203552 Advanced Soil Mechanics

Lecture No. 4

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DRAINED AND UNDRAINED STRENGTH

Soil Strength → depend highly on <u>pore pressure</u> during loading

Two extreme conditions are normally considered for design and analysis.

1. Drained Strength ; when excess pore pressure (Δu) $\cong$ 0
   during loading or after fully dissipation of Δu 2 typical eases are;
   1) Sand or gravel layers of high k → Δu dissipate fast
   2) Clayey soil with slow rate of loading → small increase of Δu and longer time of dissipation

2. Undrained Strength ; when Δu remained during loading

Δu fully developed and remained — "Fully undrained condition"

Δu partially is dissipated — "Partialy drained"

Ex.

1) Saturated clay with high rate of loading

2) Silt and fine sand with seismic or repeated load → Accumulation of pore pressure → Boiling

Mohr – Coulomb's Effective Strength Equation

$$\bar{\tau} = \bar{c} + \left(\sigma - u_s - \Delta u\right) \cdot \tan \bar{\phi}$$

u_s = Constant

Δu = varied during loading

Drained or Effective strength concept

→ usually applied for Δu = 0 or known Δu

Undrainred or Total strength concept

→ for Δu is fully deveoped or unknow Δu (high)

→ "Stress-path method" is normally used to explain drained and Undrained soil strength.

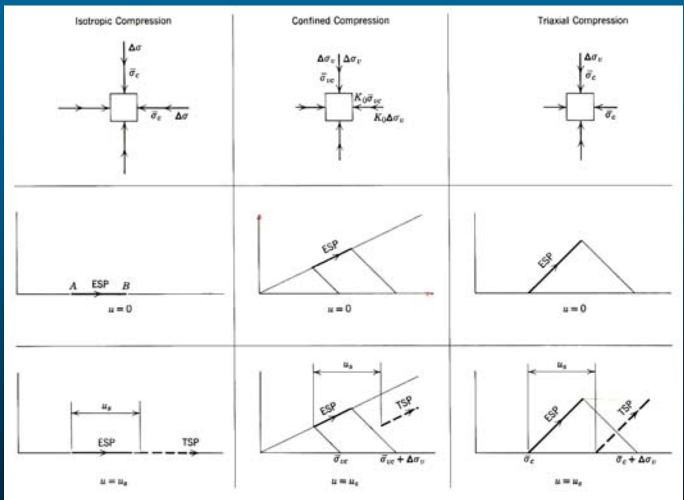
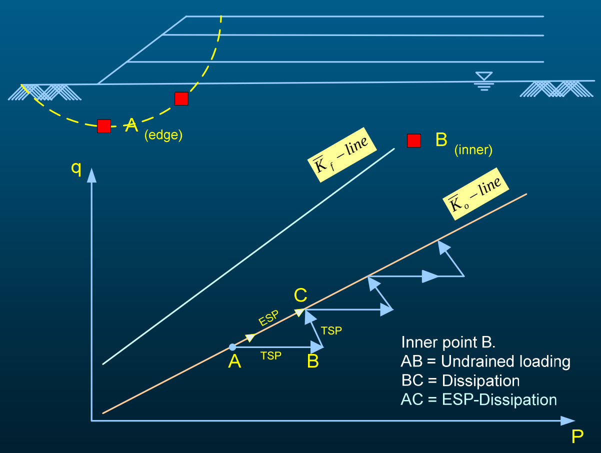


Fig. 20.1 Stress paths for compression tests.

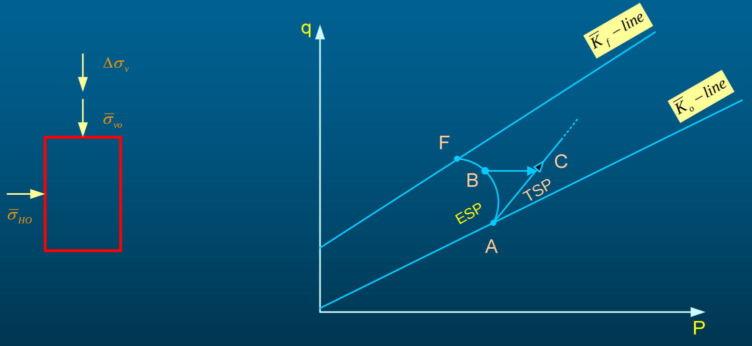
Actual Behaviors

1. Confined Compression / Consolidation



2. Triaxial Compression

Ex. At the edge of embankment

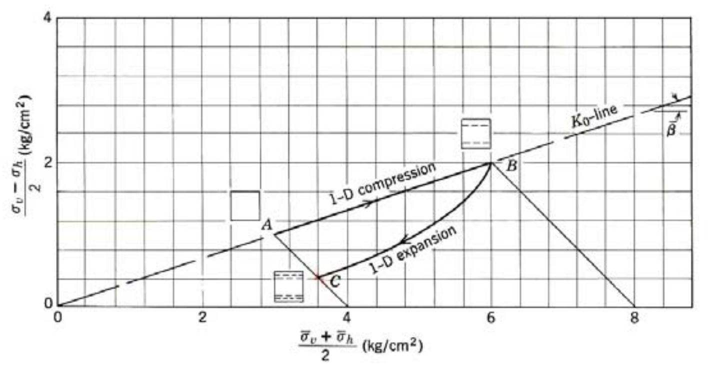


At Pt. A. For Normally Consolidated Clay at K_o-Condition

AB = Triaxial Undrained Loading (ESP)

AC = Triaxial Undrained Loading (TSP)

BC = Dissipation of Excess pore pressure



$K_0 = \frac{\bar{\sigma}_h}{\bar{\sigma}_v}$ for 1-D strain

$K_0 \approx 0.95 - \sin\bar{\phi}$ for N. C. Clay

$K_0 \approx 1.0 - \sin\bar{\phi}$ for N. C. Sand

$$\bar{\beta} = \tan^{-1}\frac{1 - K_0}{1 + K_0}$$

Fig. 20.7 Stress paths for oedometer test.

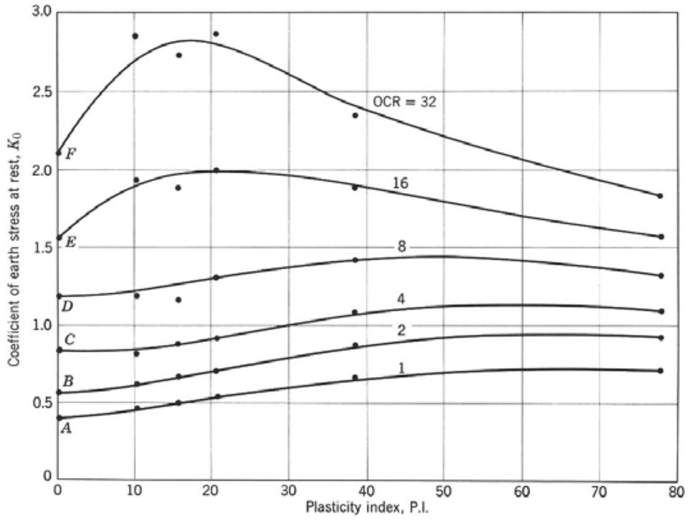


Fig. 20.8 K_0 as function of overconsolidation ratio and plasticity index. Points *A* to *F* interpolated from Hendron's data. Overconsolidation ratio—OCR (From Brooker and Ireland (1965)).

STRENGTH TEST BY TRIAXIAL COMPRESSION

Dr.Warakorn Mairaing

STRENGTH TEST BY TRIAXIAL COMPRESSION

Advantages

1. Closely simulate the aetual field stress condition.
2. Fully control the drainage condition in the sample.
3. Get more design informations. (Design Parameters)
   - c , ϕ
   - Δu
   - Δv
   - E , K_o
4. Automatic control and monitoring capability.

Disadvantages

1. Require qualified technician
2. Costly

SOIL SAMPLE

UU-Test

1. Natural Soil (Saturated)

   UCS → $\sigma_3 = 0$ → Cu

   $\phi = 0$ Concept → $T_3 > 0$ → Cu
2. Compacted Soil (Unsaturated)

   $T_3 > 0$ → Cu, ϕu, Δu

   Very difficult to obtain

CU-Test

1. Natural Soil (Saturated)

   c , ϕ , Δu
2. Compacted Soil with pore pressure

   (Δu)

   $\bar{c}, \bar{\phi}$

CD-Test

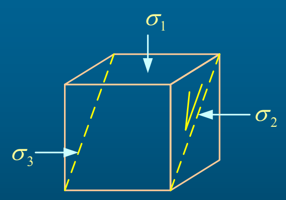
1. Gronular Soil
2. Slow rate of loading

$\bar{c}, \bar{\phi}, \Delta = 0$

$\bar{E}, \Delta v$

Types of Triaxial Test

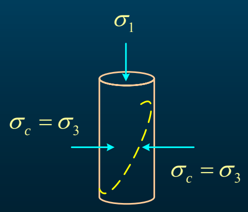
1. True Triaxial Test

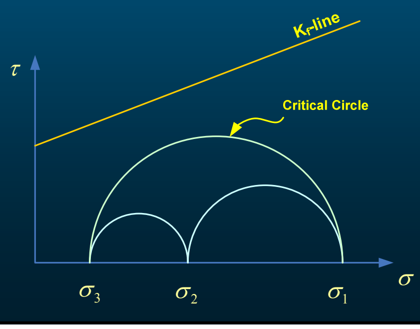


When $\sigma_1 > \sigma_2 > \sigma_3$

- Usually occur in plane-strain condition in the field
- Laboratory Test is quite complicated need special equipment

2. Conventional Triaxial Test

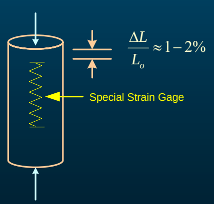




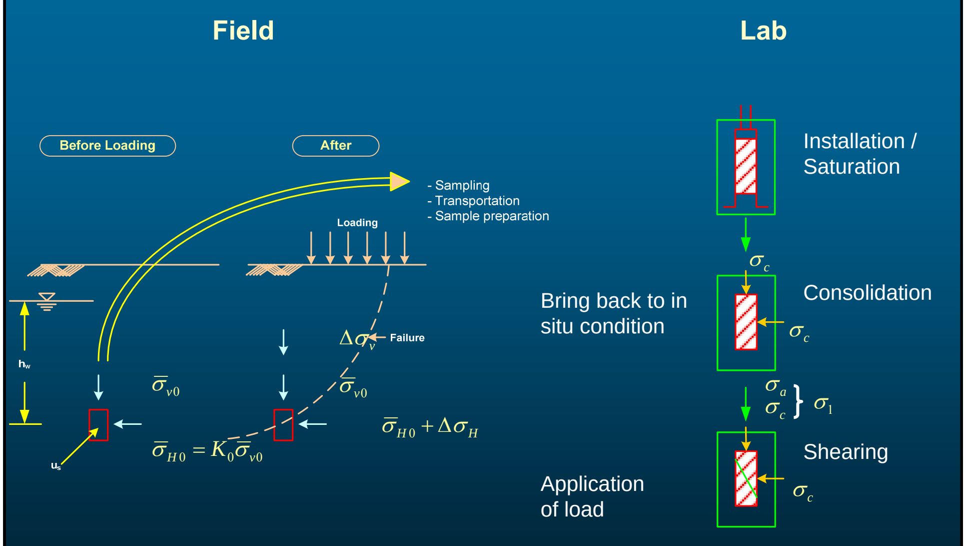
3. Repeated or Cyclic loading Triaxial Test

Simulate - earthquake
- repeated load (pavement)
- machine foundation
- Oil drilling platform

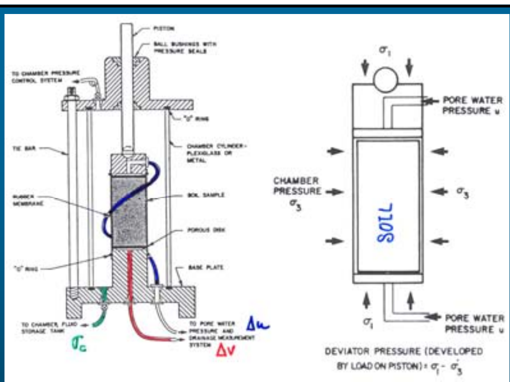
4. Low strain Triaxial Test



For
- Tunneling work
- Retaining wall



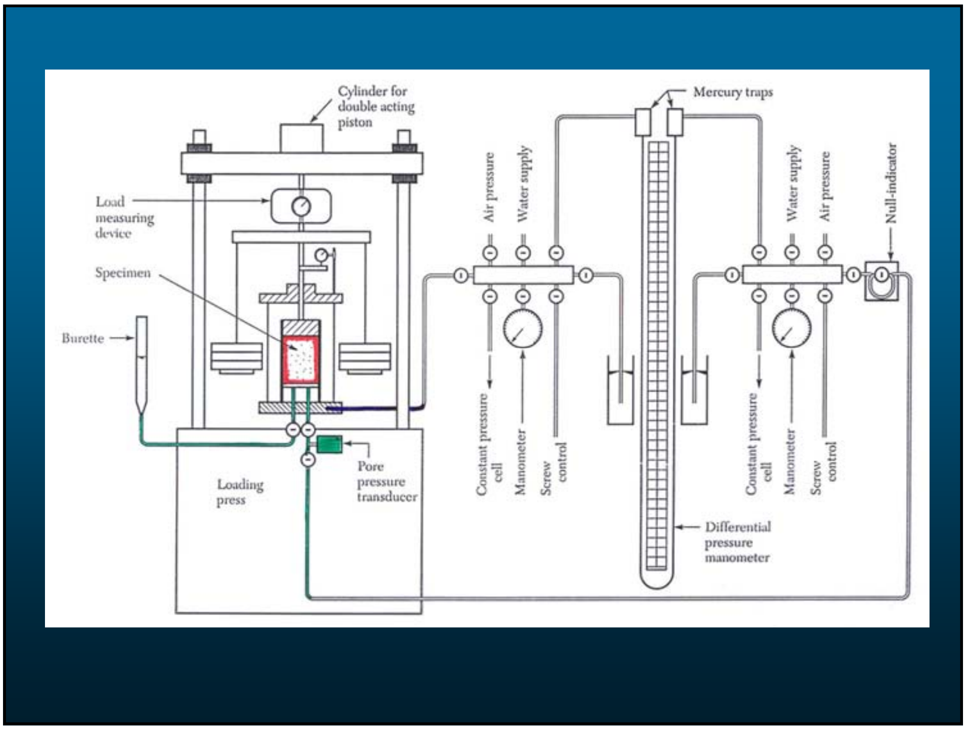
Simulation of triaxial Test

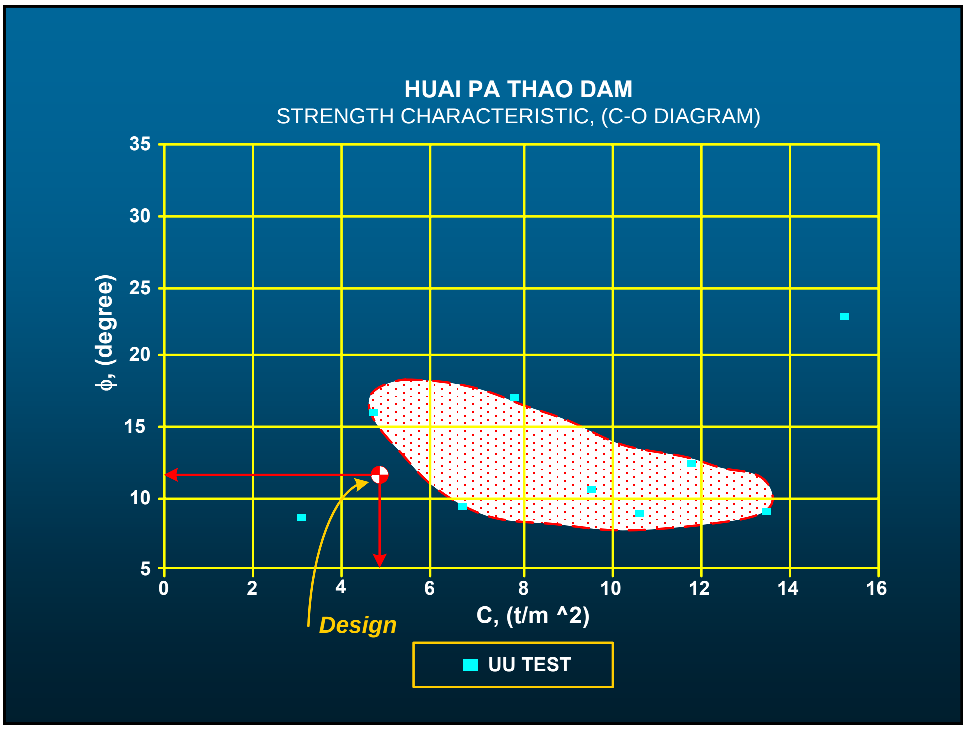


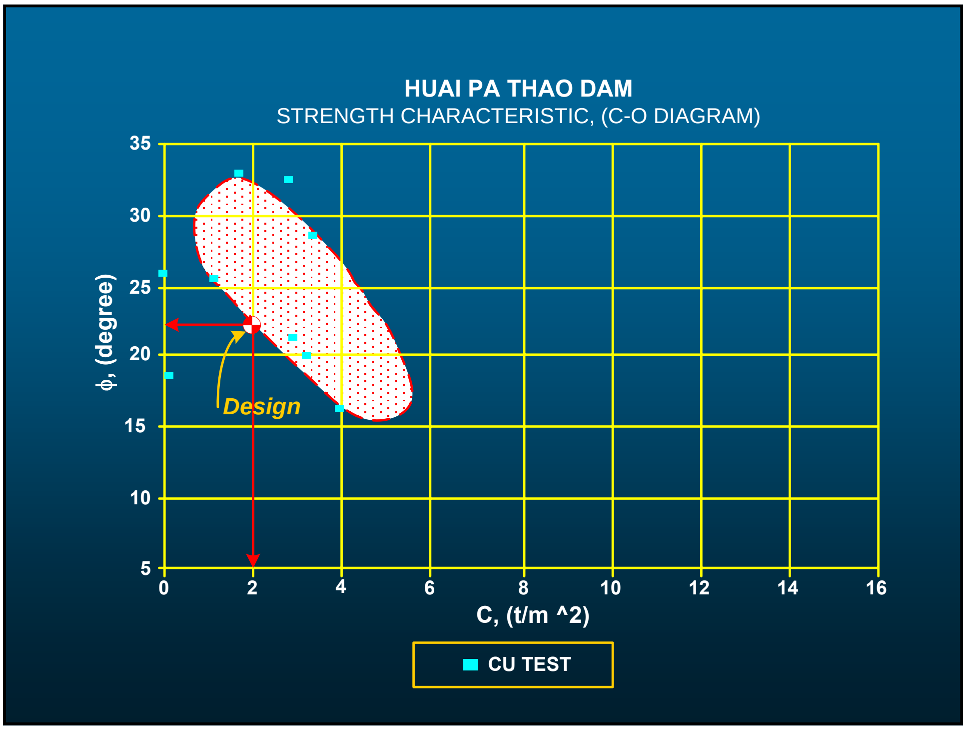
	UU-Test Unconsolidated Undrained Test	Consolidated Tests		
		Consolidation Phase	**CU-Test**	**CD-Test**
σ_3	Held Constant	Held Constant	Held Constant	Held Constant
σ_1	Gradually Increased From σ_3	Equal to σ_3 *	Gradually increased from σ_3	Very gradually increased from σ_3
u	Drainage Lines Closed	Drainage Lines Open **	No water permitted to escape. Pore pressure measured for effective stress tests.	Drainage lines open.

*** Unless anisotropic consolidation is to be effected**

**** In back pressured tests, pressure is supplied to pore lines, but drainage is permitted**


Cylinder for double acting piston
Mercury traps
Load measuring device
Specimen
Burette
Air pressure
Water supply
Null-indicator
Loading press
Pore pressure transducer
Constant pressure cell
Manometer
Screw control
Differential pressure manometer


HUAI PA THAO DAM
STRENGTH CHARACTERISTIC, (C-O DIAGRAM)
φ, (degree)
35
30
25
20
15
10
5
0
2
4
6
8
10
12
14
16
C, (t/m ^2)
Design
UU TEST



DRAINED STRENGTH

Drained and Undrained Strength

สภาพการเกิดความดันน้ำในมวลดิน ขณะที่มีการรับน้ำหนักจากภายนอกมีผลต่อกำลังรับน้ำหนักของดิน

Drained Strength เมื่อ Excess pore pressure (Δu) $\cong 0$

ตลอดช่วงระยะการรับน้ำหนัก หรือในช่วงเวลาที่ Δu ระบายออกจนหมดแล้วจะเกิดขึ้นใน 2 กรณี คือ

1. ดินทราย หรือ กรวด เมื่อ k สูง → Δu ระบายออกได้รวดเร็ว
2. ดินเหนียวที่มีการบรรทุกน้ำหนักช้า → Δu เกิดขึ้นน้อยและมีเวลาระบายได้

Undrained Strength เมื่อ Excess pore pressure (Δu) $\neq 0$

ในช่วงที่เกิดกำลังน้ำหนักสูงจนมีโอกาสพิบัติ หรือเสี่ยงต่อการพิบัติ

1. ดินเหนียว ที่มีการบรรทุกน้ำหนักเร็ว → Δu เกิดขึ้นและสะสม
2. ดินทรายละเอียด (Silt) มีการสั่นสะเทือนหรือมีแรงแผ่นดินไหว
   Δu เกิดในลักษณะ repeated load และสะสมจนเกิด Boiling

Mohr-Coulomb's Effective Strength Equation

$$\tau = \bar{c} + (\sigma - \underbrace{u_s}_{\text{Constant}} - \underbrace{\Delta u}_{\text{Varied}}) \cdot \tan \bar{\phi}$$

การศึกษา drained และ undrained strength คือการพิจารณาช่วงที่ Δu มีการเปลี่ยนแปลงในขณะบรรทุกน้ำหนักหรือขณะระบายน้ำแล้วทำให้ $\bar{\tau}$ (strength) ของดินเปลี่ยนแปลงไปด้วย

การอธิบายพฤติกรรมนี้สามารถทำได้สะดวกโดยใช้วิธี "Stress path"

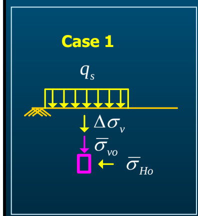
CONDITIONS OF STRESS CHANGE COMPARING TO TRIAXIAL TESTS

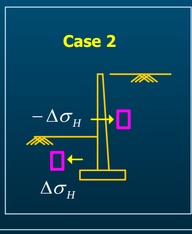
Path 1 Compression loading
σ_3 = const., σ_1 = increase

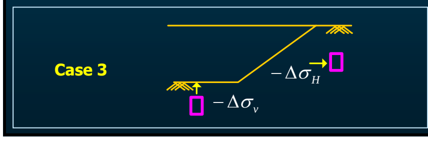
Path 2 Extension loading
σ_3 = increase , σ_1 = const.

