CIVE 3208
Geotechnical Mechanics

Course Review

Fall 2014

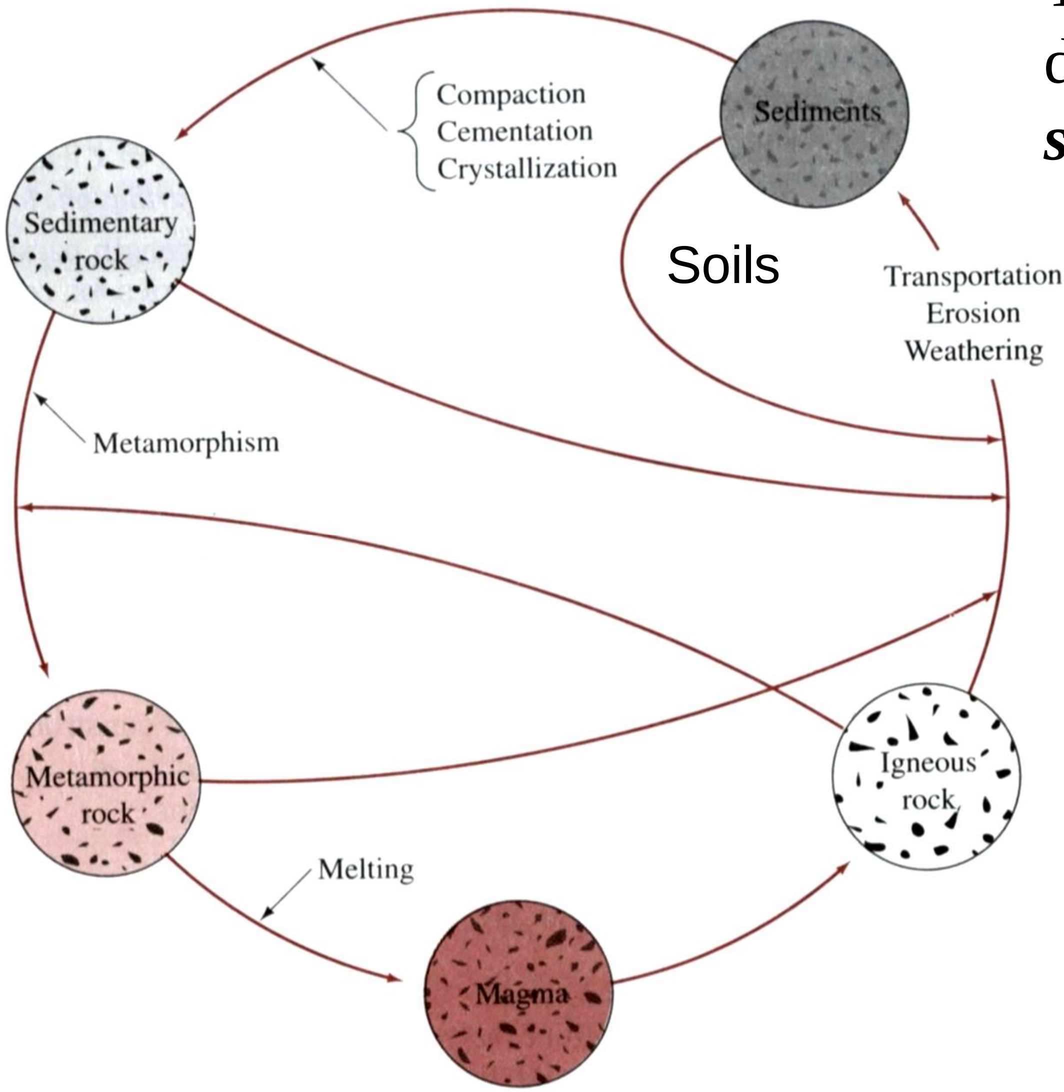
References:

Canadian Foundation Engineering Manual
Soil Mechanics & Foundations, M. Budhu, John Wiley & Sons
Principles of Foundation Engineering, B. M. Das, Thompson Publishing
Principles of Geotechnical Engineering, B. M. Das, Thompson Publishing
An Introduction to Geotechnical Engineering, Holtz & Kovacs, Prentice Hall
Foundation Design, Principels & Practices, D.P. Coduto, Prentice Hall

Review: Soil Mechanics

- Origin and Formation of Soil
- Soil Composition
  - Soil-a 3-phase material
  - Soil Characterization (particle size, soil plasticity)
- Soil Classification
- Soil Compaction
- Groundwater
- Stress (Total vs. Effective)
  - Relationship between horizontal & vertical stresses
- Compressibility and settlement
- Strength

Origin and Formation of Soil



The final products due to weathering are ***soils***

Soil Formation

Rock

Residual soil

formed by weathering and remain in their place of origin

Transported soil

formed by weathering and transported far away

Transported Soils

1. ***Alluvial Soils:*** transported by water (rivers, streams etc)

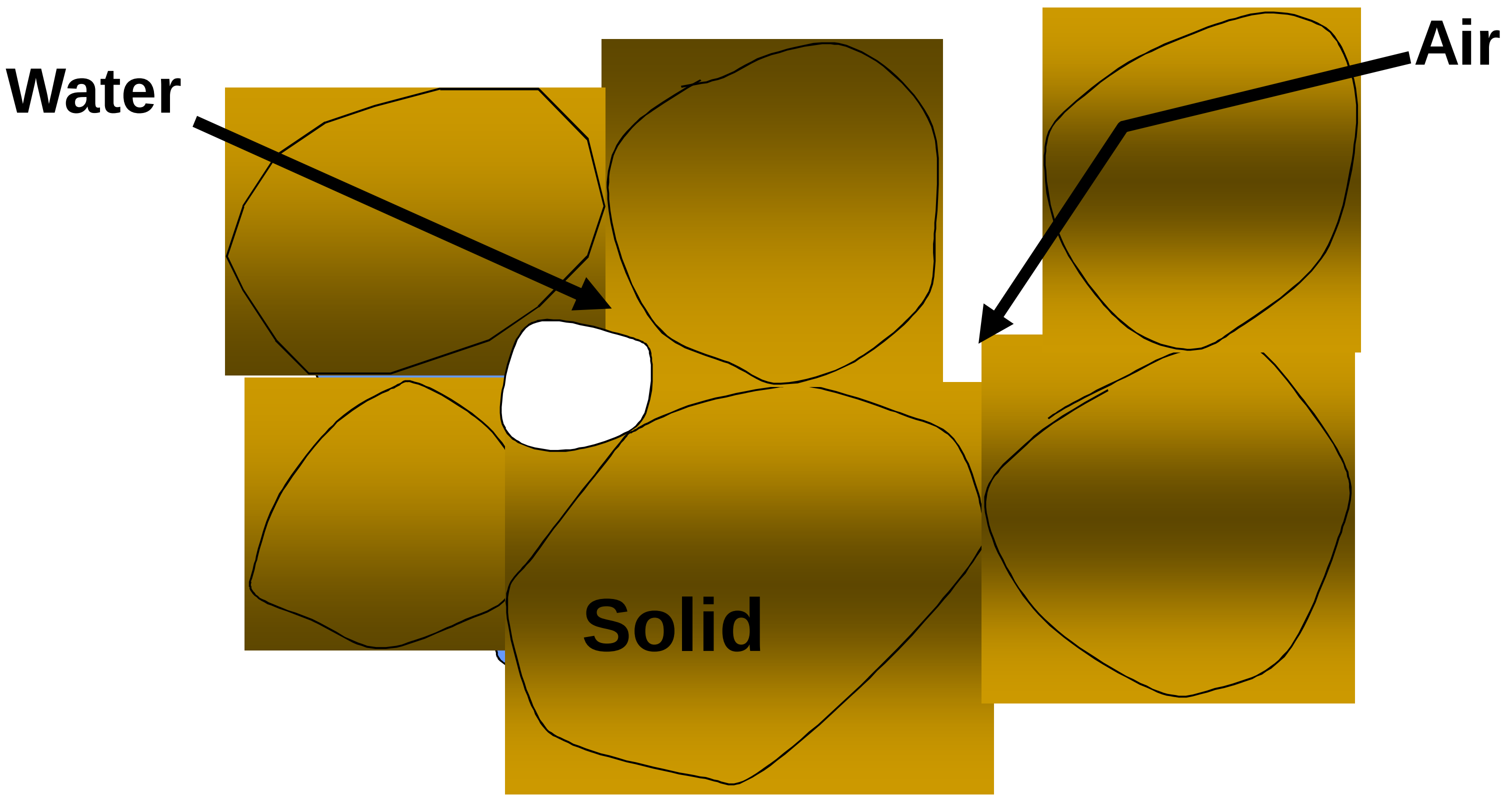
2. ***Marine soils:*** deposited in a marine environment (salt water)
   - Lacustrine soils: formed by deposition in quiet lakes (fresh water)

