Class

Principles of Foundation Engineering CEE430/530

General Information

Lecturer: Scott A. Barnhill, P.E.

Lecture Time: Thursday, 7:10 pm to 9:50 pm

Classroom: Kaufmann, Room 224

Office Hour: I have no office. Contact me to meet before class.

Email: sabarnhill@geronline.com (ODU email not working)

Cell Phone: 621-6783

Text: Principles of Foundation Engineering, 7th Edition, Braja M. Das

NOT the SI Version

Class Web Site

http://www.geronline.com/odu.php

- Class Dates
- Information to Be Covered
- Homework Answers
- Any Handouts
- Other Reading Sources

Homework

Homework is to have an answer sheet as the first page.

- Problem Number
- Answers with any sub problems answers also shown.

Show data given at top of each problem.
Show every calculation step.
No Excel or MathCad – Period!!!.

Grading

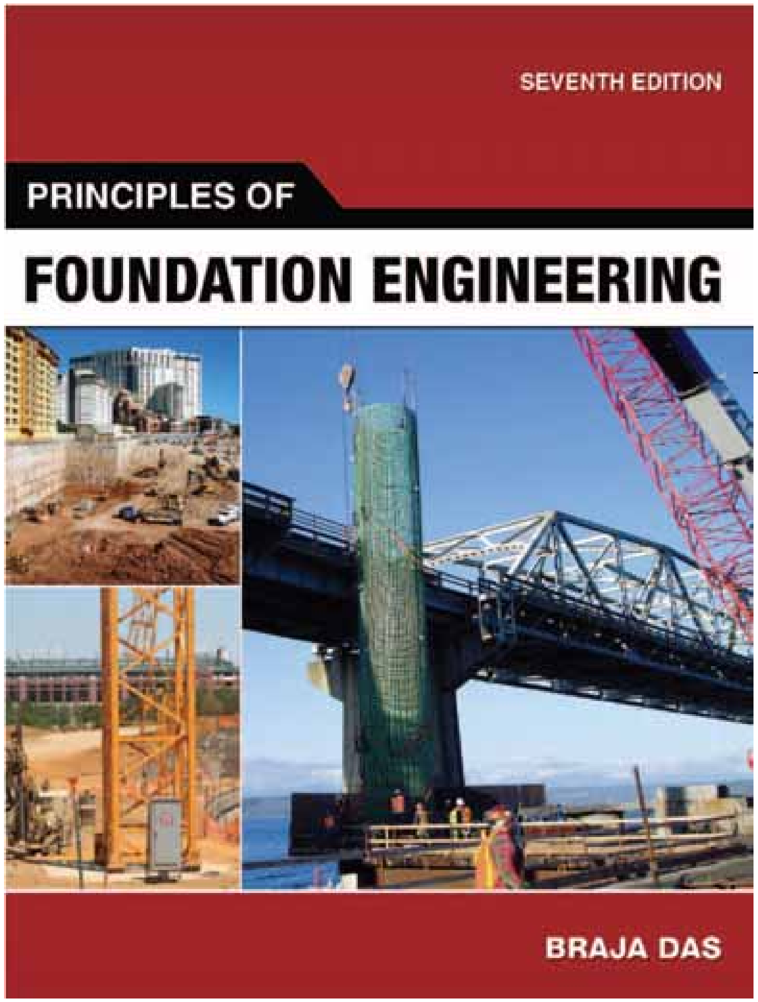
- Homework - 20%
- Mid Term Exam - 30%
- Final Exam - 30%
- Project - 20%

  Class Presentation – 10%
  Report – 10%
  Your Personal Calculations – 20%
  Peer Review – 50%
  My Opinion – 10%

Range		Letter Grade
0	54	F
55	57	D-
58	61	D
62	68	D+
69	71	C-
72	75	C
76	79	C+
80	83	B-
84	86	B
87	89	B+
90	94	A-
95	100	A

Purpose of the Class

- Familiarize you with soil properties
- Learn how subsurface soils are tested
- Learn how to apply soil properties to foundation design
- Learn about analyzing various foundation alternatives
- Learn about retaining walls
- Learn about how to improve the ground
- Apply what you have learned to actual projects



Chapter 1:

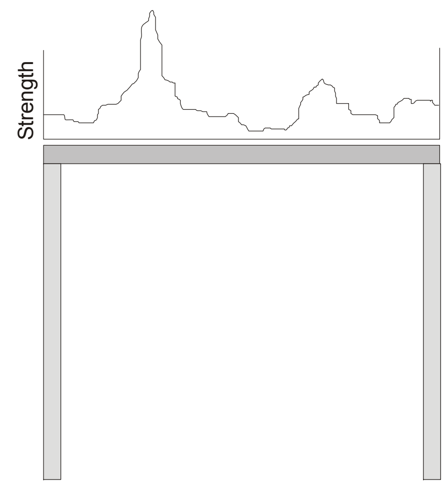
Geotechnical Properties of Soil

Your Knowledge

You are already suppose to know everything covered
in this chapter.

Strength

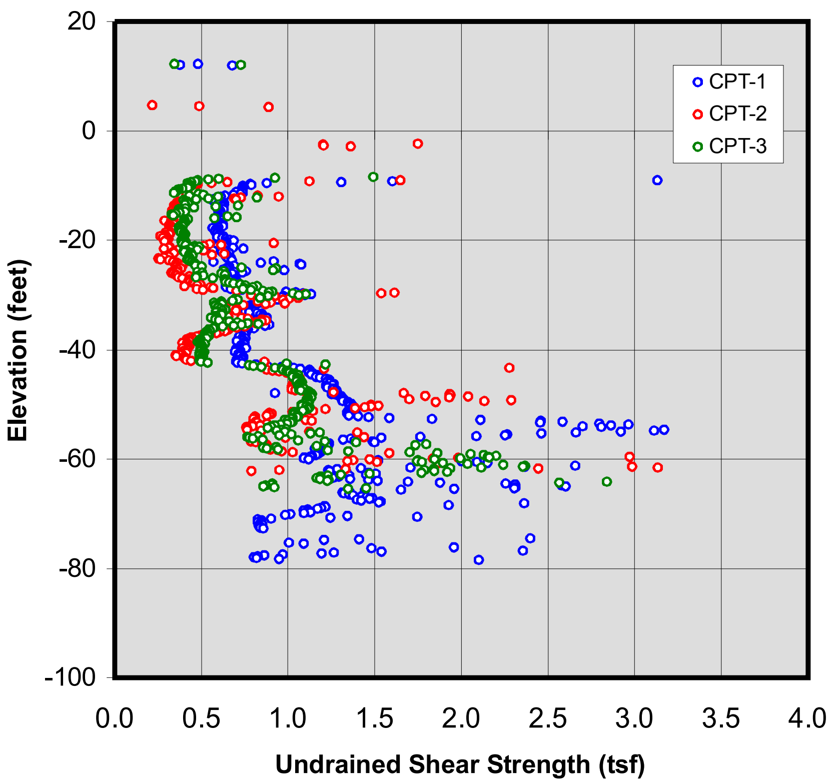
- What is the strength of steel?
- What is the strength of concrete?
- What if their strengths varied wildly even with a single column or beam.
- How would you design?

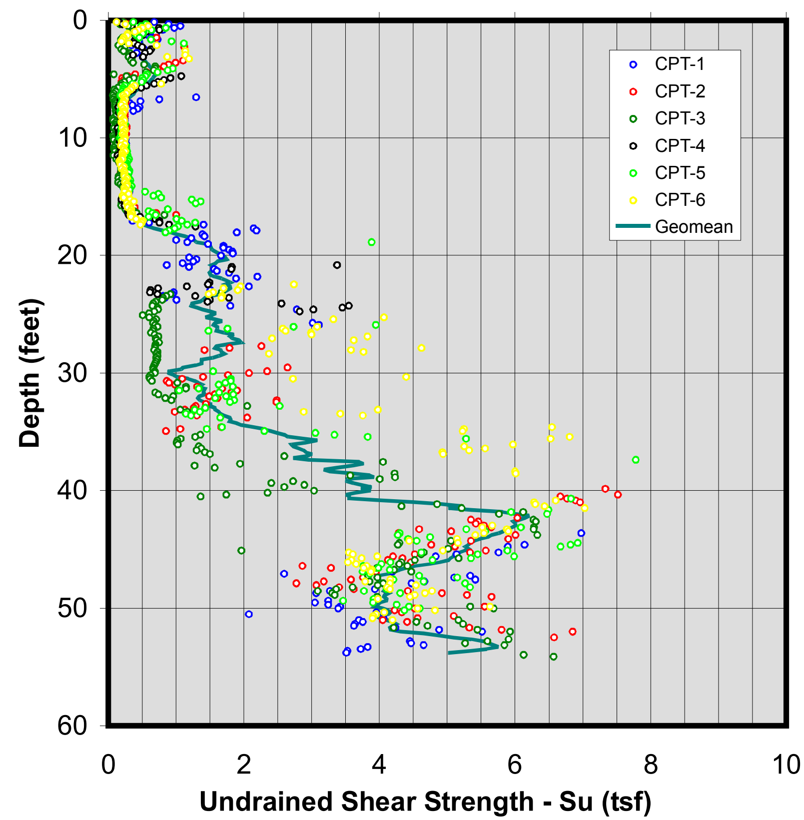


Soil

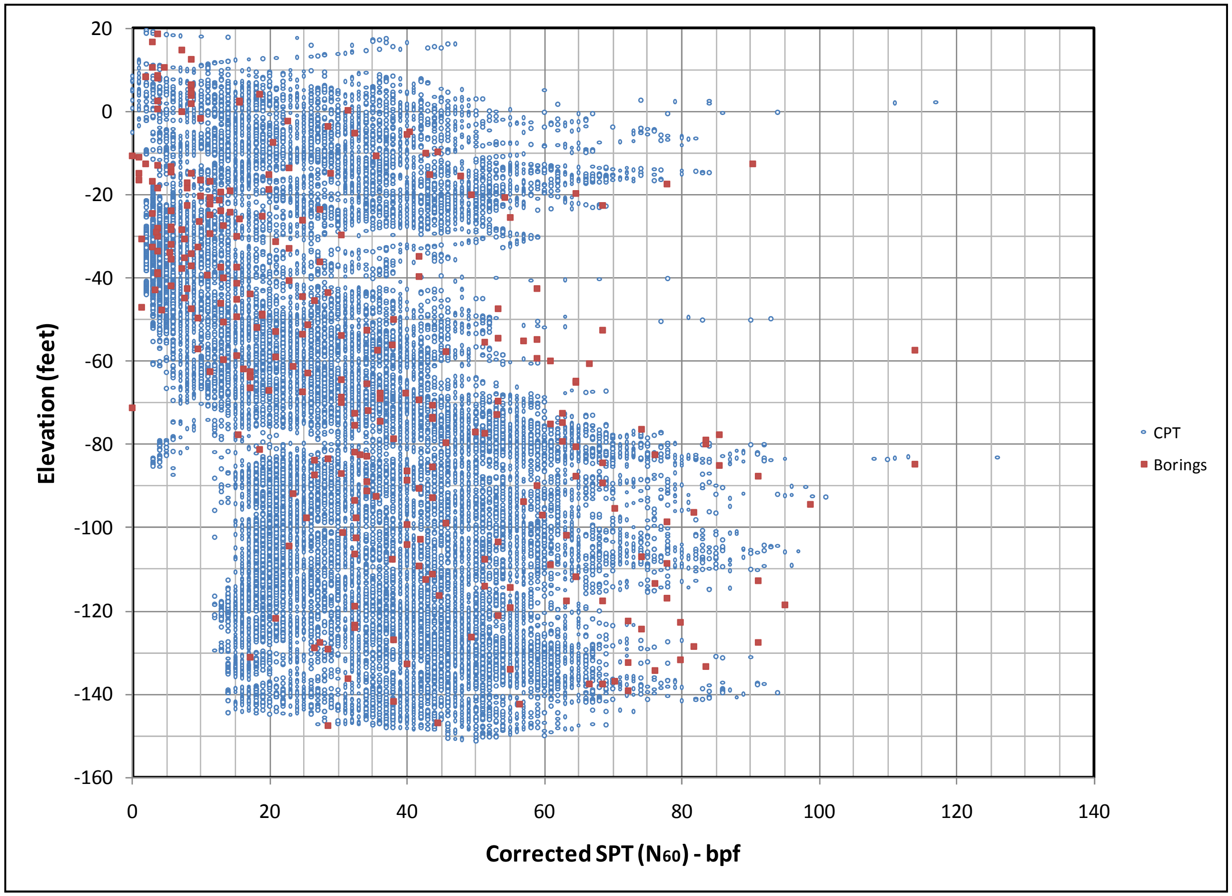
- What is the strength of soil?

Strength of Soil





Lesner Bridge



Classification Schemes

Table 1.2 Soil-Separate Size Limits

Classification system	Grain size (mm)
Unified	Gravel: 75 mm to 4.75 mm Sand: 4.75 mm to 0.075 mm Silt and clay (fines): <0.075 mm
AASHTO	Gravel: 75 mm to 2 mm Sand: 2 mm to 0.05 mm Silt: 0.05 mm to 0.002 mm Clay: <0.002 mm

Weight-Volume

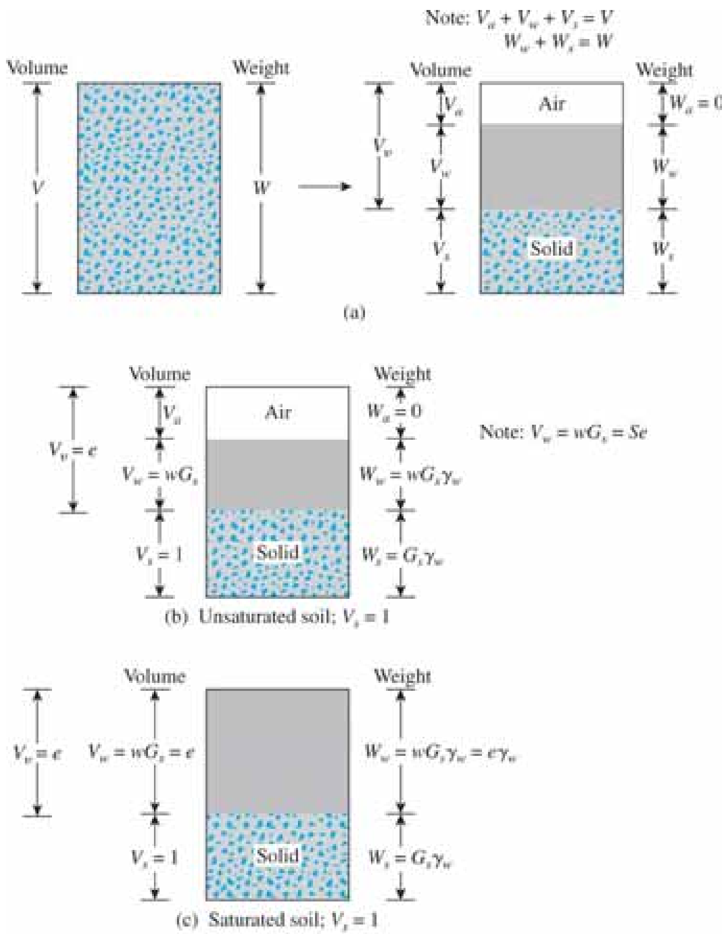


Figure 1.3 Weight–volume relationships

Unit Weight Relationships

Table 1.3 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Unit-weight relationship	Dry unit weight	Saturated unit weight
$\gamma = \dfrac{(1+w)G_s\gamma_w}{1+e}$	$\gamma_d = \dfrac{\gamma}{1+w}$	$\gamma_{sat} = \dfrac{(G_s+e)\gamma_w}{1+e}$
$\gamma = \dfrac{(G_s+Se)\gamma_w}{1+e}$	$\gamma_d = \dfrac{G_s\gamma_w}{1+e}$	$\gamma_{sat} = [(1-n)G_s+n]\gamma_w$
$\gamma = \dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	$\gamma_d = G_s\gamma_w(1-n)$	$\gamma_{sat} = \left(\dfrac{1+w}{1+wG_s}\right)G_s\gamma_w$
$\gamma = G_s\gamma_w(1-n)(1+w)$	$\gamma_d = \dfrac{G_s}{1+\dfrac{wG_s}{S}}\gamma_w$	$\gamma_{sat} = \left(\dfrac{e}{w}\right)\left(\dfrac{1+w}{1+e}\right)\gamma_w$
	$\gamma_d = \dfrac{eS\gamma_w}{(1+e)w}$	$\gamma_{sat} = \gamma_d + n\gamma_w$
	$\gamma_d = \gamma_{sat} - n\gamma_w$	$\gamma_{sat} = \gamma_d + \left(\dfrac{e}{1+e}\right)\gamma_w$
	$\gamma_d = \gamma_{sat} - \left(\dfrac{e}{1+e}\right)\gamma_w$	

Common Relationships

$$W = W_s(1 + w)$$

$$V = V_s(1 + e)$$

$$\gamma_d = \frac{\gamma_m}{1 + w}$$

$$\gamma' = \gamma_{sat} - \gamma_w$$

$$\gamma_d = \frac{G_s \cdot \gamma_w}{1 + e}$$

$$\gamma_{sat} = \gamma_d (1 + w_{sat})$$

$$e = \frac{n}{1 - n}$$

$$n = \frac{e}{1 + e}$$

$$e = \frac{w \cdot G_s}{S}$$

$$S = \frac{w}{w_{sat}}$$

$$w_{sat} = \frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}$$

W – Total Weight
V – Total Volume
w – water content
Ws – Weight of Solids
Vs – Volume of Solids
e – void ratio
γd – Dry Unit Weight
γm – Moist Unit Weight
γw – Unit Weight of Water
γ' – Bouyant Unit Weight
Gs – Specific Gravity
n – porosity
S – Saturation
w_{sat} – Saturated Moisture Content

Specific Gravity

Table 1.4 Specific Gravities of Some Soils

Type of soil	G_s
Quartz sand	2.64–2.66
Silt	2.67–2.73
Clay	2.70–2.9
Chalk	2.60–2.75
Loess	2.65–2.73
Peat	1.30–1.9

