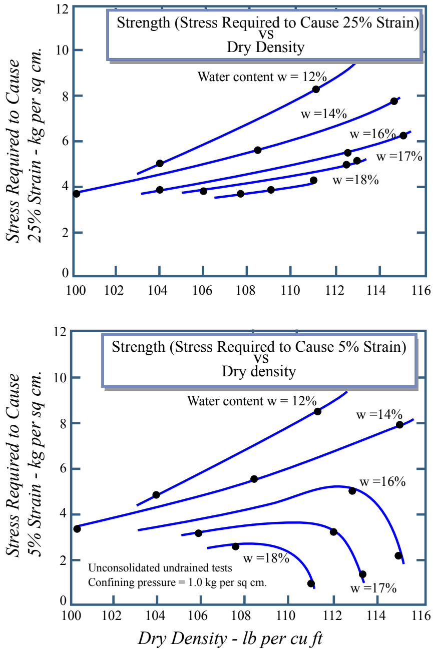
Values of b from shear strength measurements with suction control

Figure by MIT OCW.

Adapted from: Mesri & Shahien (1994)

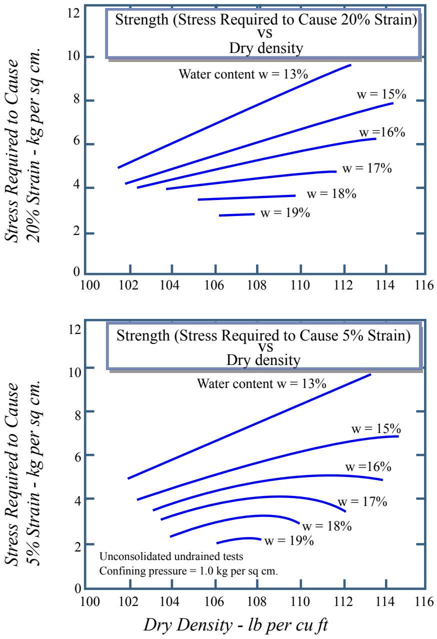
CCL 5/12/96 1.322 Compacted Clays





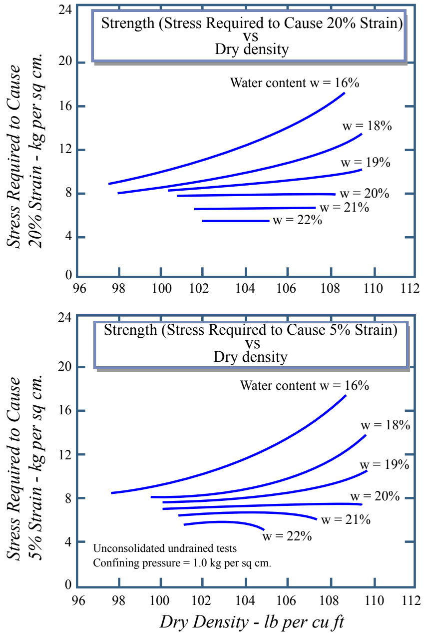
Relationship between dry density, water content and strength as compacted for samples of silty clay-kneading compaction.

$W_L = 37$ $I_P = 14$ CF = 24



Relationship between dry density, water content and strength as compacted for samples of sandy clay-kneading compaction.

$W_L = 35$ $I_P = 16$ CF = 24



Relationship between dry density, water content and strength as compacted for samples of highly plastic clay-kneading compaction.