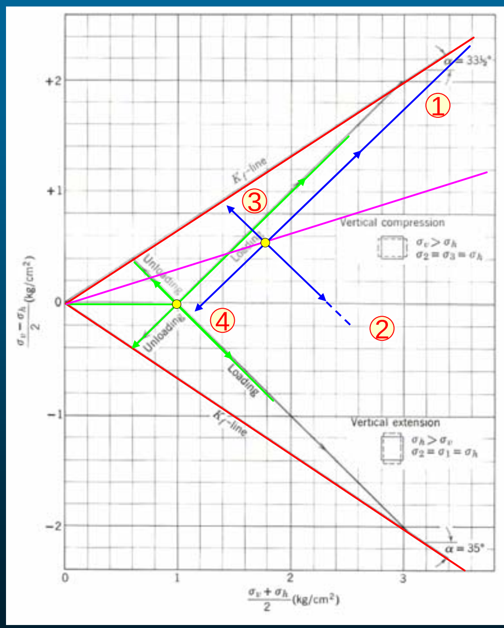
Path 3 Compression unloading
σ_3 = decrease , σ_1 = const.

Path 4 Extension unloading
σ_3 = const. , σ_1 = decrease









Stress paths for triaxial tests.

CD-TEST

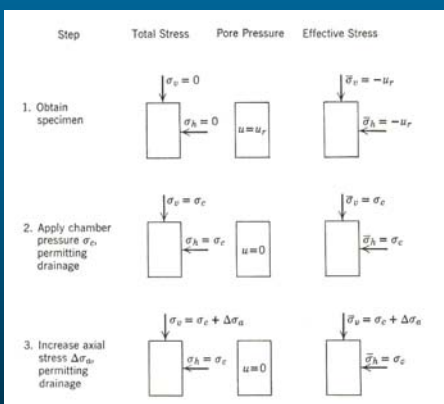
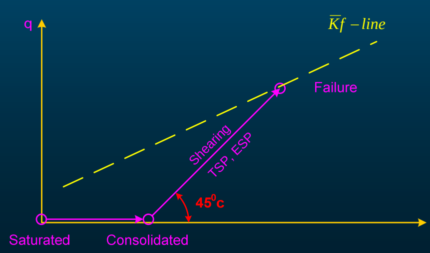


Fig. 28.3 Consolidated-drained (CD) triaxial test.



CU-TEST

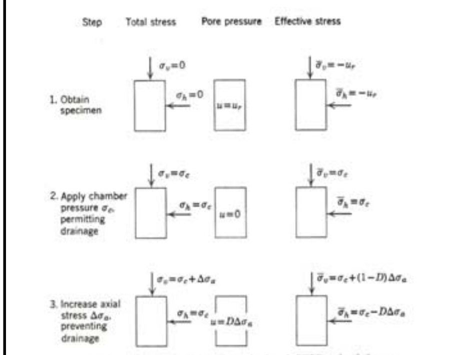
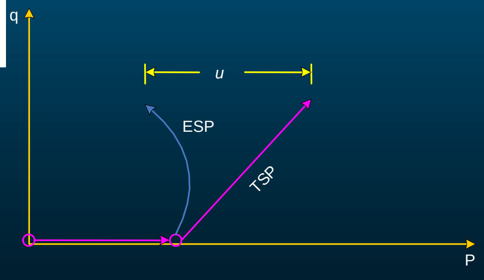


Fig. 28.5 Consolidated-undrained (CU) triaxial test.



UU Test

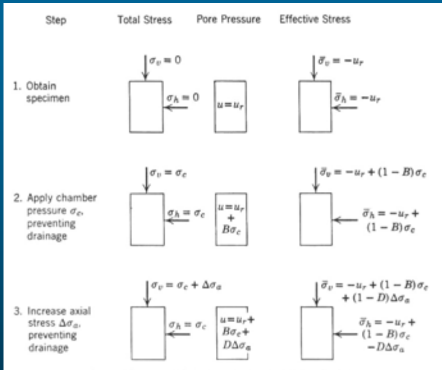
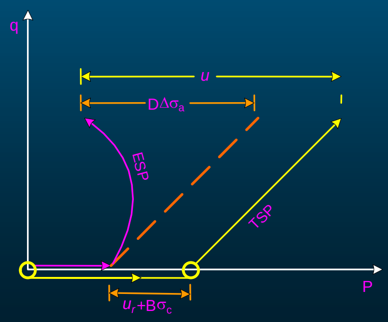
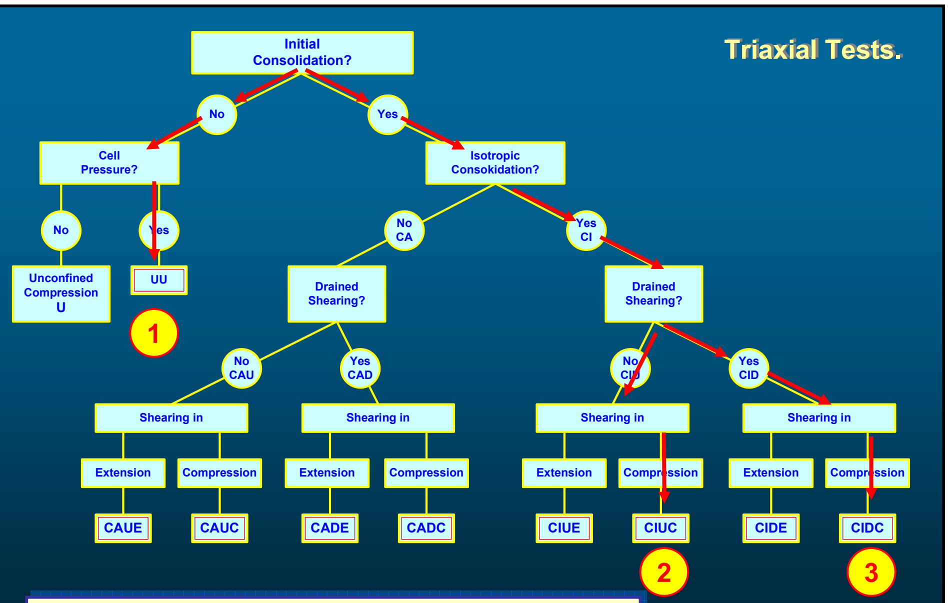


Fig. 28.4 Unconsolidated-undrained (UU) triaxial test.



Triaxial Tests.



① UU – Unconsolidated Undrained

② $\overline{\text{CU}}$ – Consolidated Undrained with pore pressure measurement

③ CD – Consolidated Drained.

DRAINED SHEAR STRENGTH

DRAINED SHEAR STRENGTH

I. FIELD CONDITION

- Granular materials , dry or partially saturated.
- Cohesive materials , slow rate of loading

II. LABORATORY CONDITION

- Slow rate of loading, drainage permitted, excess pore pressure (Δu) = 0

CD-Test or Slow Test

Case Study Triaxial Test on "Remolded Weald Clay" by Henkel (1956)

LL.	= 43%	Percent clay (<0.002)	= 40%
PL.	= 18%	Activity	= 0.6
PI.	= 25%	Specific gravity (G)	= 2.74

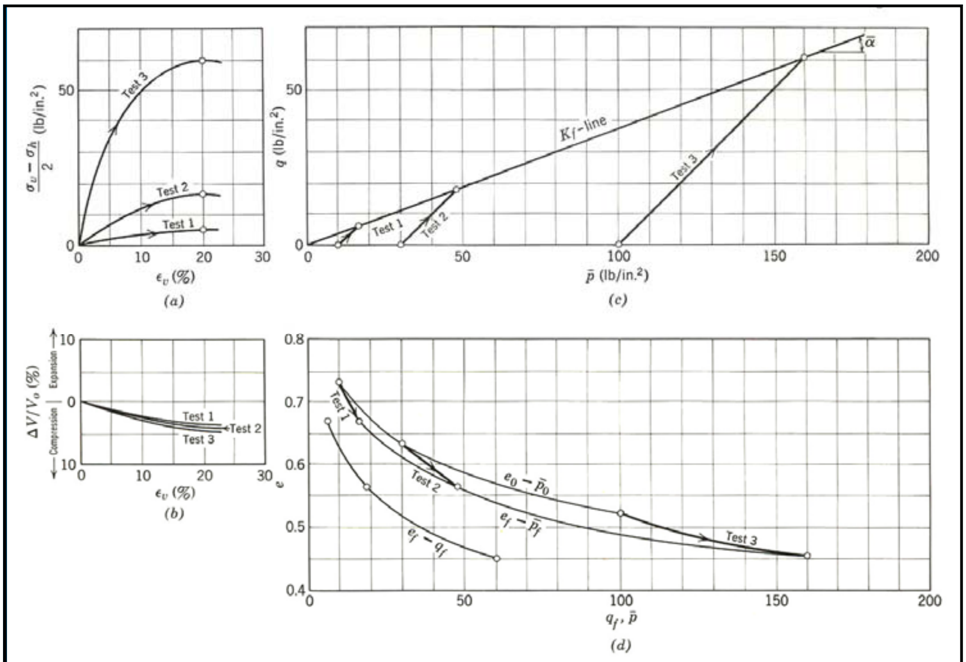
Case I. Normally Consolidated Clay (NC) , 3 Samples

Sample No.	1	2	3
Confining pressure (psi)	10	30	100

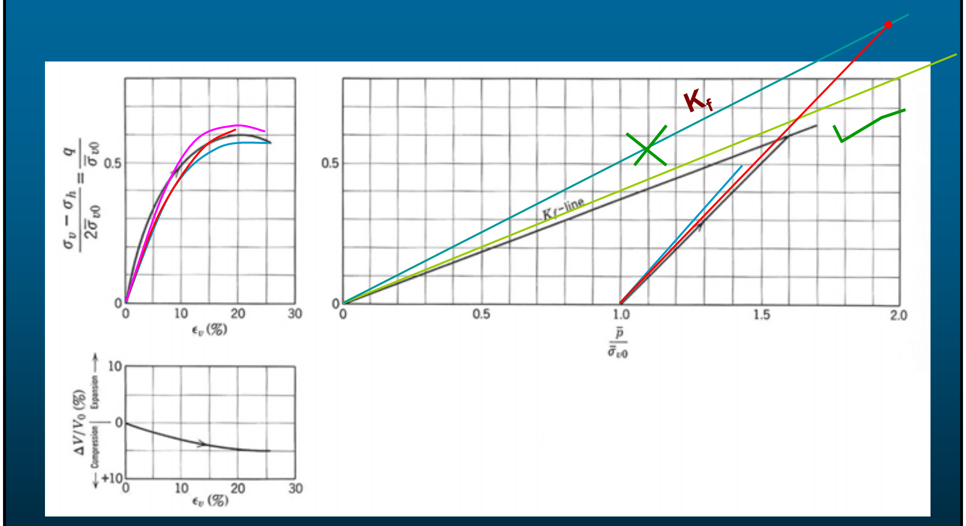
Case II. Over Consolidated Clay (OC) , 7 Samples

- Consolidated To = 120 psi
- Then rebound to test at

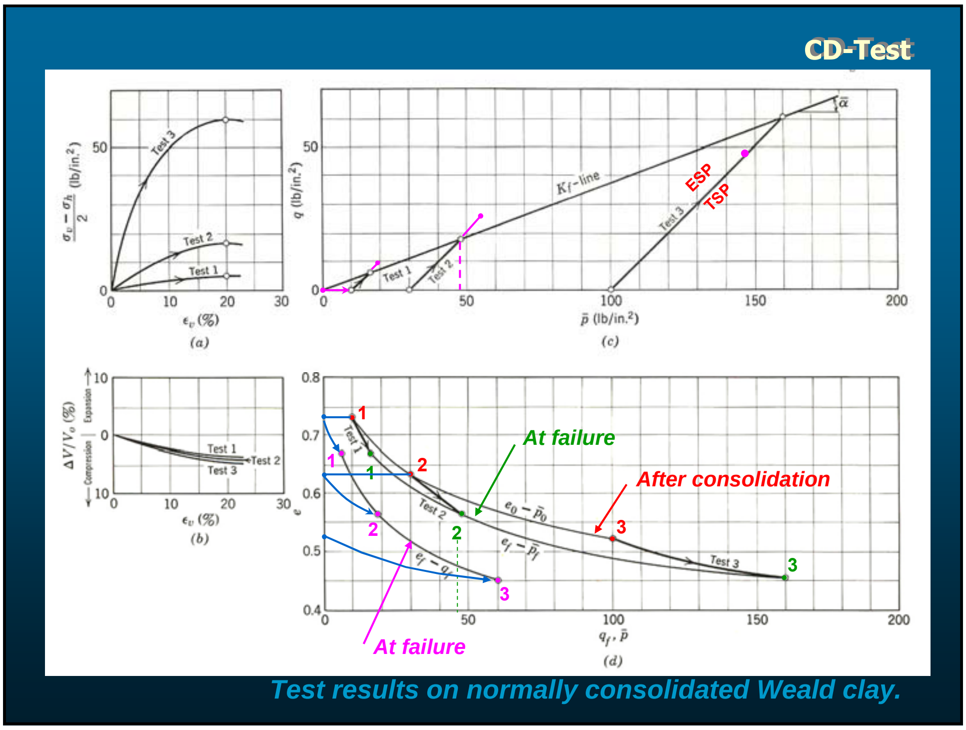
Sample No.	1	2	3	4	5	6	7
Confining p. (psi)	5	8	15	25	45	60	70
OCR.	24	15	8	4.8	2.7	2.0	1.7

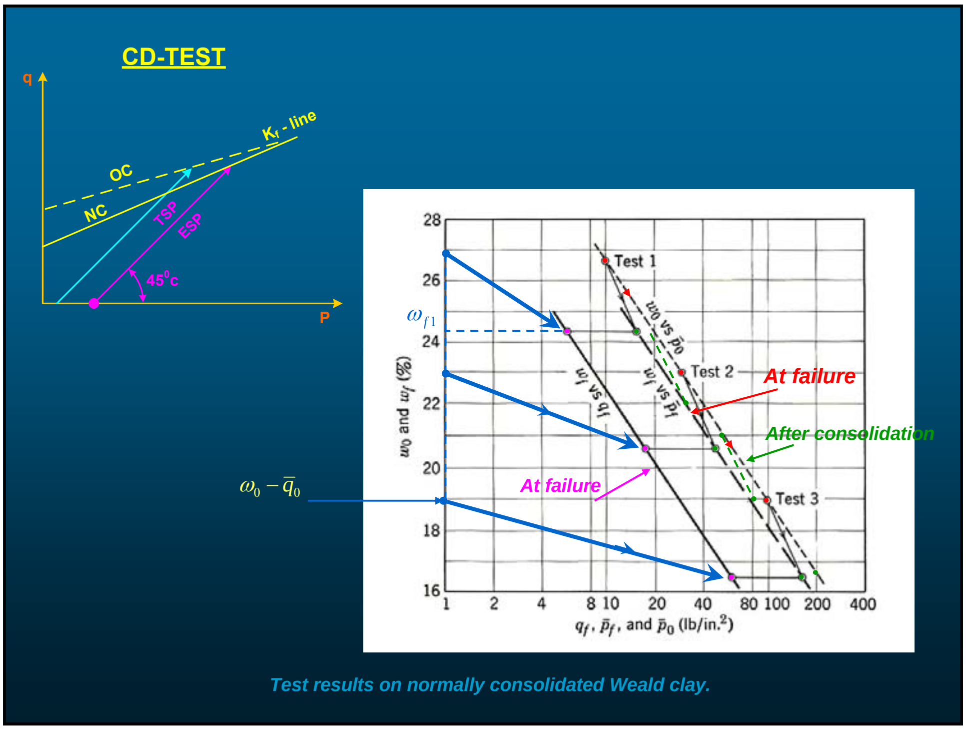


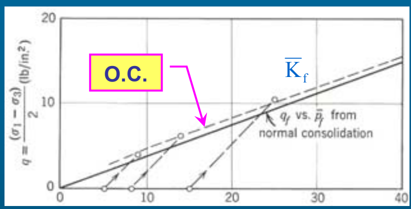
Test results on normally consolidated Weald clay.



Normalized test results.

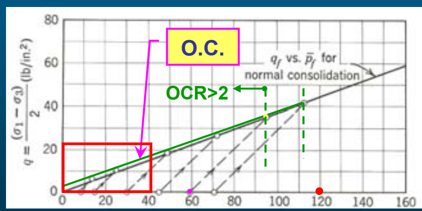

CD-Test
At failure
After consolidation
ESP
TSP
K_f-line
(a)
(b)
(c)
(d)
Test results on normally consolidated Weald clay.


CD-TEST
K_f - line
OC
NC
TSP
ESP
45°c
q
P
ω_{f1}
$\omega_0 - \bar{q}_0$
Test 1
Test 2
Test 3
At failure
After consolidation
$q_f, \bar{p}_f$, and $\bar{p}_0$ (lb/in.^{2})
Test results on normally consolidated Weald clay.



① ② ③

$\bar{p} = \frac{(\bar{\sigma}_1 + \bar{\sigma}_3)}{2}(lb/in.^2)$

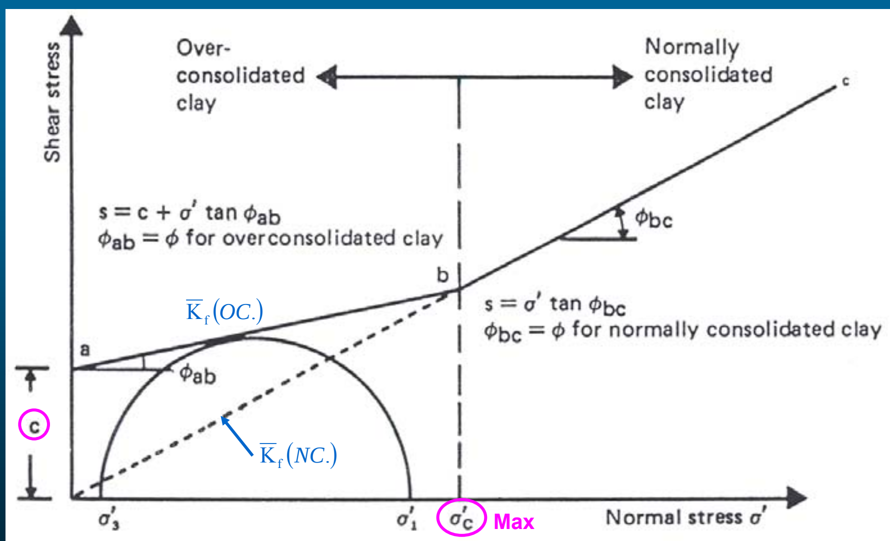


③ ④ ⑤ ⑥⑦

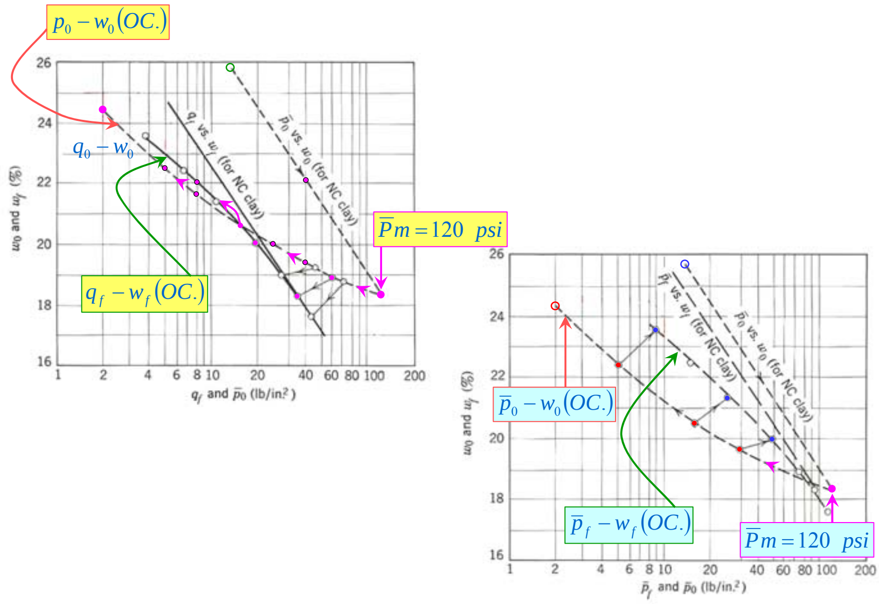
$\bar{p} = \frac{(\bar{\sigma}_1 + \bar{\sigma}_3)}{2}(lb/in.^2)$

Results of CD tests on overconsolidated Weald clay.