Atterberg Limits

Liquid Limit (LL), Plastic Limit (PL) & Plasticity Index (PI) =LL-PL

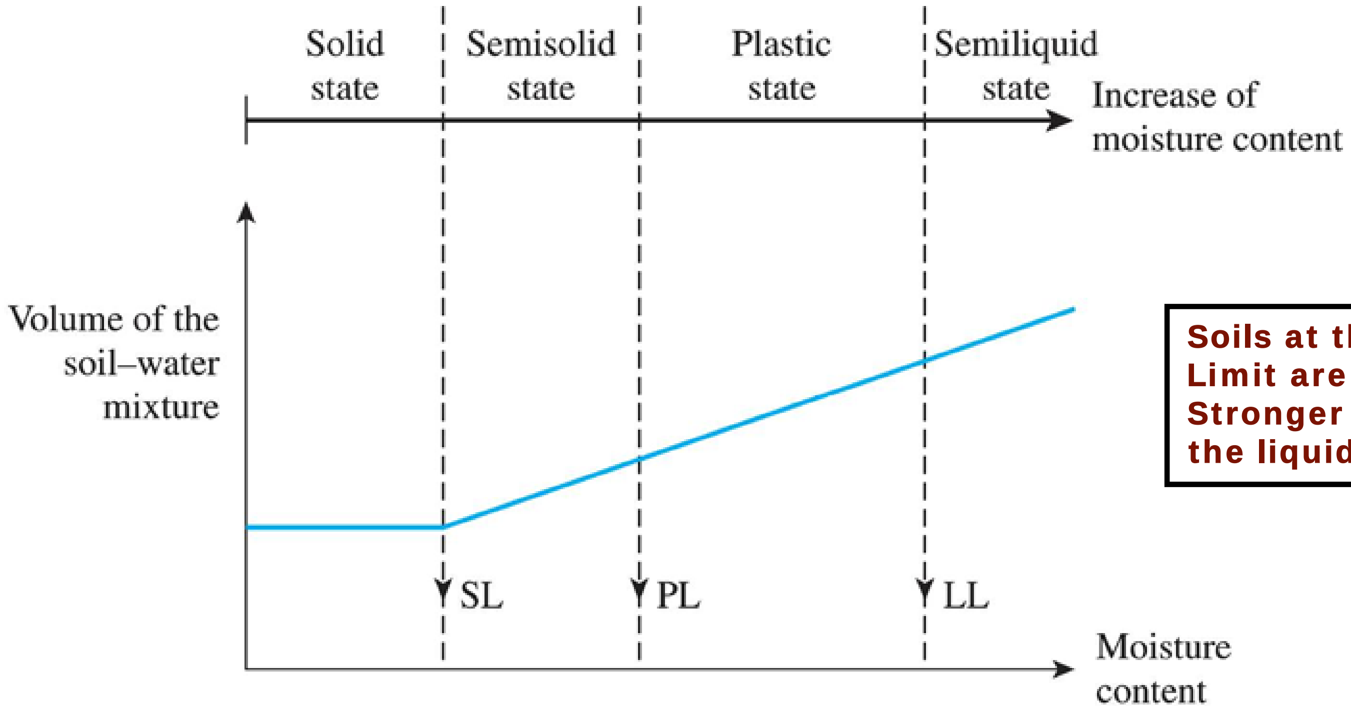


Figure 1.4 Definition of Atterberg limits

Soils at the plastic Limit are 100 times Stronger than at the liquid limit.

Atterberg Limit Relationships

- Soils with moisture content near or at the liquid limit are usually normally consolidated.
- As the moisture content moves towards the plastic limit, preconsolidation increases.
- Soils with moisture contents exceeding the liquid limit, the soils can be underconsolidated. Must know site history.
- Cohesive strength increases as moisture content moves towards the plastic limit.

Atterberg Limits Chart

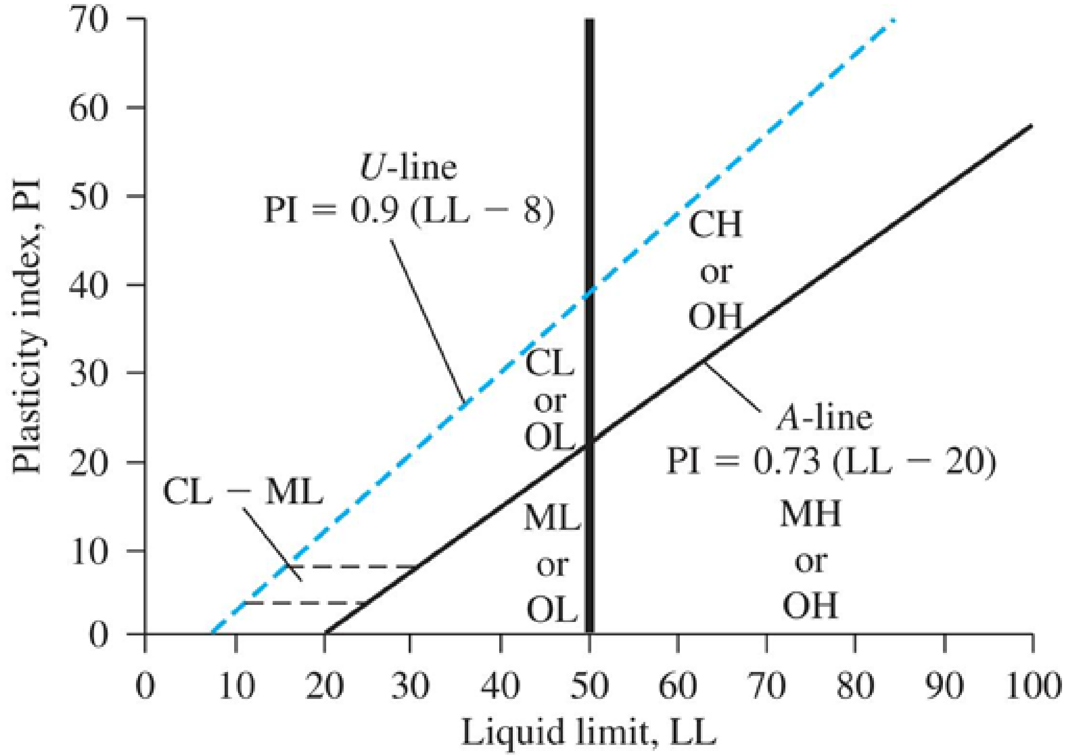


Figure 1.5 Plasticity chart

Empirical Correlations

Atterberg Limits are used in numerous empirical correlations

- Preconsolidation
- Undrained Strength
- Constrained Modulus
- Permeability
- Moist Unit Weight
- Dry Unit Weight
- Submerged Unit Weight
- ϕ'
- C_c & C_r
- Swell Pressure
- C_v
- Void ratio
- Critical State Soil Mechanics Parameters – Γ, λ, N, Λo

Use correlations to compare to more sophisticated tests. Look for how consistent the soil.

Unified Soils Classification

Table 1.8 Unified Soil Classification Chart (after ASTM, 2009) (ASTM D2487-98: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification). Copyright ASTM INTERNATIONAL. Reprinted with permission.)

Criteria for assigning group symbols and group names using laboratory tests[a]				Soil classification: Group symbol	Soil classification: Group name[b]
Coarse-grained soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[c]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[e]	GW	Well-graded gravel[f]
			$C_u < 4$ and/or $1 > C_c > 3$[e]	GP	Poorly graded gravel[f]
		Gravels with Fines More than 12% fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[d]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[e]	SW	Well-graded sand[i]
			$C_u < 6$ and/or $1 > C_c > 3$[e]	SP	Poorly graded sand[i]
		Sand with Fines More than 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
			Fines classify as CL or CH	SC	Clayey sand[g,h,i]
Fine-grained soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line[j]	CL	Lean clay[k,l,m]
			PI < 4 or plots below "A" line[j]	ML	Silt[k,l,m]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OL	Organic clay[k,l,m,n]; Organic silt[k,l,m,o]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[k,l,m]
			PI plots below "A" line	MH	Elastic silt[k,l,m]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OH	Organic clay[k,l,m,p]; Organic silt[k,l,m,q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[a] Based on the material passing the 75-mm. (3-in) sieve.
[b] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[c] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt; GW-GC well-graded gravel with clay; GP-GM poorly graded gravel with silt; GP-GC poorly graded gravel with clay.
[d] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt; SW-SC well-graded sand with clay; SP-SM poorly graded sand with silt; SP-SC poorly graded sand with clay.
[e] $C_u = D_{60}/D_{10} \quad C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$
[f] If soil contains ≥15% sand, add "with sand" to group name.
[g] If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
[h] If fines are organic, add "with organic fines" to group name.
[i] If soil contains ≥15% gravel, add "with gravel" to group name.
[j] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[k] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[l] If soil contains ≥30% plus No. 200, predominantly sand, add "sandy" to group name.
[m] If soil contains ≥30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[n] PI ≥ 4 and plots on or above "A" line.
[o] PI < 4 or plots below "A" line.
[p] PI plots on or above "A" line.
[q] PI plots below "A" line.

Coarse Grained Soils

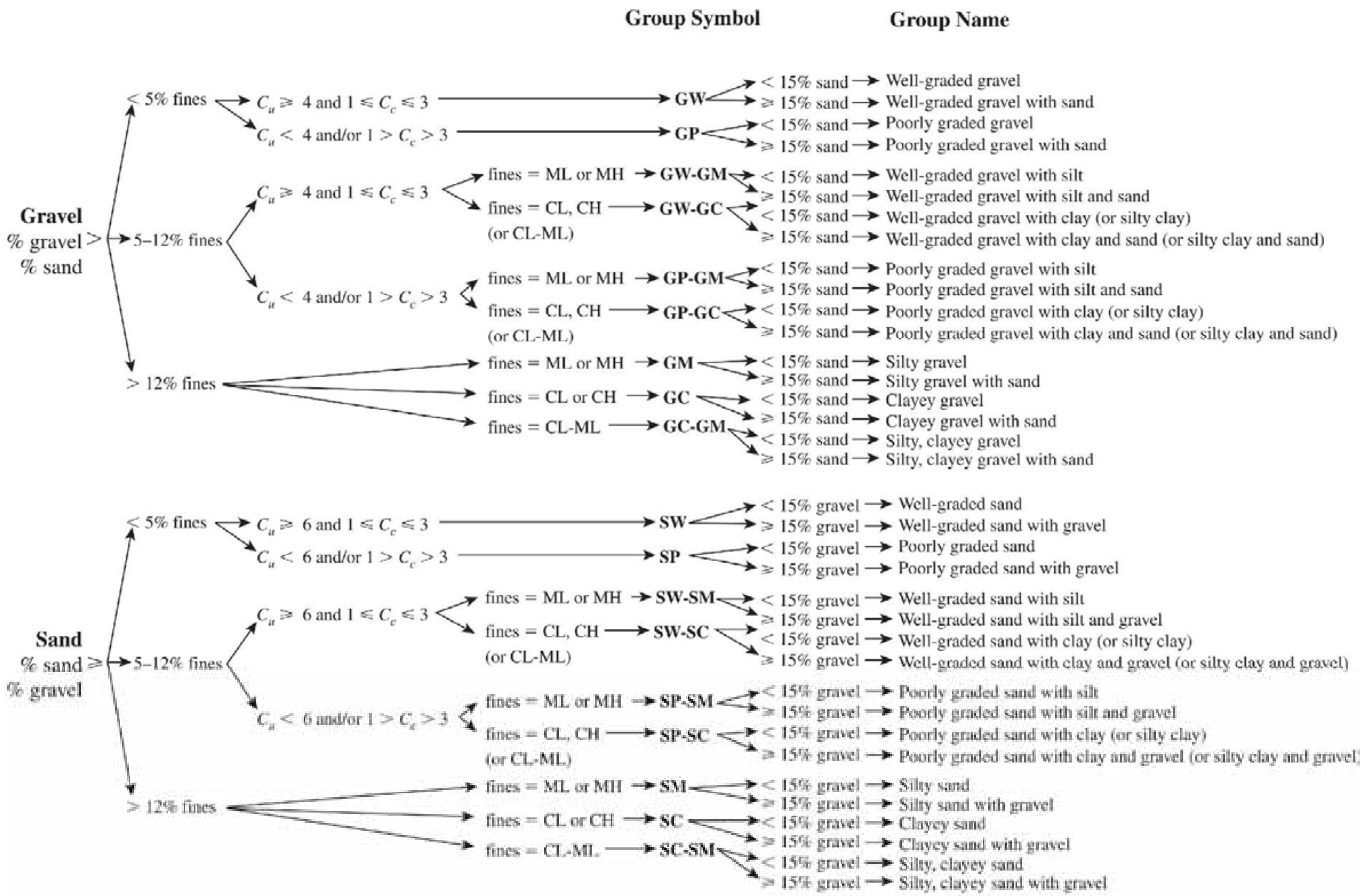


Figure 1.6 Flowchart for classifying coarse-grained soils (more than 50% retained on No. 200 Sieve) (After ASTM, 2009) (ASTM D2487-98: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification). Copyright ASTM INTERNATIONAL. Reprinted with permission.)

Fine Grained Soils

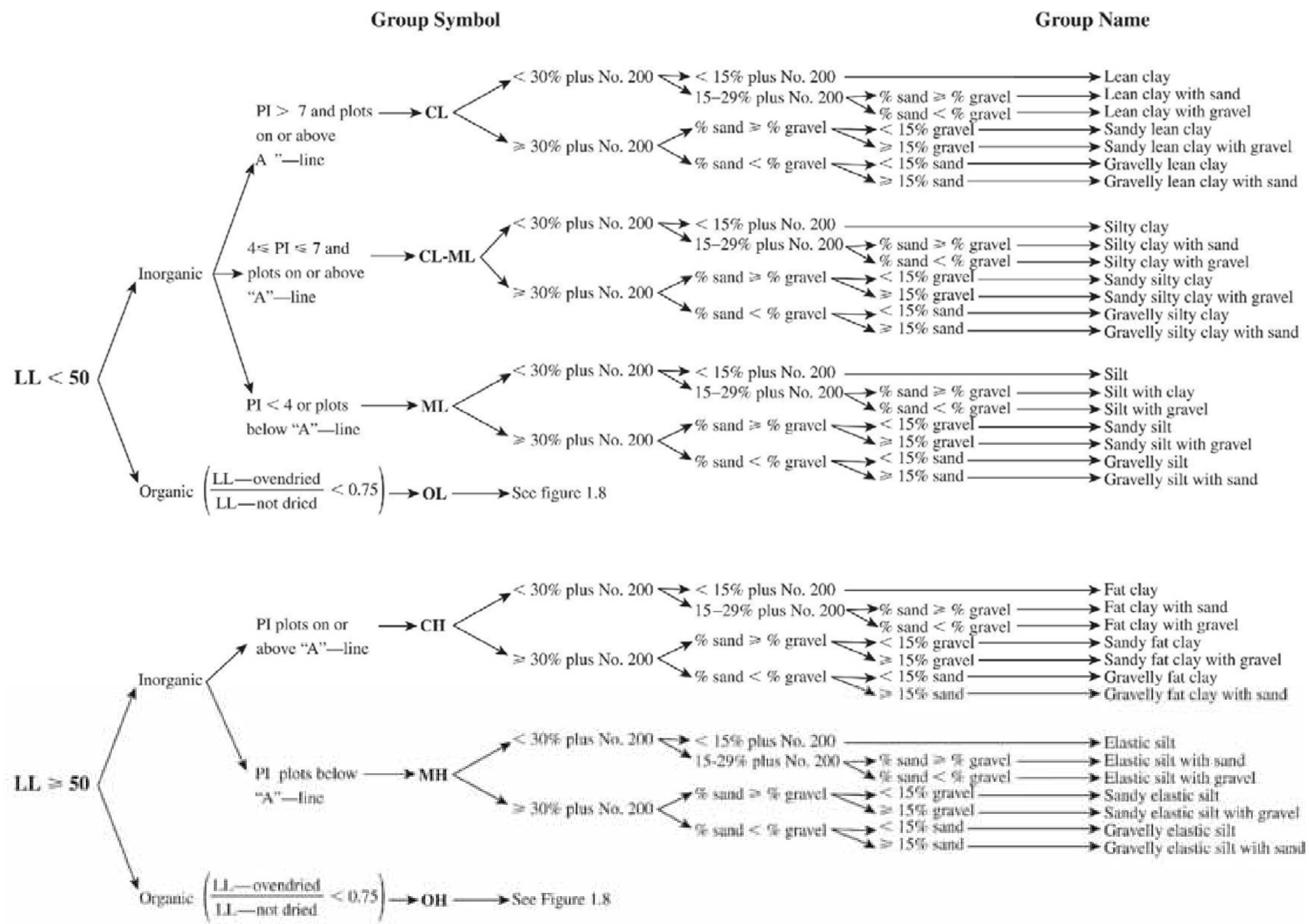


Figure 1.7 Flowchart for classifying fine-grained soil (50% or more passes No. 200 Sieve) (After ASTM, 2009)(ASTM D2487-98: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification). Copyright ASTM INTERNATIONAL. Reprinted with permission.)