3. ***Glacial Soils:*** transported and deposited by glaciers

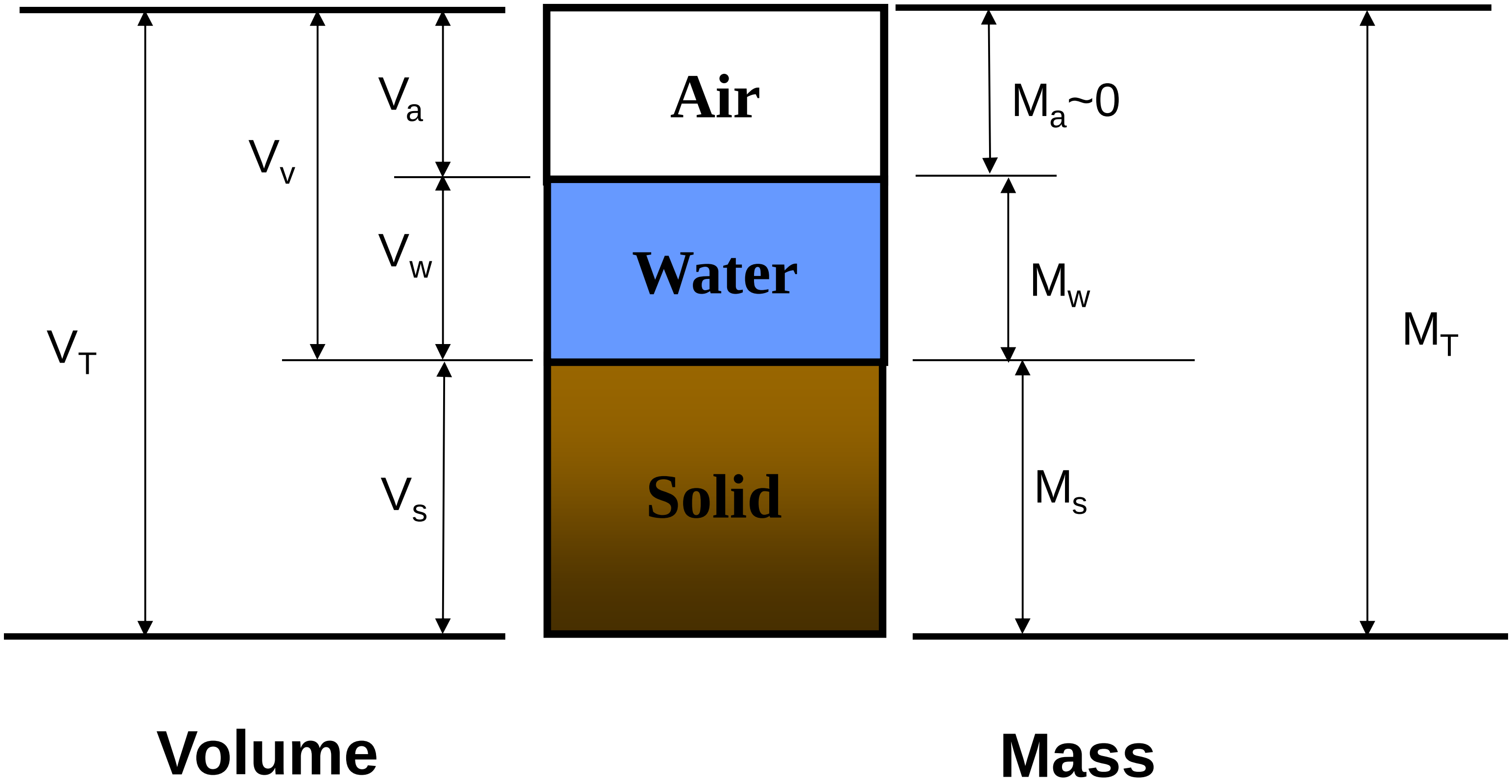
4. ***Colluvial soils:*** soils accumulated at the base of slopes/hills due to gravity (e.g. landslides)

5. ***Aeolian/Eolian soils:*** transported by wind; e.g. sand dunes, loess

Soil: A 3-Phase Material



Three Phase System



Volumetric Relationships

- Volume Components:
  - Volume of Solids = V_s
  - Volume of Water = V_w
  - Volume of Air = V_a
  - Volume of Voids = $V_a + V_w = V_v$

$$\text{Void Ratio, } e = \frac{V_v}{V_s}$$

$$\text{Porosity, } n(\%) = \frac{V_v}{V_T} \times 100\%$$

$$\text{Degree of Saturation, } S(\%) = \frac{V_w}{V_V} \times 100\%$$

Mass Relationships

- <u>Mass Components</u>:
  - Mass of Solids = M_s
  - Mass of Water = M_w
  - Mass of Air ~ 0

$$\boxed{Water\ \ Content,\ \ w(\%) = \frac{M_w}{M_s} \times 100\%}$$

$$\boxed{Density, \rho = \frac{M}{V}}$$

Soil Unit weight (kN/m^3)

- <u>Bulk (or Total) Unit weight</u>

$$\gamma = W_T / V_T$$

- <u>Dry unit weight</u>

$$\gamma_d = W_s / V_T$$

- <u>Buoyant (submerged) unit weight</u>

$$\gamma' = \gamma - \gamma_w$$

- Unit weight of Water, γ_w
  - γ_w = 1.0 g/cm^3 (strictly accurate at 4° C)
  - γ_w = 62.4 pcf
  - γ_w = 9.81 kN/m^3

More Phase Relations

- several of the definitions and formulae given above can be combined to give some of the following commonly used formulae

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

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

$$S = \frac{V_w}{V_v} = \frac{wG_s}{e}$$

$$\gamma = \frac{G_S + Se}{1+e}\gamma_W$$

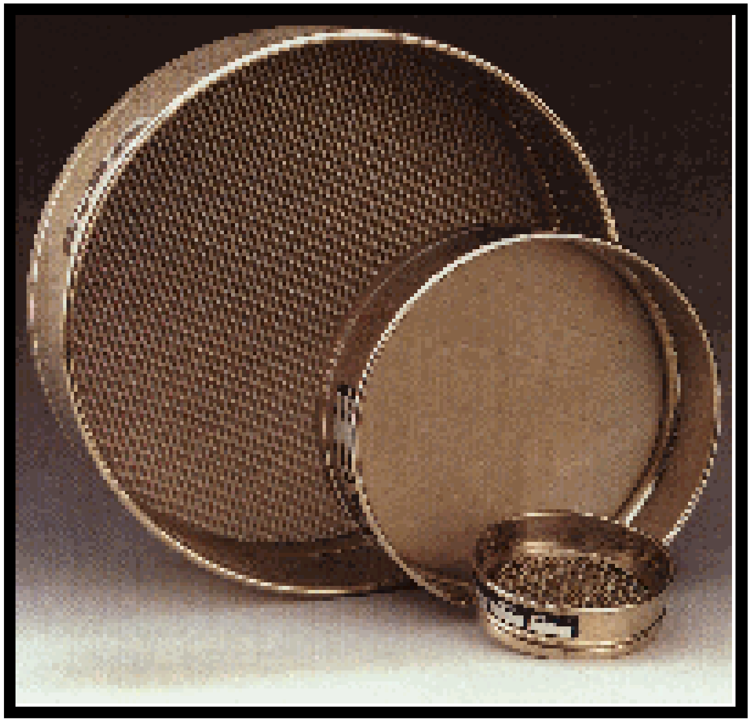
$$\gamma = \frac{G_S(1+w)}{1+e}\gamma_W$$

Example 1

- A soil sample obtained below GWT has a moisture content of 38% and Gs=2.70. Compute the void ratio, dry unit weight and buoyant unit weight of this soil.

Fine-Grained vs. Coarse-Grained Soils

- U.S. Standard Sieve - *No. 200*
  - 0.0029 inches
  - 0.075 mm



- "*No. 200*" means...