$\bar{p}_m = 120 \ lb/in.^2$



Failure envelope of a clay with preconsolidation pressure = $\sigma_c{}'$



Stress-volume relations for overconsolidated Weald $\bar{p}_m = 120\ lb/in.^2$

Summary of CD-Test Resuts of Weald Clay

Unique Characteristic of $q_f - \bar{p}_f - w_f$ **Relationship**

For Normally Consolidated Clay (NC)

For each soil , when CD-Test was test then the following results can be summarized

1. The water content after consolidation is linearly proportioned to log p_0. Consolidation pressure.

Curve : W_o – log (p_o) is straight line

2. After drained shearing of soil samples, the final water contents (W_f) at failure are linearly proportioned to g_f and $\bar{p}_f$ (in log. Scale)

W_f – log(g_f) are straight lines

W_f – log(p_f) Parallel to each others

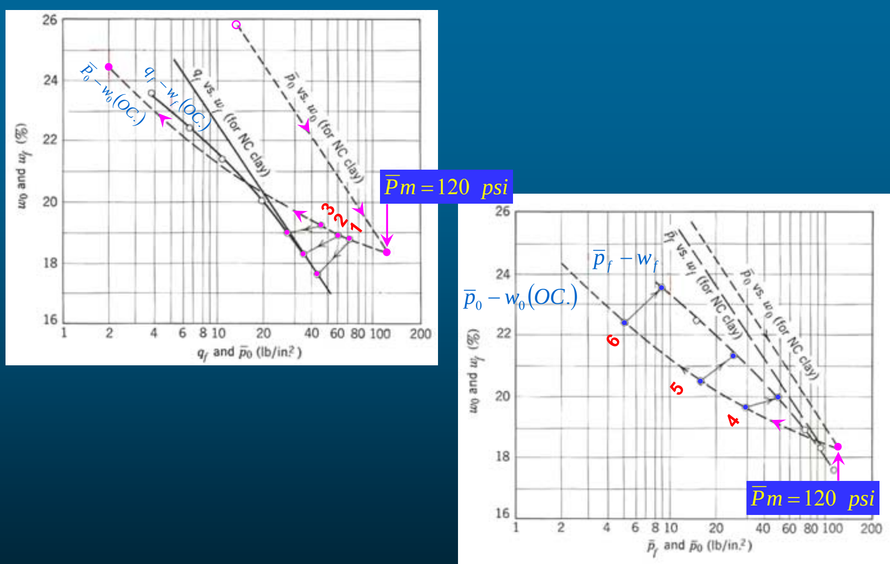
For Overconsolidated Clay (O.C)

3. After preconsolidation pressures, the samples were rebound to a curtain confining pressure. The w_o - will show the lower value than w_o of N.C. due to OC. Clay has inelastic property of soil.

4. The unique but not linear relationships between

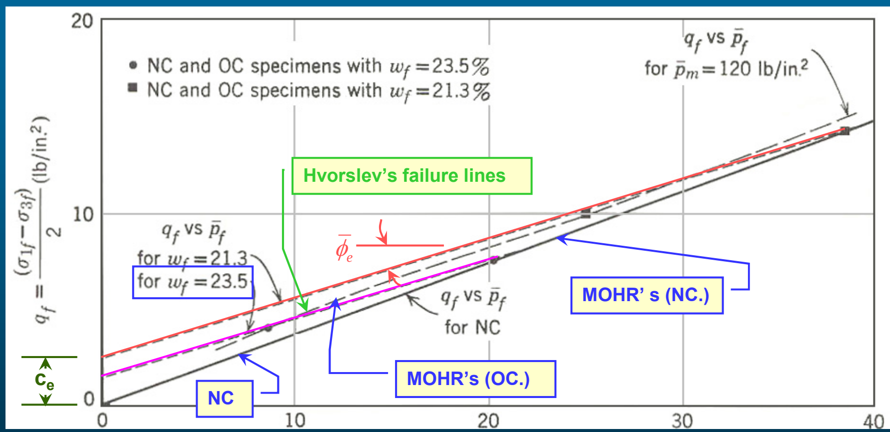
$$w_o - \log(p_o), \qquad w_f - \log(q_f), \qquad w_f - \log(\bar{p}_f)$$

were obtained for each soil at each $\bar{P}_m$



Stress-volume relations for overconsolidated Weald $\bar{p}_m = 120\ lb/in.^2$

Hvorslev's parameters.



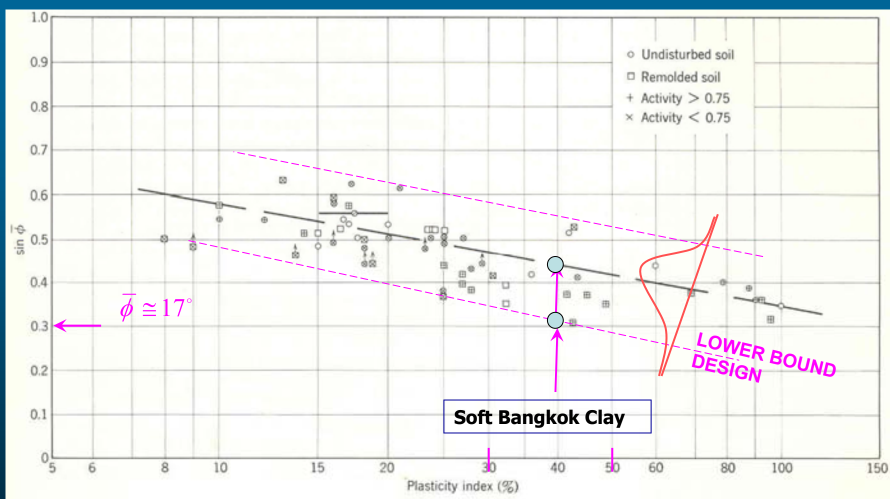
$$\bar{p}_f = \frac{(\bar{\sigma}_{1f} + \bar{\sigma}_{3f})}{2} (lb/in.^2)$$

Hvorslev's Strength Parameter (1937)

- Combine effect of NC. And OC. to w_f and $\bar{\sigma}_f$

$$\tau_f = f(w_f) + f(\bar{\sigma}_f)$$

$$\bar{\tau}_f = \bar{c}_e + \bar{\sigma}_f \cdot \tan\bar{\phi} \quad @ \quad W_{fi}$$



Relationship between sin ϕ and plasticity index for normally consolidated soils (From Kenney, 1959).

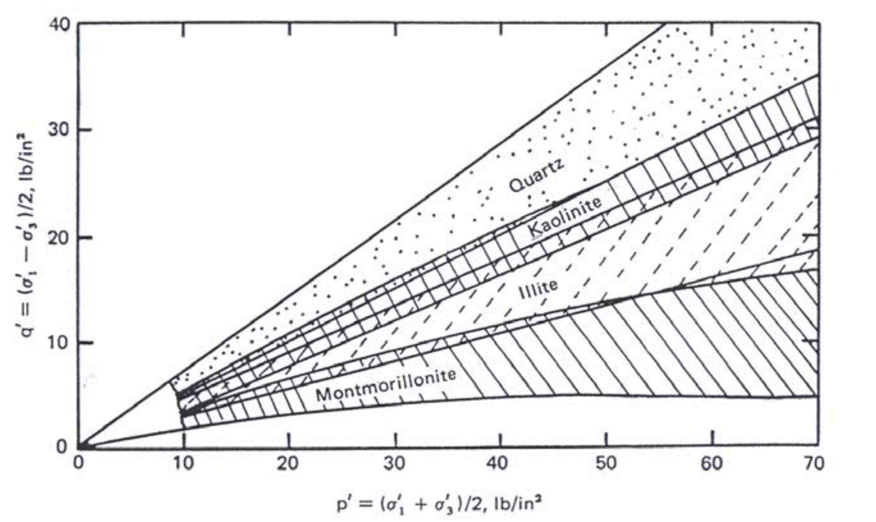


Fig. 7.24 Modified Mohr's failure envelope for quartz and clay minerals. (Note: 1 lb/in² = 6.9 kN/ n.) (*Replotted after R. E. Olson, Shearing Strength of Kaolinite, Illite and Montmorillonite,* J. Geotech. Eng. Div., ASCE, *vol. 100, no. GT11, 1974.)*

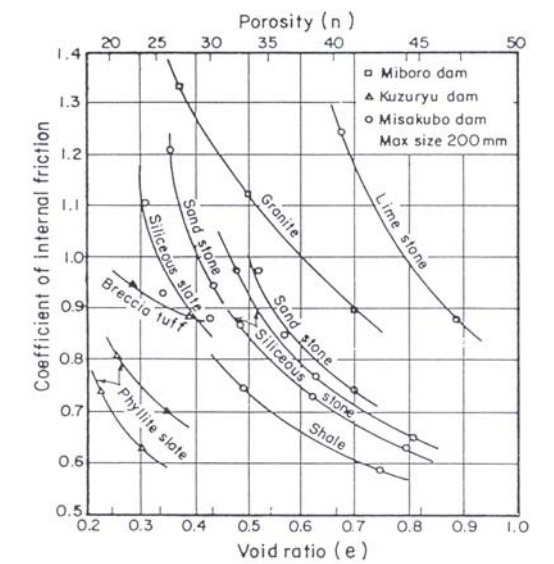


Fig. 11. Relationship between void ratio and tan ϕ of various rock materials.

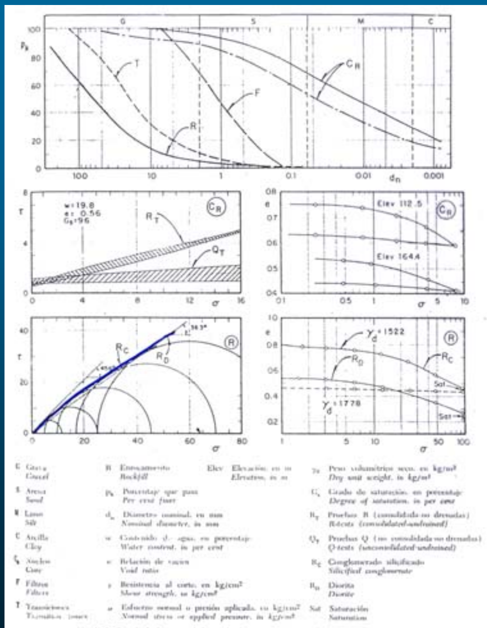
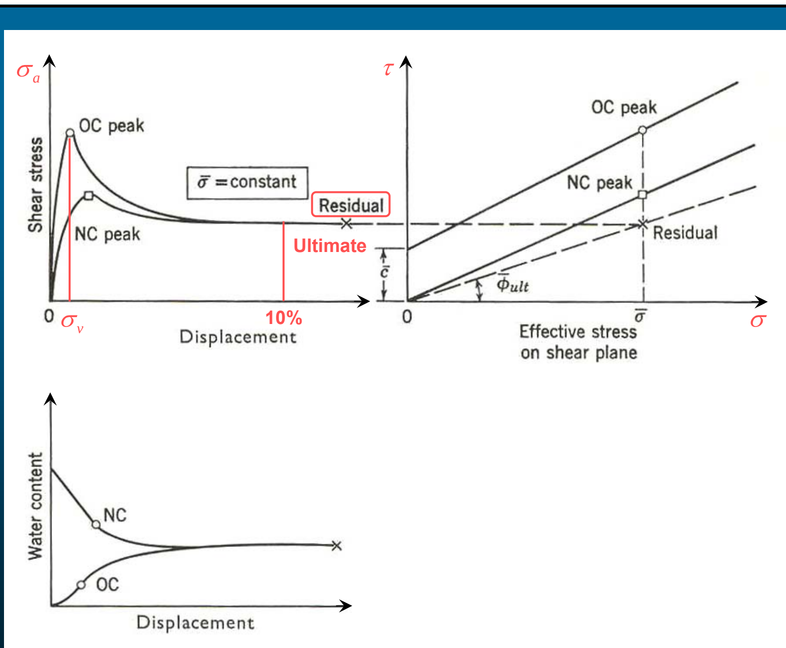


Fig 7.9 Propiedades mecánicas de los materiales colocados en la presa
Mechanical properties of materials for the dam



Relationship between peak and ultimate conditions.

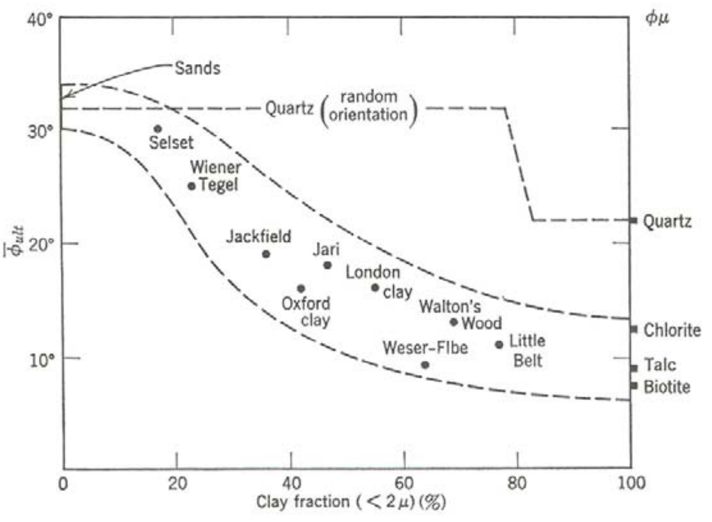
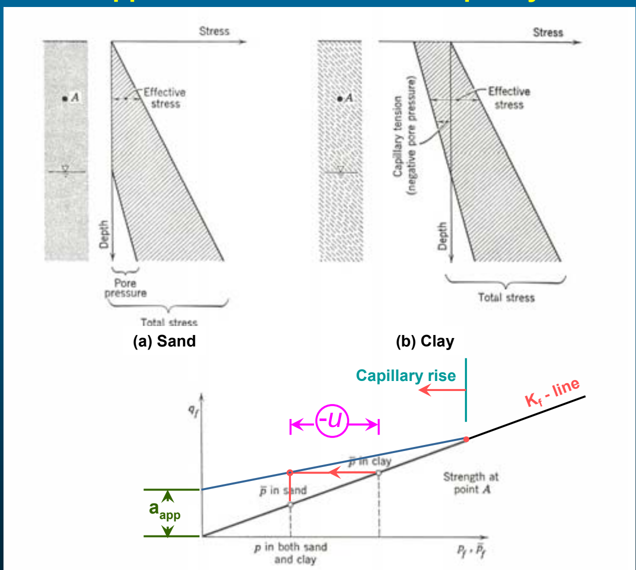
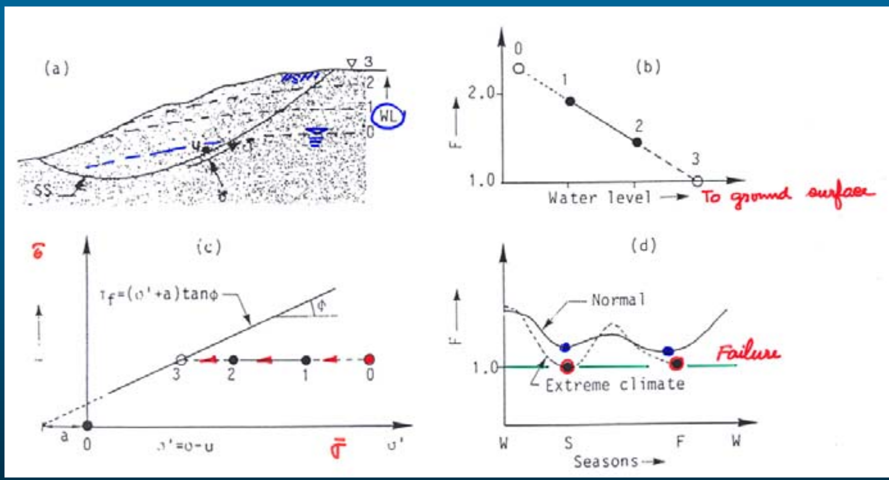


Fig. 21.14 Relations between $\bar{\phi}_{ult}$ and clay content (From Skempton, 1964).

Apparent Cohesion due to capillary



Effect of capillary tensions on effective stress and strength. (a) Sand. (b) Clay.



Effect of variable ground water conditions.

UNDRAINED SHEAR STRENGTH

Undrained Shear Strength

Sort strength is highly depended on the drainage condition in soil mass. During undrained condition when excess pore pressure is fully developed and no time to dissipate, the strength is called **"undrained strength"**

Normally, the undrained strength is lower than drained strength due to the present of pore pressure. The behavior of undrained strength can also explain by the theory of effective stress and represented by **"Stress path"**

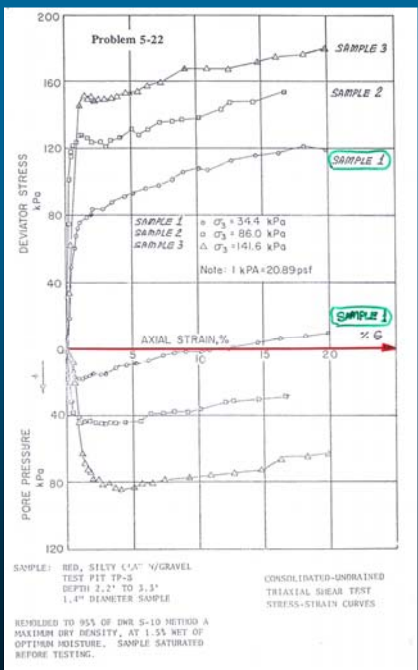
1. Theoritical or laboratory
   Fully undrained conditions can be simulated.

2. Practical or field
   Partially drained condition is usually occurred.

We should consider the worst case for the design.

Loading → drained strength higher then undrained strength is used.

Unloading → drained strength is lowest.



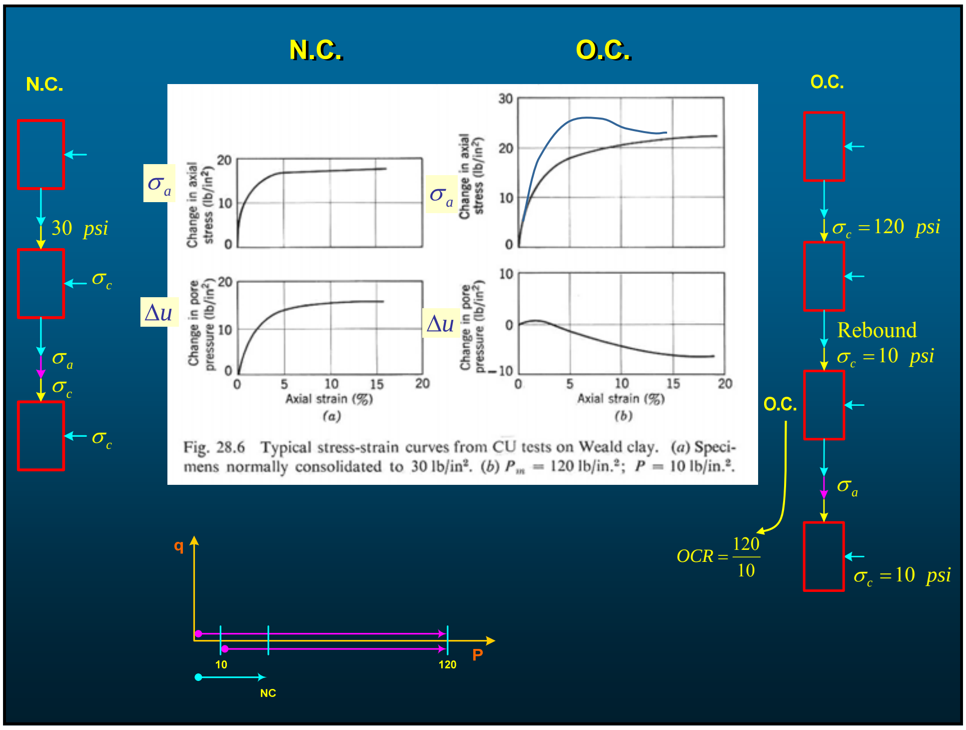


Fig. 28.6 Typical stress-strain curves from CU tests on Weald clay. (a) Specimens normally consolidated to 30 lb/in². (b) P_m = 120 lb/in.²; P = 10 lb/in.².

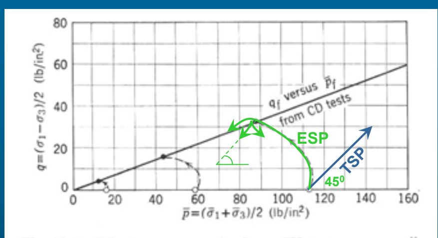


Fig. 28.7 Effective stress paths from CU tests on normally consolidated Weald clay.

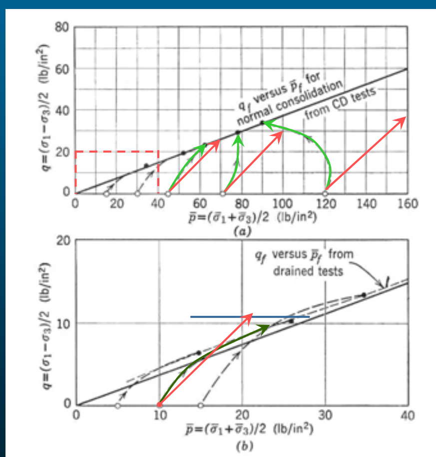


Fig. 28.8 Effective stress paths from CU tests on overconsolidated Weald clay. $\bar{p}_m$ = 120 lb/in.².

▸ Example 28.1

Given. Normally consolidated Weald clay with $\bar{p}_0 = 30$ psi.
Find. σ_1, $\bar{\sigma}_1$, σ_3, $\bar{\sigma}_3$, and u when
a. $q = 5$ psi.
b. q is at its peak value.
Solution. Figure E28.1 is a blown-up version of the q versus $\bar{p}$ diagram of Fig. 28.7, using an effective stress path interpolated between those for $\bar{p}_0 = 16$ lb/in.² and $\bar{p}_0 = 59$ lb/in².