Organic Soils

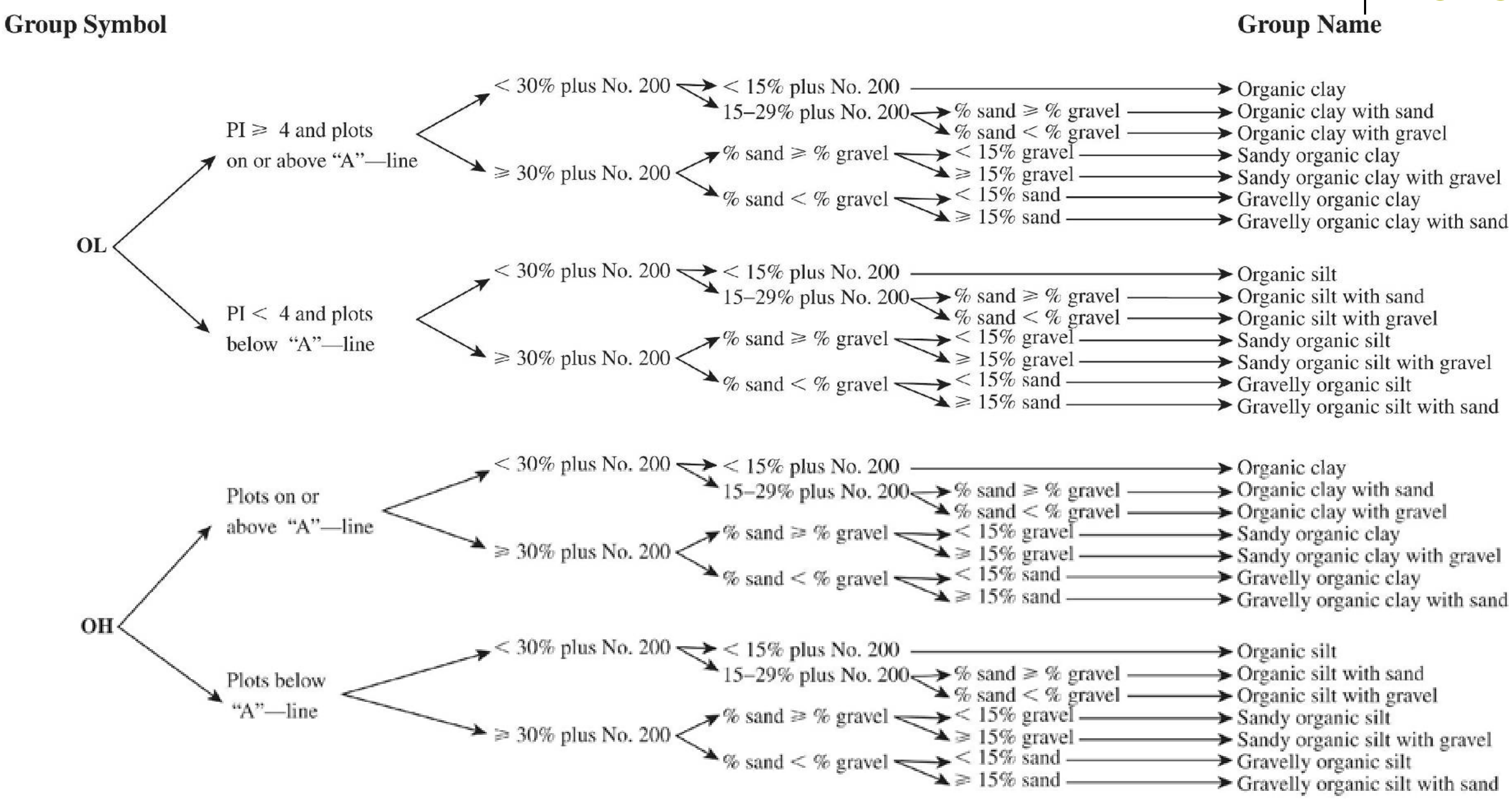


Figure 1.8 Flowchart for classifying organic fine-grained soil (50% or more passes No. 200 Sieve) (After ASTM, 2009) (ASTM D2487-98: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification). Copyright ASTM INTERNATIONAL. Reprinted with permission.)

Effective Stress

If you cannot master the concept of effective stress and cannot calculate it accurately, you will not get a good grade in this class. All foundation design requires it.

Calculation of Effective Stress

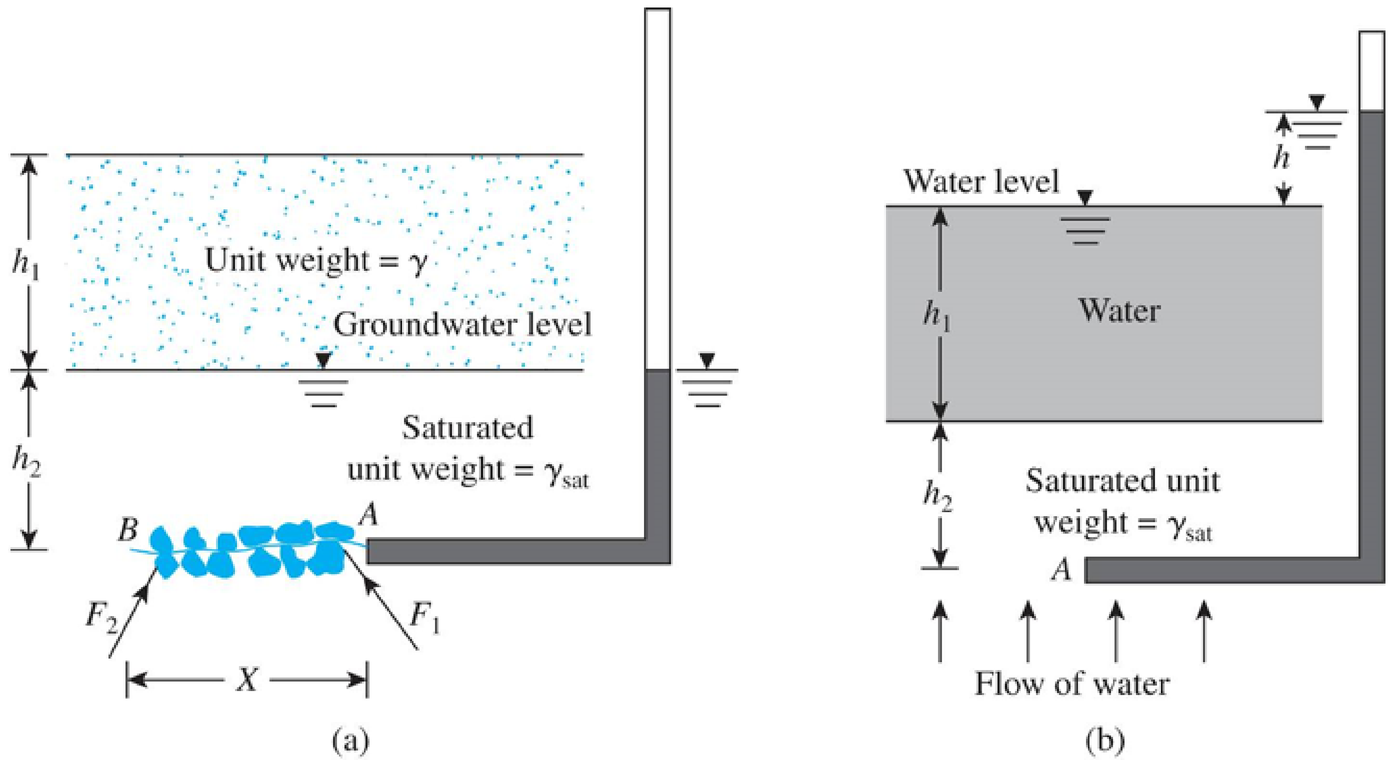


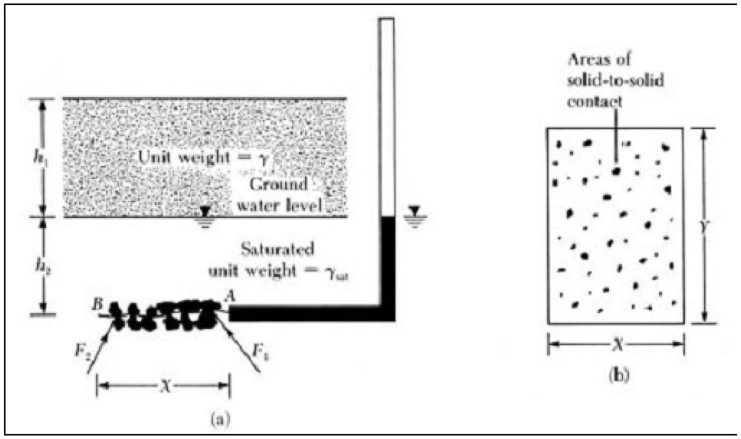
Figure 1.13 Calculation of effective stress

You must understand the concept of effective stress. It is so fundamental to foundation engineering that you will simply not be able to complete almost every design problem.

Effective Stress – No Seepage

In Figure, pore pressure at Point A is $u = h_2 \bullet \gamma_w$

Where γ_w is the unit weight of water (62.4 pcf)



$$\sigma' = \sigma - u = (h_1 \bullet \gamma_m + h_2 \bullet \gamma_{sat}) - h_2 \bullet \gamma_w$$
$$= h_1 \bullet \gamma_m + h_2(\gamma_{sat} - \gamma_w) = h_1 \bullet \gamma_m + h_2 \bullet \gamma'$$

Where γ' = effective, or submerged, unit weight of soil

$$\gamma_{sat} = \frac{G_s \cdot \gamma_w + e \cdot \gamma_w}{1 + e}$$

$$\gamma' = \gamma_{sat} - \gamma_w = \frac{G_s \cdot \gamma_w + e \cdot \gamma_w}{1 + e} - \gamma_w = \frac{\gamma_w (G_s - 1)}{1 + e}$$

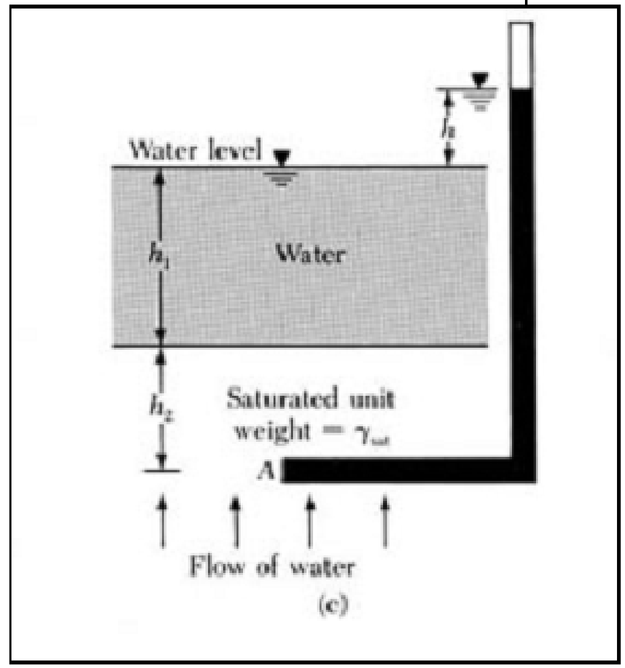
Effective Stress – Seepage

In this problem, there is upward seepage of water.
For this case, the effective stress at Point A is

$\sigma = h_1 \bullet \gamma_w + h_2 \bullet \gamma_{sat}$
$u = (h1 + h2 + h) \bullet \gamma_w$

$\sigma' = \sigma - u = (h_1 \bullet \gamma_w + h_2 \bullet \gamma_{sat}) - (h_1 + h_2 + h) \bullet \gamma_w$
$\quad = h_2(\gamma_{sat} - \gamma_w) - h \bullet \gamma_w = h_2 \bullet \gamma' - h \bullet \gamma_w$
or

$$\sigma' = h_2\left(\gamma' - \frac{h}{h_2} \cdot \gamma_w\right) = h_2(\gamma' - i \cdot \gamma_w)$$

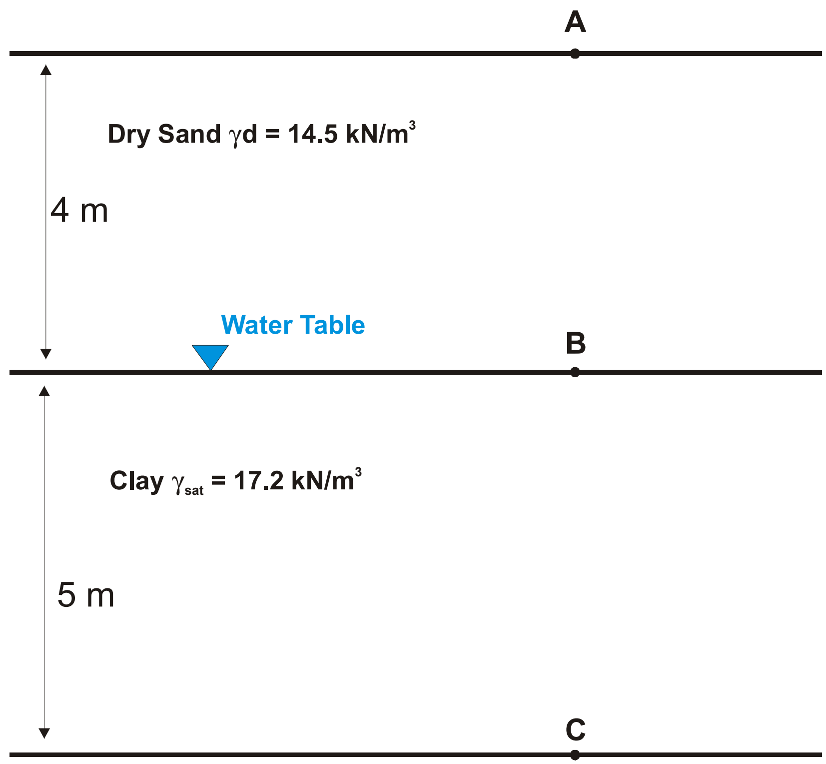


"i" is the hydraulic gradient. If "i" is very high so that $\gamma' - i \bullet \gamma_w = 0$, the effective stress = 0. There Will be no contact between soil particles. This is referred to as the quick condition (quick sand), or failure by heave.

Critical Gradient

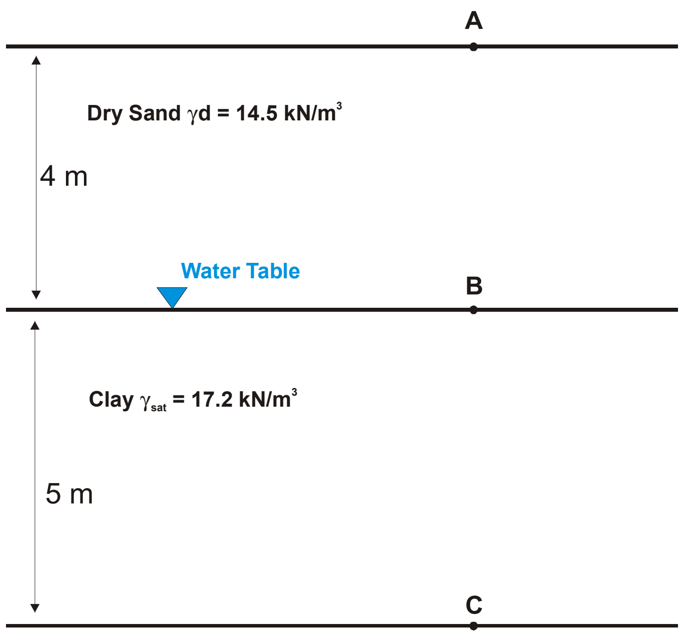
$i = i_{cr} = \gamma' / \gamma_w$
$\quad = (G_s - 1) / (1 + e_o)$

Variation of effective stress in a soil profile

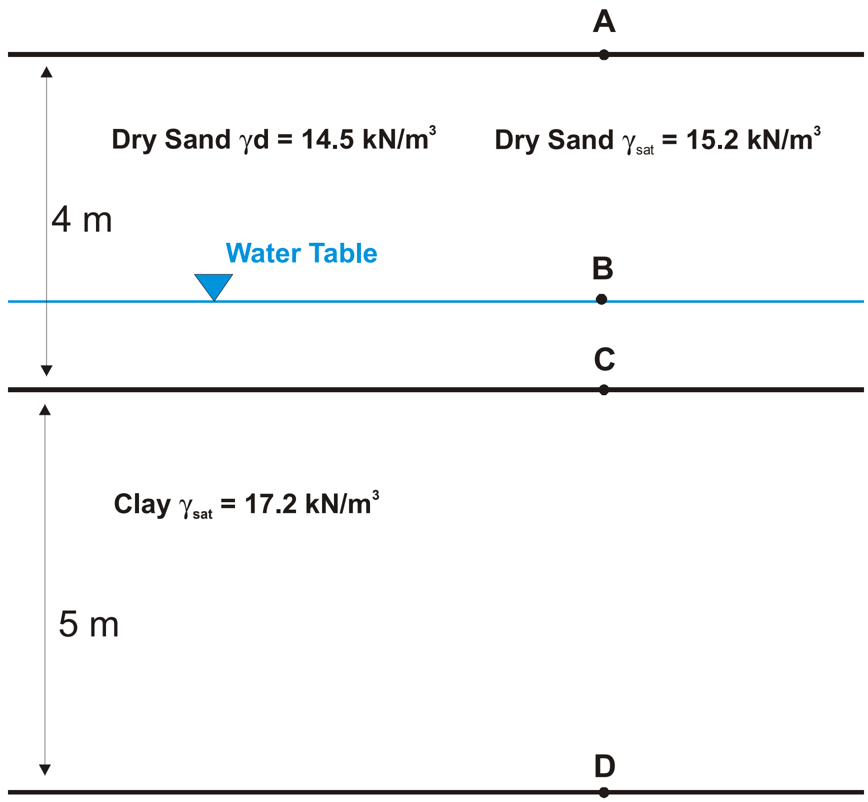


Effective Stress Solution

Point	Depth (m)	σ (kN/m3)	u (kN/m2)	σ' (kN/m2)
A	0	**0**	**0**	0
B	4	(4)(γd) = (4)(14.5) = 58	**0**	58 - 0 = **58**
C	9	58 + (5)(γsat) = (5)(17.2) = 144	(5)(γw) = (5)(9.81) = 49.05	144 - 49.5 = **94.95**