Engineering Characterization of Soils

Soil Properties that Control its Engineering Behavior

Particle Size

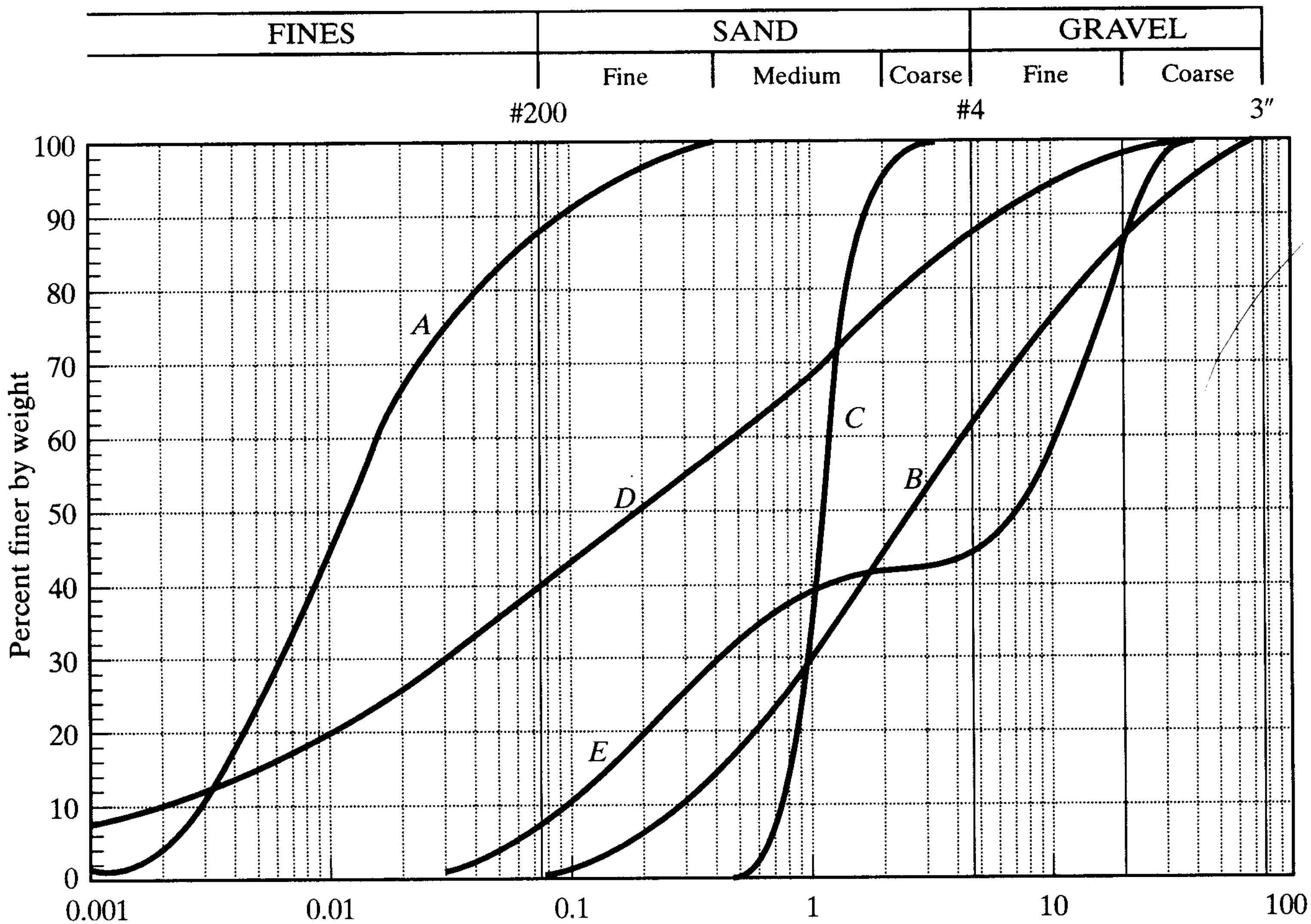
coarse-grained

- Particle/Grain Size Distribution
- Particle Shape

fine-grained

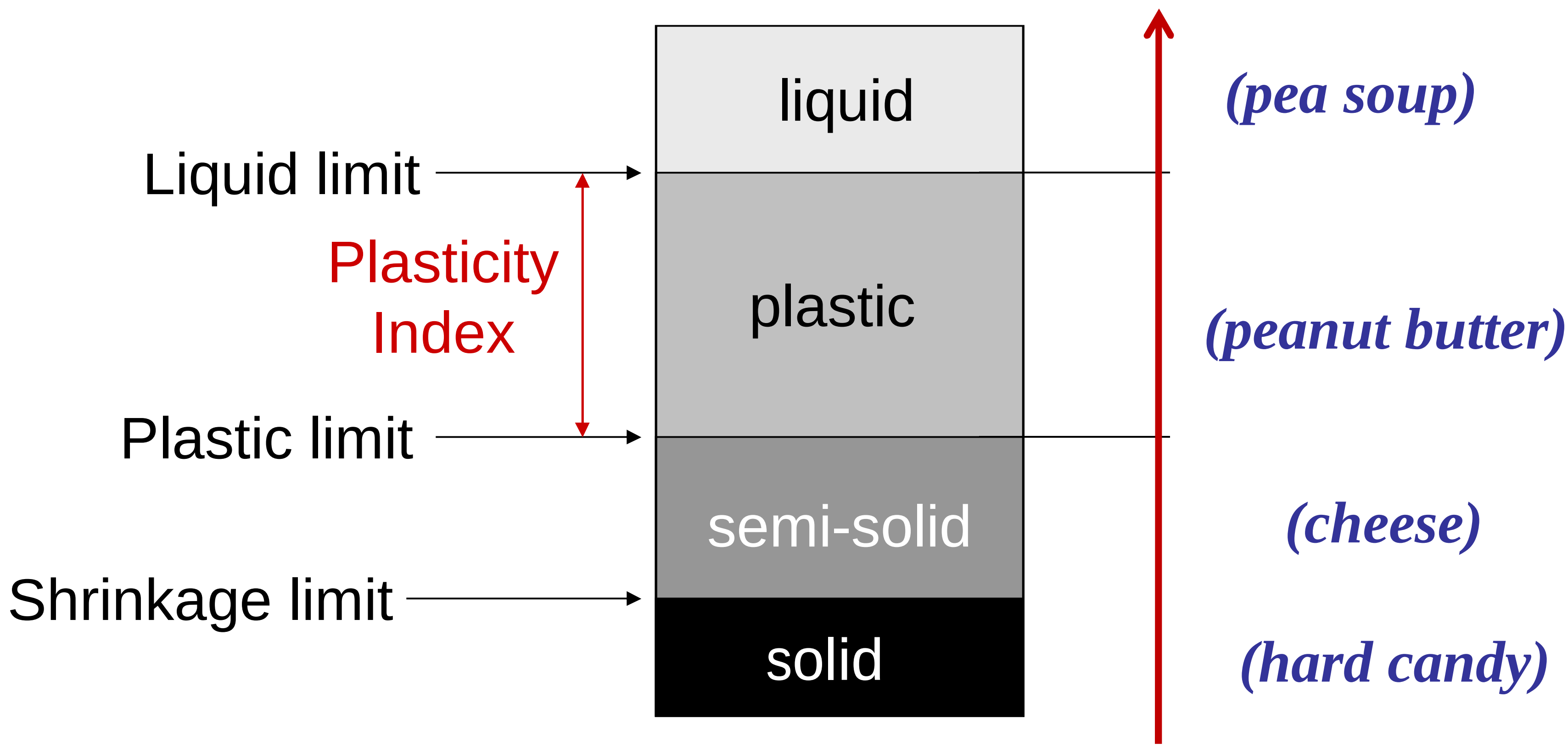
- Soil Plasticity

Grain Size Distribution Curves



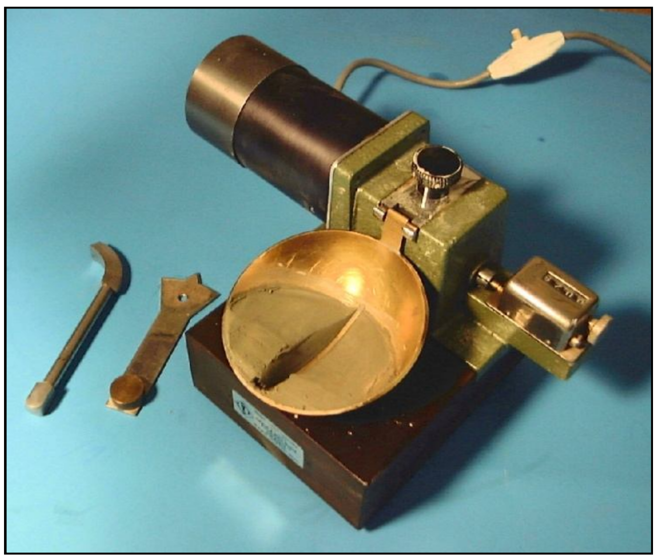
Atterberg Limits

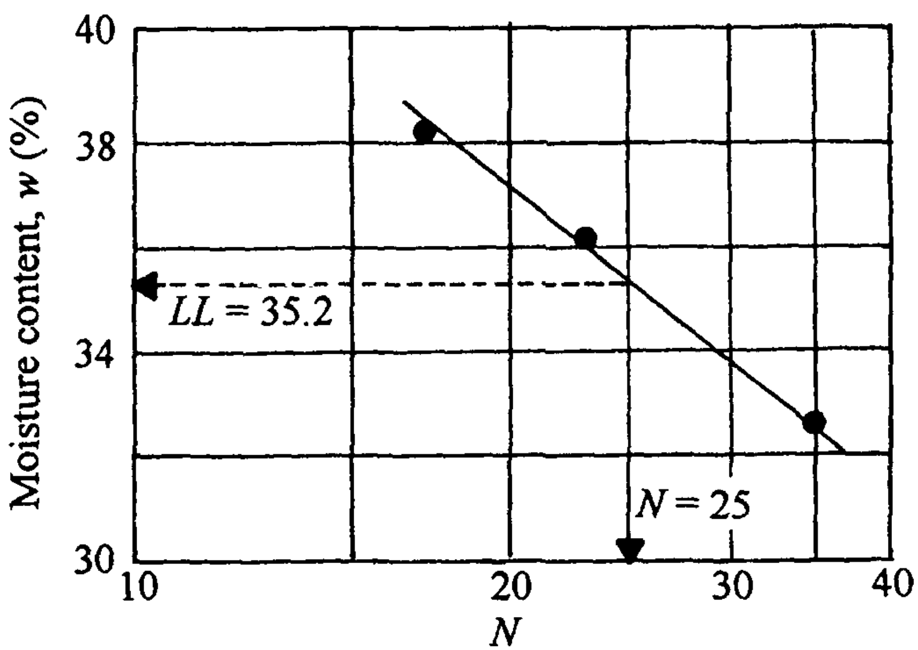
- Consistency of fine-grained soil varies in proportion to the water content



Liquid Limit (*LL or w_L*)

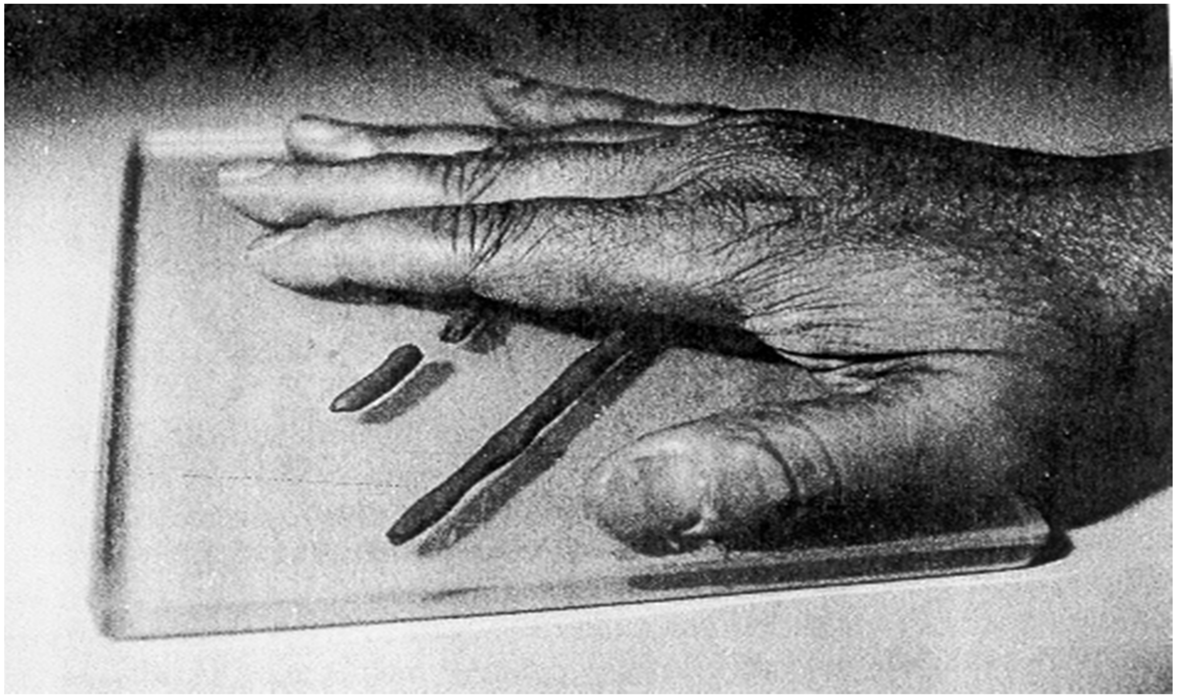
- The moisture content at which a 2 mm-wide groove in a soil pat will close for a distance of 0.5 in when dropped 25 times in a standard brass cup falling 1 cm each time at a rate of 2 drops/sec in a standard liquid limit device