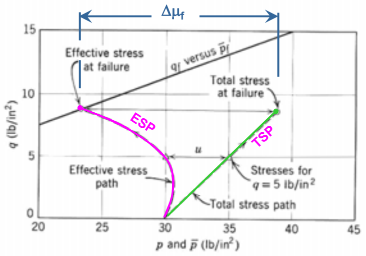


Fig. E28.1

Stress (lb/in.²)	(a)	(b)	
q	5	8.7	given
p	35	38.7	from graph
σ_1	40	47.4	$p + q$
σ_3	30	30.0	$p - q$
$\bar{p}$	30	23.3	from graph
$\bar{\sigma}_1$	35	31.0	$\bar{p} + q$
$\bar{\sigma}_3$	25	14.6	$\bar{p} \mp q$
$u = \Delta u$	5	15.4	$p - \bar{p}$
A	0.50	0.89	$\Delta u / \Delta(\sigma_1 - \sigma_3)$

$A = \frac{\Delta u}{2q}$

$A_f = \frac{\Delta u_f}{2q_f}$

Example 28.2

Given. A specimen of Weald clay with $\bar{p}_m = 120$ psi and $\bar{p}_0 = 10$ psi. → $OCR = \frac{120}{10} = 12$
Find. σ_1, $\bar{\sigma}_1$, σ_3, $\bar{\sigma}_3$, u, and A at failure.
Solution. Figure E28.2 is copied from Fig. 28.8.

$$
\begin{aligned}
q_f &= 10.2 \text{ lb/in.}^2 \\
p_f &= 20.2 \\
\sigma_{1f} &= 30.4 \\
\sigma_{3f} &= 10.0 \\
\bar{p}_f &= 26.0 \\
\bar{\sigma}_{1f} &= 36.2 \\
\bar{\sigma}_{3f} &= 15.8 \\
u_f &= -5.8 \\
A_f &= -0.28
\end{aligned}
$$

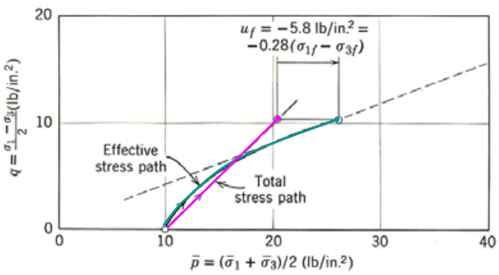


Fig. E28.2

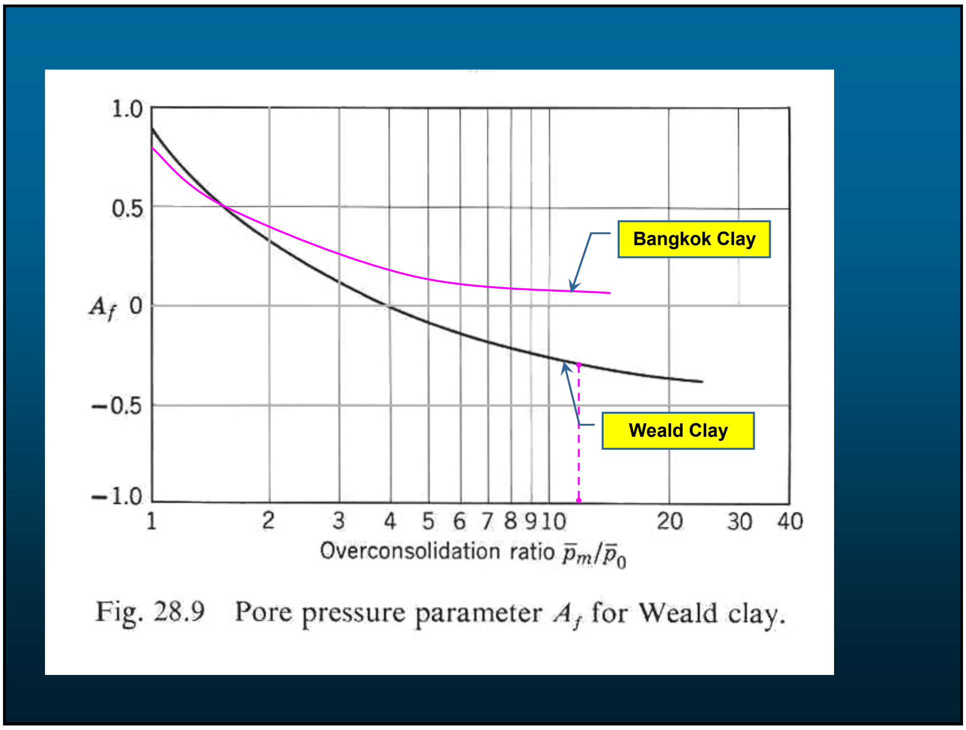


Fig. 28.9 Pore pressure parameter A_f for Weald clay.

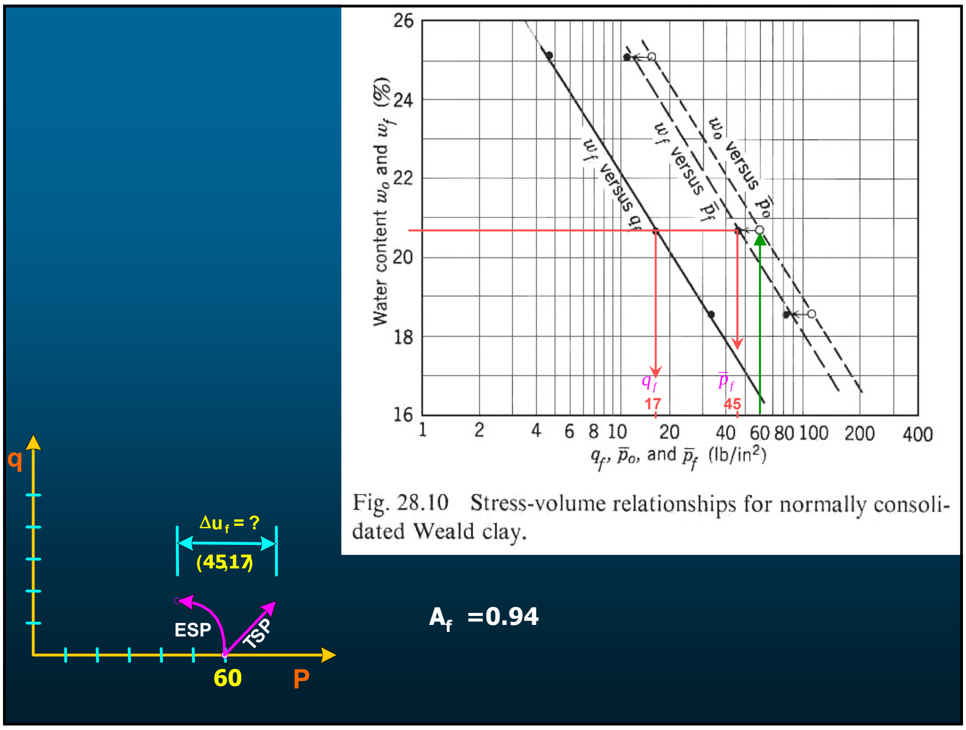


Fig. 28.10 Stress-volume relationships for normally consolidated Weald clay.

▸ **Example 28.3**

Given. Normally consolidated Weald clay with $\bar{p}_0 = 30$ psi.

Find. q_f and w_f for both drained and undrained shear with σ_1 increasing while σ_3 remains contants.

Solution.

Drained shear: Construct the effective stress path and find $\bar{p}_f$ and q_f. Then find w_f using either the $\bar{p}_f$ versus w_f or q_f versus w_f relations (see Fig. E28.3).

$$\bar{p}_f = 48 \text{ lb/in.}^2, \qquad \bar{q}_f = 18 \text{ lb/in.}^2, \qquad w_f = 20.6\%$$

Undrained shear: Enter the stress-volume diagram with the given $\bar{p}_0$ and find w_f. From q_f versus w_f, find q_f. $\bar{p}_f$ can be found from q_f versus $\bar{p}_f$.

$$w_f = 23.0\%, \qquad q_f = 8.7 \text{ lb/in.}^2, \qquad \bar{p}_f = 23.5 \text{ lb/in.}^2$$

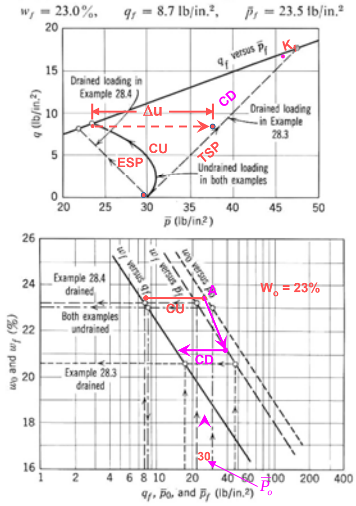


Fig. E28.3

CD Test

q_f = 17.5 psi

from $w_f - q_f -$

find w_f = 20.6%

w_o = 23%

CU Test

q_f = 8.5 psi

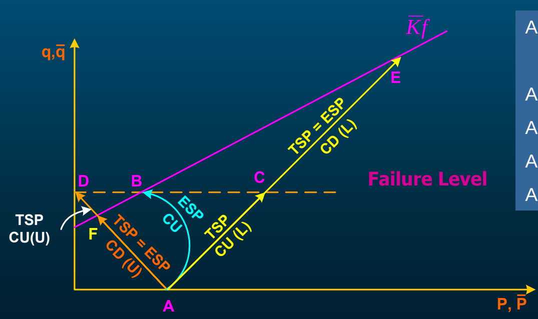
w_f = 23%

▸ **Example 28.4**

Repeat Example 28.3 with σ_3 decreased and σ_1 constant.

Solution. Follow same steps as in Example 28.3. The undrained strength is the same for both examples. Note also that $q_f = 0.29\bar{p}_0$.

	$\bar{p}_f$ (lb/in.²)	w_f (%)	q_f (lb/in.²)
Drained loading	48	20.6	18
Undrained loading and unloading	23.5	23.0	8.7
Drained unloading	22	23.2	8.0



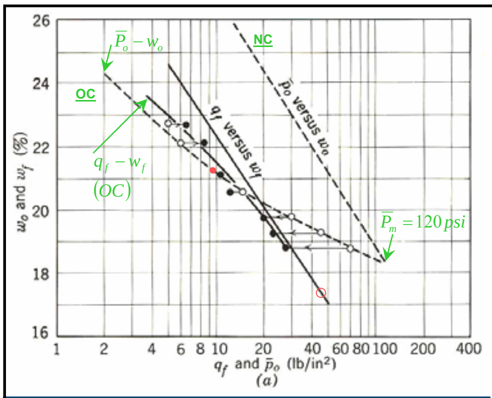
AB = ESP for Undrained T.A. $(\overline{CU})$ - Loading and Unloading.

AC = TSP for CU – Loading

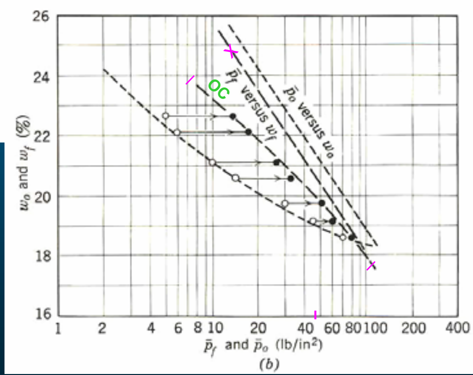
AE = TSP = ESP for CD – Loading

AF = TSP = ESP for CD – Unloading

AD = TSP for CU – Unloading.



OC-Clay → Consolidate – pm = 120 psi



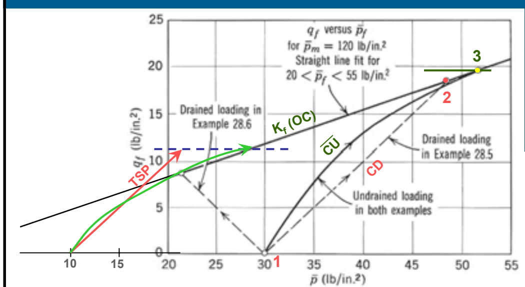
Stress-volume relations for over-consolidated Weald clay $\overline{p}_m = 120\ lb/in.^2$

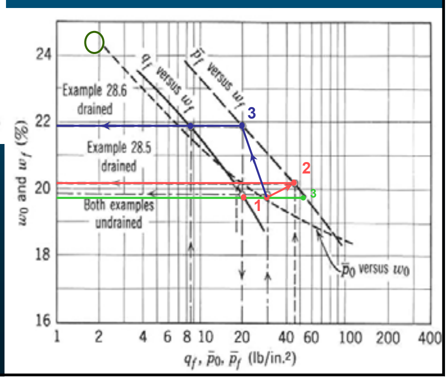
Example 28.5

Given. Overconsolidated Weald clay with $\overline{p}_m = 120\,psi$ *and* $\overline{p}_o = 30\,psi$

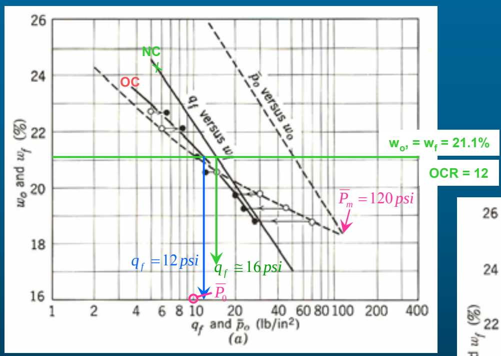
Find. q_f and w_f for both drained and undrained shear with σ_1 , increasing while σ_3 remains constant.

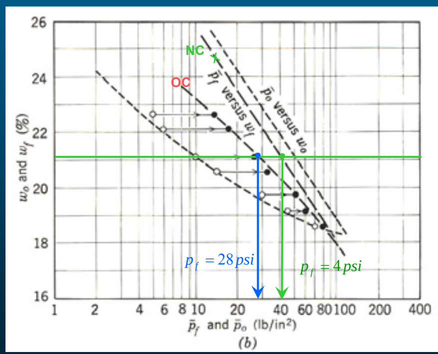
Solution. Follow same steps as in Example 28.3. The diagrams are given in Fig. E28.5 and the answers appear in the table in Example 28.6





If $\bar{P}_o$ = 10 psi → Construct ESP, TSP, Find Δu, A_f



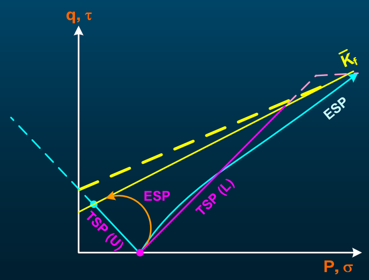


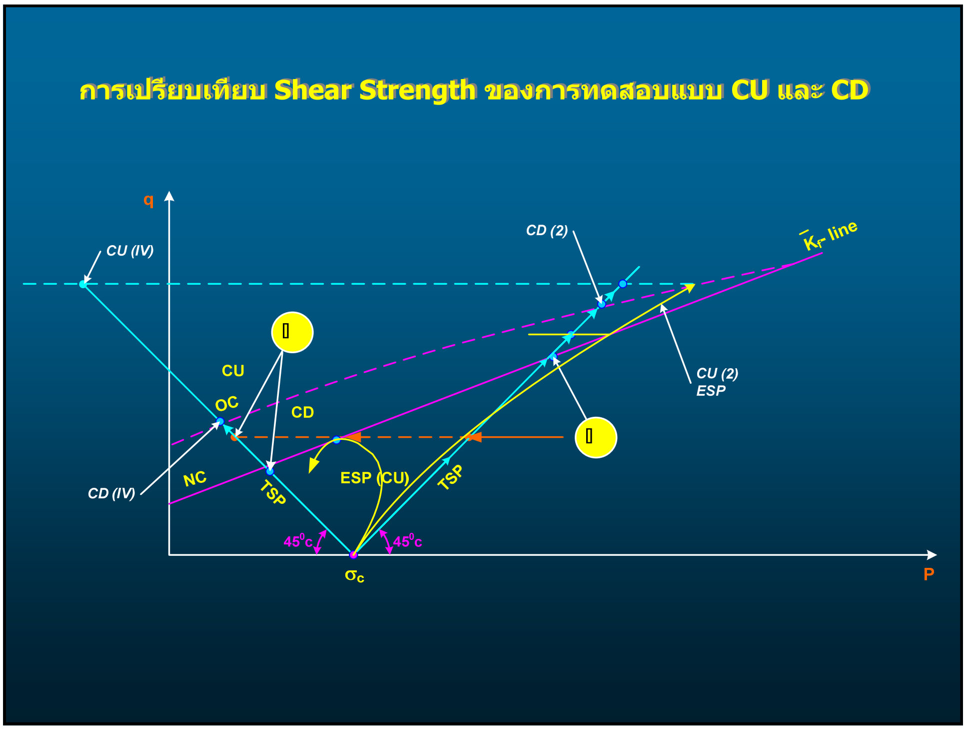
Stress-volume relations for overconsolidated Weald clay $\bar{p}_m = 120\ lb/in.^2$

Relative Magnitude of Drained and Undrained Strength

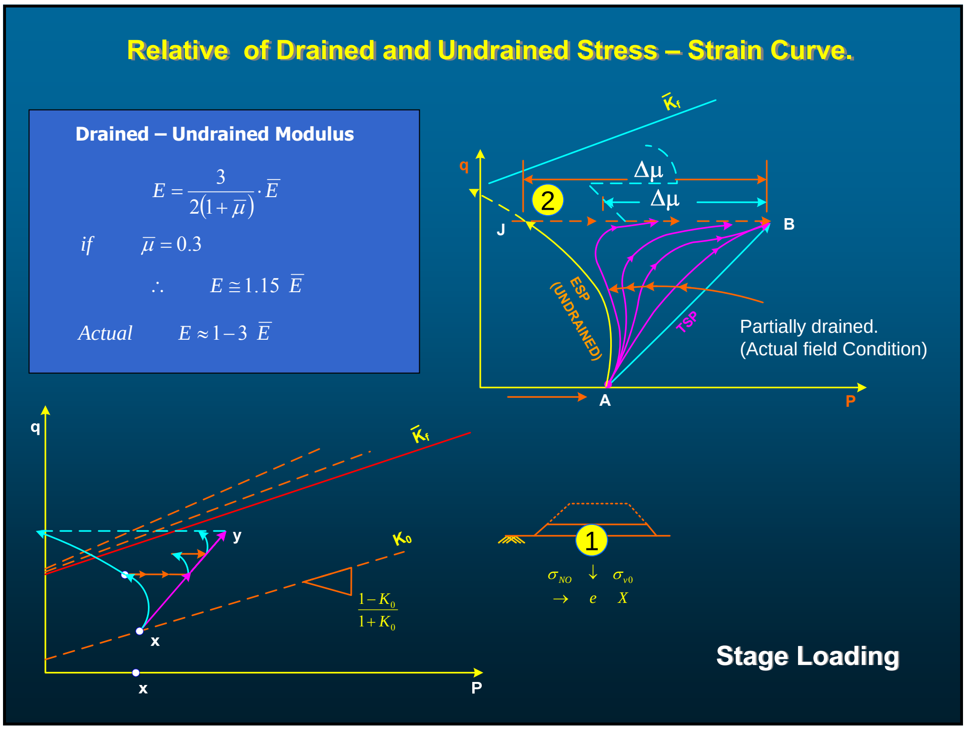
	Normally Consolidated Clay	Heavily Overconsoldated Clay
Triaxial compression loading (σ_1 increasing with σ_3 constant)	1 CD > CU	2 CU ≈ CD
Triaxial Compression unloading (σ_1 constant with σ_3 decreasing)	3 CU ≈ CD	4 CU >> CD

Note. These comparisons apply for specimens with the same initial effective stress.





การเปรียบเทียบ Shear Strength ของการทดสอบแบบ CU และ CD
q
CU (IV)
CD (2)
$\bar{K}_f$ - line
CU
OC
CD
CU (2)
ESP
CD (IV)
NC
TSP
ESP (CU)
TSP
45^0c
45^0c
σ_c
P



Relative of Drained and Undrained Stress – Strain Curve.
Drained – Undrained Modulus
$E = \frac{3}{2(1+\bar{\mu})} \cdot \bar{E}$
if $\bar{\mu} = 0.3$
$\therefore \quad E \cong 1.15 \ \bar{E}$
Actual $E \approx 1-3 \ \bar{E}$
$\bar{K}_f$
q
$\Delta\mu$
2
$\Delta\mu$
J
B
ESP (UNDRAINED)
TSP
Partially drained.
(Actual field Condition)
A
P
q
$\bar{K}_f$
y
K_0
x
$\frac{1-K_0}{1+K_0}$
x
P
1
σ_{NO} σ_{v0}
e X
Stage Loading

หาความสัมพันธ์ของ Drained และ Undrained Modulus

If **E = Undrained Young's Modulus**

E = Drained Young's Modulus

While is $E = f(\overline{E})$

From Eq. 12.5 (a) for Triaxial Loading $E_x = \frac{1}{E}\left[\sigma_x - u\left(\sigma_y + \sigma_z\right)\right]$

For Total Stress - Vertical loading $\sigma_x = \sigma_v,\ \sigma_y = \sigma_z = 0$

$$E_v = \frac{1}{E}\sigma_v \qquad \text{...(1)}$$

For Effective Stress - vertical loading $\overline{\sigma}_x = \overline{\sigma}_v,\ \overline{\sigma}_y = \overline{\sigma}_z = \overline{\sigma}_h = -\mu$

$$E_v = \frac{1}{E}\left(\sigma_v - 2\overline{\mu}\,\overline{\sigma}_n\right) \qquad \text{...(2)}$$

When $\overline{u}$ = Poisson's ratio of mineral skeleton.