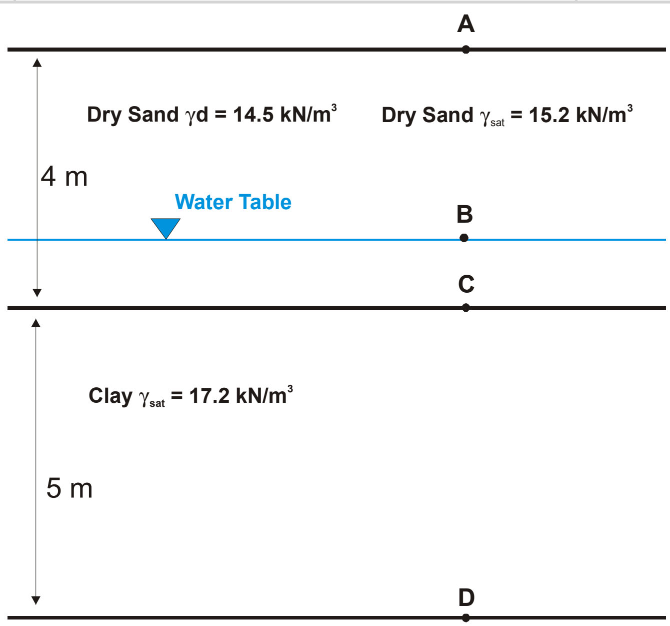


Effective Stress #2



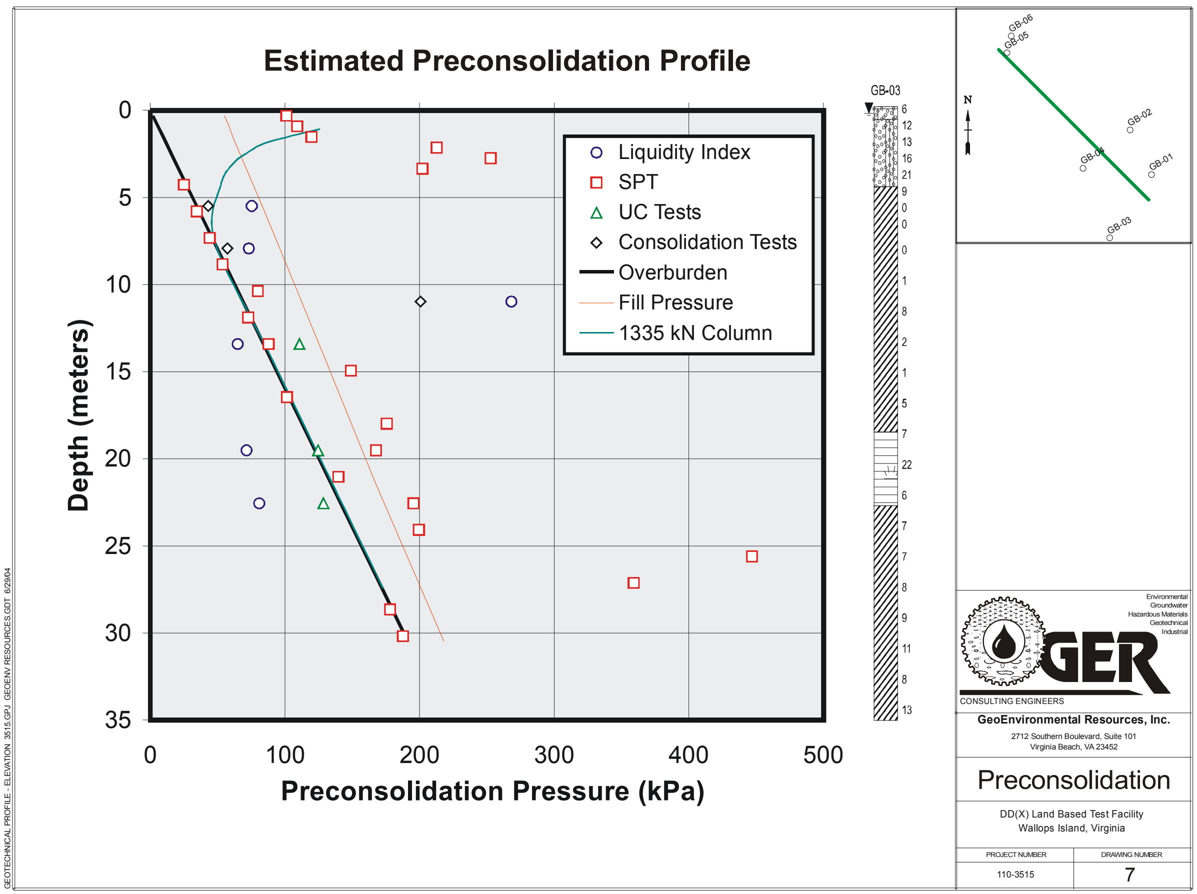
Effective Stress Solution #2

A	0	**0**	**0**	0
B	3	(3)(γd) = (3)(14.5) = **43.5**	**0**	43.5 - 0 = **43.5**
C	4	43.5 + (1)(γsat1) = 43.5 + (1)(15.2) = **58.7**	(1)(γw) = (1)(9.81) = **9.81**	58.7 - 9.81 = **48.9**
D	9	58.7 + (5)(γsat2) = 58.7 + (5)(17.2) = **144.7**	9.81+(5)(γw) = 9.81+(5)(9.81) = **58.86**	144.7 - 58.6 = **86.1**

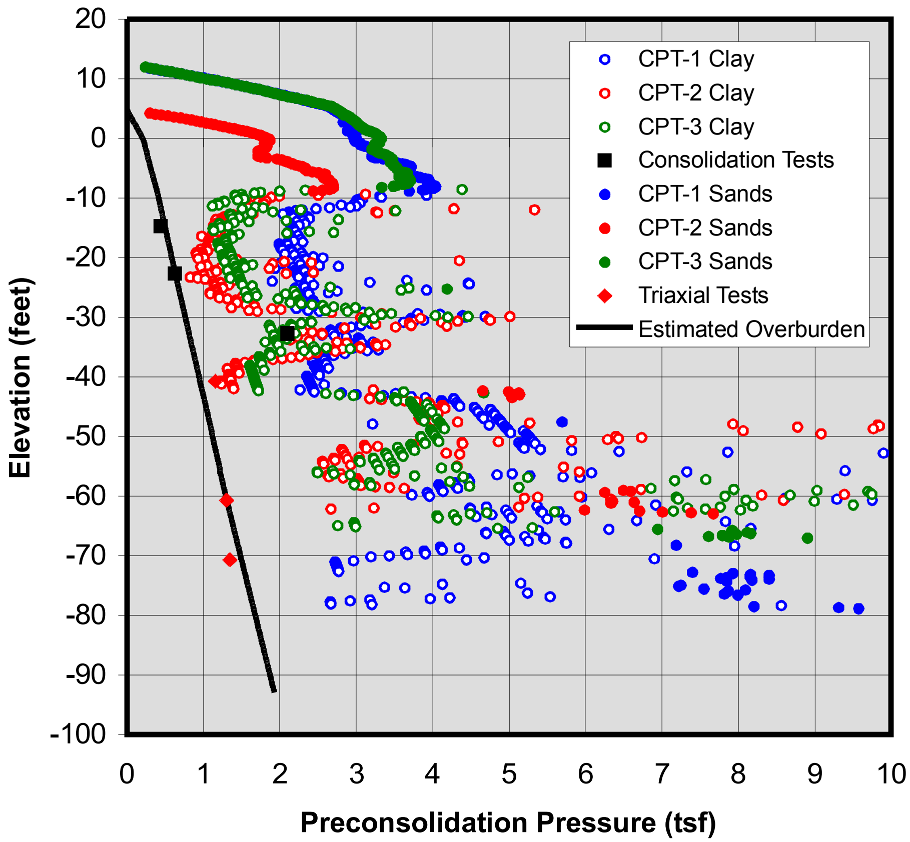


What happens of the groundwater changes in the future?

Preconsolidation Profile



Preconsolidation Profile - CPT



Consolidation

Consolidation is the movement of pore water out of the soil.

Initially the applied load is carried by the pore water. This creates an increase in pore water pressure.

As pore water moves out of the soil which is controlled by the permeability, pore pressure dissipates and the soil matrix begins to carry the load. The soil then compresses.

Principles of Consolidation

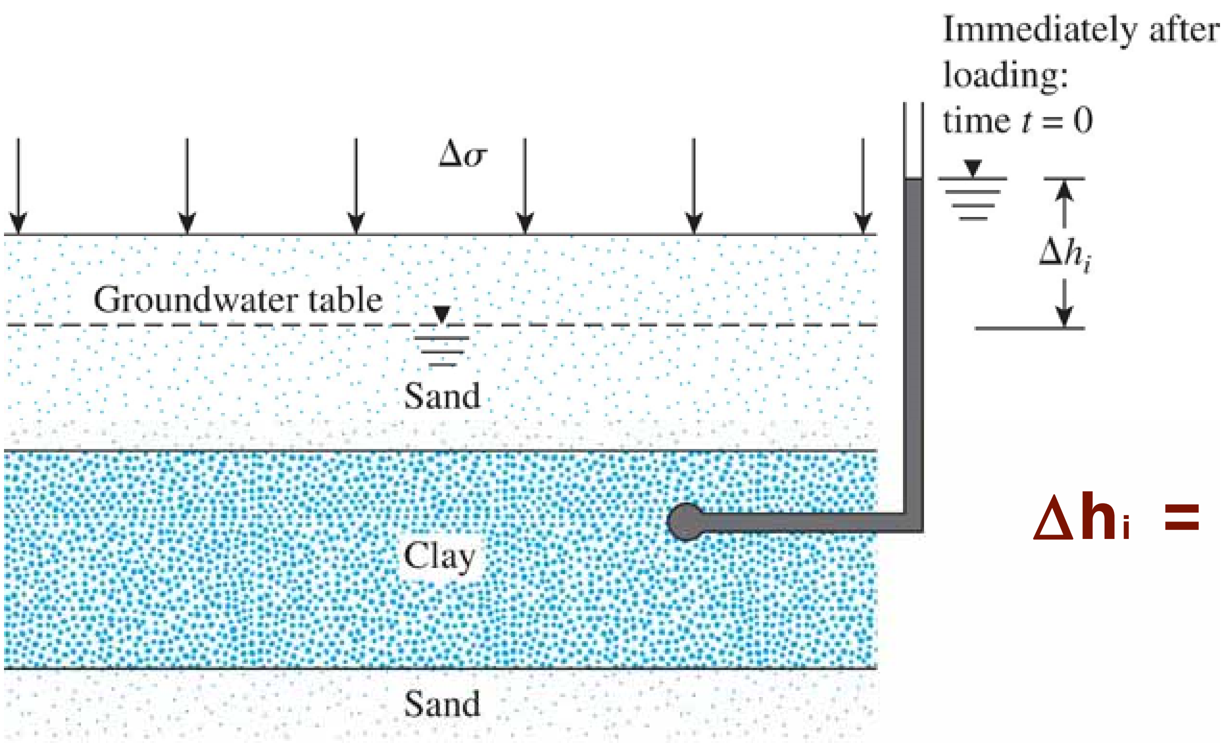


Figure 1.14 Principles of consolidation

$\Delta h_i = \Delta\sigma$

Consolidation Testing

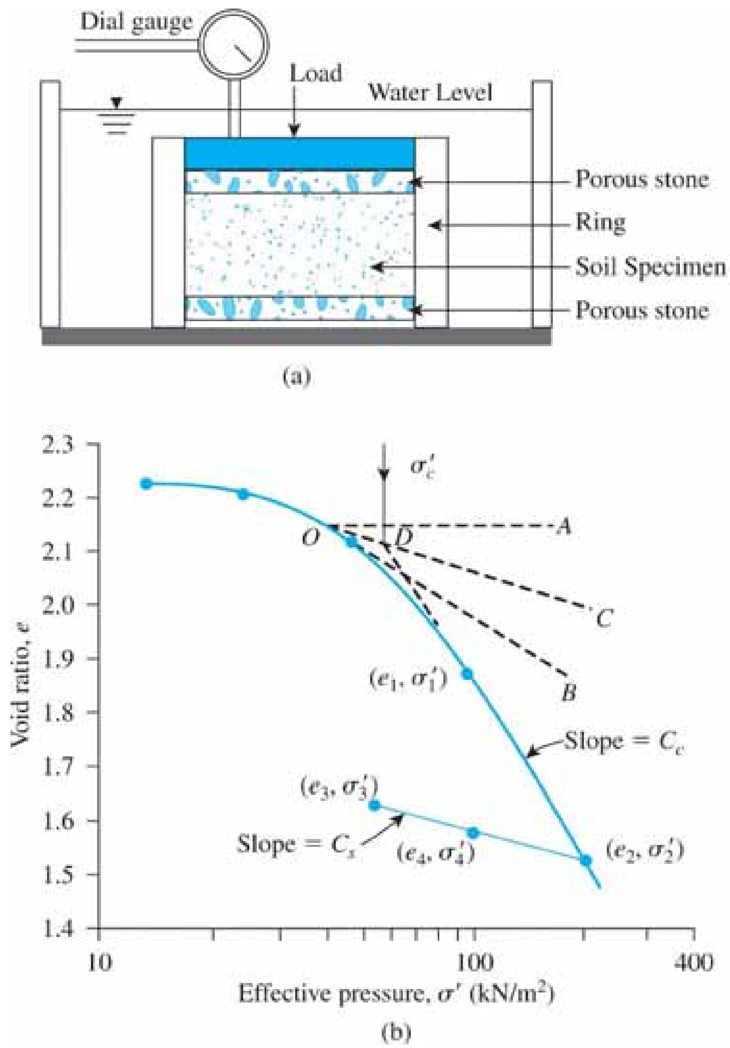


Figure 1.15 (a) Schematic diagram of consolidation test arrangement; (b) e–log σ' curve for a soft clay from East St. Louis, Illinois (*Note:* At the end of consolidation, $\sigma = \sigma'$)

Normally Consolidated Clay

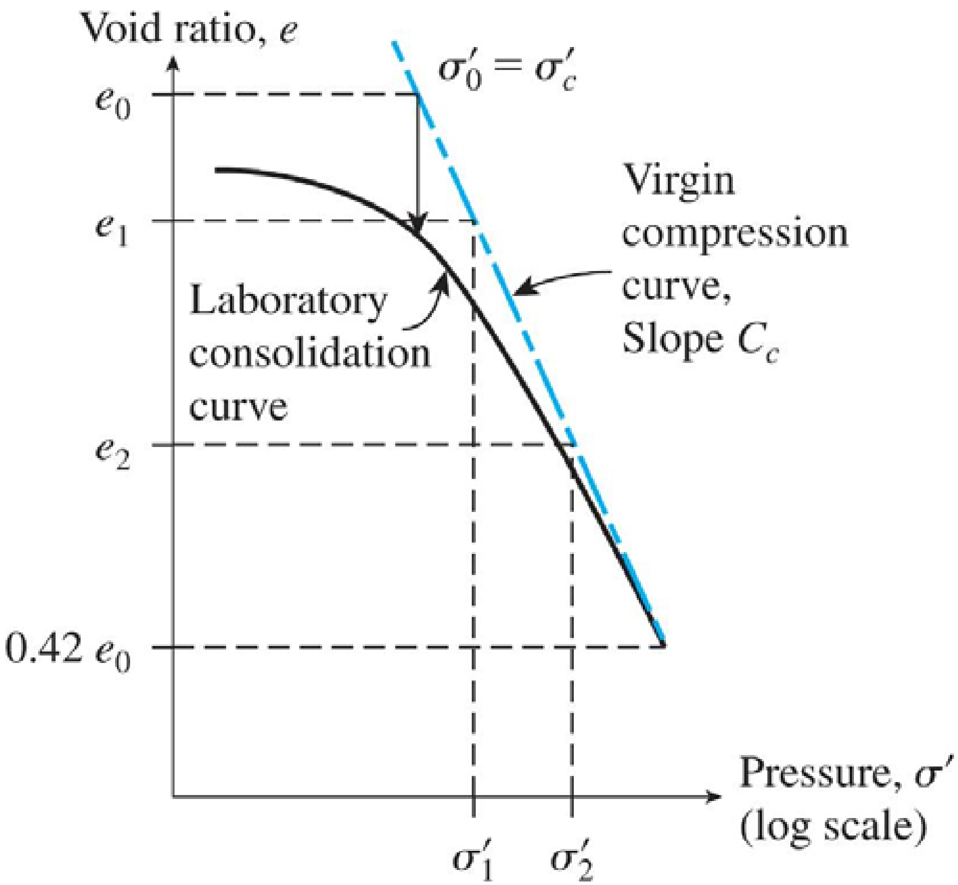


Figure 1.16 Construction of virgin compression curve for normally consolidated clay

Overconsolidated Clay

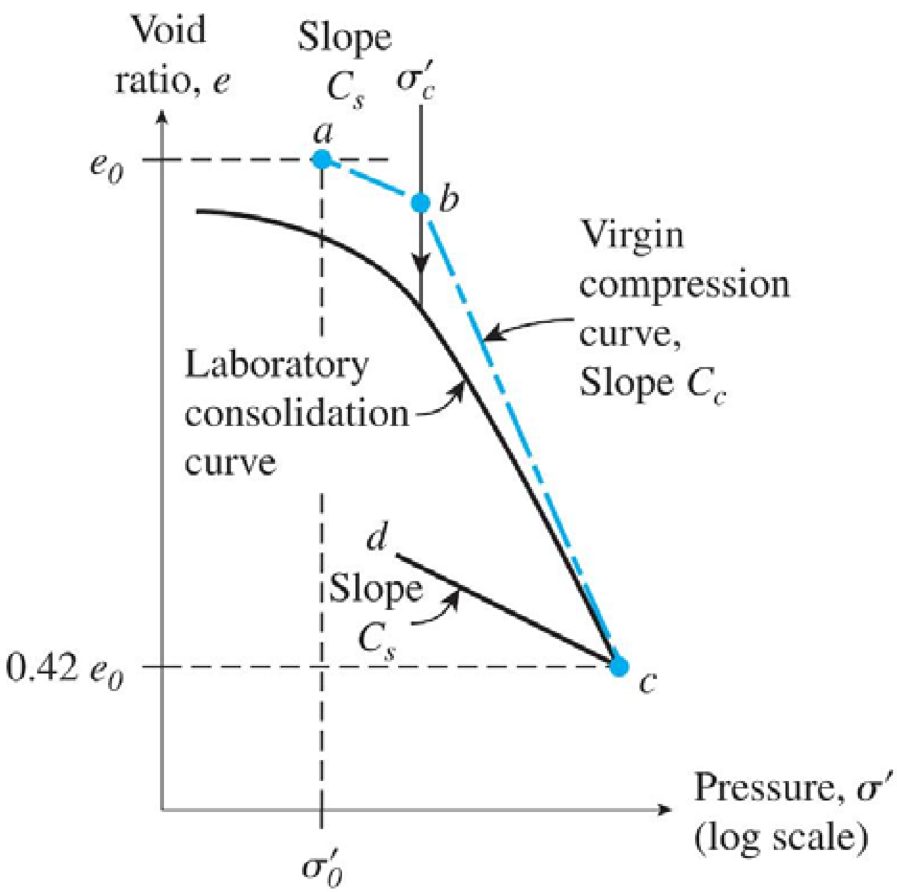
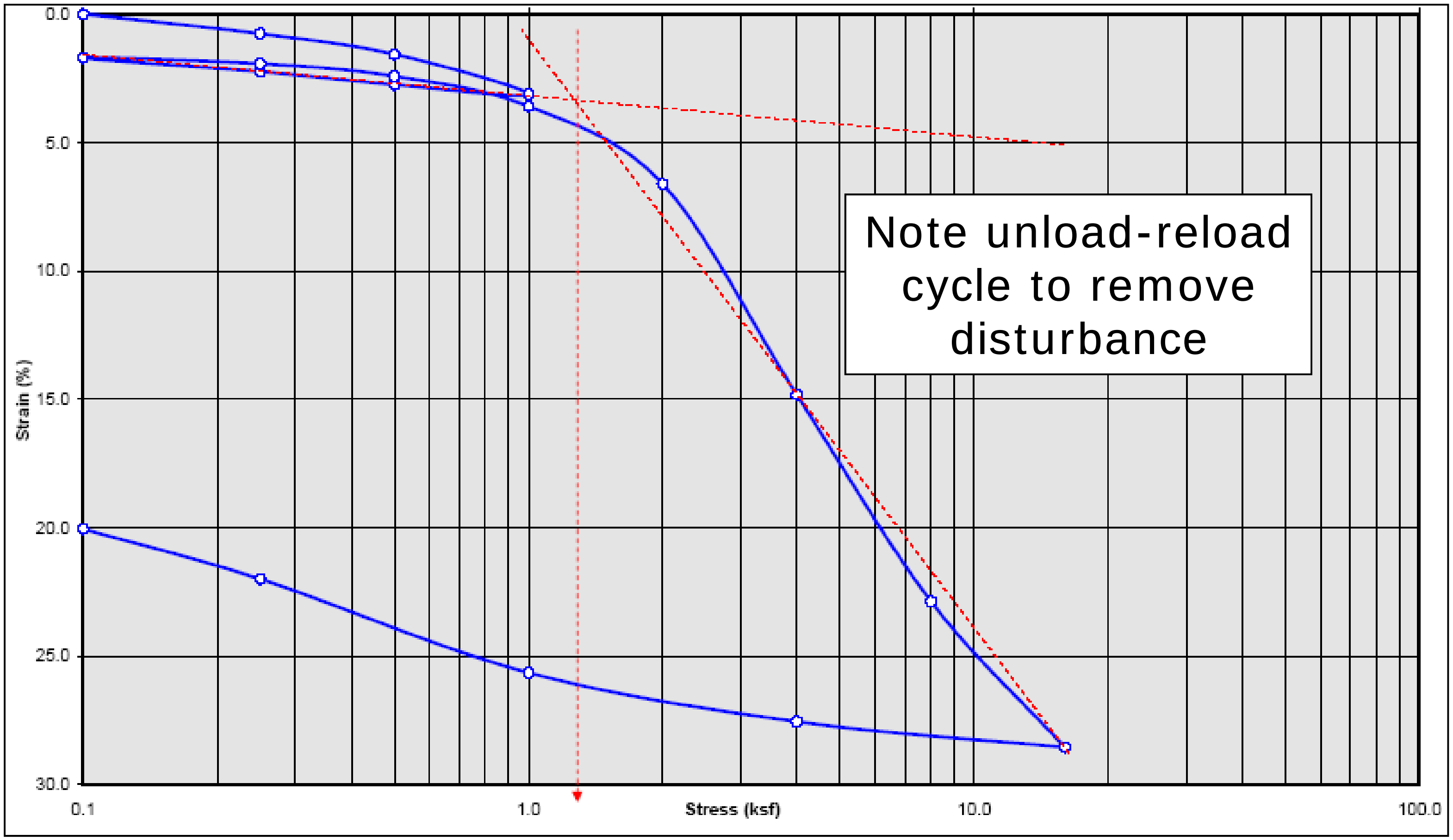