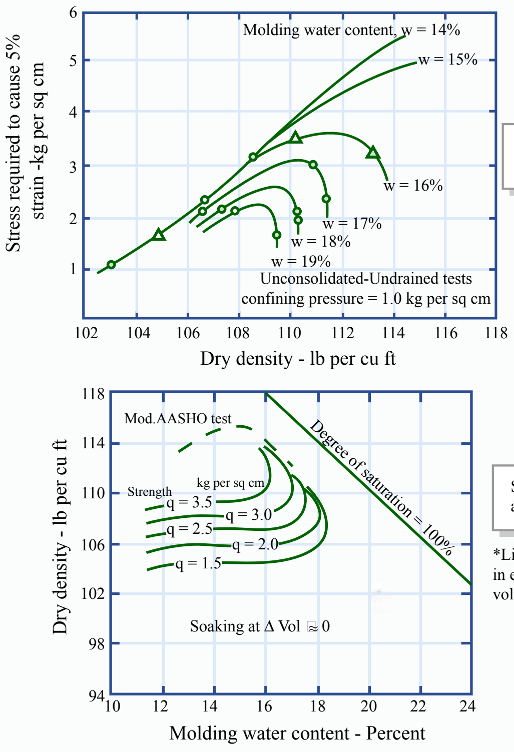
$W_L = 59$ $I_P = 32$ CF = 46

Figure by MIT OCW.

Effect of w_m & γ_d on As-Compacted UUC Strength

UUC Data from Saad & Chen (10/59) JSMFD, 85(5)

G1



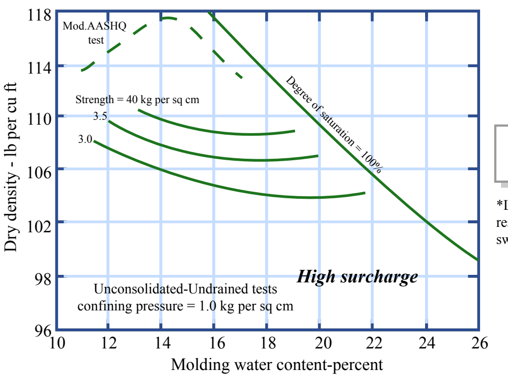
Strength after soaking (stress required to cause 5% strain) vs Dry density

Strength after soaking - determined as stress required to cause 5% strain

*Lines show initial composition resulting in equal strengths after soaking at constant volume.

Relationship between initial composition and strength after soaking at constant volume for samples of silty clay prepared by kneading compaction.

Figure by MIT OCW.

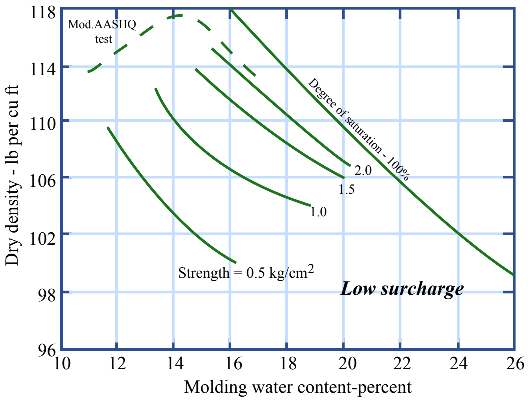


Relationship between initial composition and strength after soaking under surcharge pressure of 35 PSI for samples of sandy clay prepared by kneading compaction.

Strength determined as stress required to cause 25% strain

*Lines show initial composition resulting in equal strengths after swelling

Figure by MIT OCW.



Relationship between initial composition and strength after swelling under surcharge pressure of 1 PSI for samples of sandy clay prepared by kneading compaction.

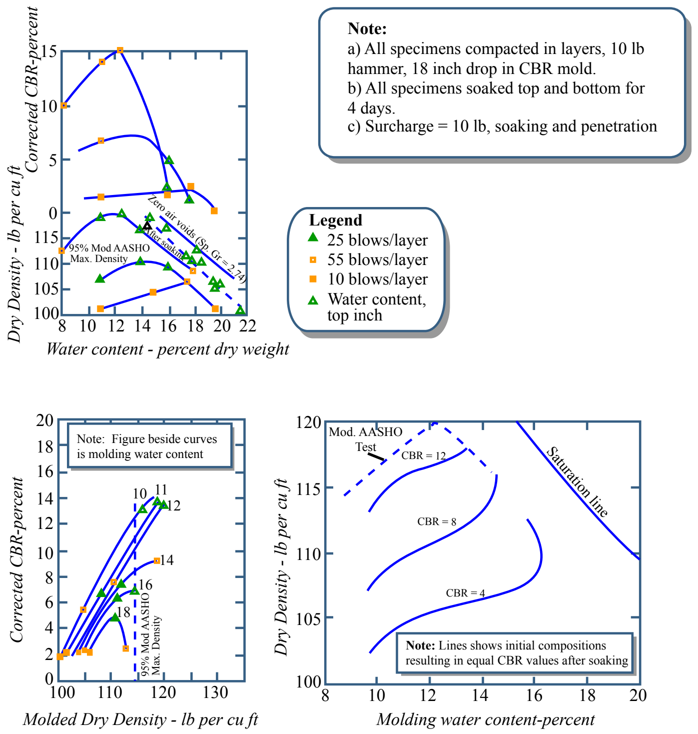
Strength determined as stress required to cause 25% strain