For Isotropic mineral skeleton and axial loading from

Eq. 26.5 $A = \frac{1}{1+2\left(C_s/C_c\right)} \xrightarrow{C_s=C_c} A = \frac{1}{3}$

Then

$$\overline{\sigma}_v = \sigma_v - A\sigma_v = \sigma_v - \frac{1}{3}\sigma_v = \frac{2}{3}\sigma_v \qquad \text{...(3)}$$

$$\overline{\sigma}_h = 0 - A\sigma_v = -\frac{\sigma_v}{3} \qquad \text{...(4)}$$

From Eq. (2)

$$G_v = \frac{1}{E}\left(\frac{2}{3}\sigma_v + \frac{2}{3}\overline{\mu}\sigma_v\right) = \frac{2}{3}\frac{\sigma_v}{\overline{E}}\left(1+\overline{\mu}\right) \qquad \text{...(5)}$$

Or (1) = (5) Then

$$E = \frac{3}{2\left(1+\overline{\mu}\right)}\overline{E} \qquad \text{...(6)}$$

Final Strains are depended on stress-path

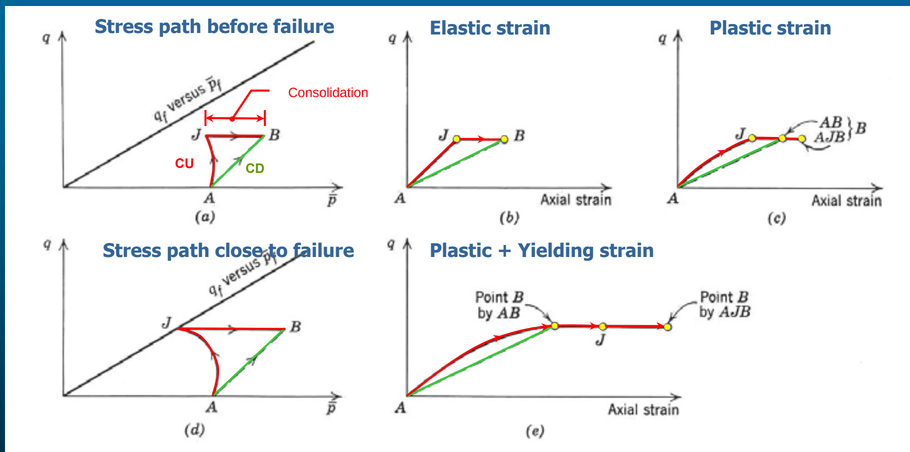
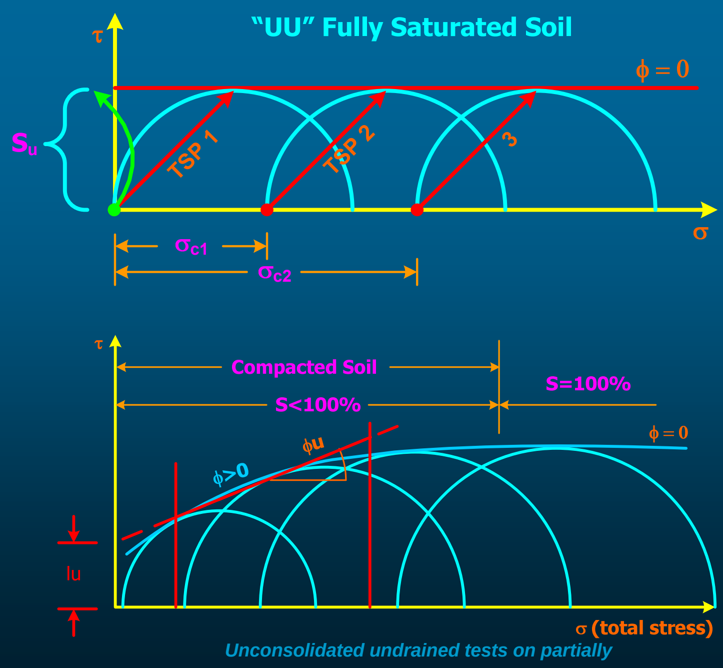
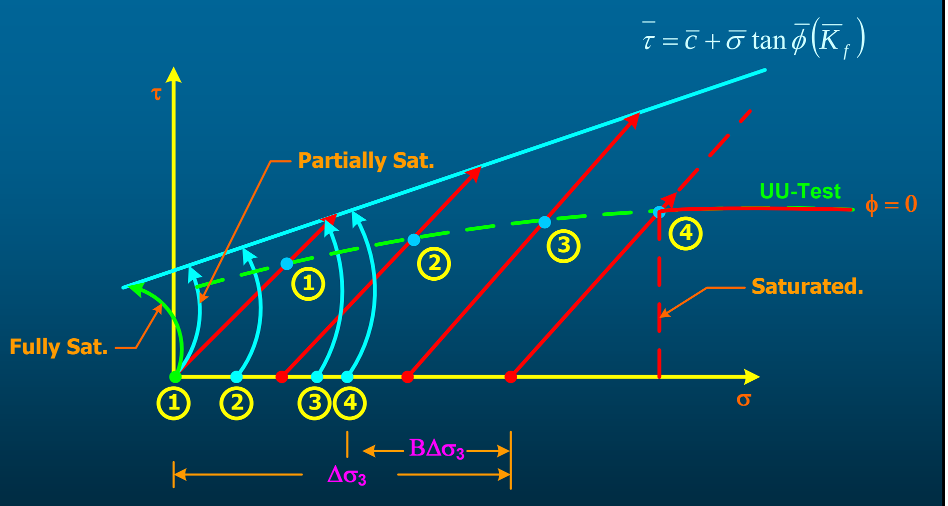


Fig. 28.15 Axial strains as a function of stress path. (*a*) Non-failure loading. (*b*) Elastic response to non-failure loading. (*c*) Nonelastic response to non-failure loading. (*d*) Failure loading. (*e*) Response to failure loading.

1. Larger $\bar{p}$ during loading → cause smaller final strain (E_a)
2. Stress-path (ESP) closer to K_f - line → cause larger final strain due to plastic + yielding behaviors.





Unconsolidated undrained tests on partially

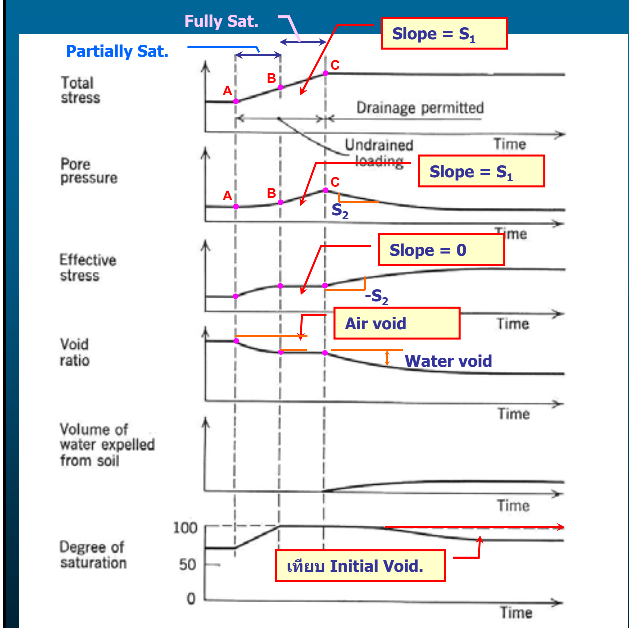


Fig. 28.16 Compression of partially saturated soil in an oedometer.

Effect of Partially saturated soil ต่อ
- **Strength**
- **Deformation**
- **Consolidation**
- **% Saturation**

ในระหว่าง Undrained - Drained Loading.

Application
1. **Compacted embankment**
2. **Earth dam**
3. **Landfill**

Undrained Shear Strength

1. Undrained Shear Strength of Saturated Sand

Undrained loading on saturated sand during fast or repeated loading conditions

Generally

1. **Loose sand similar to N.C. or Soft Clay**
2. **Dense sand similar to O.C. or Stiff Clay**

Exception

For loose sand after peak, soil can maintain it failure condition and pore pressure start to decrease due to dilatency effect.

2. Cavitation of Pore Water

For dense sand during searing, soil mass tend to expound and develop negative pore pressure.

If $u < -1.0$ ATM. (-14.4 psi), then pore water will cavilate

Solution

Back pressure of about 1 ATM (or more) is applied in soil sample and confining pressure.

Medium Dense Sand --> No Cavitation

$\phi = 0$

Dense Sand -->Cavitation @ Low Confining Pressure

Unsaturated due to cavitation

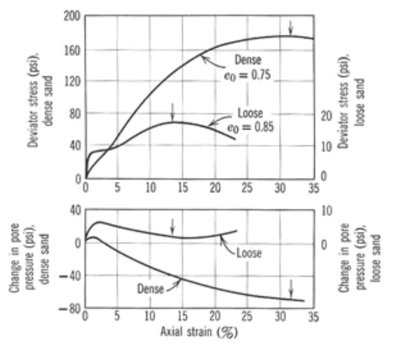


Fig. 29.1 Stress-strain curves for undrained triaxial compression of a saturated sand (from Leonards, 1962).

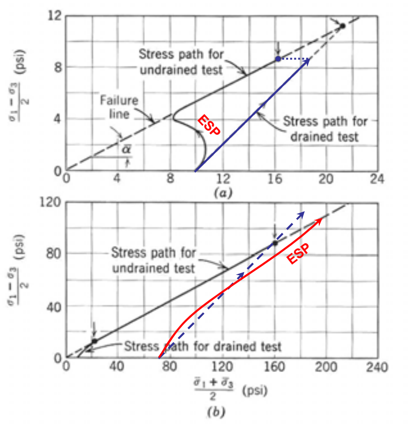


Fig. 29.2 Stress-paths for undrained triaxial compression of a saturated sand. (a) Loose, $e_0 = 0.85$. (b) Moderately dense, $e_0 = 0.75$.

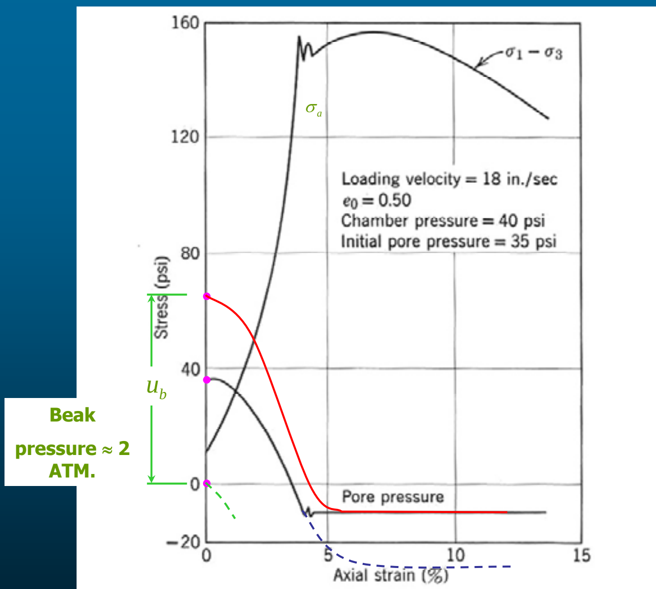
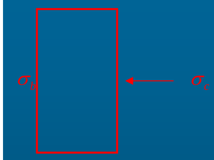


Fig. 29.3 Cavitation during undrained triaxial test of a dense sand (From Whitman and Healy, 1962).



$\bar{\sigma}_c = \sigma_c - \sigma_b = (\sigma_c + \Delta\sigma_c) - (\sigma_b + \Delta\sigma_b)$

3. Sensitive Clay and Very Loose Sands.

The Collapsing soil structure results the rearrangement of soil particles after peak strength. Then, the excess pore pressure continue to increase and cause the ultimate K-line larger than peak K-line

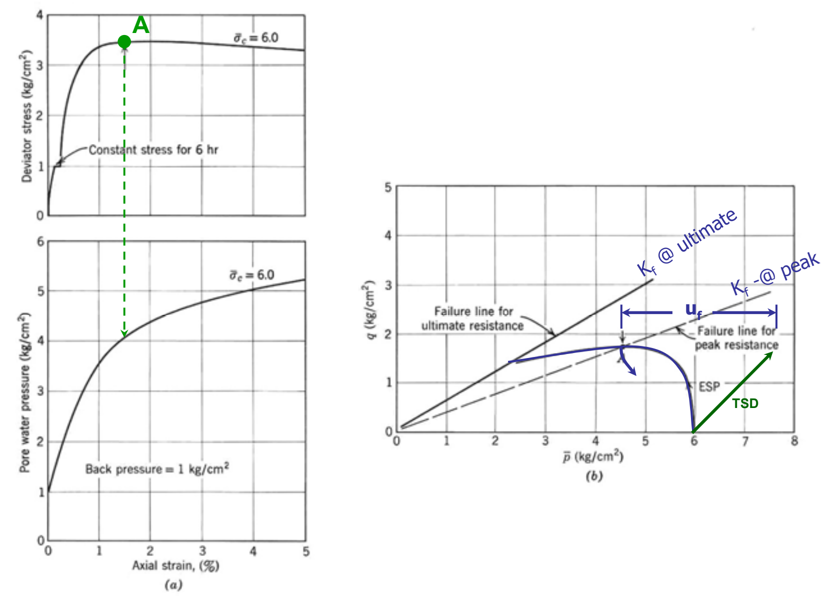


Fig. 29.5 Behavior of sensitive clay during undrained shear (after Crawford, 1959).

More Meta Structure

Unstable Structure

Stable Structure

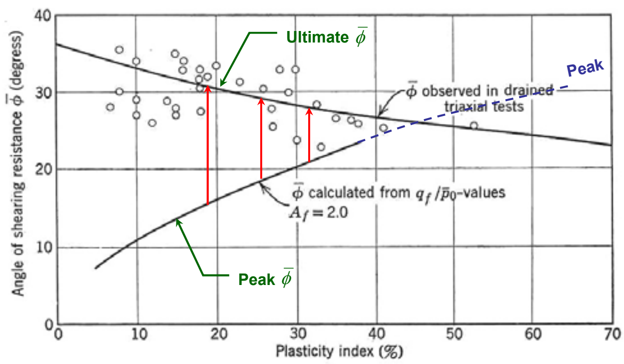


Fig. 29.7 Comparison of friction angles mobilized at peak resistance in drained and undrained tests.

4. Strength after repeated load or seismic load

Fatigue strength (cycle / repeated loading)	<	static strength (Single loading)

Due to

1. Accumulation of pore pressure
2. Rearrangement of soil particles.
3. Reduction of cementing bonds.

Generally we want to find the "Fatigue Limit" which is (the number of load application (cycle) until) the failure strength below peak single strength occurred.

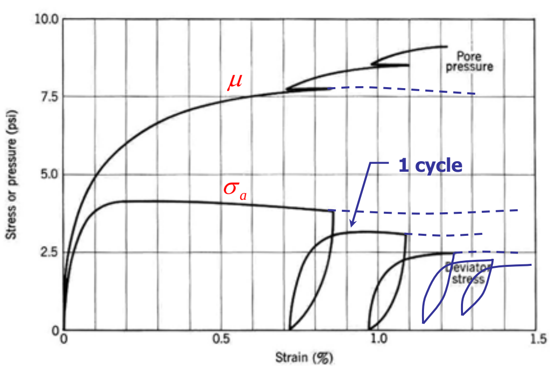


Fig. 29.8 Effect of repeated loading on undrained strength of very loose saturated sand. Specimen consolidated to 10 psi; void ratio = 0.834. (From Healy, 1963.)

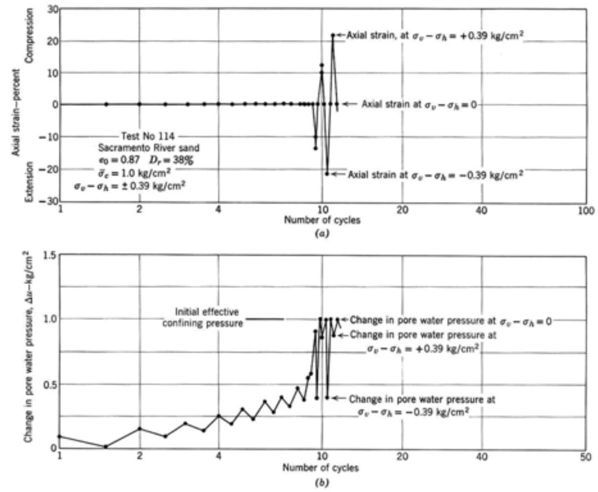


Fig. 29.9 Pore pressure and axial strain versus number of cycles during repeated triaxial loading of loose saturated sand. (*a*) Axial strain versus number of cycles. (*b*) Observed change in pore water pressure versus number of cycles. (From Seed and Lee, 1966.)

Photo 1. Sand boil in Port Island

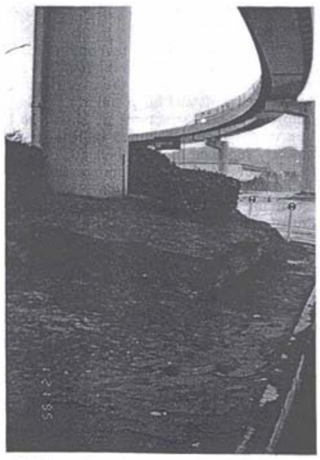


Photo 2. Extraction of the Pier Foundation in Port Island

Photo Failure of the Container Berths in Rokko Island

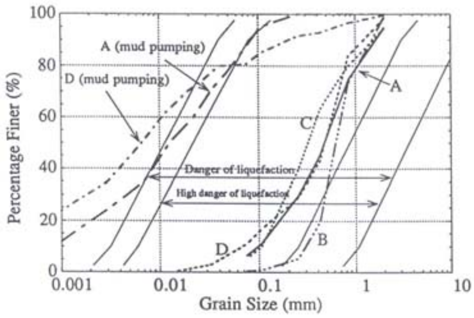


Fig. Grain Size Distribution of Sand Boils

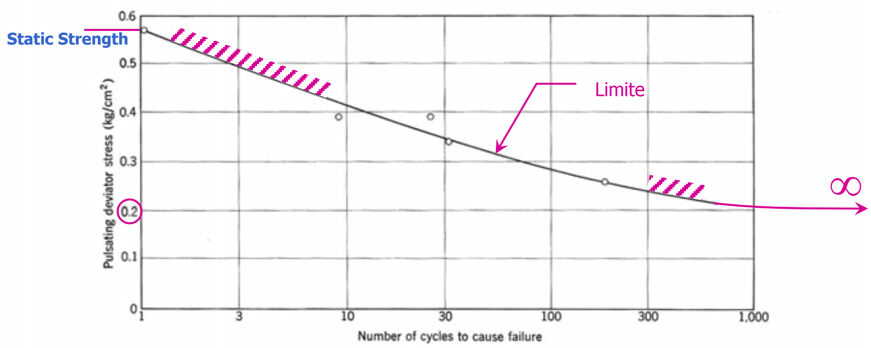


Fig. 29.12 Relationship between pulsating deviator stress and number of cycles required to cause failure Sacremento River sand initial void ratio = 0.87; initial confining stress = 1.0 kg/cm². (from Seed and Lee, 1967.)

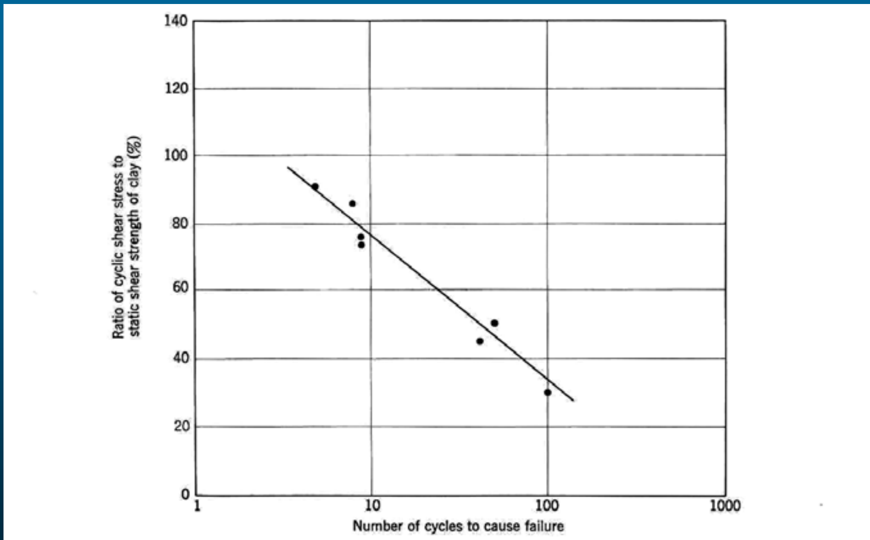


Fig. 29.13 Strength of samples of silty clay under cyclic loading conditions (from Seed and Wilson, 1967).

5. Anisotropic Consolidation before Shearing.

Generally

1. **Typical isotropic T.A. Test**
2. **Ansisotropic T.A. Test**

Both tests will show the different stress paths, but the same failure line (same kf-line)

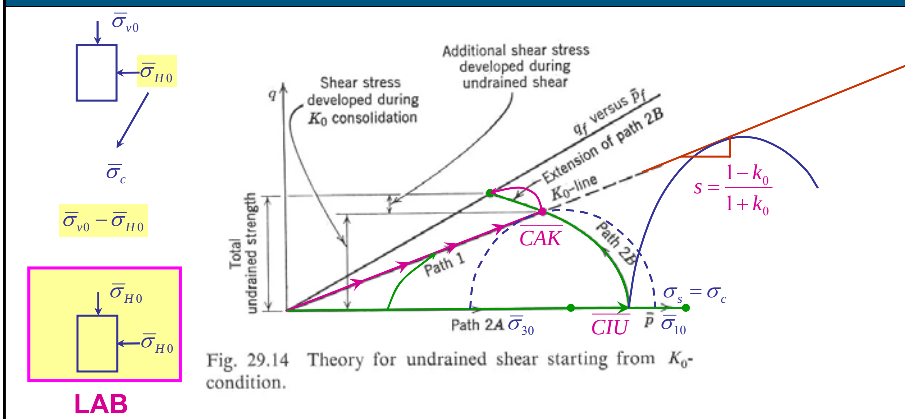


Fig. 29.14 Theory for undrained shear starting from K_0-condition.