Plastic Limit (*PL*, w_P)

- The moisture content at which a thread of soil just begins to crack and crumble when rolled to a diameter of 1/8 inches



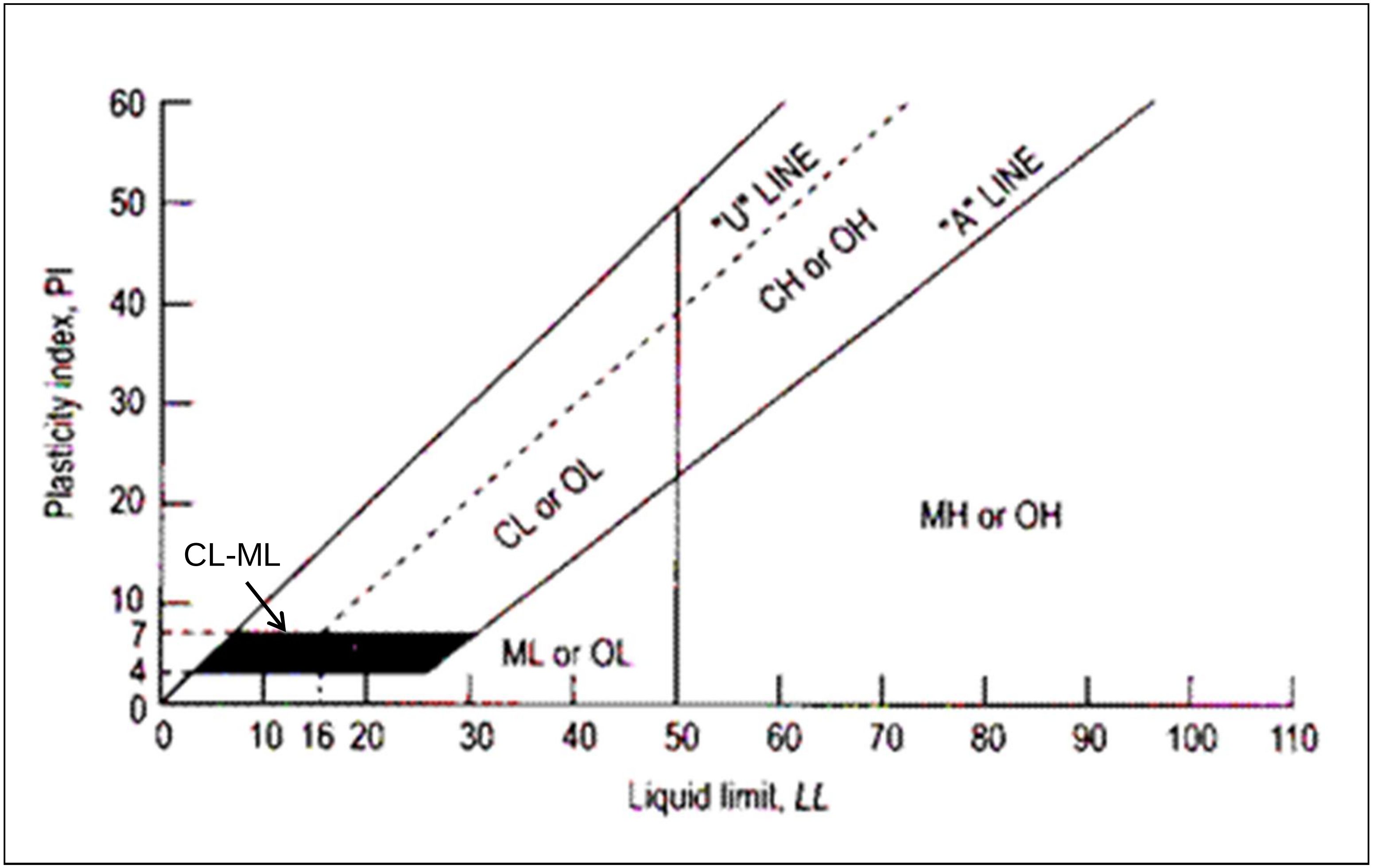
USCS Classification Chart

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM DESIGNATION D-2487)

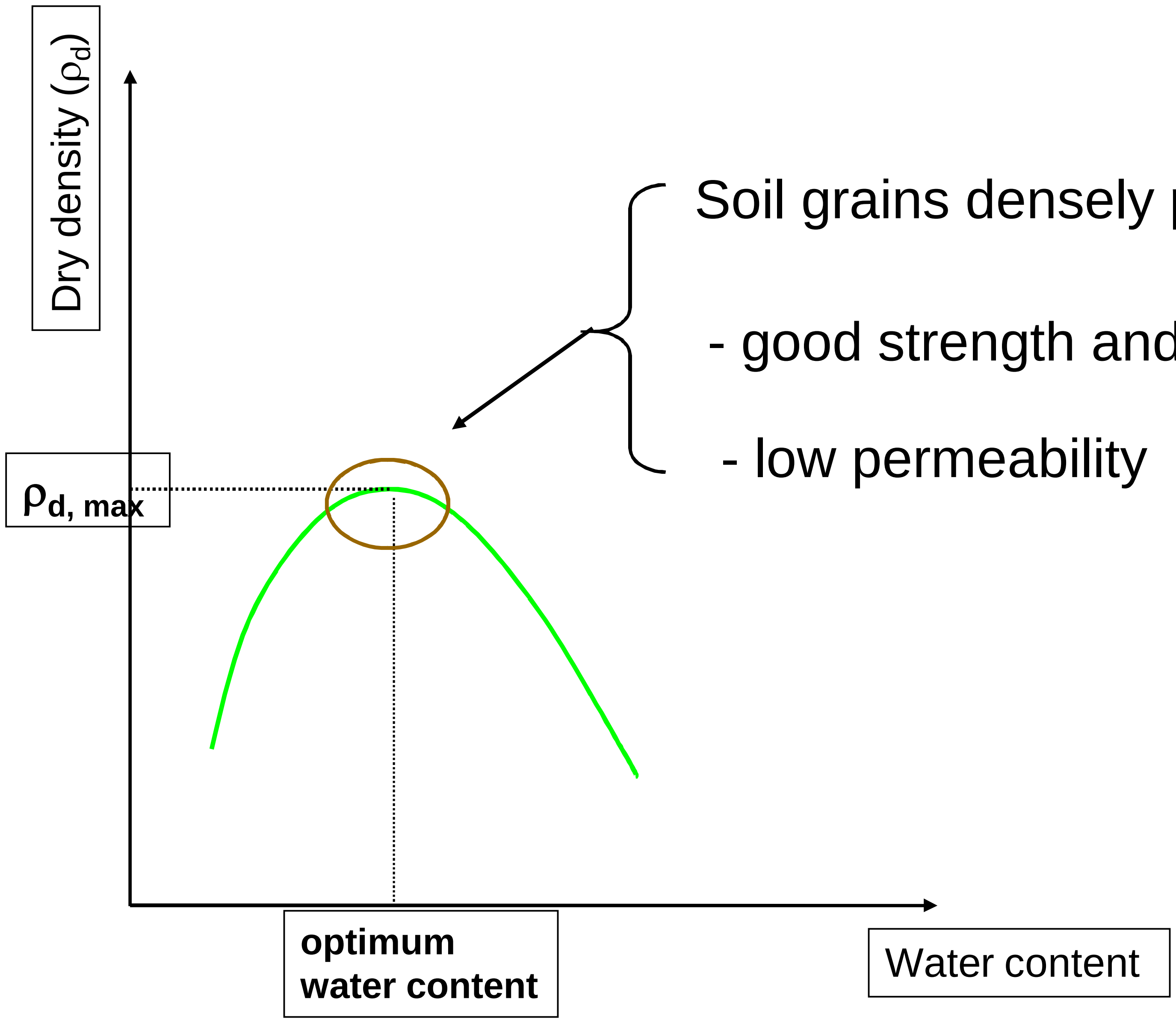
SOIL CLASSIFICATION CHART

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	Cleans 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 $C_c > 3$[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	Cleans 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]
		Sands 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 pass 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	(Liquid Limit - oven dried) / (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	(Liquid Limit - oven dried) / (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

Plasticity Chart



Compaction Curve



Soil grains densely packed

- good strength and stiffness

- low permeability

Laboratory Compaction Test

- to obtain the compaction curve and define the optimum water content and maximum dry density for a specific compactive effort.