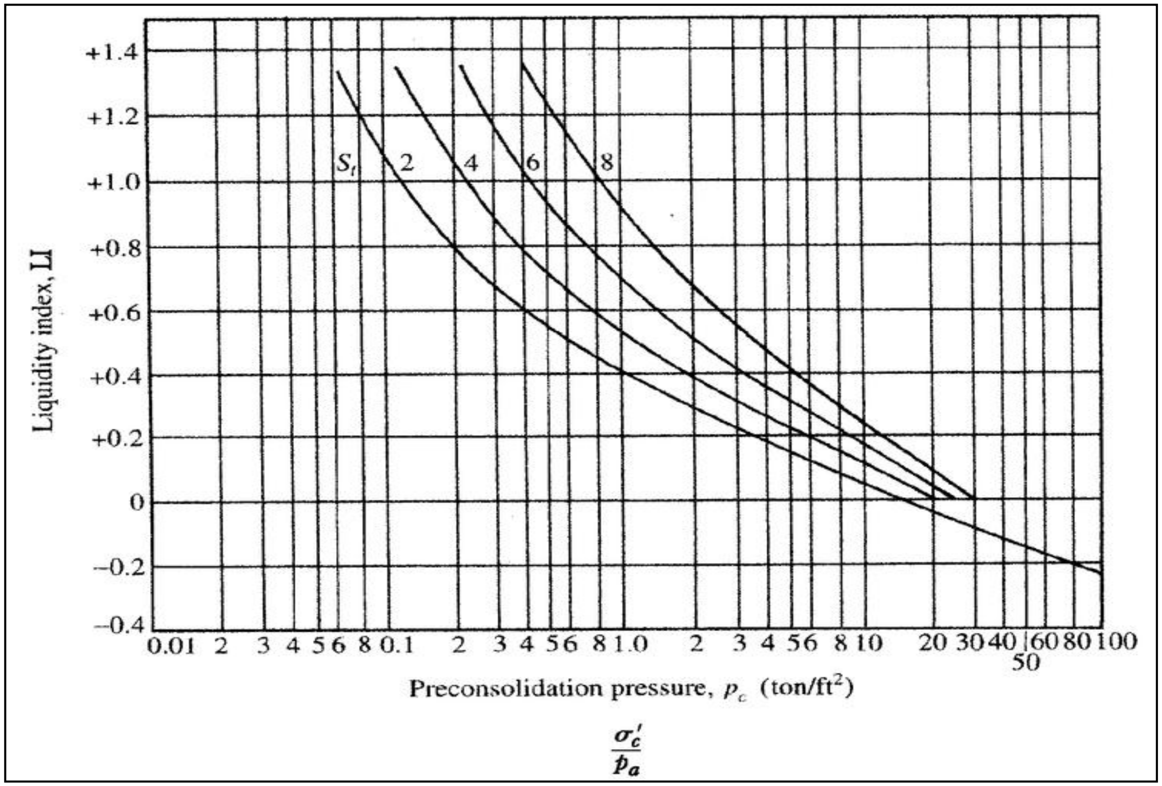
Figure 1.17 Construction of field consolidation curve for overconsolidated clay

Real World



0.0
5.0
10.0
15.0
20.0
25.0
30.0
Strain (%)
0.1
1.0
10.0
100.0
Stress (ksf)
Note unload-reload cycle to remove disturbance

NAVFAC LI Versus P'c



LI = (w-PL)/PI

Note: p_a = atmospheric pressure [~100 KN/m2 (1 U.S. ton/ft2)]

One Dimensional Consolidation

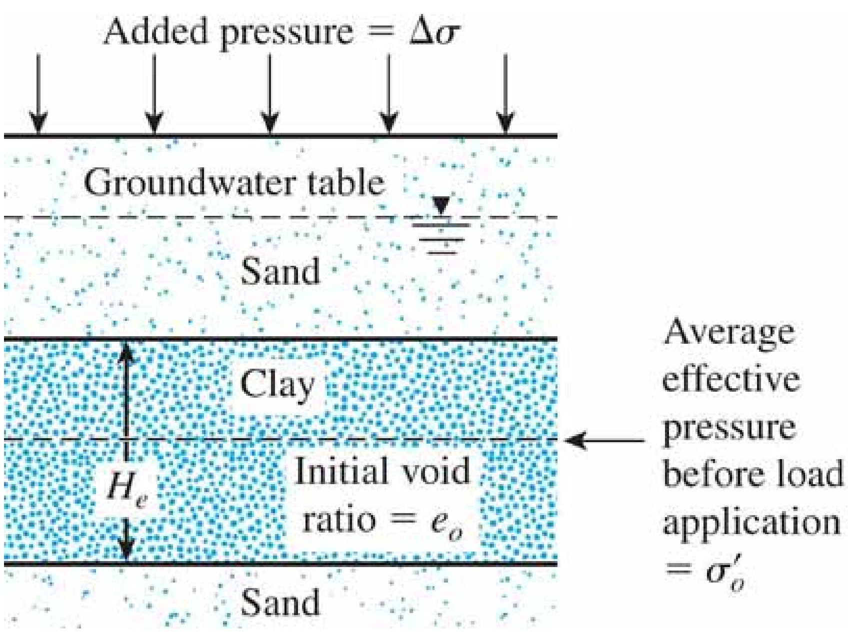


Figure 1.18 One-dimensional settlement calculation

Primary Consolidation Settlement

Equation 1.65

=0 for normally consolidated soil

Cr

$$S := \frac{(C_s \cdot H)}{1+e} \cdot \log\left(\frac{\sigma'_c}{\sigma'_o}\right) + \frac{(C_c \cdot H)}{1+e} \cdot \log\left(\frac{\sigma'_o + \Delta\sigma}{\sigma'_c}\right)$$

For normally consolidated soils $\sigma_c = \sigma'_o$ Therefore $\log(\sigma_c / \sigma'_o) = 0$ and the first quantity goes to zero as well.

Pore Pressure Dissipation

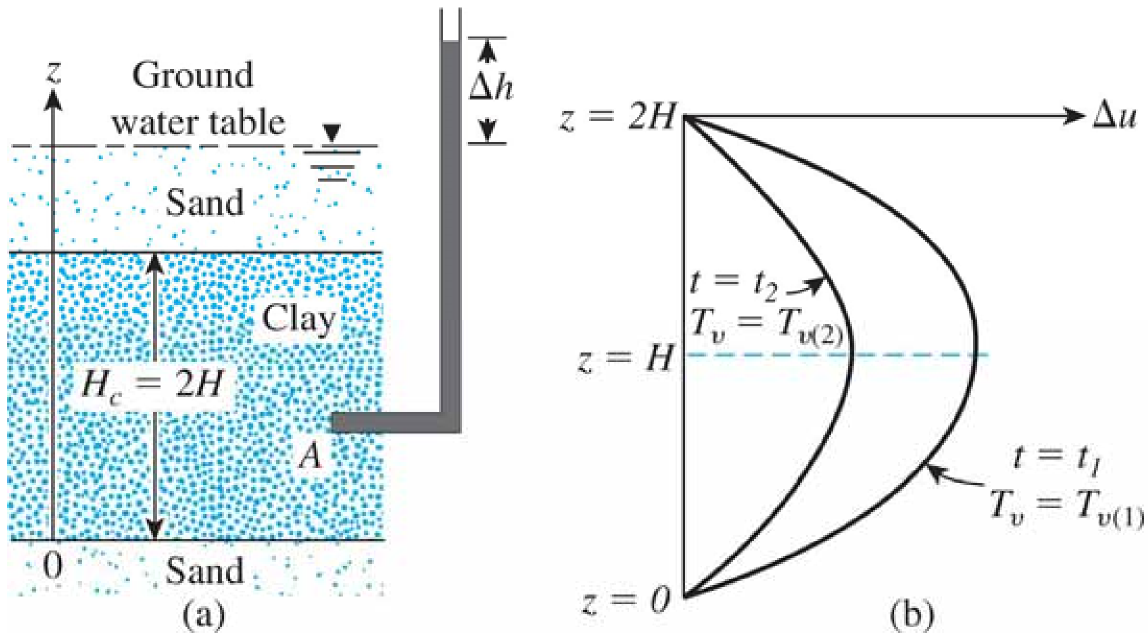


Figure 1.19 (a) Derivation of Eq. (1.68); (b) nature of variation of Δu with time

Drainage Conditions

H = 0.5 Hc

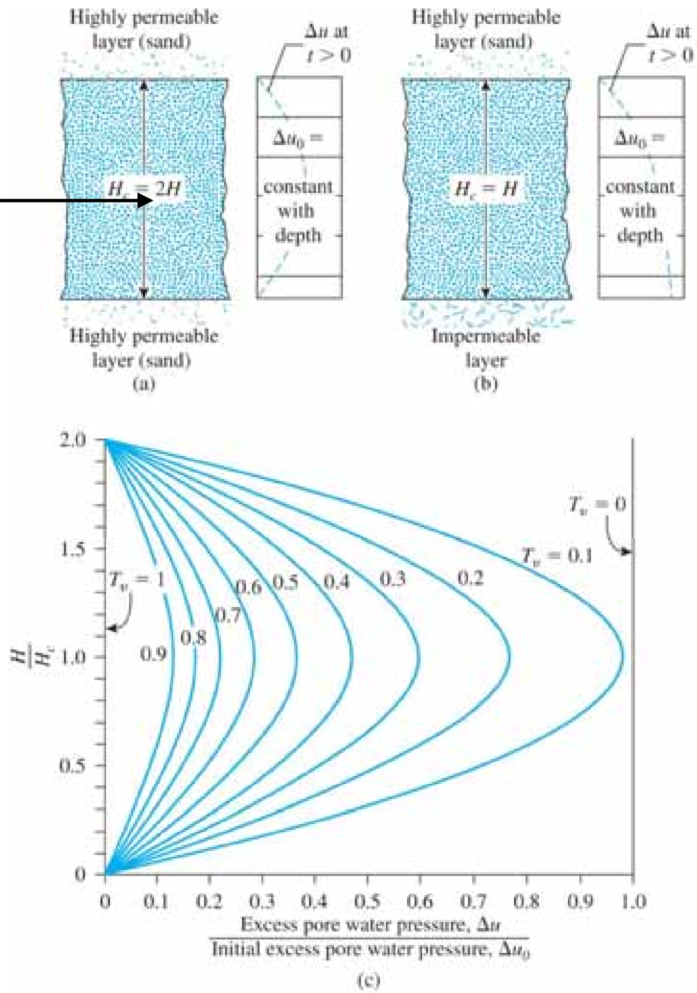
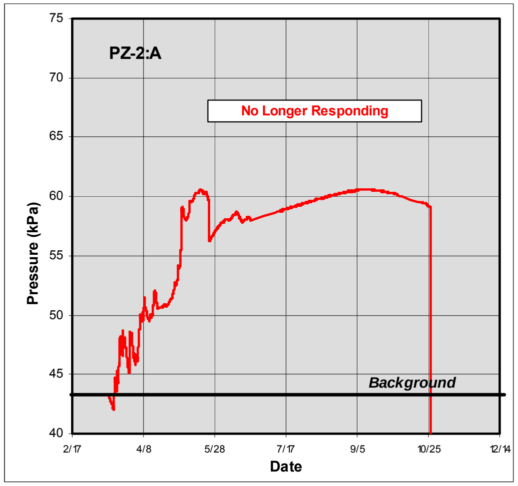
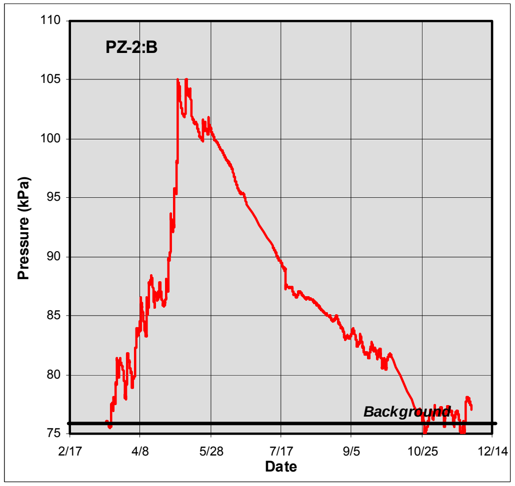


Figure 1.20 Drainage condition for consolidation: (a) two-way drainage; (b) one-way drainage; (c) plot of $\Delta u / \Delta u_0$ with T_v and H/H_c

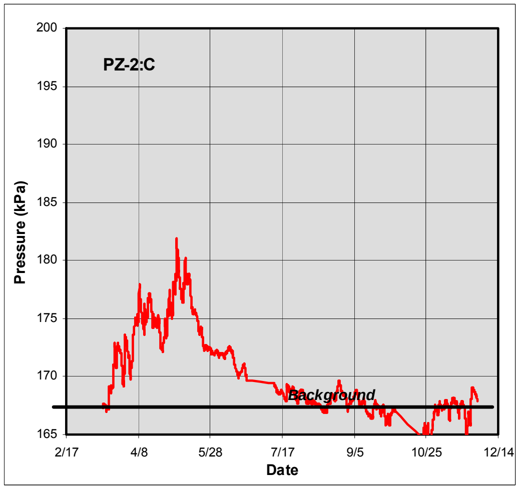
Field Data



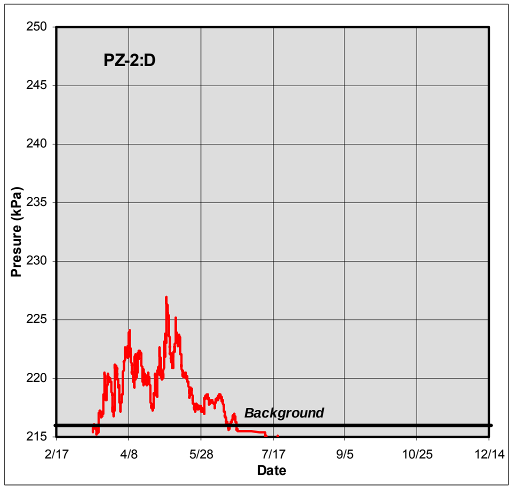
PZ-2:A
No Longer Responding
Background
Pressure (kPa)
Date
75
70
65
60
55
50
45
40
2/17
4/8
5/28
7/17
9/5
10/25
12/14



PZ-2:B
Background
Pressure (kPa)
Date
110
105
100
95
90
85
80
75
2/17
4/8
5/28
7/17
9/5
10/25
12/14