*Lines show initial composition resulting in equal strengths after swelling

Figure by MIT OCW.

UUC Data on Compacted Clays After Soaking from Seed & Chan (12/59) JSMFD 85(6)

CCL 5/12/96 1.322 Compacted Clays

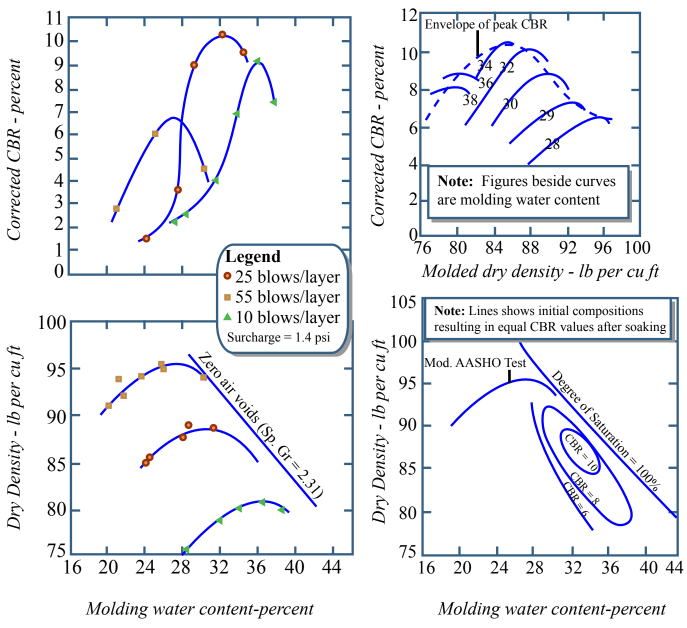


Note:
a) All specimens compacted in layers, 10 lb hammer, 18 inch drop in CBR mold.
b) All specimens soaked top and bottom for 4 days.
c) Surcharge = 10 lb, soaking and penetration

Note: Lines shows initial compositions resulting in equal CBR values after soaking

Relationship between initial composition and soaked CBR for samples of lean clay.

(Data from Corps of Engineers Manual on Airfield Pavement Design)

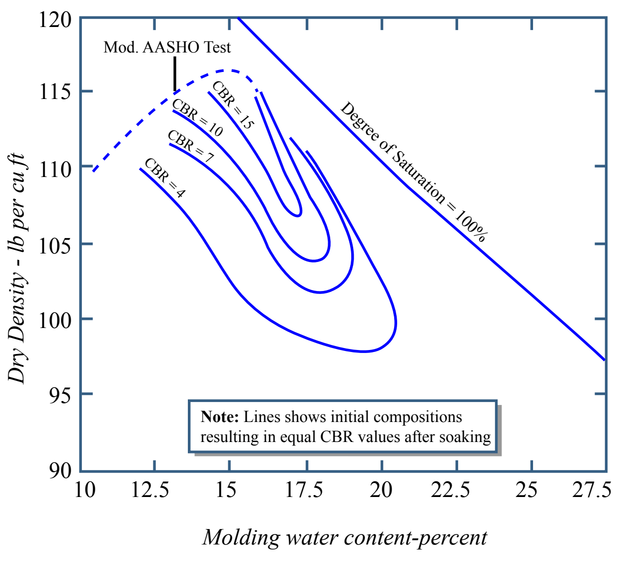


Note: Figures beside curves are molding water content

Note: Lines shows initial compositions resulting in equal CBR values after soaking

Relationship between initial composition and soaked CBR for samples of highly plastic clay.