Standard Proctor:

- 3 layers
- 25 blows per layer
- 2.7 kg hammer
- 300 mm drop

hammer

1000 ml compaction mould

Modified Proctor:

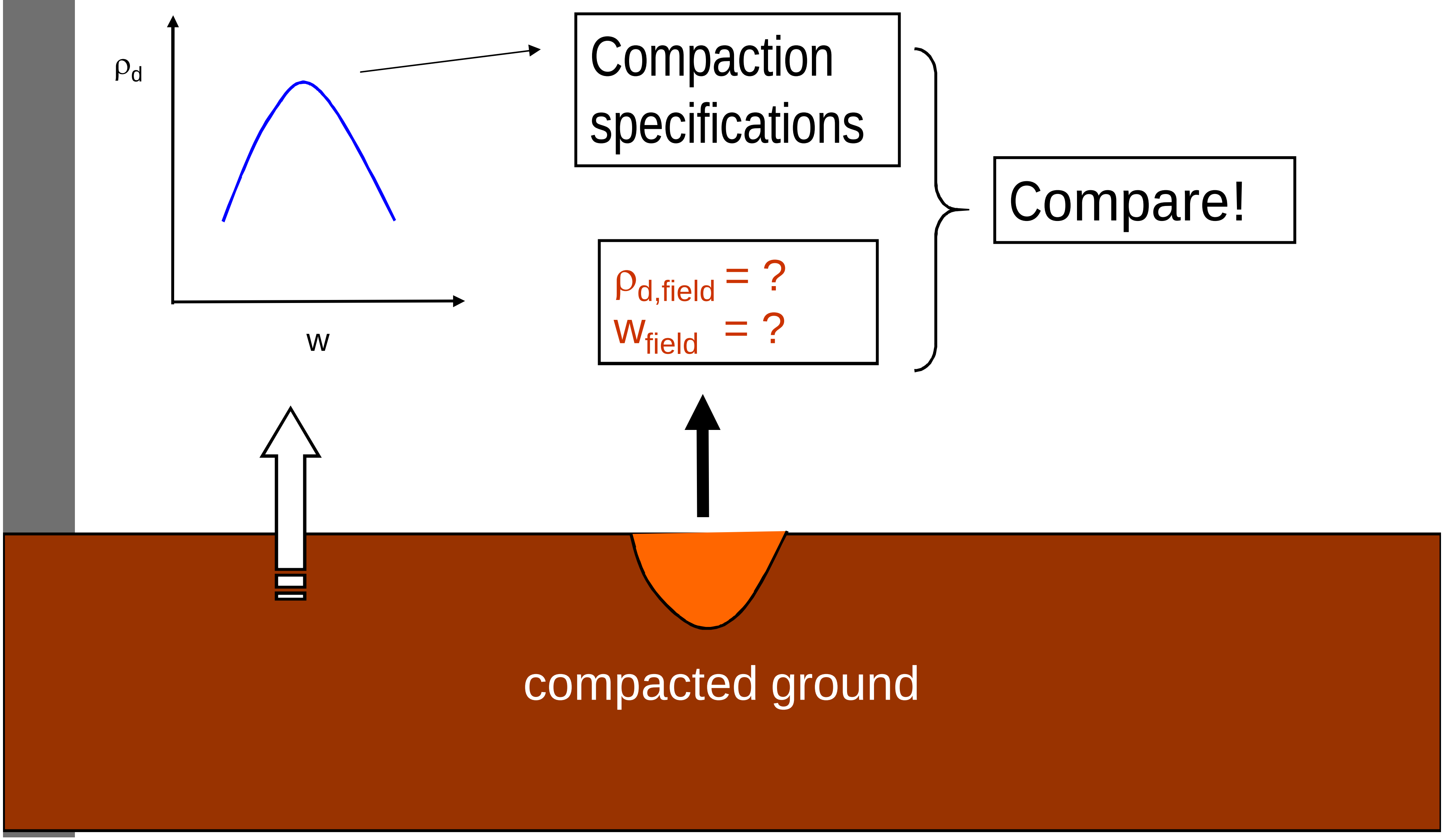
- 5 layers
- 25 blows per layer
- 4.9 kg hammer
- 450 mm drop

Compaction

Different types of rollers (clockwise from right)

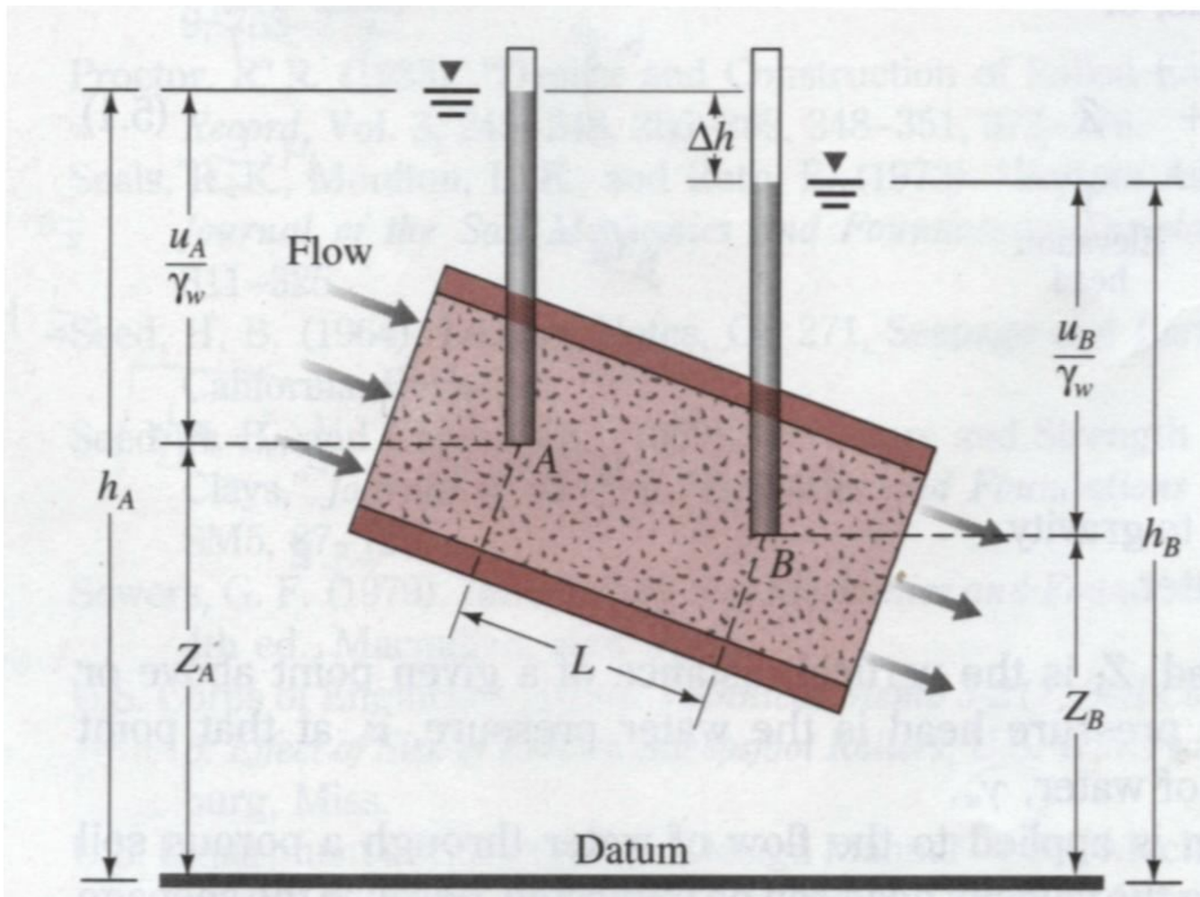
- Smooth-wheel roller
- Vibratory roller
- Pneumatic rubber tired roller
- Sheepsfoot roller

Compaction Control Test



Flow of Water in Soil

$$H = Z + \frac{u}{\gamma_w} + \frac{v^2}{2g_v} = h_z + h_p + h_v$$



Darcy Law

$$v = ki$$

$$q = A \times v = A \times k \times \frac{\Delta h}{L}$$

- Hydraulic Conductivity
  - » Average particle size
  - » Particle size distribution
  - » Void ratio (or density)
  - » Fabric/structure of the soil, layering, fissuring