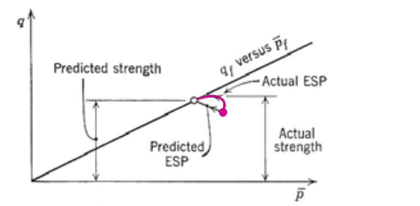


Fig. 29.15 Actual typical effective stress path for undrained shear starting from K_0-condition.

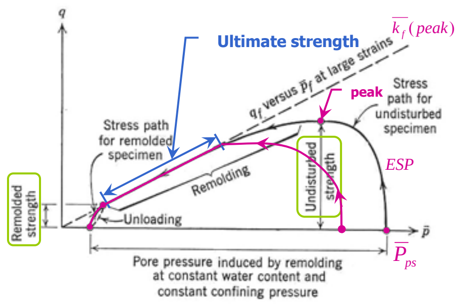


Fig. 29.16 Stress paths for undisturbed and remolded soils.

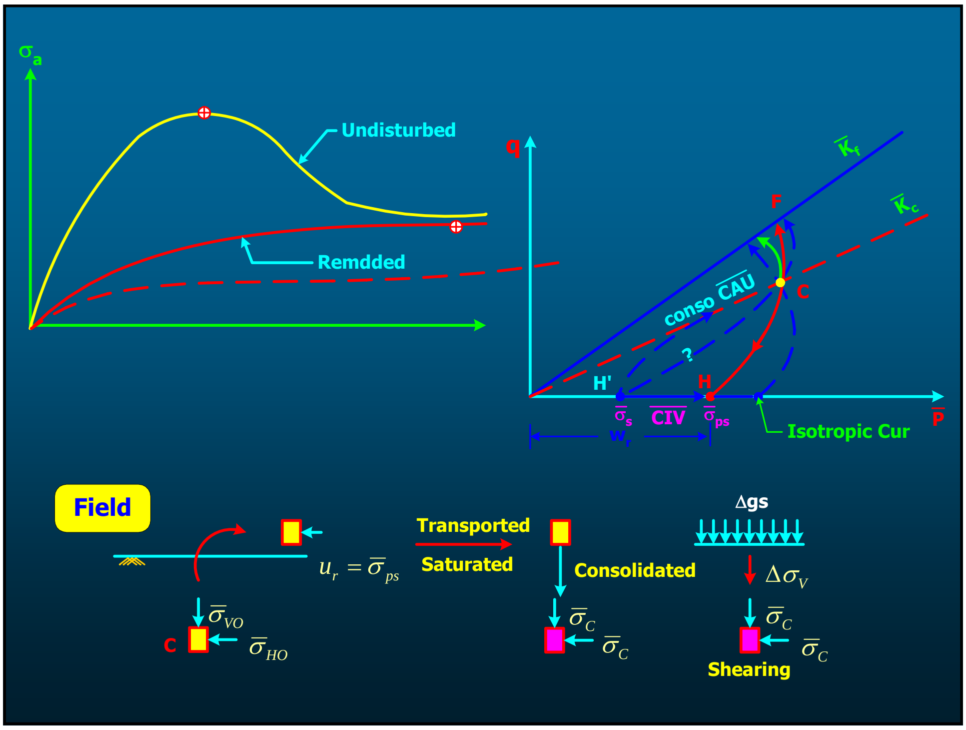
6. Remolding and Disturbance.

Undisturbed strength → soil in natural forming structure

remolded strength → Soil is completely disturbed (or destroy the original structure.)

disturbed strength → partially disturbed to some certain degree.

$$\text{Sensitivity} = \frac{\text{Undisturbed strength}}{\text{Remolded strength}}$$



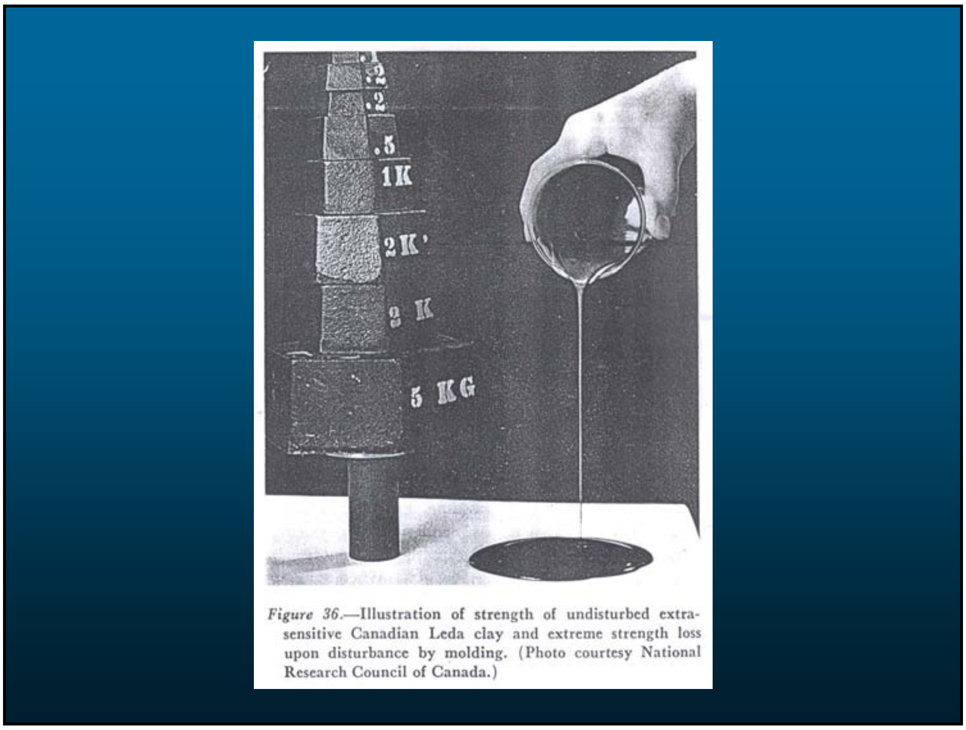
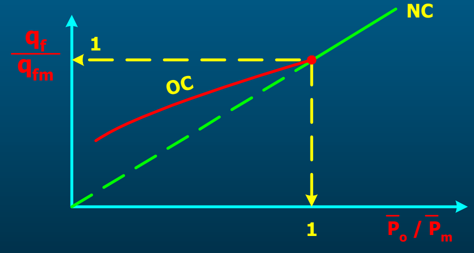


Figure 36.—Illustration of strength of undisturbed extra-sensitive Canadian Leda clay and extreme strength loss upon disturbance by molding. (Photo courtesy National Research Council of Canada.)

7. Stress History

- Overconsolidated OC
- Normally Consolidated NC



Normalized undrained strength by SHANSEP

SHANSEP Theory by Ladd and Foott (1974)

Stress History And Normalized Soil Engineering Properties

When clayey soil is naturally sedimented or artificially sedimented in laboratory. The undrained shear strength (Su) is proportioned to $\bar{P}_o$ as

$$\frac{S_u}{\bar{P}_o} = S_o \cdot (OCR)^m \quad \text{...(1)}$$

When $S_o = \frac{S_u}{\bar{P}_o}$ **@ Normally Consolidation State (OCR = 1)**

For Bangkok Clay $S_o \approx 0.25$

m = Constant $\approx$ 0.8 ± 0.05

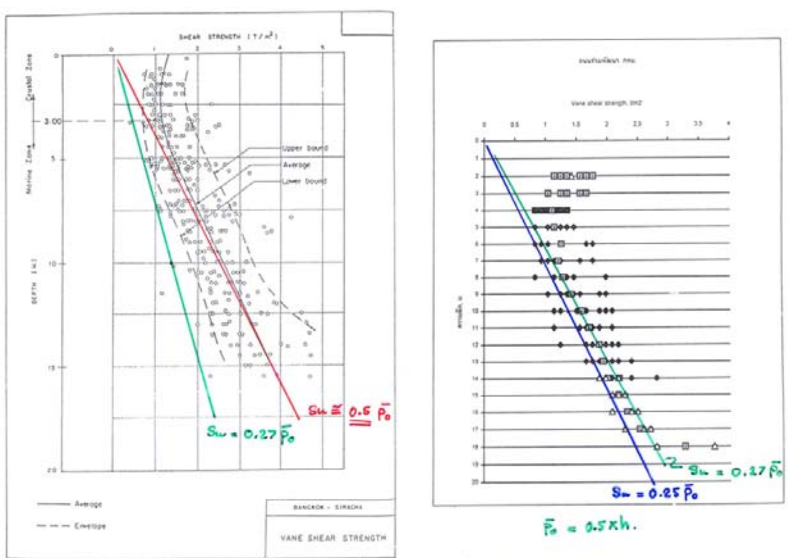
OCR = Overconsolidation Ratio

Laboratory Procedures

1. Obtain soil sample (Undisturbed or slightly disturbed)
2. Consolidate in triaxial to $\bar{P}_o$ > $\bar{P}_m$
3. Release consolidation pressure to test at different OCR.
4. Obtain Su (or q_f) at various $\bar{P}_o$ and OCR, Then solve for <u>m</u> and <u>S_o</u>

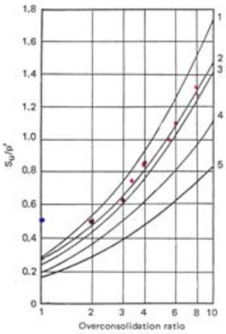
Application

1. Use for stability analysis for large {embankment excavation}
2. Use for field test quality control.



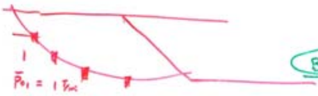
รูปที่ 3.2 ความสัมพันธ์ระหว่างค่า Undrained shear strength และความลึกของดินตามทางสายกรุงเทพ - ศรีราชา

รูปที่ 3.3 ความสัมพันธ์ระหว่างค่า Undrained shear strength และความลึกของดินตามถนนร่วมพัฒนา



Curve no.	Soil	W_n, %	PI
1	Maine organic clay	65	34
2	Bangkok clay	65	41
3	Atchafalaya clay	45	75
4	Boston blue clay	41	21
5	Connecticut Varve clay Clay layer Silt layer	 65 35	 39 12

Fig. 7.42 S_u/p' for several clays. *(Redrawn after C. C. Ladd and R. Foot, New Design Procedures for Stability of Soft Clays, J. Geotech. Eng. Div., ASCE, vol. 100, no. GT7, 1974.)*



$$\frac{S_u}{\bar{P}_o} = S \cdot (OCR)^m$$

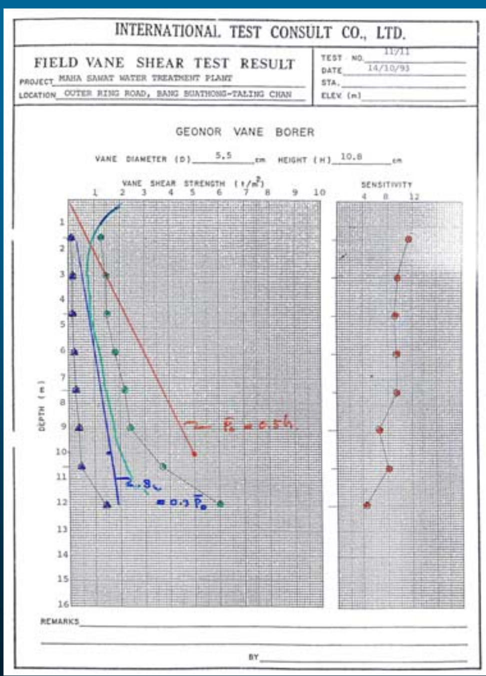
BKK - Clay

$$\frac{S_u}{\bar{P}_o} = (0.25 - 0.30) \cdot (OCR)^{0.8}$$

$$\frac{S_u}{\bar{P}_o} = 0.3$$

$$S_u = 0.3 \times 1 = 0.3\ T/m^2$$

$$\frac{S_u}{\bar{P}_o} = 0.3 \leftarrow \text{N.C.}\ OCR = 1 \leftarrow \frac{S_u}{\bar{P}_o} = 0.3\ (OCR)^{0.8}$$


INTERNATIONAL TEST CONSULT CO., LTD.

FIELD VANE SHEAR TEST RESULT

PROJECT MAHA SAWAT WATER TREATMENT PLANT

LOCATION OUTER RING ROAD, BANG BUATHONG-TALING CHAN

TEST NO. 11/11

DATE 14/10/93

STA.

ELEV. (m)

GEONOR VANE BORER

VANE DIAMETER (D) 5.5 cm HEIGHT (H) 10.8 cm

REMARKS

BY

Table : Common Methods for Measuring undrained Strength

Method	Comment
In-situ measurements	
1. Vane test	Usually considered to give best result, but is limited as to strength of soil with which it can be used
2. Soft to Medium Penetration test (SPT)	Gives crude correlation to strength
3. Static Conre Pure Test (CPT)	
Measurements upon undisturbed samples	
1. Unconfined compression	Best general purpose test; underestimates strength because disturbance decreases effective stress
2. UU test at in situ confining pressure	Most representative of laboratory tests, because of compensating errors.
3. CU test at in situ confining pressure	Overestimates strength, because disturbance leads to smaller water content upon reconsolidation

STRESS – STRAIN RELATIONSHOP FOR $\overline{\text{CU}}$

Applications

1. Immediate Settlement of Loaded Area
2. Movement of Tunnel in soil
3. Excavation heaving and lateral movement

Parameters

- Young's Modulus, E (Elastic modulus)
- Shear modulus, G
- Poisson's Raton, μ or υ

From Eq. 12.4

$$G = \frac{E}{2(1+\mu)} \qquad \ldots(1)$$

For saturated soil in undrained condition $\mu \approx 0.5$, then

$$G = \frac{E}{3} \qquad \ldots(2)$$

Shear modulus G can be measured quite accurate by shear wave propagation velocity. Ex. Shear column, seismic survey etc.

Hardin and Black (1968) estimated G for sand and clay.

$$G = 1230\frac{(2.973-e)^2}{1+e}\cdot\sqrt{\bar{\sigma}_c} \qquad \ldots(3)$$

G and $\bar{\sigma}_c$ in psi

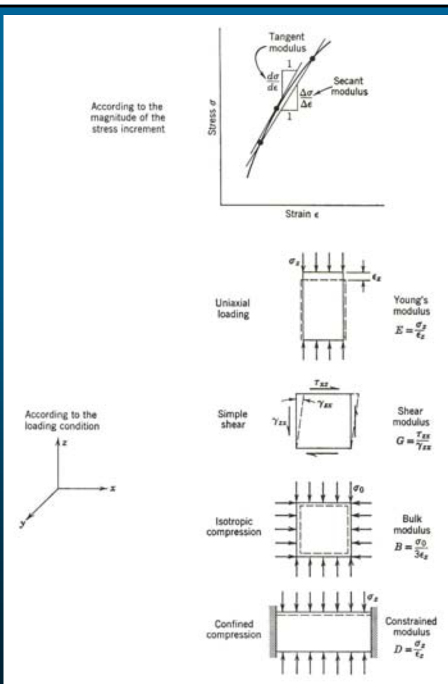
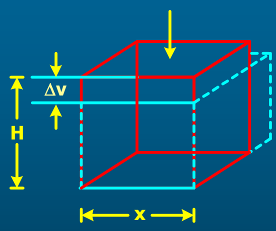


Fig. 12.1 Various types of modulus.

Table : ค่า Poisson's Ratio

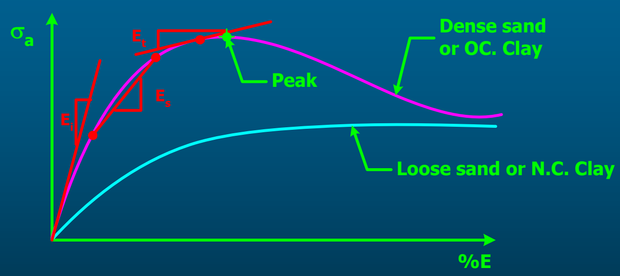
Type of Soil	V (G)
Clay, Saturated	0.4 – 0.5
Clay, Unsaturatd	0.1 – 0.3
Sandy Clay	0.2 – 0.3
Silt	0.3 – 0.35
Sand, Gravelly Sand	0.3– 0.4
Rock	0.1 – 0.4
Loess	0.1 – 0.3
Ice	0.36
Concrete	0.15
ที่มา : Bowles, J.E. (1968)	



$$\frac{\Delta x}{x_0} = 0.5\frac{\Delta v}{H}$$

$$\frac{\Delta y}{y_0} = 0.5\frac{\Delta v}{H}$$

Young's Modulus flow stress – strain Carve



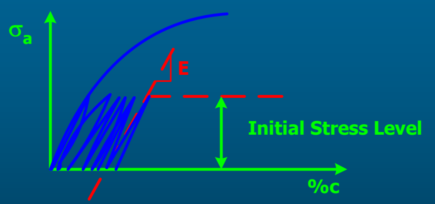
E_i = Initial Tangential Modulus

E_s = Secant Modulus

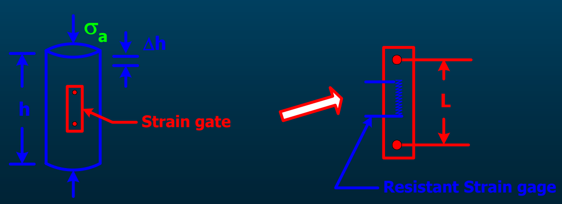
E_t = Tangential Modulus

Test for small strain modulus

1. Repeated load test



2. Using lorv strain strain - gage



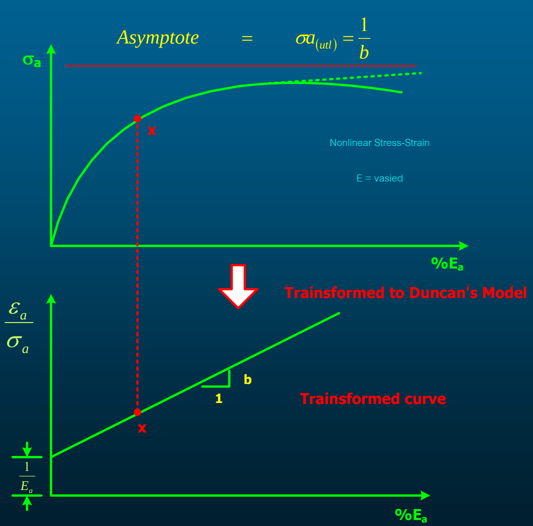
$$\varepsilon = \frac{\Delta L}{L}$$

$$E = \frac{\sigma_a}{\varepsilon}$$

$$\rightarrow \Delta L$$

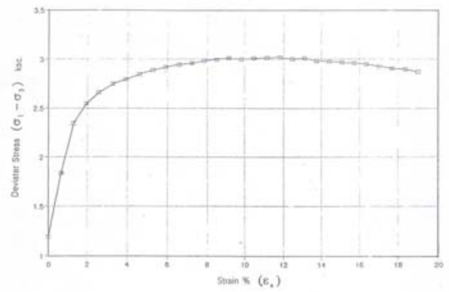
Duncan's Stress – Strain Model (FEM)

Due to non-linearity of soil stress-strain curve, Duncan and Kulhawy (1969) proposed hyperbolic model for stress-strain

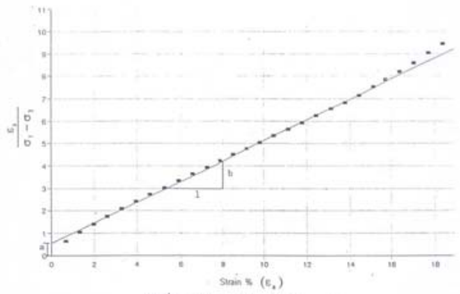


$$\therefore E_t = f\left(E_i, R_f, K, n...\right) \qquad ...(4)$$

R_f = Failure ratio
K = Axial modulus member
N = Modulus exponent



ภาพที่ 28 ตัวอย่างความสัมพันธ์ของหน่วยแรงและความเครียด



ภาพที่ 29 Transformed Curve ของ stress-strain

Strain – Contours in Stress - path

During series of $\overline{CU}$ - triaxial Test, if strain levels C% axial strain; are plotted along in stress – paths. The readial lines of the same strain can be drawn as "strain – contours".

Applications

1. Prediction of undrained settlement. (Elastic sett)
2. Estimation of consolidation settlement after dissipation
   Using Ev v.s. Consolidation pressure
   wo v.s. $\bar{P}_o$
3. Estimation of Failure vertical strain during loading

Methods for prediction of settlement from Triaxial Test.

1. Direct method.

- **Predict stress – path due to insitu and construction loadings.**
- **Obtain the underturbed representative soil samples.**
- **Run triaxial test follow the predicted stress – path.**
- **Direct measure the vertical strain in the sample**

$$E_v = E_v\,(\text{undrained}) + E_v\,(\text{Consolidation})$$

$$S = Ev \cdot \frac{H}{L} \qquad \text{...(5)}$$

When H = soil layer thickness
L = sample hight.