(Data after J.R. Bell)



Note: Lines shows initial compositions resulting in equal CBR values after soaking

Relationship between initial composition and soaked CBR for samples of silty clay.

(Data after W.J Turnbull and CR Foster)

G3

Summary of CBR Data After Soaking from Seed & Chan (12/59) JSMFD 85(6)

Figures by MIT OCW.

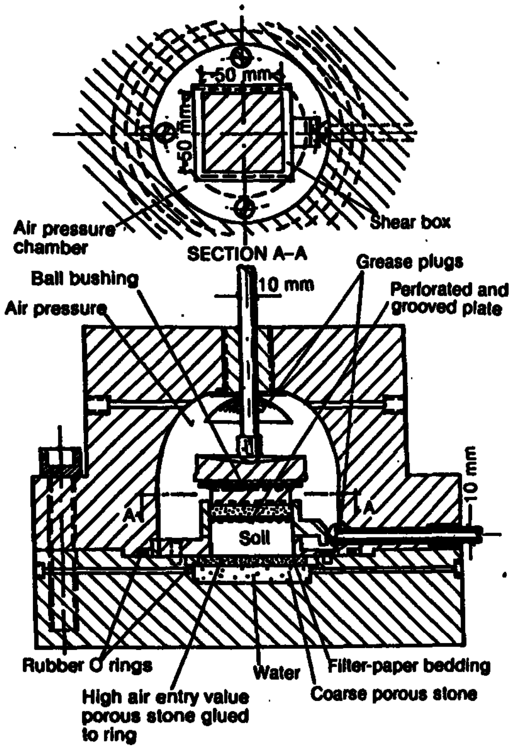
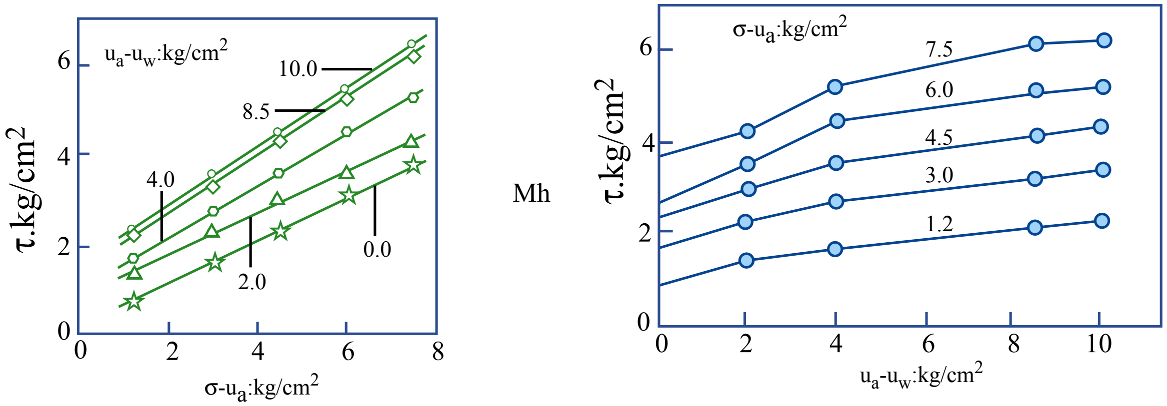