TABLE 2.7 Coefficient of Permeability for Common Soil Types

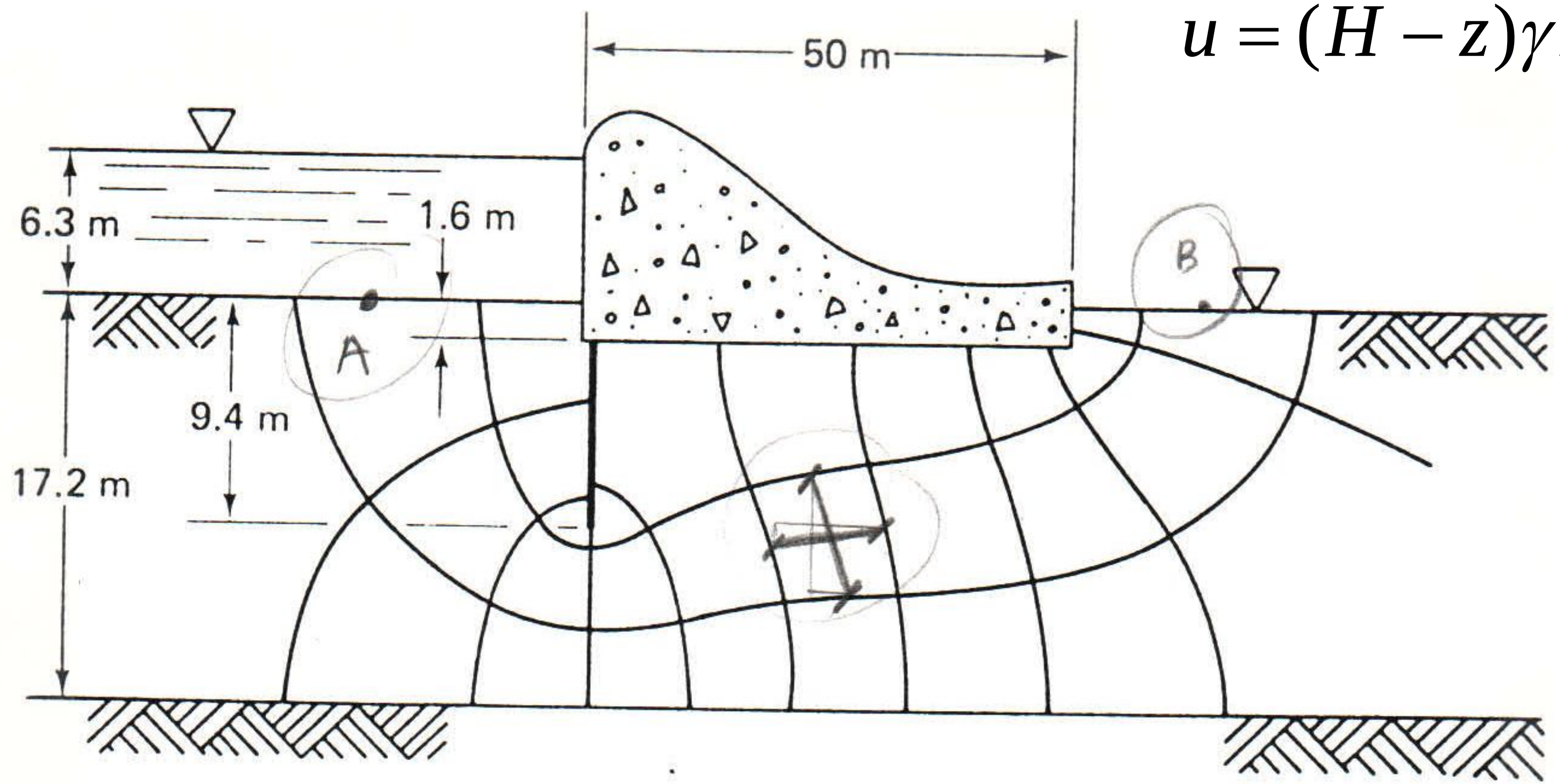
Soil type	k_z (cm/s)
Clean gravel	>1.0
Clean sands, clean sand and gravel mixtures	1.0 to 10^{-3}
Find sands, silts, mixtures comprising sands, silts, and clays	10^{-3} to 10^{-7}
Homogeneous clays	$<10^{-7}$

2D Flow

$$\frac{\partial^2 h}{\partial^2 x} + \frac{\partial^2 h}{\partial^2 y} = 0$$

$$q = k \times \Delta h \times L \times \frac{N_F}{N_D}$$

$$u = (H - z)\gamma_w$$

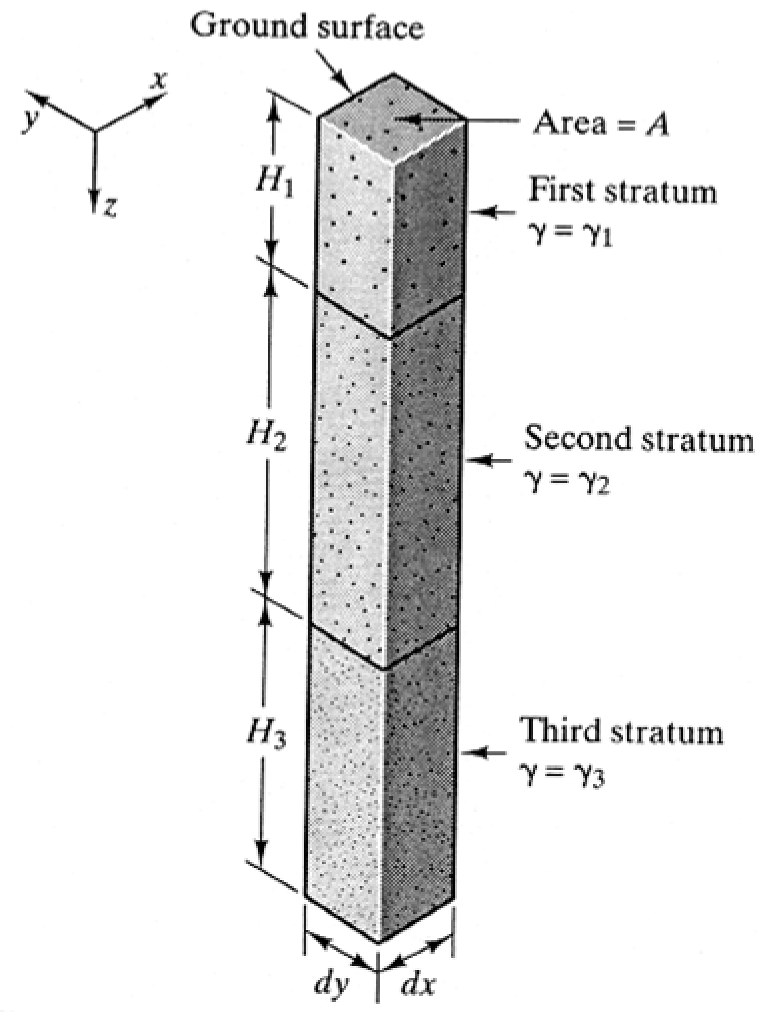


Sources of stress in the ground

- Geostatic Stress
  - Occur due to the weight above the point being evaluated

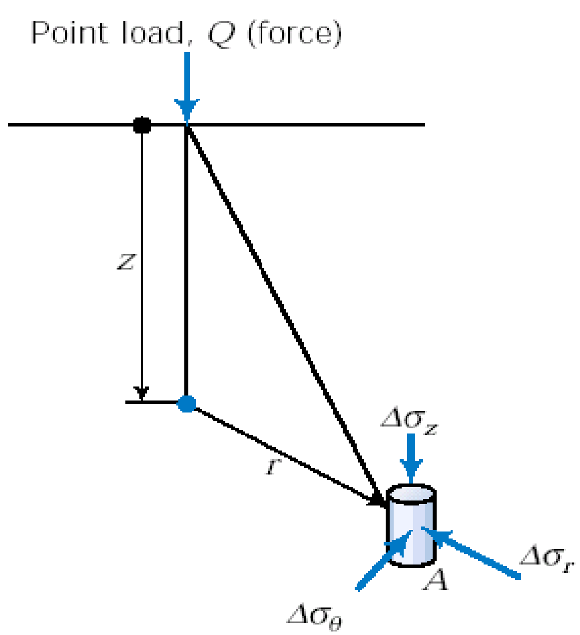
- Induced Stress (stress due to surface load)
  - Soils are generally loaded at the surface (or just below the surface)
  - External loads: structures, equipment, etc

Geostatic Stresses



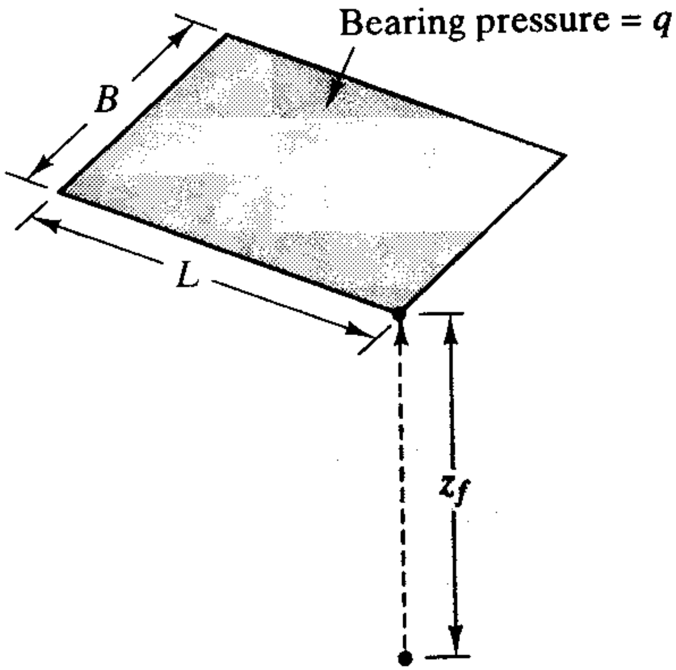
Stress distribution due to point load

- Boussinesq (1885) solution
  - For a homogeneous, isotropic, linear elastic, semi-infinite medium
  - Solutions for stress increments in the vertical, radial and the transverse directions yield

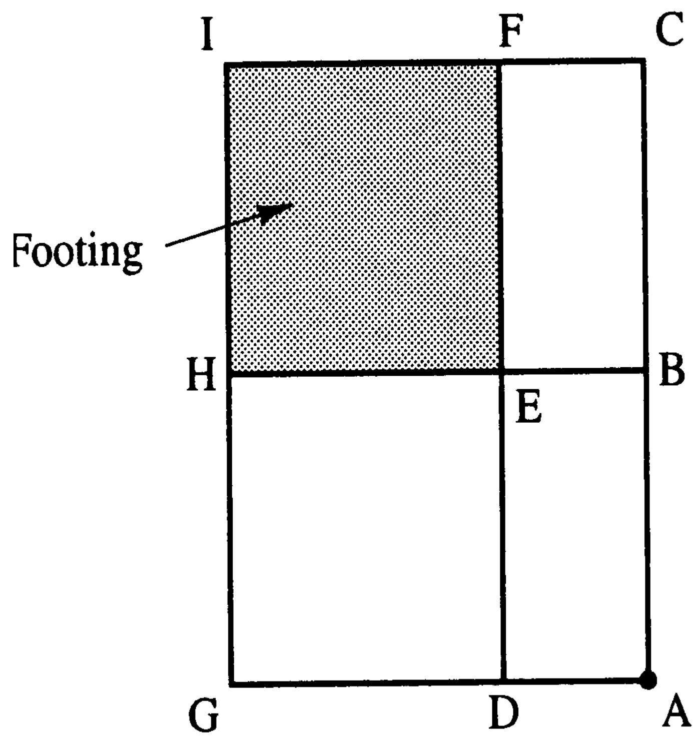


$$\Delta\sigma_z = \frac{3Q}{2\pi z^2}\left(\frac{1}{1+(r/z)^2}\right)^{5/2} = \frac{3Qz^3}{2\pi L^5}$$