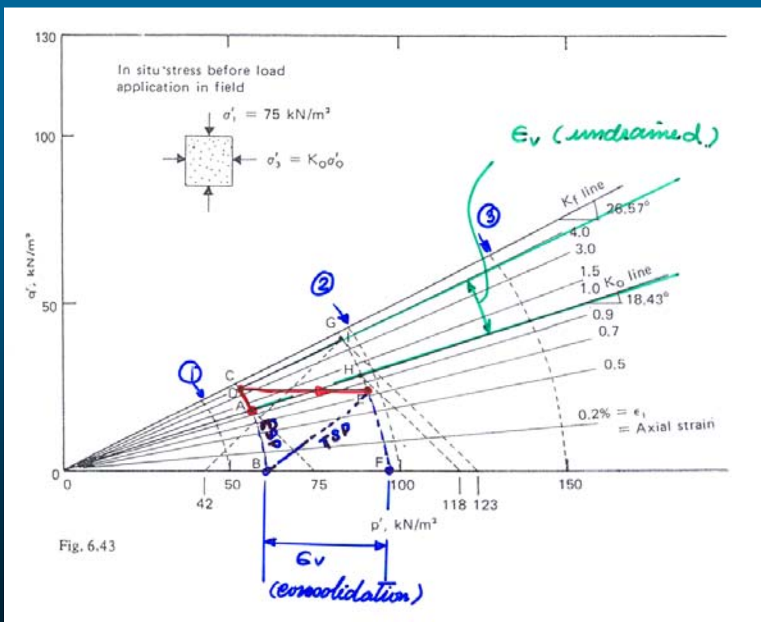


Fig. 6.43

2. Indirect Method

- **Test a series of triaxial $(\overline{CU})$ tests**
  Ex. @ σ_c = 50, 100, 150 kN/m³ as in Fig
- **Establish the strain contours**
- **Establish the relationship between $\overline{P}_o$ v.s. e_o (or w_o)**
- **Construct stress path follow actual loading.**
- **E_v (undrained) is obtained from strain contour.**
- **E_v (consolidation)** $\cong \frac{1}{3} E_{(vol)}$

  E_{vol} calculate from $\overline{P}_o$ v.s. e_o (or w_o)

  E_v = E_v (undrained) + E_v (consolidation)

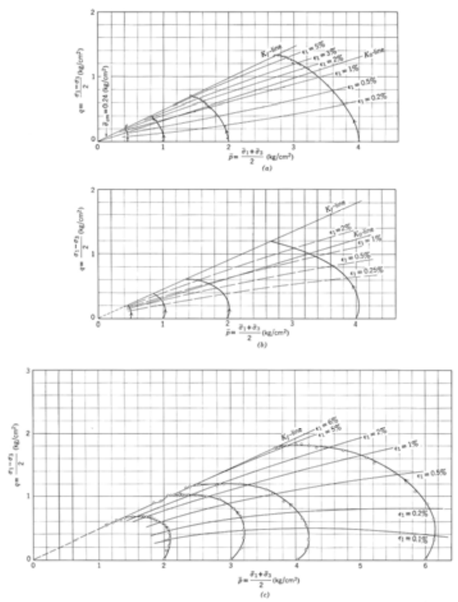
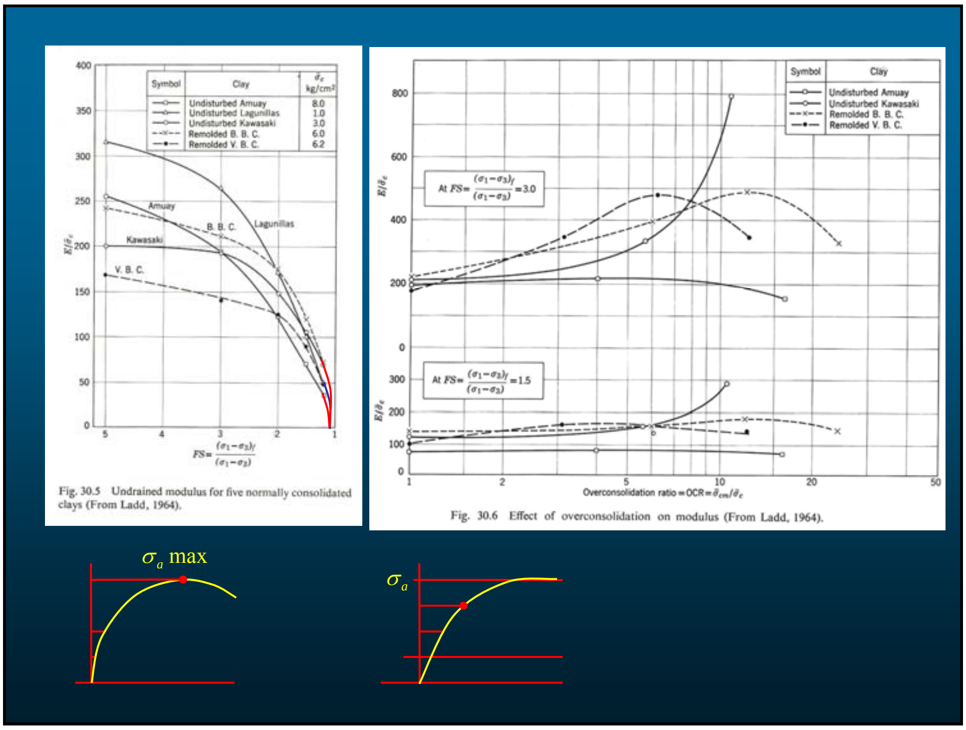
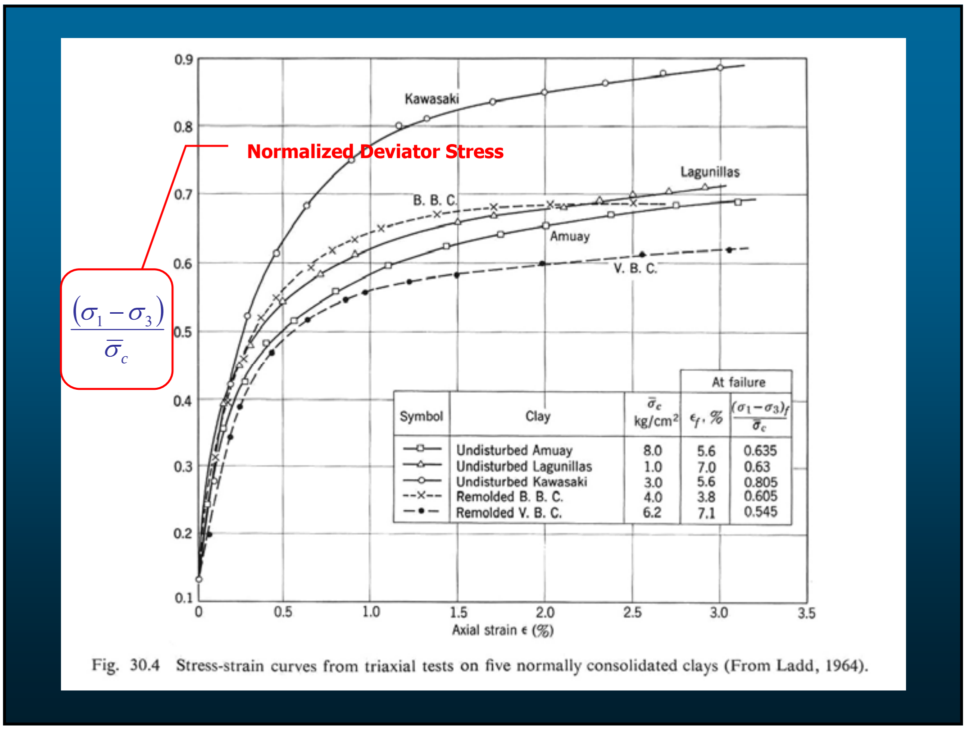


Fig. 30.3 Contours of equal strain for three normally consolidated clays. (a) Amuay clay. (c) Remolded Boston blue clay. (b) Lagunillas clay. (From Lambe, 1964.)

457


Symbol
Clay
$\bar{\sigma}_c$ kg/cm²
Undisturbed Amuay 8.0
Undisturbed Lagunillas 1.0
Undisturbed Kawasaki 3.0
Remolded B. B. C. 6.0
Remolded V. B. C. 6.2
Amuay
B. B. C.
Lagunillas
Kawasaki
V. B. C.
$E/\bar{\sigma}_c$
$FS = \frac{(\sigma_1-\sigma_3)_f}{(\sigma_1-\sigma_3)}$
Fig. 30.5 Undrained modulus for five normally consolidated clays (From Ladd, 1964).
Symbol
Clay
Undisturbed Amuay
Undisturbed Kawasaki
Remolded B. B. C.
Remolded V. B. C.
At $FS = \frac{(\sigma_1-\sigma_3)_f}{(\sigma_1-\sigma_3)} = 3.0$
At $FS = \frac{(\sigma_1-\sigma_3)_f}{(\sigma_1-\sigma_3)} = 1.5$
Overconsolidation ratio = OCR = $\bar{\sigma}_{cm}/\bar{\sigma}_c$
Fig. 30.6 Effect of overconsolidation on modulus (From Ladd, 1964).
σ_a max
σ_a


Kawasaki
Normalized Deviator Stress
$\frac{(\sigma_1-\sigma_3)}{\bar{\sigma}_c}$
Lagunillas
B. B. C.
Amuay
V. B. C.
At failure
Symbol | Clay | $\bar{\sigma}_c$ kg/cm² | ϵ_f, % | $\frac{(\sigma_1-\sigma_3)_f}{\bar{\sigma}_c}$
Undisturbed Amuay 8.0 5.6 0.635
Undisturbed Lagunillas 1.0 7.0 0.63
Undisturbed Kawasaki 3.0 5.6 0.805
Remolded B. B. C. 4.0 3.8 0.605
Remolded V. B. C. 6.2 7.1 0.545
Axial strain ϵ (%)
Fig. 30.4 Stress-strain curves from triaxial tests on five normally consolidated clays (From Ladd, 1964).

Factors stress – strain behaviors (E)

1. Consolidation pressure $\bar{P}_o$, $\bar{P}_o$

N.C. Clay → E is proportional to $\bar{P}_o$ or σ_c Then the plot of normalized stress – strain curves is unique for each clay. Figure 30.4

O.C. Clay → E is depended on OCR and stress level (F.S.) Figure 30.6

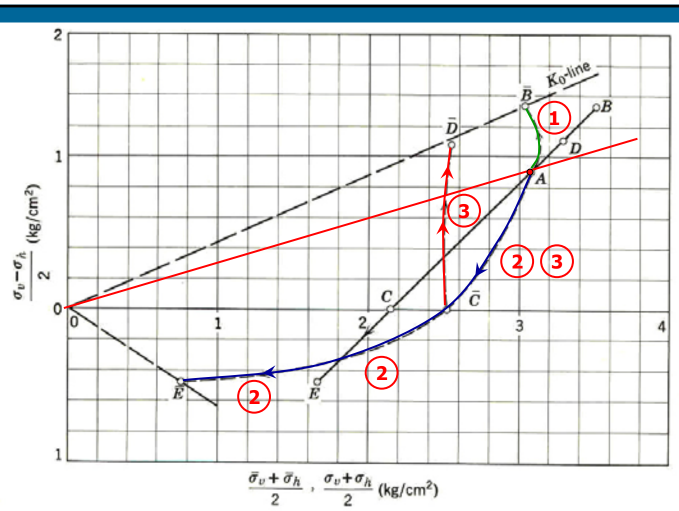
2. Loading Rate and Loading cycle

- Faster loading rate → higher (on table 30.3)
- Second or subsequent loading cycles E ≈ 1.5 E (first loading)

3. Time and Aging Effects

- Thixtropic Effects → Soil stronger with time for remolded and high liquidity index.
- Consolidation Time → longer time → secondary consolidation → stronger soil.
- Strain Rate Effect → Dynamic stronger than Static E (dynamic) ≈ 1.5 to 2.0 E (Static)

Note : without considering of excess pore pressure.



TEST	TSP	ESP	TYPE OF TEST	
1	AB	$A\bar{B}$	CAU	← Compression loading.
2	ACE	$A\bar{C}\bar{E}$	CAU-RE	← Excavation
3	ACD	$A\bar{C}\bar{D}$	CA-UU	← Perfect sample

Fig. 30.7 Strength tests on Boston blue clay.

4. Loading patterns

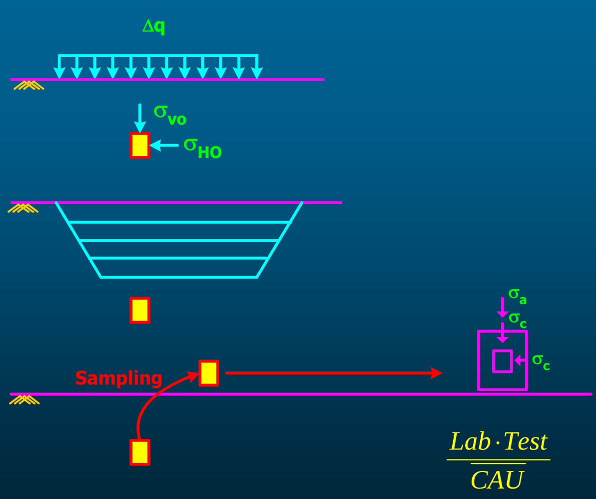
Stress – strain (E) behaviors of soil are highly depended on loading Patten (or stress paths)

Ex.

Test 1. → Compression loading from K_o
(Spread footing, embankment …)

Test 2. → Unloading from K_o
(Excavation pit, deep foundation …)

Test 3. → unloading from sampling and compression loading in laboratory
(Soil sampling and compression Test)



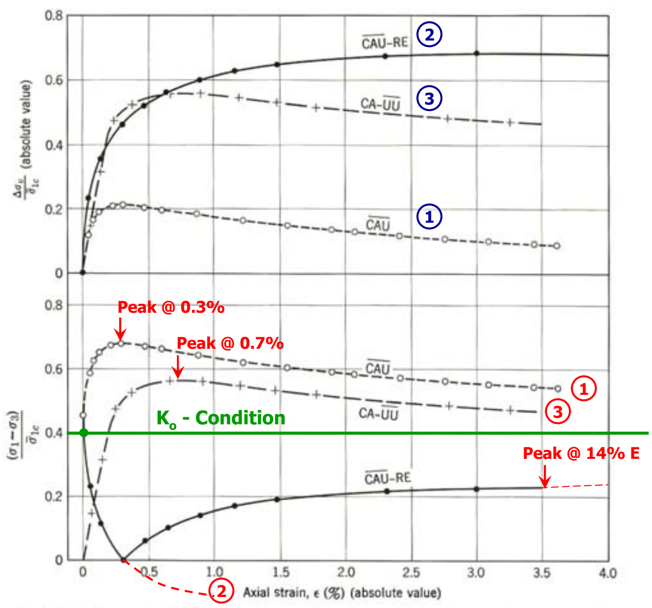
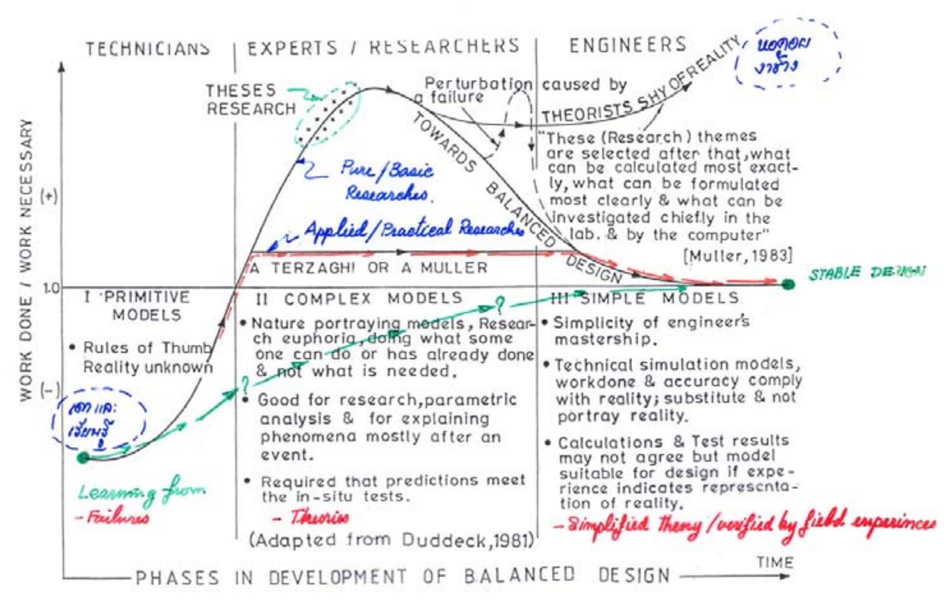


Fig. 30.8 Stress-strain data on Boston blue clay. *Note.* Zero strain on CA–UU test taken at $\sigma_1 - \sigma_3 = 0$. (From Ladd, 1964.)

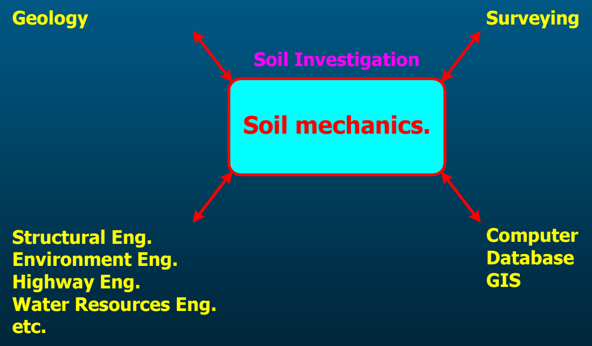


Application of Advanced soil Mechanics

1. Soil Investigation
2. Pile Foundation
3. Excavation
4. Land Reclamation
5. Slope Stability
6. Embankment and Dam
7. Tunneling
8. Geotechnical Monitoring
9. Seepage and Filter
10. Soil Dynamic and Earthquake
11. Soil Engineering Database

Soil Investigation (Site Characterization)

เพื่อประเมินสภาพชั้นดินและคุณสมบัติดิน (และหิน) เพื่อการศึกษาวิเคราะห์ออกแบบ และแก้ไขปัญหาทางวิศวกรรมปฐพี



เนื้อหาจาก Advanced Soil Mechanics ที่เกี่ยวข้อง

1. Soil Formation, Weathering Processes, Sedimentation, Deposition
2. Soil Physical Properties and Classification
3. Soil and Clay Mineralogy
4. Site Investigation → Soil Boring, Sampling. Etc.
5. Laboratory Test → Physical, Strength, Compressibility
6. Database, zoning, GIS → Statistic, Computer Graphic, Soil profile Model.

Pile Foundation (Deep foundation)

Problems concerned

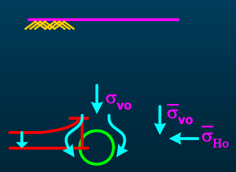
1. Pile Capacity
   - Strength
   - Stress - Strain
2. Consolidation – Settlement
   - Stress - Strain (Elastic)
   - Consolidation
   - Stress - distribution
3. Method of Installation
   - Driver → pore pressure, soil displacement
   - Bored
   - Pre bored
4. Pile group / Mat foundation
   - Group efficiency → Stress overlapping
   - Relative stiffness
   - Differential Settlement

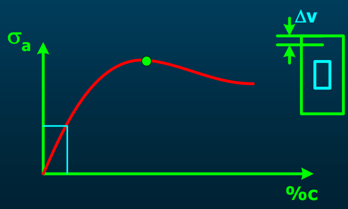
5. Caisson / Shaft
- End bearing
- Seepage
- Horizontal pressure , Friction

6. Bored pile / Barrette Wall (Slurry)
- Bentonite properties
- Trench Stability
- Seepage.

7. Tunneling

Problem 1. Strength
2. Insitu – Stress, Stress Release
3. Seepage
4. Rock bolting – Soil nailing, living
5. Blasting, Tunneling machine





8. Geotechnical monitoring

Problem
1. Pore pressure
2. Stress, load
3. Movements
4. Temperature
5. Permeability

9. Excavation

Problem
1. Soil Strength - Stability
2. Seepage
3. Lateral earth pressure – lateral movement.
4. Pressure relief - heaving

10. Land Reclamation

Problem
1. Settlement
2. Soil Strength - Stability
3. Compaction - Soil improvement

11. Slope Stability (Natural – Manmade)

Problem
1. Soil Strength
2. Pore pressure – drainage - infiltration
3. Erosion

12. Embankment and Dams

Problem
1. Soil strength
2. Compaction
3. Seepage - drainage
4. Settlement

13. Seepage and Filter
Problem 1.

14. Soil Dynamic and Earthquake
Problem 1.

15. Soil Engineering Database
Problem 1.

Design Pile Foundation

Pile Types

1. Short pile (3-12 m)
Timber, R/C

2. Long P/C pile
Section 15 → 62.5 cm.
Length 26 m. Maximum

3. Bored Pile
- Dry Process d = 35 → 200 cm.
- Wet Process
- Micro pile (grouting technique)

4. Steel Pile

Pile Capacity Calculation

1. Static method from soil strength

$$Q_f = P \cdot \sum_{i=1}^{n} (\beta_i \cdot C_i \cdot L_i)$$

$$Q_p = Ap \cdot \overline{\sigma}_v \cdot N_q$$

Where

P	=	Skin friction parameter length
β_i	=	adhesion factor
c_i	=	cohesion
L_i	=	thickness of soil layer
A_p	=	section area of pile
$\overline{\sigma}_v$	=	effective overburden pressure

2. Dynamic method

- for construction monitoring and cross – checking only

3. Static method from Dutch Cone

$$P_u = \sum_{i=1}^{n} (\alpha_i \cdot q_{fi} \cdot L_i \cdot P) + \lambda \cdot A_p \cdot q_c$$

Where

n	=	no. of soil layer
α_i	=	adhesion factor
q_{fi}	=	local friction from Dutch Cone
L_i	=	thickness of soil layer
λ	=	point bearing factor
q_c	=	point cone resistance within 4D

4. Design by B.M.A. Code

$$P_a = q_f \cdot P \cdot L$$

When

q_f = skin friction equal to 800 kg/m² (0-7) m.

800 + 200L_1 (> 7) m.

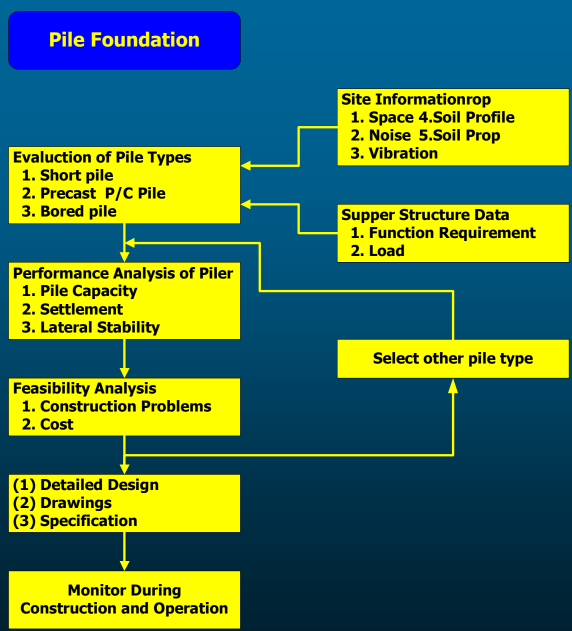
L_1 = pile length for longer than 7 m.