Fig. 1. Direct shear test apparatus with controlled suction

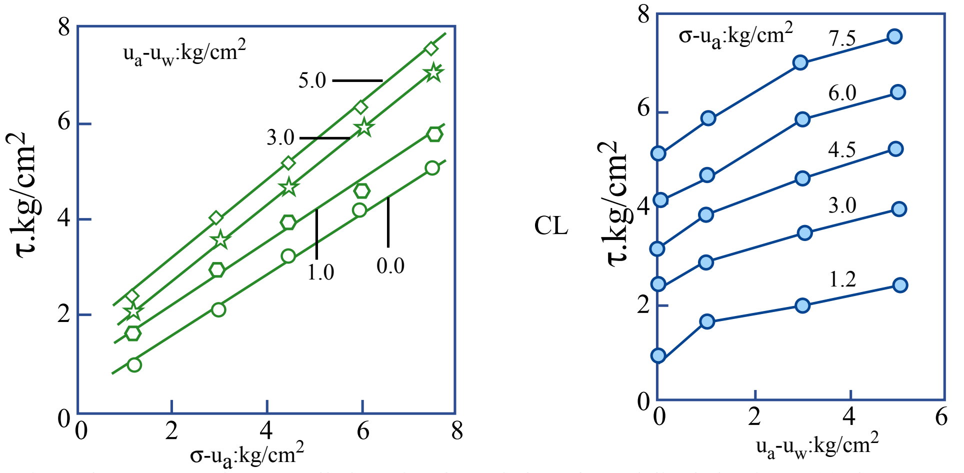
Table 1. Soil characteristics, initial conditions, consolidation time and rate of shear of the samples tested

	Madrid grey clay 'Peñuela'	Red clay of Guadalix de la Sierra	Madrid clayey sand 'Arena de miga'
Atterberg limits			
w_L	71	33	32
PI	35	13·6	15
Sieve analysis: % passing	MH	CL	SC
10			100
16	—	100	94
40	100	97	48
200	99	86·5	17
Standard Proctor test			
γ_{max}: g/cm^3	1·33	1·80	1·91
w_{opt}: %	33·7	17·0	11·5
Initial conditions			
γ: g/cm^3	1·33	1·80	1·91
w: %	29	13·6	9·2
Suction: kg/cm^2	8·5	2·8	0·7
Time of consolidation under surcharge and suction applied: days	4	4	4
Rate of shear: mm/day	2·4	2·4	2·4
Time to failure: days	2·5–3·0	2–3	1–2

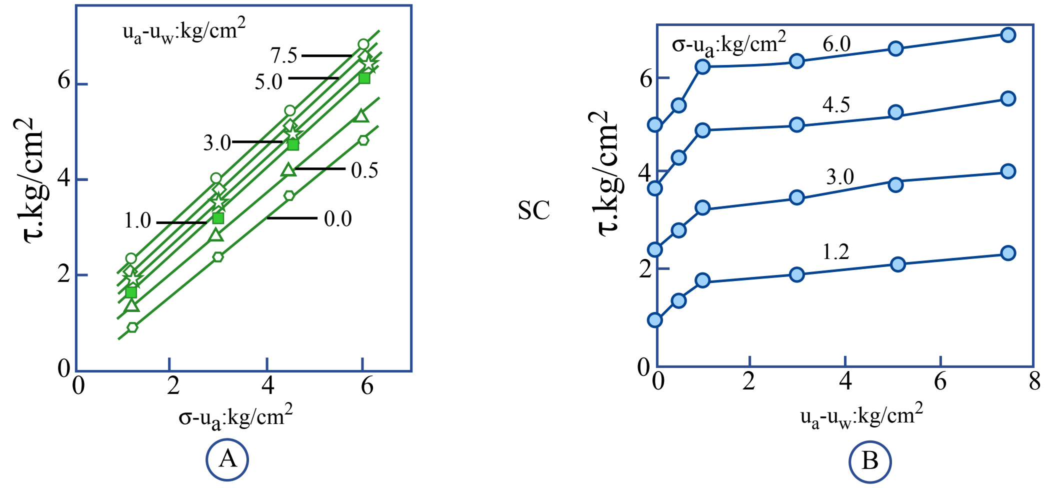
Escario & Saez (1986) Geot. 36(3)



Direct shear tests under controlled suction for Madrid grey clay 'Peñuela': (a) shear strength versus normal stress for different values of the suction; (b) shear strength versus suction for different values of the normal stress.



Direct shear tests under controlled suction for red clay of Guadalix de la Sierra: (a) shear stress versus normal stress for different values of the suction; (b) shear stress versus suction for different values of the normal stress.



Direct shear tests under controlled suction with Madrid clayey sand ('Arena de miga'): (a) shear stress versus normal stress for different values of the suction; (b) shear stress versus suction for different values of the normal stress

Figure by MIT OCW.

CD Direct Shear on 3 Compacted Soils

Adapted from: Escario & Saez (1986)