Boussinesq's Method



$$\Delta\sigma_z = I_z q$$



To Compute Stress at Point A Due to Load from Footing EFHI:

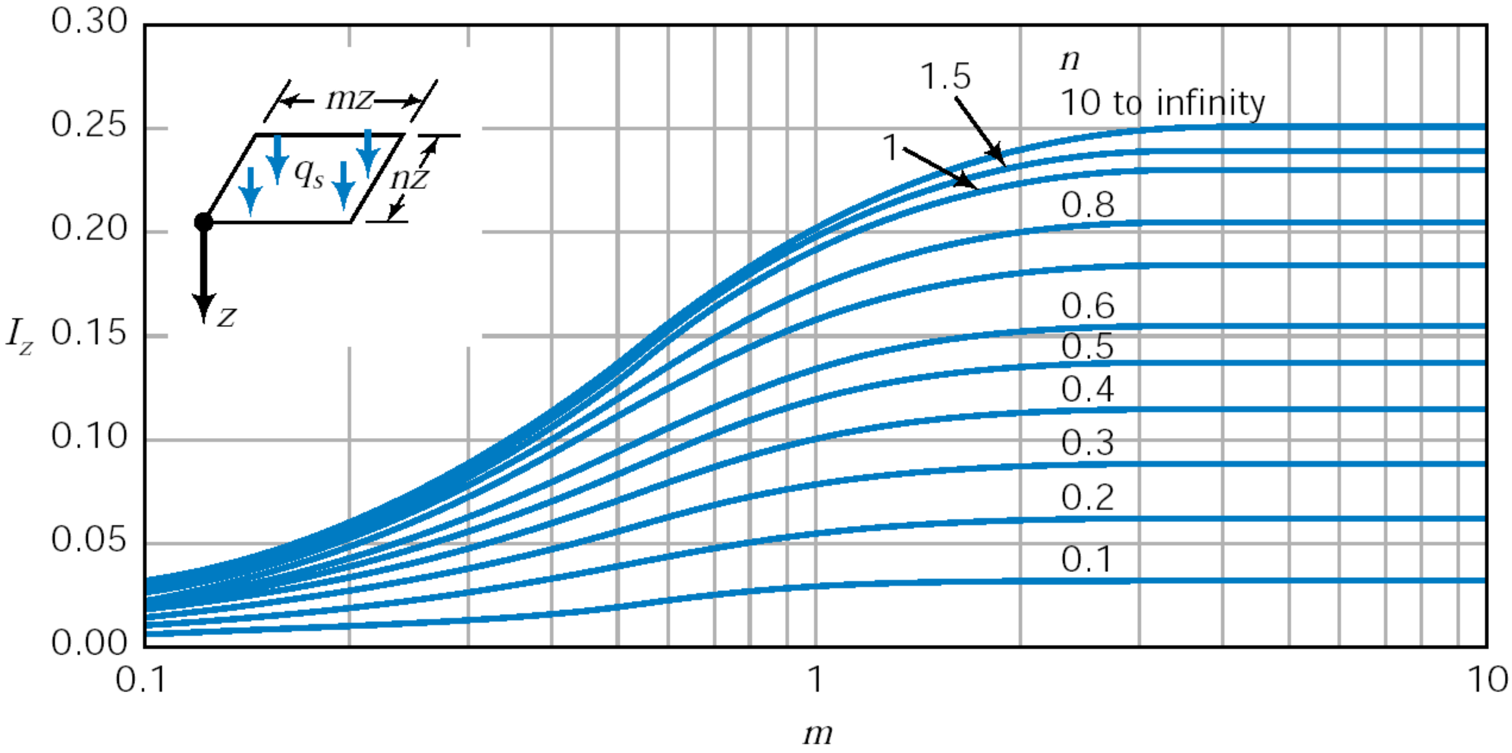
$$(\Delta\sigma_v')_A = (\Delta\sigma_v')_{ACGI} - (\Delta\sigma_v')_{ACDF} - (\Delta\sigma_v')_{ABGH} + (\Delta\sigma_v')_{ABDE}$$

Boussinesq's Method

- Integration by Newmark (1935) to obtain I_z yielded

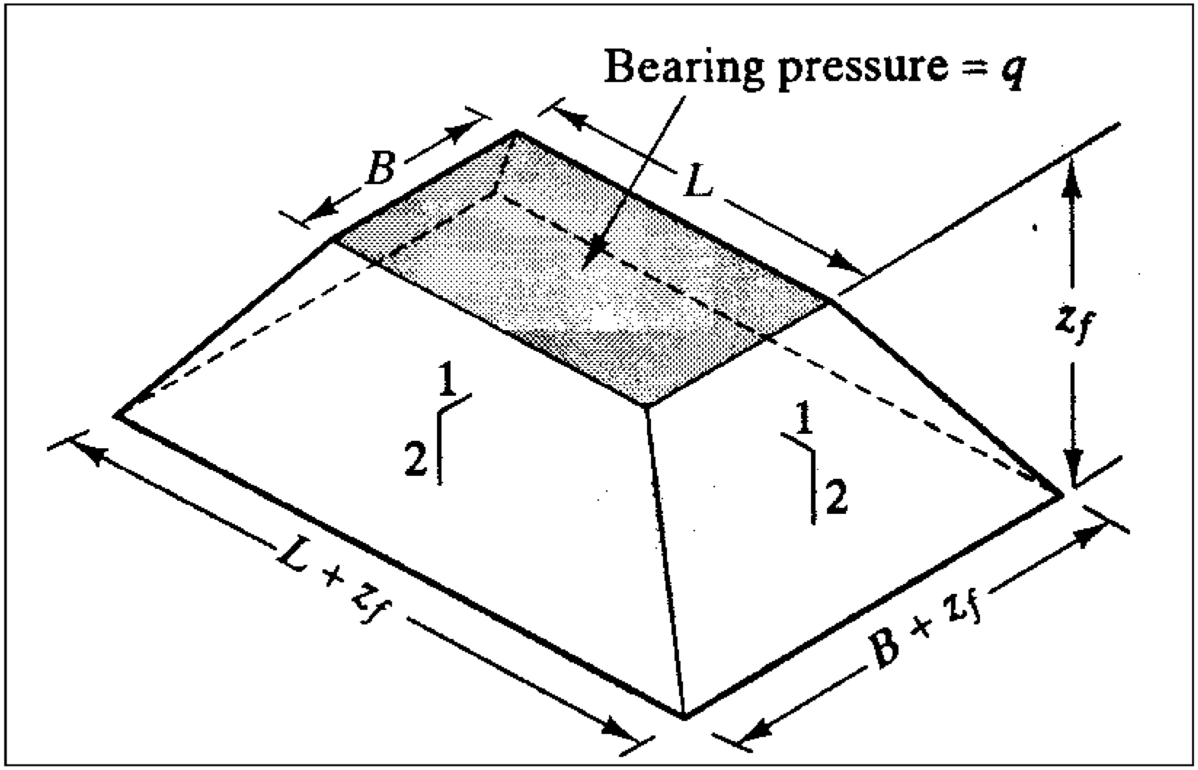
$$I_z = \frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right)\right] + \tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2-m^2n^2+1}\right)$$

(for $m^2+n^2-m^2n^2+1>0$) Where, $m = B/z$, & $n = L/z$



Approximate Methods

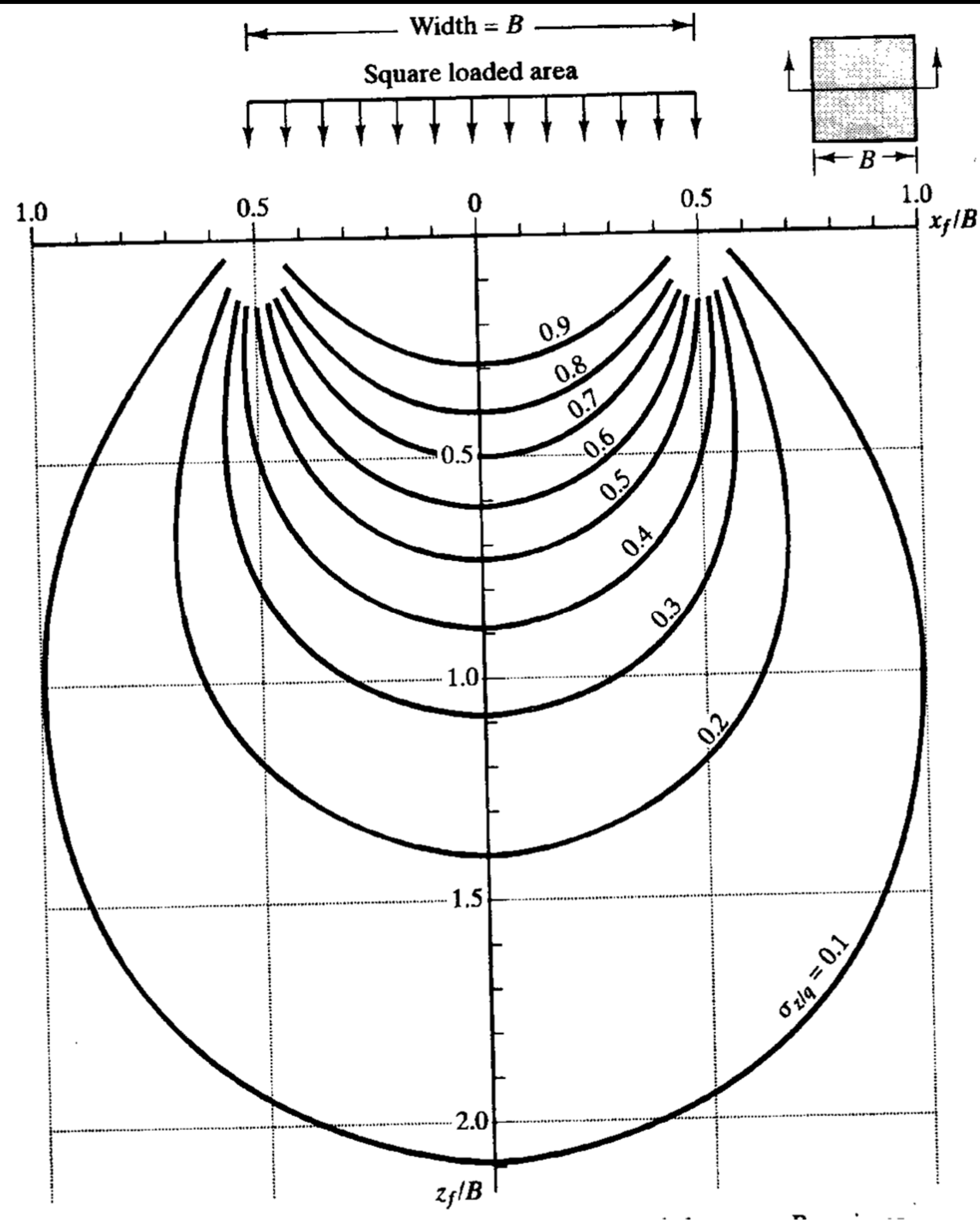
- 1:2 method (60° approximation)