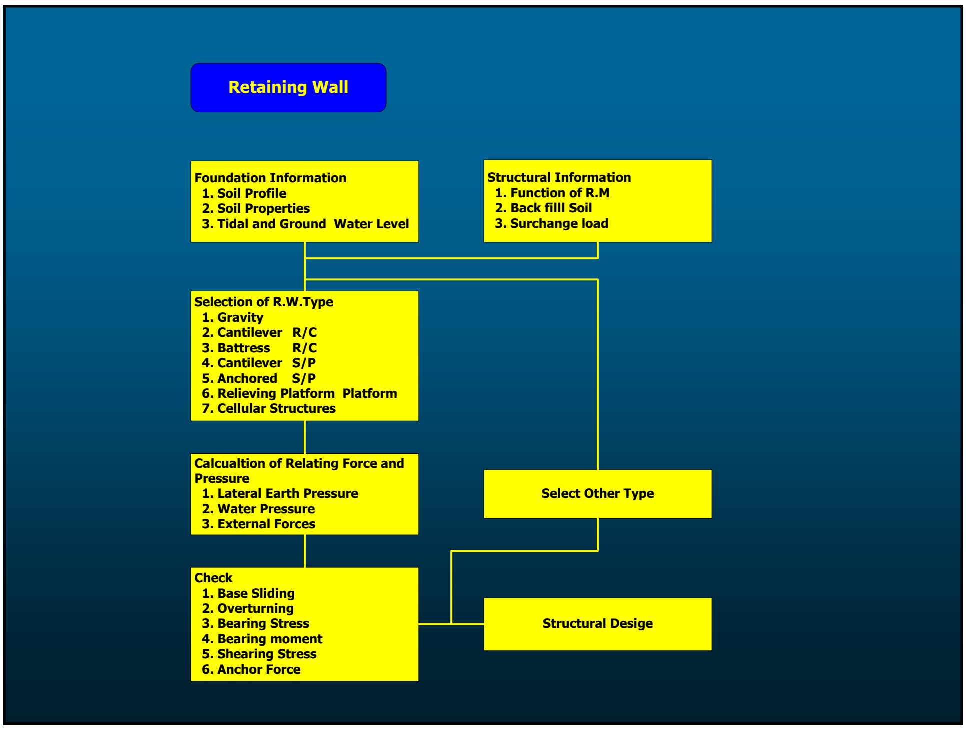
Pile Group Reduction

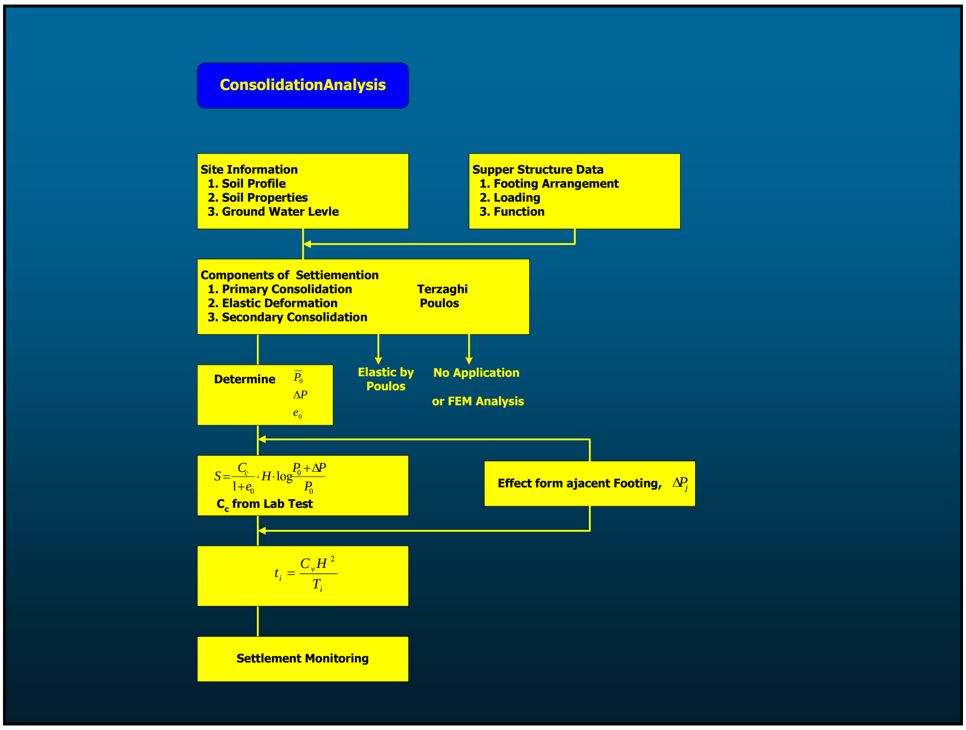
- Feld Rule
- Friction Area Ratio

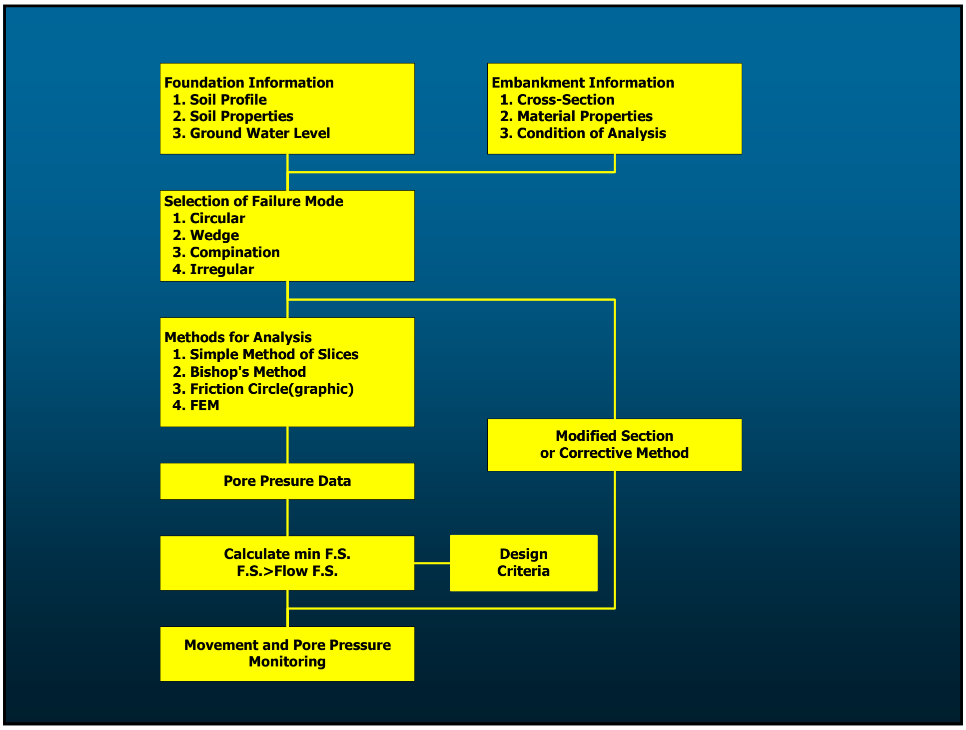
Lateral Loaded pile

- Brom's Theory

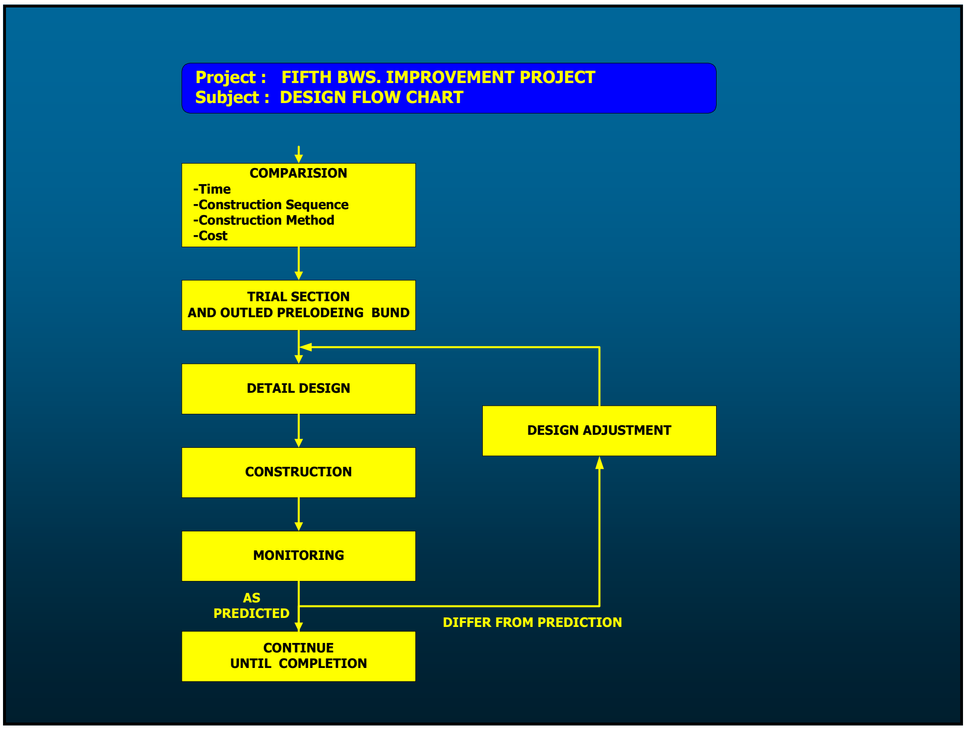



Retaining Wall
Foundation Information
1. Soil Profile
2. Soil Properties
3. Tidal and Ground Water Level
Structural Information
1. Function of R.M
2. Back filll Soil
3. Surchange load
Selection of R.W.Type
1. Gravity
2. Cantilever R/C
3. Battress R/C
4. Cantilever S/P
5. Anchored S/P
6. Relieving Platform Platform
7. Cellular Structures
Calcualtion of Relating Force and Pressure
1. Lateral Earth Pressure
2. Water Pressure
3. External Forces
Select Other Type
Check
1. Base Sliding
2. Overturning
3. Bearing Stress
4. Bearing moment
5. Shearing Stress
6. Anchor Force
Structural Desige

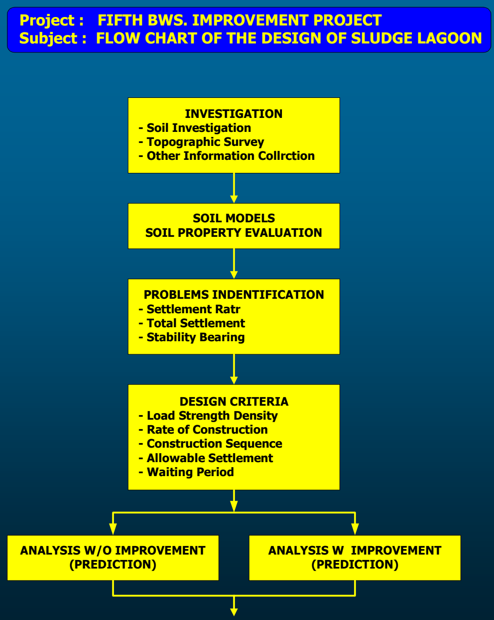

ConsolidationAnalysis
Site Information
1. Soil Profile
2. Soil Properties
3. Ground Water Levle
Supper Structure Data
1. Footing Arrangement
2. Loading
3. Function
Components of Settiemention
1. Primary Consolidation
2. Elastic Deformation
3. Secondary Consolidation
Terzaghi
Poulos
Determine $\bar{P}_0$ ΔP e_0
Elastic by Poulos
No Application
or FEM Analysis
$S = \frac{C_c}{1+e_0} \cdot H \cdot \log \frac{P_0 + \Delta P}{P_0}$
C_c from Lab Test
Effect form ajacent Footing, ΔP_j
$t_i = \frac{C_v H^2}{T_i}$
Settlement Monitoring



Foundation Information
1. Soil Profile
2. Soil Properties
3. Ground Water Level
Embankment Information
1. Cross-Section
2. Material Properties
3. Condition of Analysis
Selection of Failure Mode
1. Circular
2. Wedge
3. Compination
4. Irregular
Methods for Analysis
1. Simple Method of Slices
2. Bishop's Method
3. Friction Circle(graphic)
4. FEM
Modified Section
or Corrective Method
Pore Presure Data
Calculate min F.S.
F.S.>Flow F.S.
Design
Criteria
Movement and Pore Pressure
Monitoring



Project : FIFTH BWS. IMPROVEMENT PROJECT
Subject : DESIGN FLOW CHART
COMPARISION
-Time
-Construction Sequence
-Construction Method
-Cost
TRIAL SECTION
AND OUTLED PRELODEING BUND
DETAIL DESIGN
DESIGN ADJUSTMENT
CONSTRUCTION
MONITORING
AS
PREDICTED
DIFFER FROM PREDICTION
CONTINUE
UNTIL COMPLETION



Geotechnical Engineering involvement

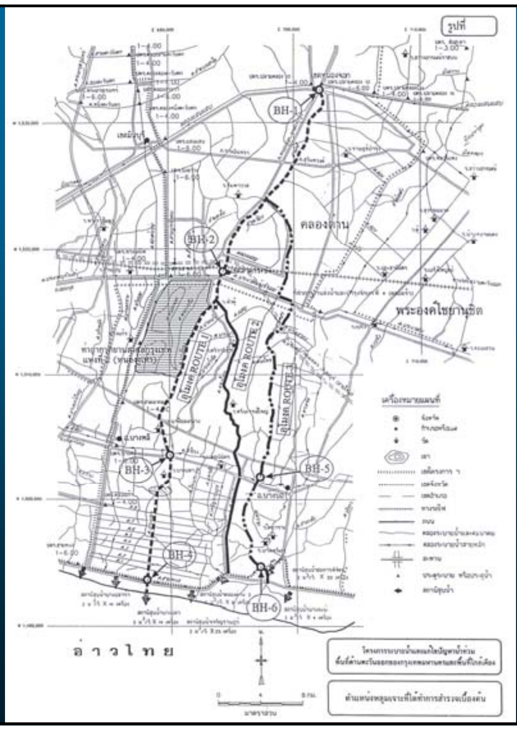
Engineering Structures	Related Geotechnical Engineering Topics										
	Bearing Capacities	Settlement	Stability	Lateral Pressures	Lateral Movement	Seepage and Drainage	Compaction	Excavation	Grouting and Deep Improvement	Surface Soil Improvement	Surface Erosion
1. EARTH AND ROCK FILLED DAMS											
1.1 Dam Embankment		○	○		□	○	○				○
1.2 Dam Foundation	○	□	□			○		○	○	□	
1.3 Dam Abutments			○		○	○		○			
1.4 Appurtenant Structures	○	○		○		○	○	○			
2. CANALS AND PIPELINES											
2.1 Conveyance Structures		○			○	○	○	○			○
2.2 intake Structure	○	○						○			
2.3 Storage or Surge Tanks	○	○									
2.4 Receiving Structure			○			○	○	○			
3. DEEP EXCAVATON											
3.1 Free Slope			○		○						○
3.2 Cantilever Sheet piling			○	○	○			○			
3.3 Braced Excavation				○	○			○			
3.4 Anchored Retaining Wall			○	○	○				○		

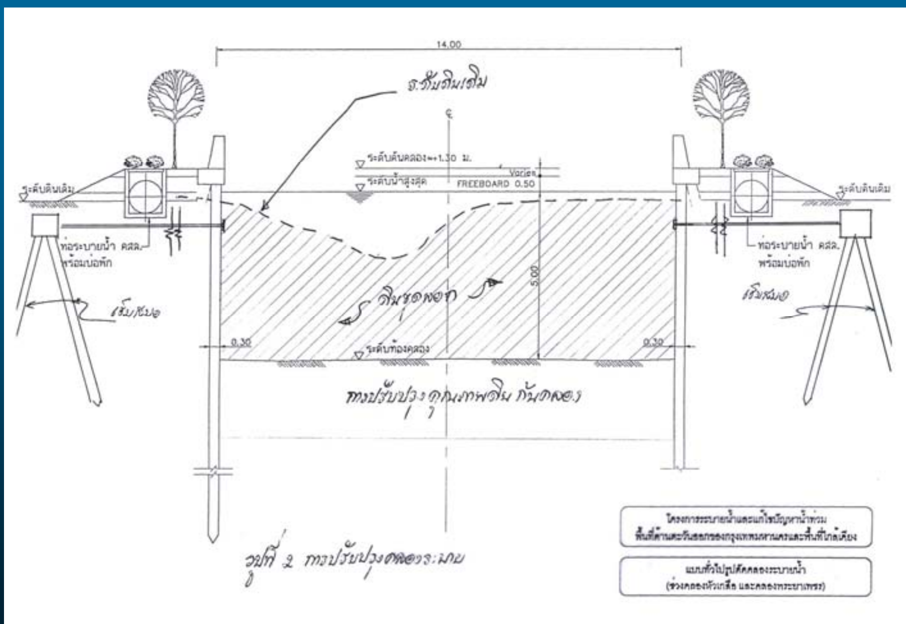
Geotechnical Engineering involvement (ต่อ)

Engineering Structures	Related Geotechnical Engineering Topics										
	Bearing Capacities	Settlement	Stability	Lateral Pressures	Lateral Movement	Seepage and Drainage	Compaction	Excavation	Grouting and Deep Improvement	Surface Soil Improvement	Surface Erosion
4. Building and Bridge Foundations											
4.1 Shallow footing	○	○						○			
4.2 Pile Foundation	○	○			○				○		
4.3 Caison or Deep massive footing	○	○	○								
4.4 Micro pile or Root pile	○	○					○	○	○		
5. Embankments, highway and runway									○		
5.1 Highway, Railway	○	○					○			○	
5.2 Runway, Taxiway and Apron	○	○					○			○	
5.3 Land Reclamation (Coastal)	○	○	○				○	○		○	
6. Tunneling											
6.1 Rock Tunneling		○		○	○	○		○	○		
6.2 Underground Opening		○		○	○	○		○	○		
6.3 Soil Tunneling	○	○		○	○	○		○	○		
6.4 Cut and Cover			○	○	○		○	○	○		

Geotechnical Engineering involvement (ต่อ)

Engineering Structures	Related Geotechnical Engineering Topics										
	Bearing Capacities	Settlement	Stability	Lateral Pressures	Lateral Movement	Seepage and Drainage	Compaction	Excavation	Grouting and Deep Improvement	Surface Soil Improvement	Surface Erosion
7. Soil and Rock Slopes	○										
7.1 Natural Slope			○		○	○					○
7.2 Cut Slope			○			○		○			○
7.3 Fill Slope	○		○				○				○
8. Offshore or Near shore Structures											
8.1 Oil Drilling Platform	○			○							
8.2 Jetty and quay wall	○	○		○							
8.3 Dry Dock	○			○	○			○	○		


BH-1
BH-2
BH-3
BH-4
BH-5
BH-6
คลองด่าน
พระองค์ไชยานุชิต
อ่าวไทย


14.00
ระดับคันคลอง≈+1.30 ม.
ระดับน้ำสูงสุด
FREEBOARD 0.50
ระดับดินเดิม
ท่อระบายน้ำ คสล.
พร้อมบ่อพัก
5.00
0.30
ระดับท้องคลอง

ตารางลักษณะโครงสร้างและปัญหาทางด้านวิศวกรรมปฐพีที่ต้องพิจารณาในการออกแบบ

ลักษณะโครงสร้าง	Consoli-dation	Pile Capacity	Bearing Capacity	Stability	Lateral Earth Pressure	Seepage & Drainage
อุโมงค์	○		○		○	□
คลองผันน้ำแบบมีกำแพงกันดิน	□	○		○	○	□
คลองผันน้ำสี่เหลี่ยมคางหมูที่ขยายจากคลองเดิมหรือขุดใหม่	□		□	○		□
คันกั้นน้ำภายนอกและภายในพื้นที่พร้อมอาคารประกอบ	○	□	○	○		○
การขุดบ่อชลอน้ำเป็นพื้นที่แก้มลิง				○		□
ท่อลอด (Siphon) และสะพาน	○	○	○		□	
อาคารบังคับน้ำตลอดความยาวคลอง	○	○				

○ **Primary Problem**

□ **Secondary Problem**

ตารางรายละเอียดการเจาะสำรวจทางและการทดสอบทางวิศวกรรมปฐพีที่

ลักษณะโครงสร้าง	ความลึกหลุมเจาะ	Field Vane Shear	Basic Properties (Seive Analysis, atterberg's Limit, Wn, etc.)	Unconfined Comp. Test	Triaxial on Test	Consoli-dation on Test	Lateral Load Test
อุโมงค์	50 ม.	-	✓	✓	✓	✓	✓
คลองผันน้ำแบบมีกำแพงกันดิน	30 ม.	15 ม.	✓	✓	-	✓	-
คลองผันน้ำสี่เหลี่ยมคางหมูที่ขยายจากคลองเดิมหรือขุดใหม่	20 ม.	15 ม.	✓	✓	-	-	-
คันกั้นน้ำภายนอกและภายในพื้นที่พร้อมอาคารประกอบ	20 ม.	15 ม.	✓	✓	-	✓	-
การขุดบ่อชลอน้ำเป็นพื้นที่แก้มลิง	20 ม.	15 ม.	✓	✓	-	-	-
ท่อลอด (Siphon) และสะพาน	30 ม.	-	✓	✓	-	✓	-
อาคารบังคับน้ำตลอดความยาวคลอง	30 ม.	-	✓	✓	-	✓	-