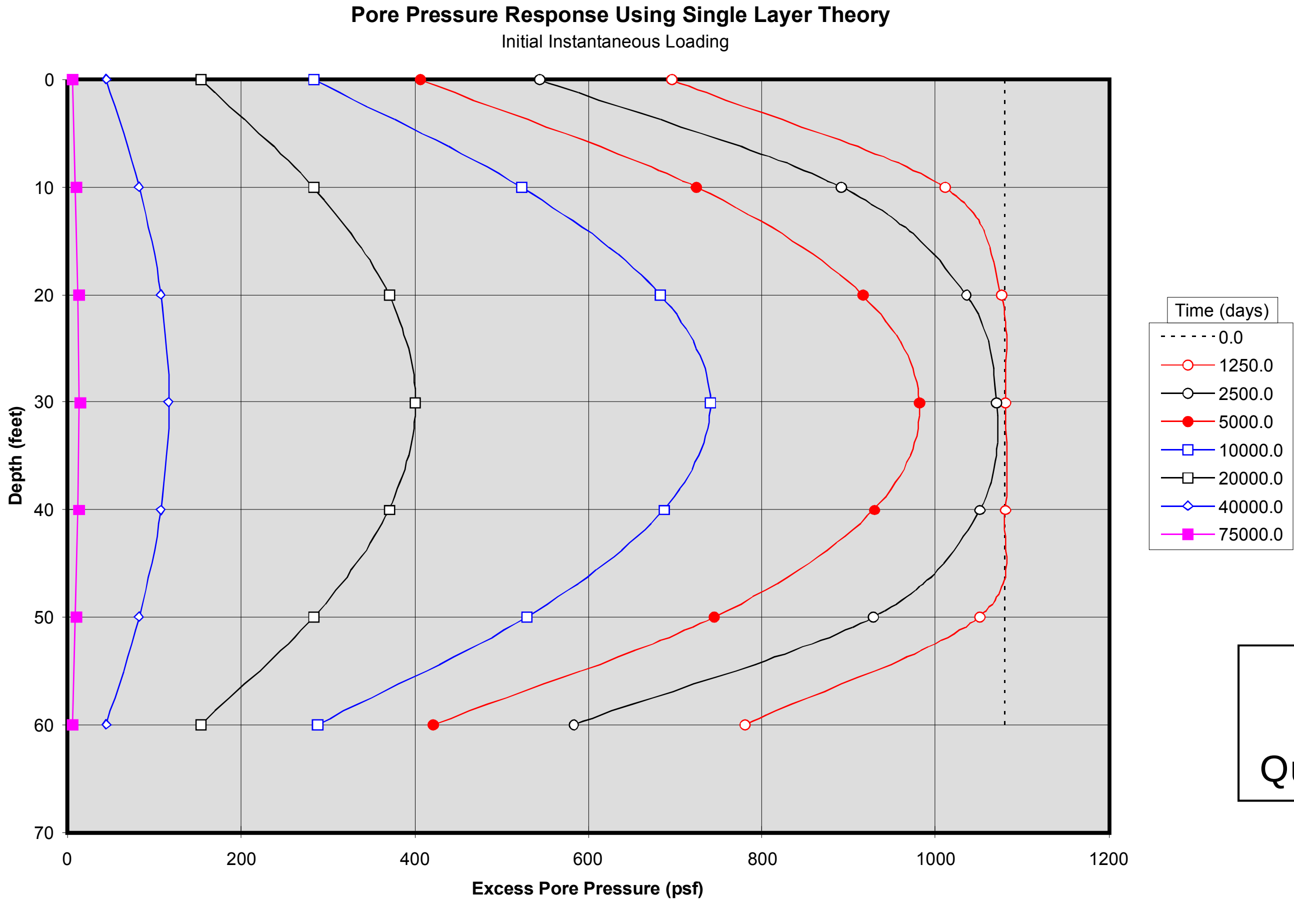


PZ-2:C
Background
Pressure (kPa)
Date
200
195
190
185
180
175
170
165
2/17
4/8
5/28
7/17
9/5
10/25
12/14



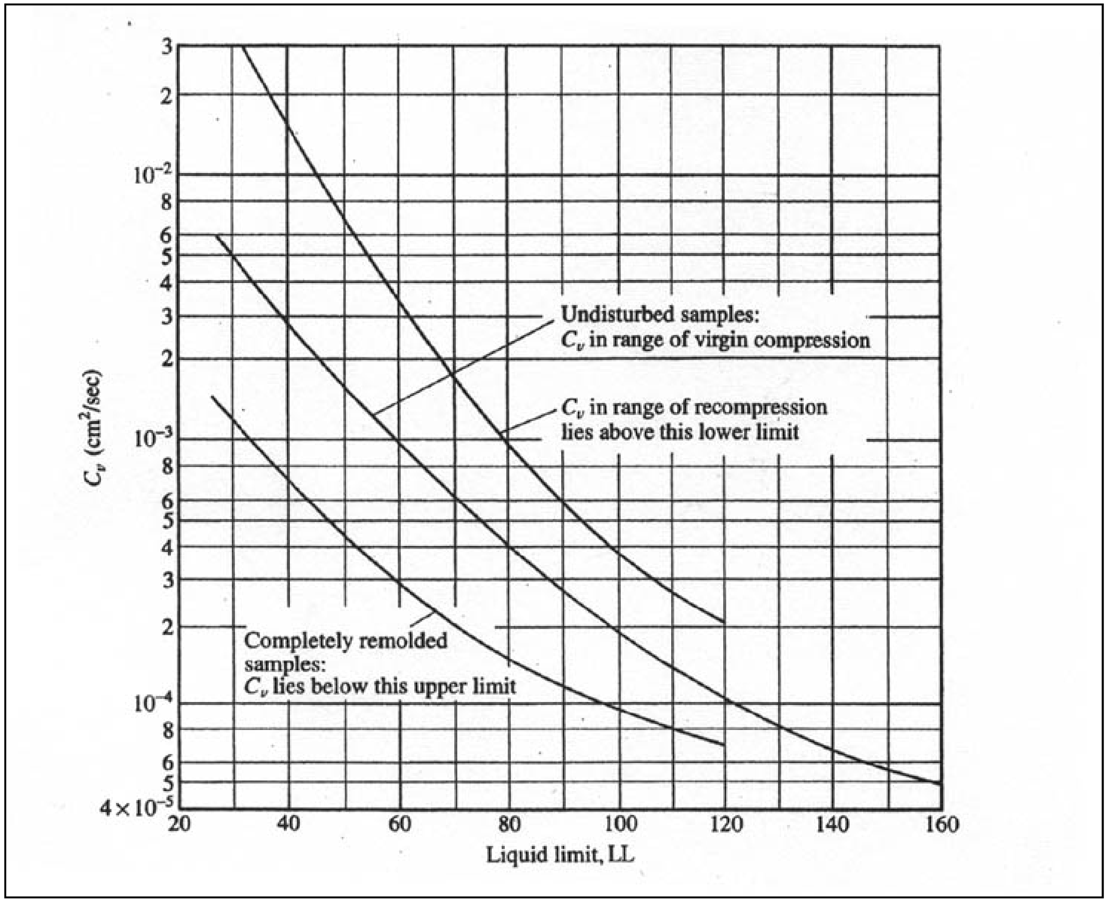
PZ-2:D
Background
Presure (kPa)
Date
250
245
240
235
230
225
220
215
2/17
4/8
5/28
7/17
9/5
10/25
12/14

Constant Cv Modeling



Multilayer Model Indicates Quicker Dissipation

Range of C_v (after U.S. Dept. of Navy)



Provided by consolidation test and varies with pressure. Use pressure at effective stress.

T_v Versus %U

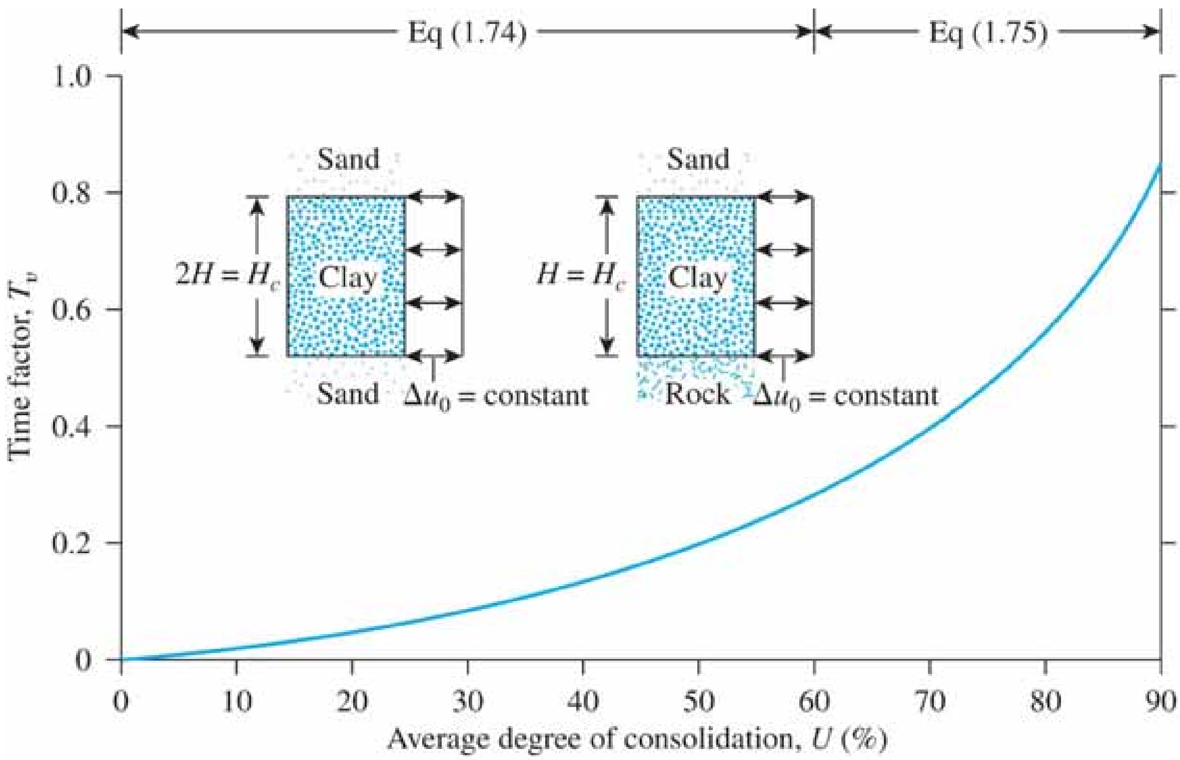


Figure 1.21 Plot of time factor against average degree of consolidation (Δu_0 = constant)

Time factor against average degree of consolidation (Δu_0 = constant)

$$T_v := \frac{\pi}{4}\cdot\left(\frac{U\%}{100}\right)^2$$ For U= 0 to 60%

$$T_v := 1.781 - 0.933\cdot\log(100 - U\%)$$ For U>60%

$$T_v := \frac{\left[\frac{\pi}{4}\cdot\left(\frac{U\%}{100}\right)^2\right]}{\left[1 - \left(\frac{U\%}{100}\right)^{5.6}\right]^{0.357}}$$

For U = 0 to 100%

T_v Versus %U

Table 1.11 Variation of T_v with U

U (%)	T_v	U (%)	T_v	U (%)	T_v	U (%)	T_v
0	0	26	0.0531	52	0.212	78	0.529
1	0.00008	27	0.0572	53	0.221	79	0.547
2	0.0003	28	0.0615	54	0.230	80	0.567
3	0.00071	29	0.0660	55	0.239	81	0.588
4	0.00126	30	0.0707	56	0.248	82	0.610
5	0.00196	31	0.0754	57	0.257	83	0.633
6	0.00283	32	0.0803	58	0.267	84	0.658
7	0.00385	33	0.0855	59	0.276	85	0.684
8	0.00502	34	0.0907	60	0.286	86	0.712
9	0.00636	35	0.0962	61	0.297	87	0.742
10	0.00785	36	0.102	62	0.307	88	0.774

T_v Versus %U

Table 1.11 *(Continued)*

U (%)	T_v	U (%)	T_v	U (%)	T_v	U (%)	T_v
11	0.0095	37	0.107	63	0.318	89	0.809
12	0.0113	38	0.113	64	0.329	90	0.848
13	0.0133	39	0.119	65	0.304	91	0.891
14	0.0154	40	0.126	66	0.352	92	0.938
15	0.0177	41	0.132	67	0.364	93	0.993
16	0.0201	42	0.138	68	0.377	94	1.055
17	0.0227	43	0.145	69	0.390	95	1.129
18	0.0254	44	0.152	70	0.403	96	1.219
19	0.0283	45	0.159	71	0.417	97	1.336
20	0.0314	46	0.166	72	0.431	98	1.500
21	0.0346	47	0.173	73	0.446	99	1.781
22	0.0380	48	0.181	74	0.461	100	∞
23	0.0415	49	0.188	75	0.477		
24	0.0452	50	0.197	76	0.493		
25	0.0491	51	0.204	77	0.511		

Time for Compression

$$Tv := \frac{(Cv \cdot t)}{H^2}$$

$$t := \left(Tv \cdot \frac{H^2}{Cv} \right)$$

Typical client wants to know how long
he has to wait before starting construction.

For U% = 90% Tv = 0.849

With H = 10 feet & double drainage
H/2 = 5 feet
Cv = 0.2 ft^2/day

t= $0.849(5)^2/0.2$ = 106 days

For single drainage H=10 feet
t = $0.849(10)^2/0.2$ = 424.5 days

Ramp or Construction Loading

A new layer of structural fill or building structure cannot be loaded instantaneously on the ground.

For this reason, the increase in loading gradually rises to the maximum load.

This gives time for excess pore water pressure to begin dissipating.

One-dimensional consolidation due to single ramp loading

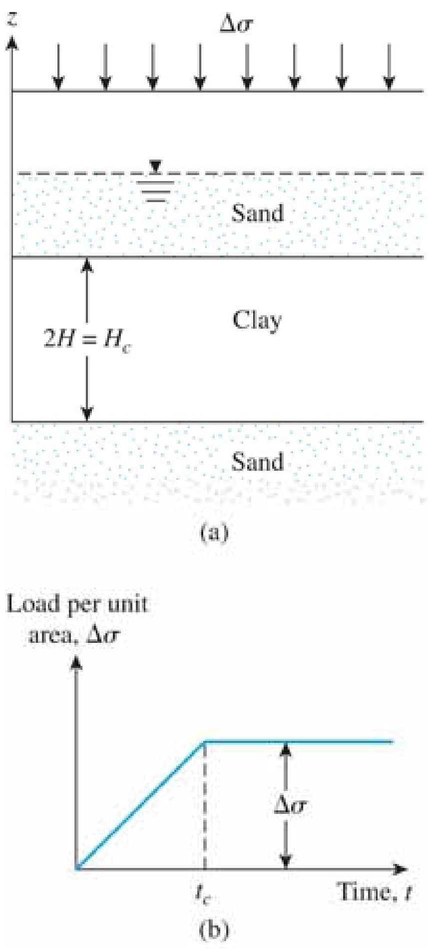


Figure 1.22 One-dimensional consolidation due to single ramp loading

Ramp Loading Parameters

$$T_c := C_v \cdot \frac{t_c}{H^2}$$

Remember double
or single drainage

Where do we get t_c? Construction Schedule.

Olson's Ramp Loading

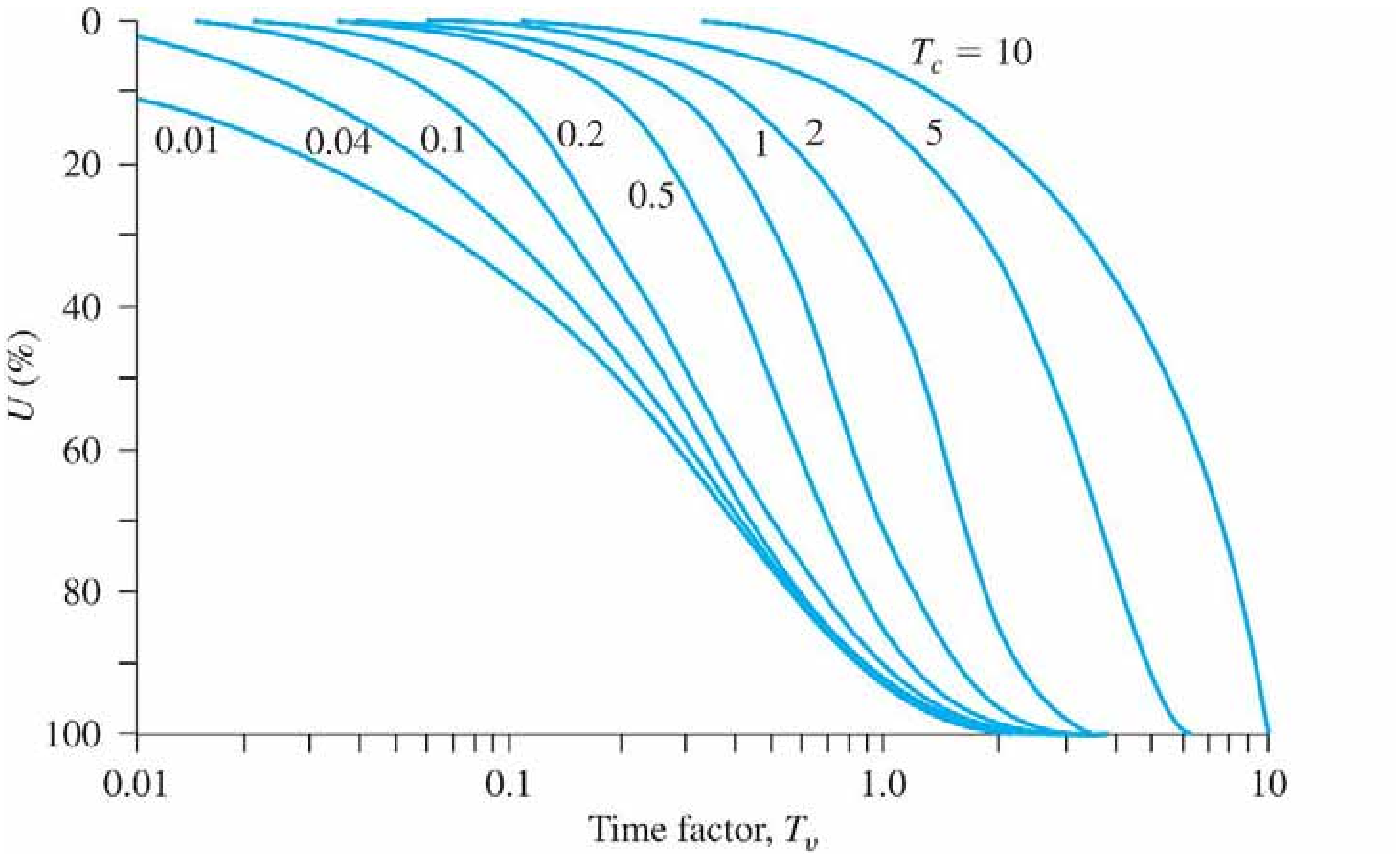


Figure 1.23 Olson's ramp-loading solution: plot of U versus T_v (Eqs. 1.78 and 1.79)

Ramp Loading Example

$$T_c := C_v \cdot \frac{t_c}{H^2}$$

<u>Example 1.10</u>

With H = 10 feet & double drainage H/2 = 5 feet
C_v = 0.2 ft^2/day
t_c = 15 days, what %U at 50 days

$T_c = 0.2(15/(5)^2) = 0.12$
$T_v = 0.2(50/(5)^2) = 0.4$

From chart U% = 70%.

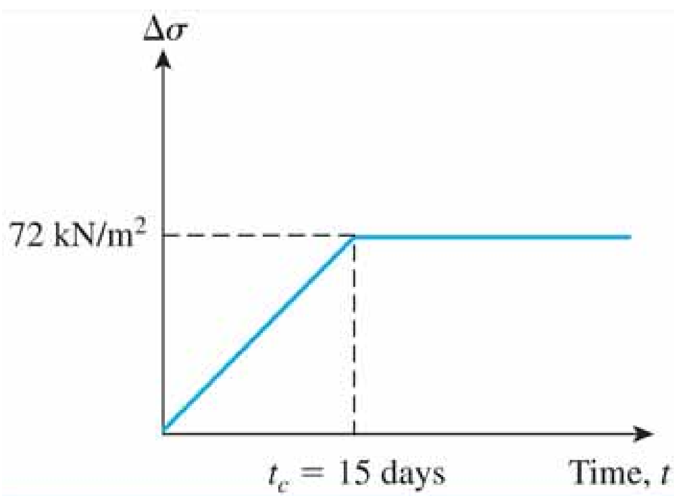


Figure 1.24 Ramp loading

Shear Strength

$S = c' + \sigma' \cdot \tan(\phi')$

Shear Strength Tests

Unconfined Compression Tests
Direct Shear Tests
Direct Simple Shear
Triaxial Shear Tests

- Unconsolidated – Undrained (UU) $\Delta u = 0$
- Consolidated – Undrained w/PPM (CU)
- Consolidated – Drained (CD)

Each test will yield a different value of Su
Drained Strength typically > Undrained Strength

Direct Shear Test

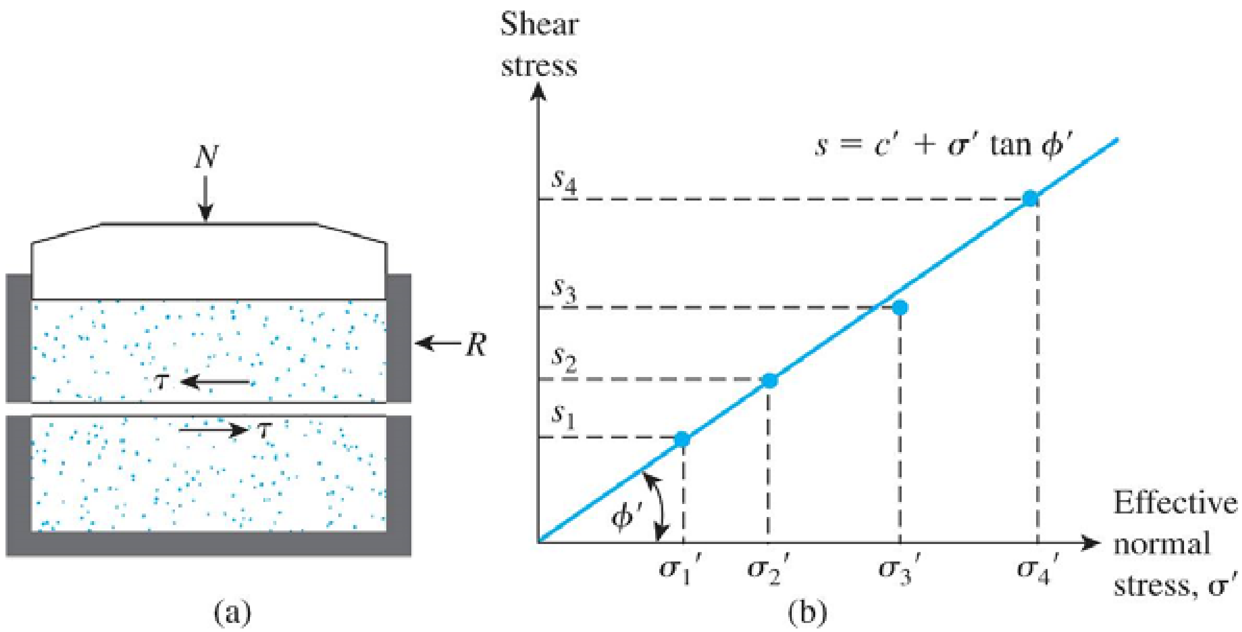


Figure 1.25 Direct shear test in sand: (a) schematic diagram of test equipment; (b) plot of test results to obtain the friction angle ϕ'

Typical Values of ϕ

Table 1.12 Relationship between Relative Density and Angle of Friction of Cohesionless Soils

State of packing	Relative density (%)	Angle of friction, ϕ' (deg.)
Very loose	<20	<30
Loose	20–40	30–35
Compact	40–60	35–40
Dense	60–80	40–45
Very dense	>80	>45

Triaxial Shear Test

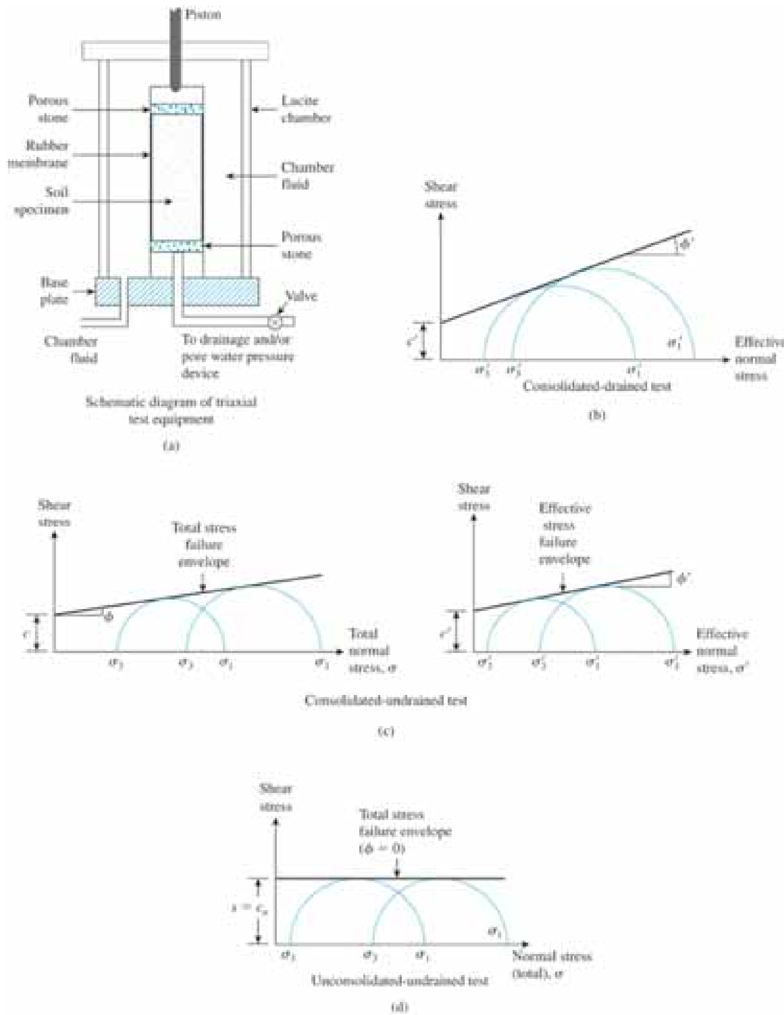


Figure 1.26 Triaxial test

Sequence of Stress Application

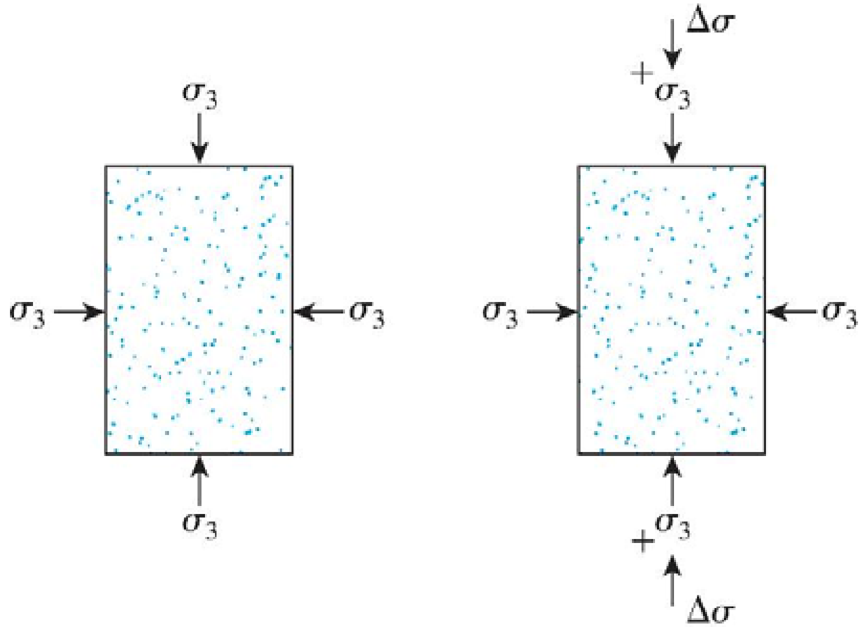


Figure 1.27 Sequence of stress application in triaxial test

Unconfined Shear Test

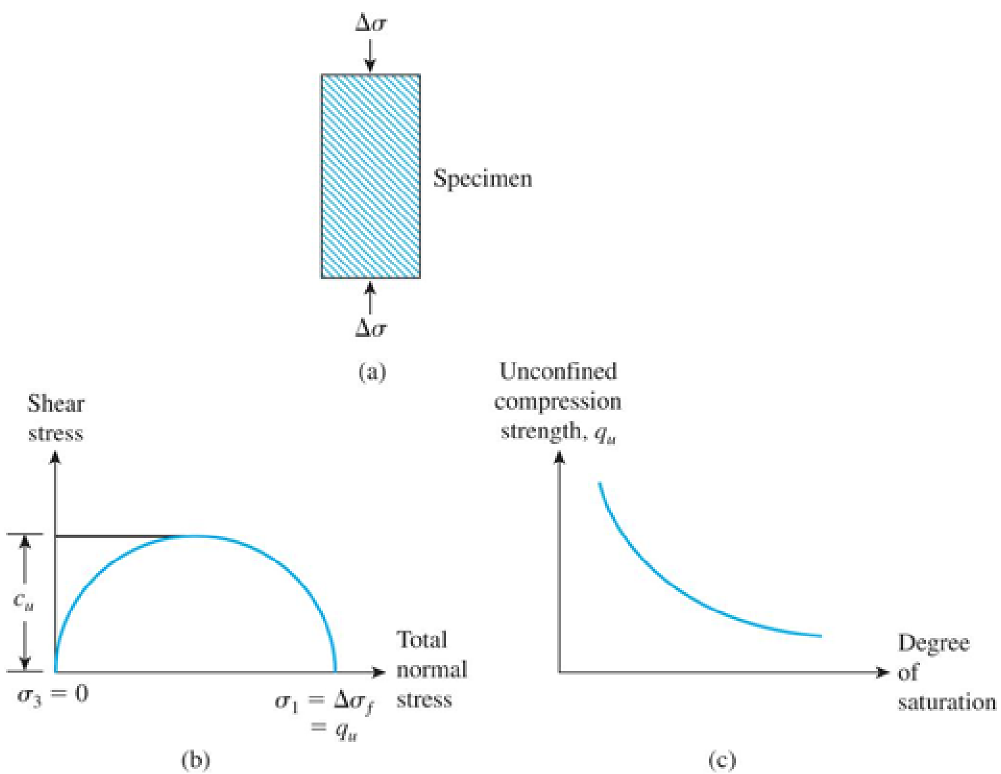


Figure 1.28 Unconfined compression test: (a) soil specimen; (b) Mohr's circle for the test; (c) variation of q_u with the degree of saturation

(a) soil specimen;
(b) Mohr's circle for the test;
(c) variation of q_u with the degree of saturation

$S_u = c_u = q_u/2$

Triaxial Test in Progress

ϕ Versus Void Ratio

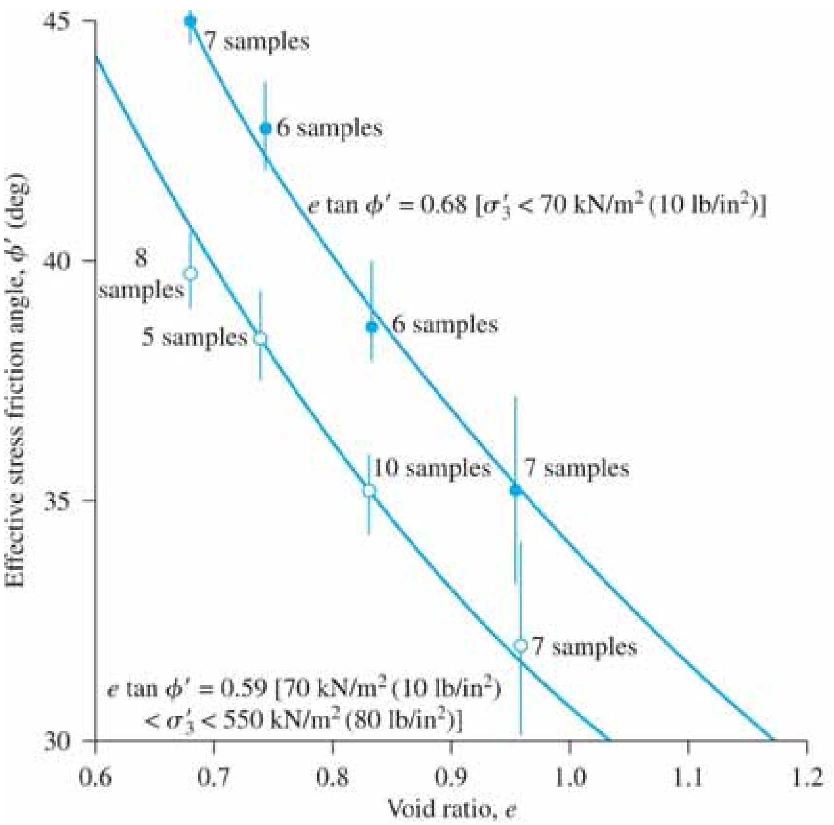


Figure 1.29 Variation of friction angle ϕ' with void ratio for Chattachoochee River sand (After Vesic, 1963) (From Vesic, A. B. Bearing Capacity of Deep Foundations in Sand. In Highway Research Record 39, Highway Research Board, National Research Council, Washington, D.C., 1963, Figure 11, p. 123. Reproduced with permission of the Transportation Research Board.)

ϕ' Versus Plasticity Index

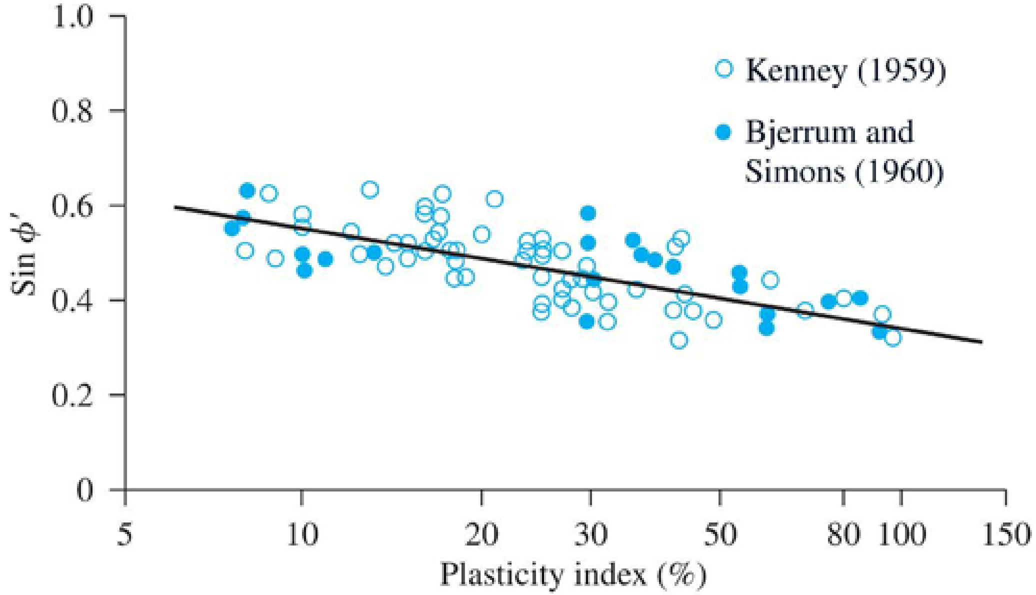
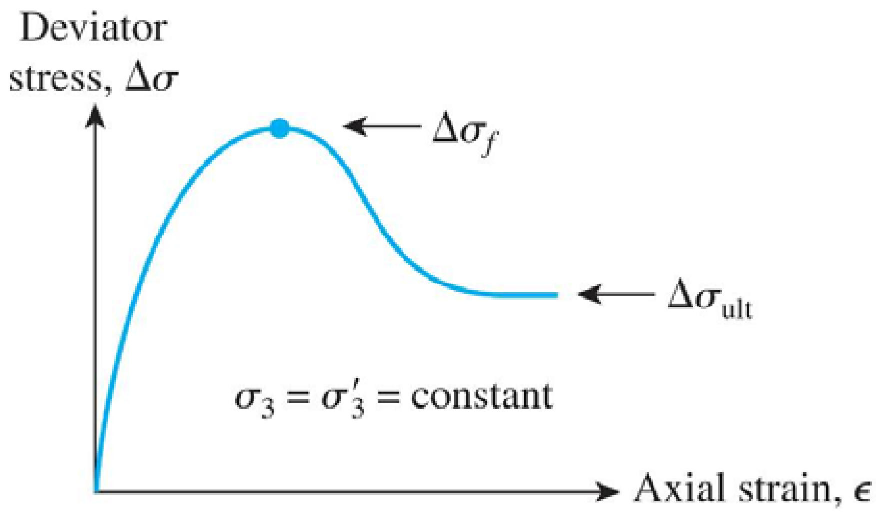


Figure 1.30 Variation of sin ϕ' with plasticity index (PI) for several normally consolidated clays

Note scatter

$$\phi = 0.0011 \cdot PI^2 - 0.2603 \cdot PI + 35.975$$

Deviator stress vs. axial strain-drained triaxial test



Strain Softening

Figure 1.31 Plot of deviator stress versus axial strain–drained triaxial test

Peak & Residual Strength Envelopes for Clays

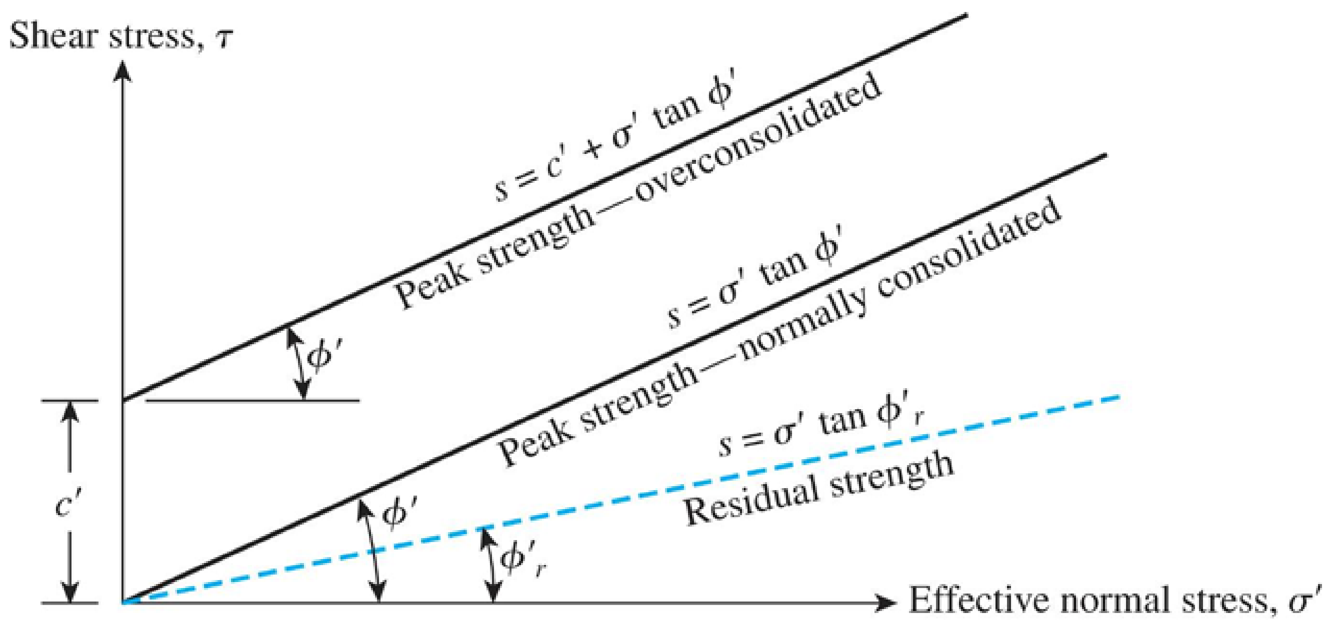


Figure 1.32 Peak- and residual-strength envelopes for clay

Variation of ϕ'_r with CF

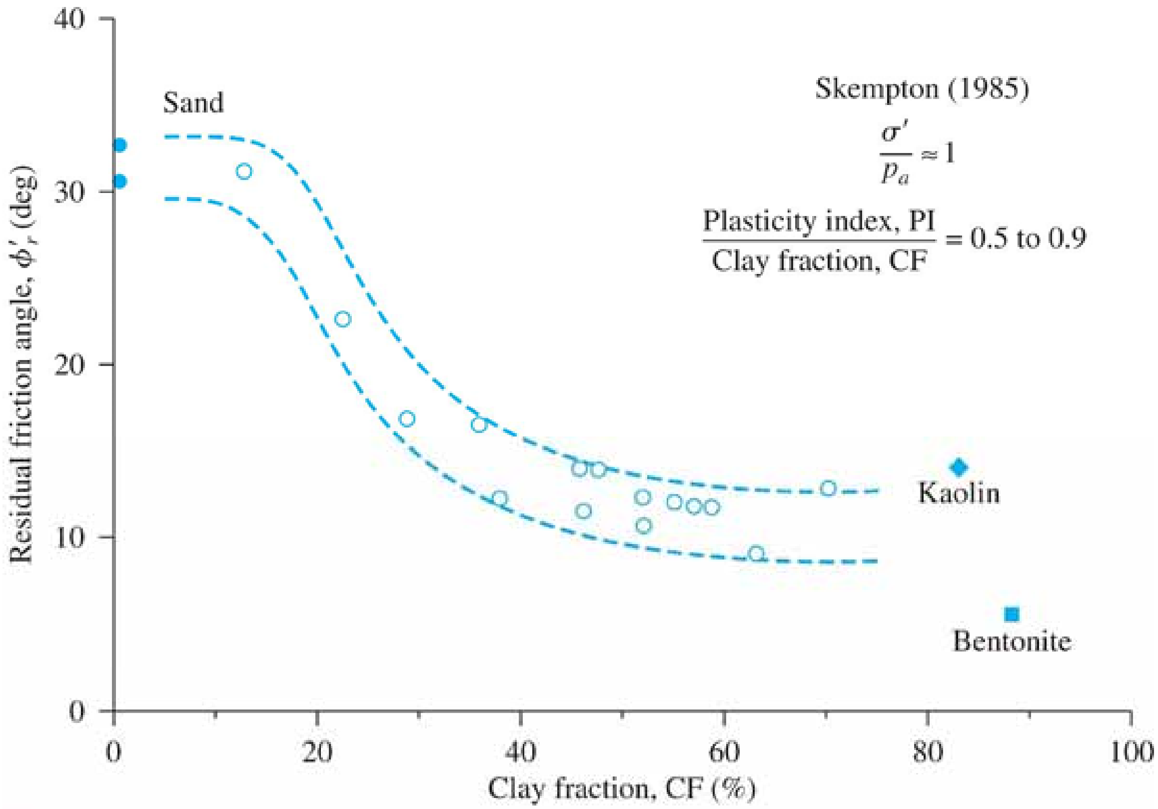


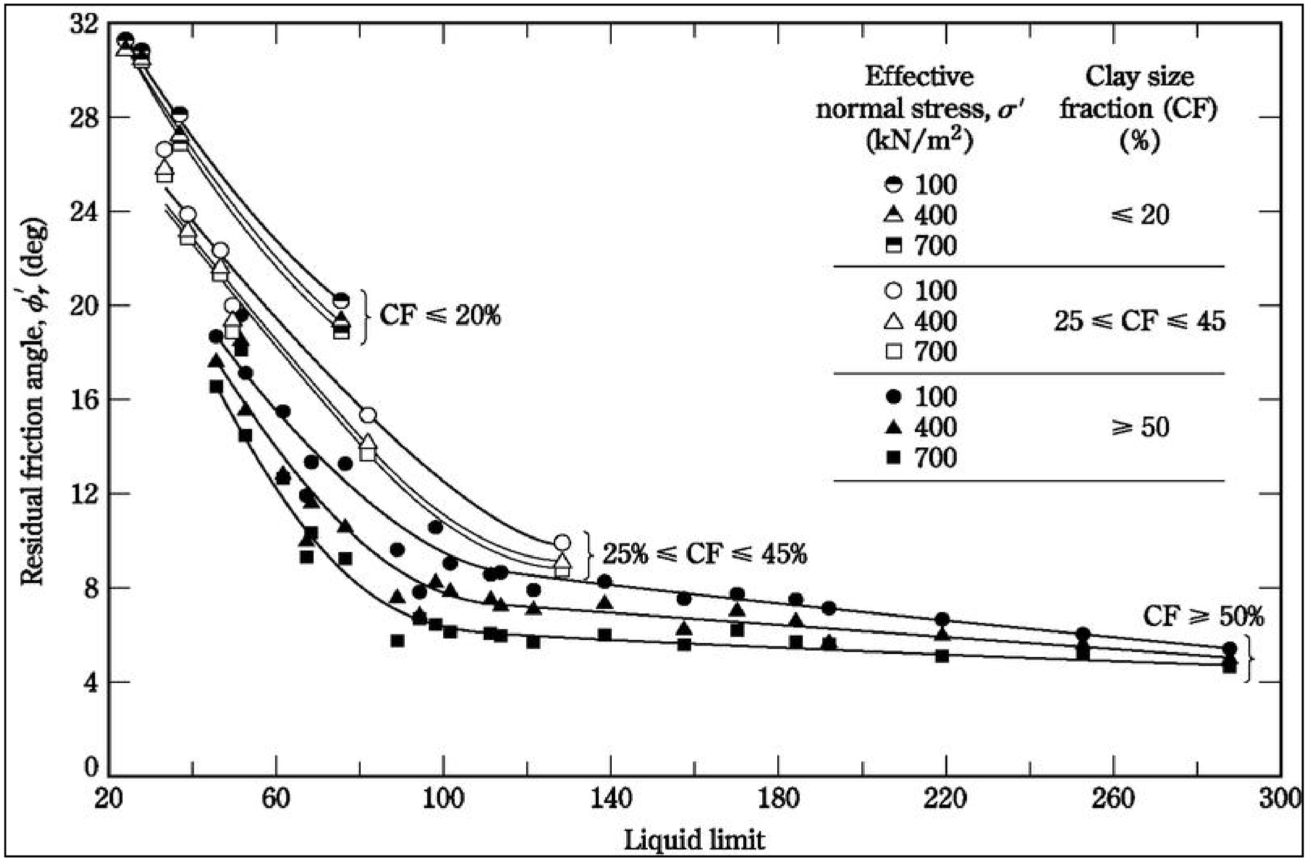
Figure 1.33 Variation of ϕ'_r with CF (*Note:* p_a = atmospheric pressure)

Empirical Correlations

Table 1.13 Empirical Equations Related to c_u and σ'_0

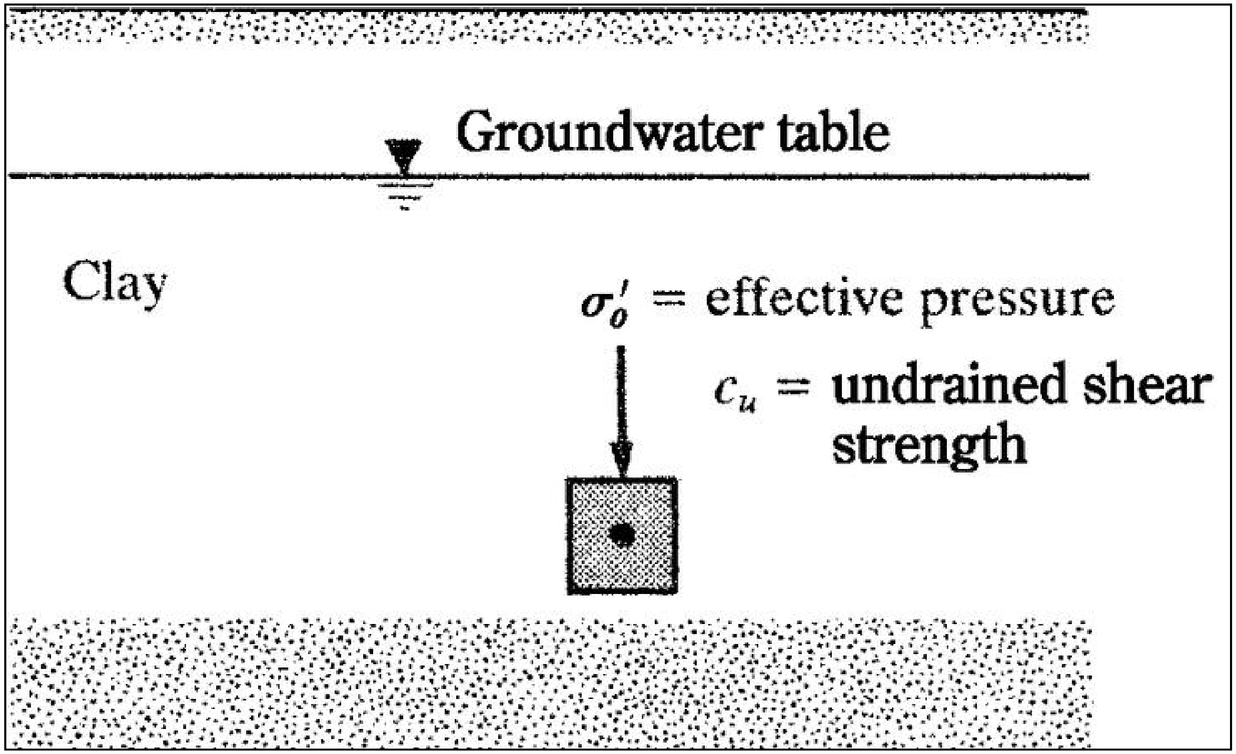
Reference	Relationship	Remarks
Skempton (1957)	$\frac{c_{u(\text{VST})}}{\sigma'_0} = 0.11 + 0.00037\,(\text{PI})$ PI = plasticity index (%) $c_{u(\text{VST})}$ = undrained shear strength from vane shear test	For normally consolidated clay
Chandler (1988)	$\frac{c_{u(\text{VST})}}{\sigma'_c} = 0.11 + 0.0037\,(\text{PI})$ σ'_c = preconsolidation pressure	Can be used in overconsolidated soil; accuracy ±25%; not valid for sensitive and fissured clays
Jamiolkowski, *et al.* (1985)	$\frac{c_u}{\sigma'_c} = 0.23 \pm 0.04$	For lightly overconsolidated clays
Mesri (1989)	$\frac{c_u}{\sigma'_0} = 0.22$	
Bjerrum and Simons (1960)	$\frac{c_u}{\sigma'_0} = 0.45\left(\frac{\text{PI\%}}{100}\right)^{0.5}$ for PI > 50%	Normally consolidated clay
	$\frac{c_u}{\sigma'_0} = 0.118\,(\text{LI})^{0.15}$ for LI = liquidity index > 0.5	Normally consolidated clay
Ladd, *et al.* (1977)	$\frac{\left(\frac{c_u}{\sigma'_0}\right)_{\text{overconsolidated}}}{\left(\frac{c_u}{\sigma'_0}\right)_{\text{normally consolidated}}} = \text{OCR}^{0.8}$ OCR = overconsolidation ratio = σ'_c/σ'_0	

Variation of ϕ'_r with liquid limit for some clays



(after Stark, 1995)

Variation With Depth - Clay Deposit



In normally consolidated clays undrained shear strength increases almost linearly with effective overburden pressure

S_u/σ' ratio

S_u/σ' Relationships

Use consolidated-undrained triaxial tests at different levels of stress to determine Su/σ' ratio. Can be estimated by:

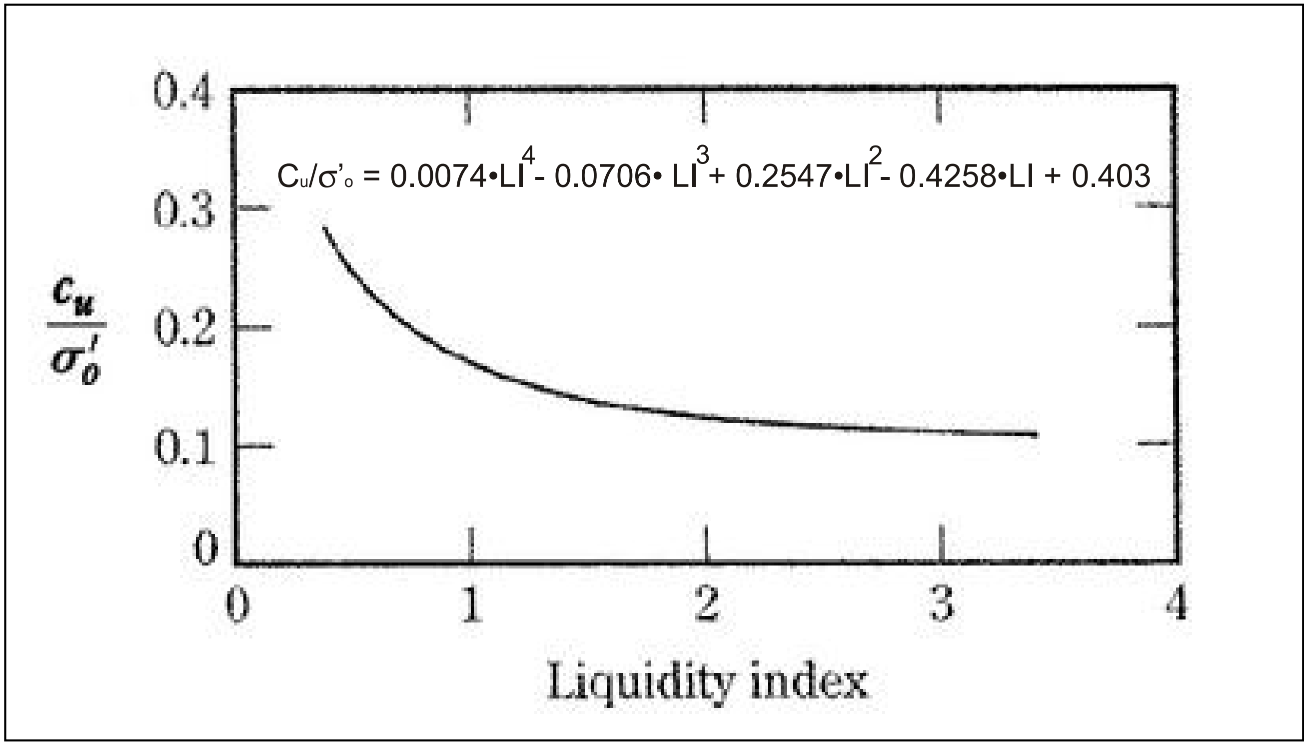
$$\left(\frac{S_u}{\sigma'}\right)_{NC} = 0.11 + 0.0037PI$$

$$\left(\frac{S_u}{\sigma'}\right)_{OC} = \left(\frac{S_u}{\sigma'}\right)_{NC} \cdot OCR^{0.8}$$

Critical State
Pore Pressure Parameter Λo
Varies with Soil

$$\Lambda_o := 1 - \frac{C_s}{C_c}$$

Variation of Cu with LI



(based on Bjerrum and Simons 1960)

LI = (w-PL)/PI

Homework

From Chapter 1

CE 420

- 1.11
- 1.12
- 1.13
- 1.14

CE 520

- All of CE 420 plus
- 1.15
- 1.19

Read Chapter 2