Pressure Bulbs

Newmark's Solution of Boussinesq Method

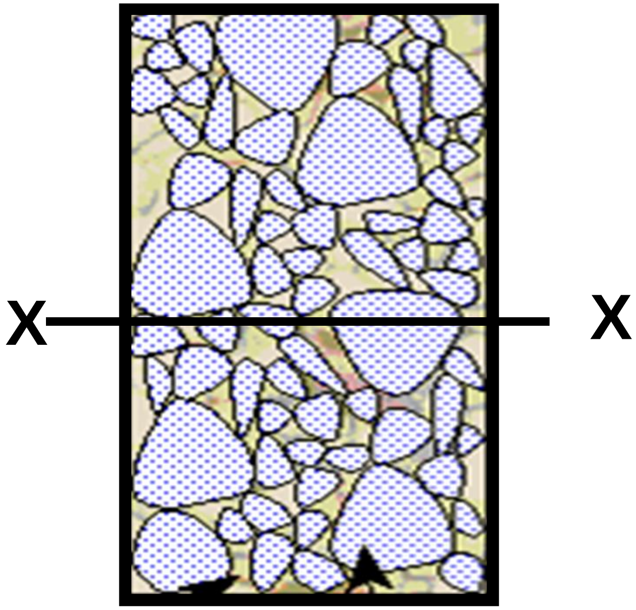
$$\Delta\sigma_z = I_z q$$



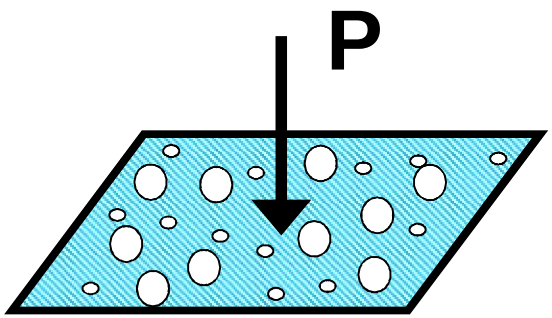
Example 2

- A column load of 400 kN will act on a rectangular footing with length of 3 m and width of 2 m. Compute the induced vertical stress at depth of 1 m below the centre of this footing.

Stresses in Soil Masses



Soil Unit



Area = A

$$\sigma = P/A$$

Assume the soil is fully saturated, all voids are filled with water.

Example 3

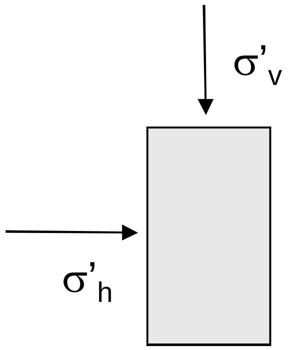
- A layer of saturated clay 4m thick is overlain by sand 5m deep. The GWT is located at a depth of 3m below the ground surface. ($\gamma_{sat\ (clay)}$=18 kN/m3, $\gamma_{sat\ (sand)}$=19 kN/m3; $\gamma_{(sand)}$=17 kN/m3) . Determine the effective stress distributions.

- If the sand above water table is saturated to a height of 1m above GWT due to capillary rise, how will this affect the stress distributions?

Relationship between Horizontal and Vertical Stresses

- Horizontal stresses in a soil mass are often expressed as a percent of the vertical stresses
- The pressure at a point in a fluid is the same in all directions
- This is not applicable to soils

$$\sigma_h = K\sigma_V$$



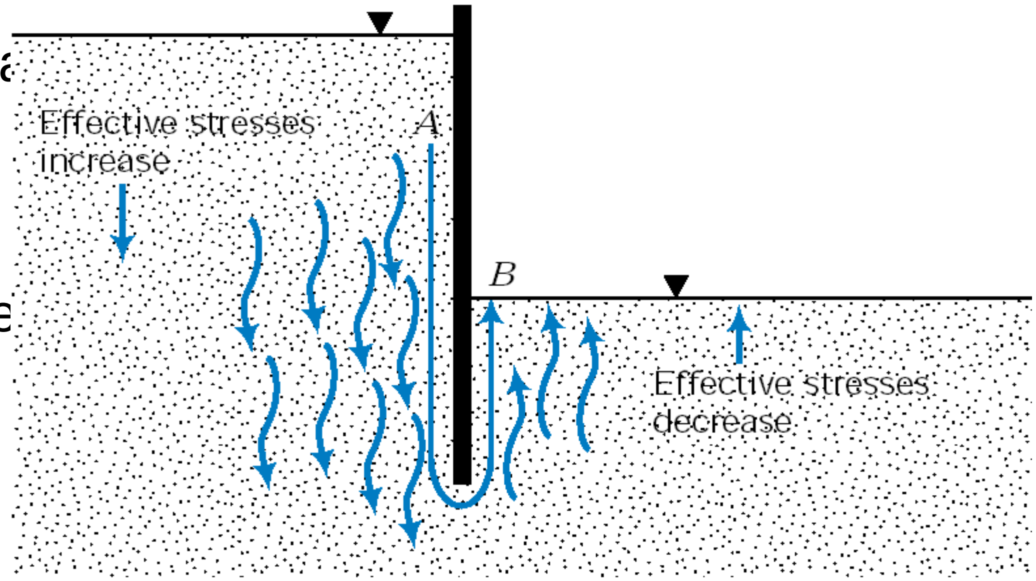
where *K* is the **earth pressure coefficient**

$$K_0 = 1 - \sin(\phi)$$

Application

There is a transfer of energy from the moving water to the soil particles. This transfer of energy is referred to as the *seepage force*.

- Seepage Forces play an important role in geotechnical design
- Left side of the wall
  - Effective stresses increase
  - Lateral forces on the wall increase



- Right side
  - Effective stresses decrease,
  - Resistance decreases

$$\sigma_z' = \gamma' z + iz\gamma_w \qquad \sigma_z' = \gamma' z - iz\gamma_w$$

Critical Hydraulic gradient

- The effective stress would decrease as the upward seepage velocity increase

$$\sigma_z^{'} = \gamma' z - (i\gamma_w)z$$

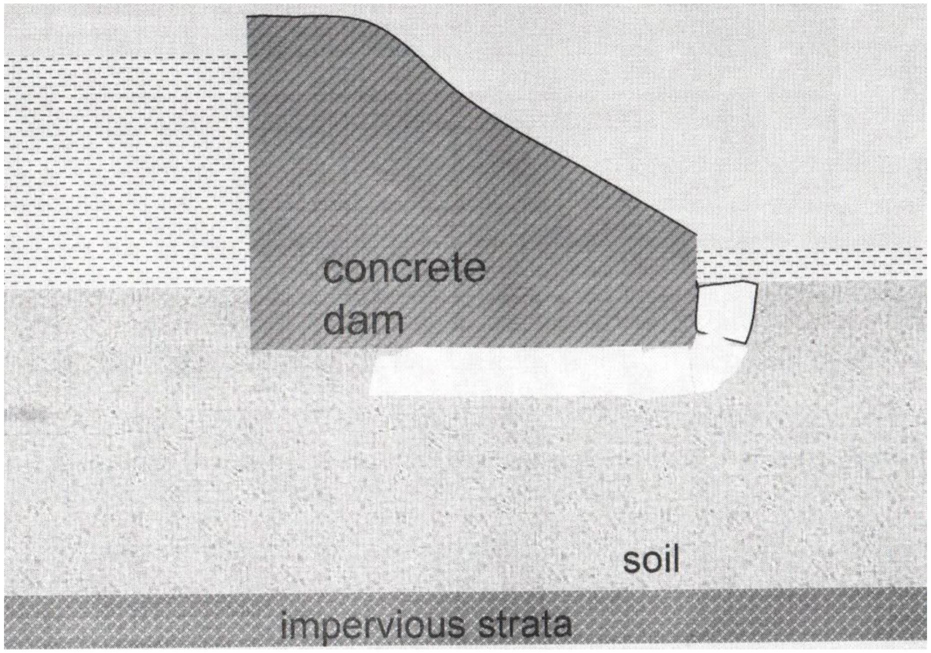
$$\text{for } i = i_{cr} = \gamma' / \gamma_w , \quad \sigma_z^{'} = 0$$

- This is called the “quick” (piping or boiling) condition

Piping in Granular Soils

- Piping is a very serious problem. It leads to downstream flooding which can result in loss of lives.

- Therefore, provide adequate safety factor against piping.



$$F.S_{Piping} = \frac{i_{cr}}{i_{exit}}$$

Typically 5-6

Increase in Vertical Effective Stress

- Due to a Placement of a fill

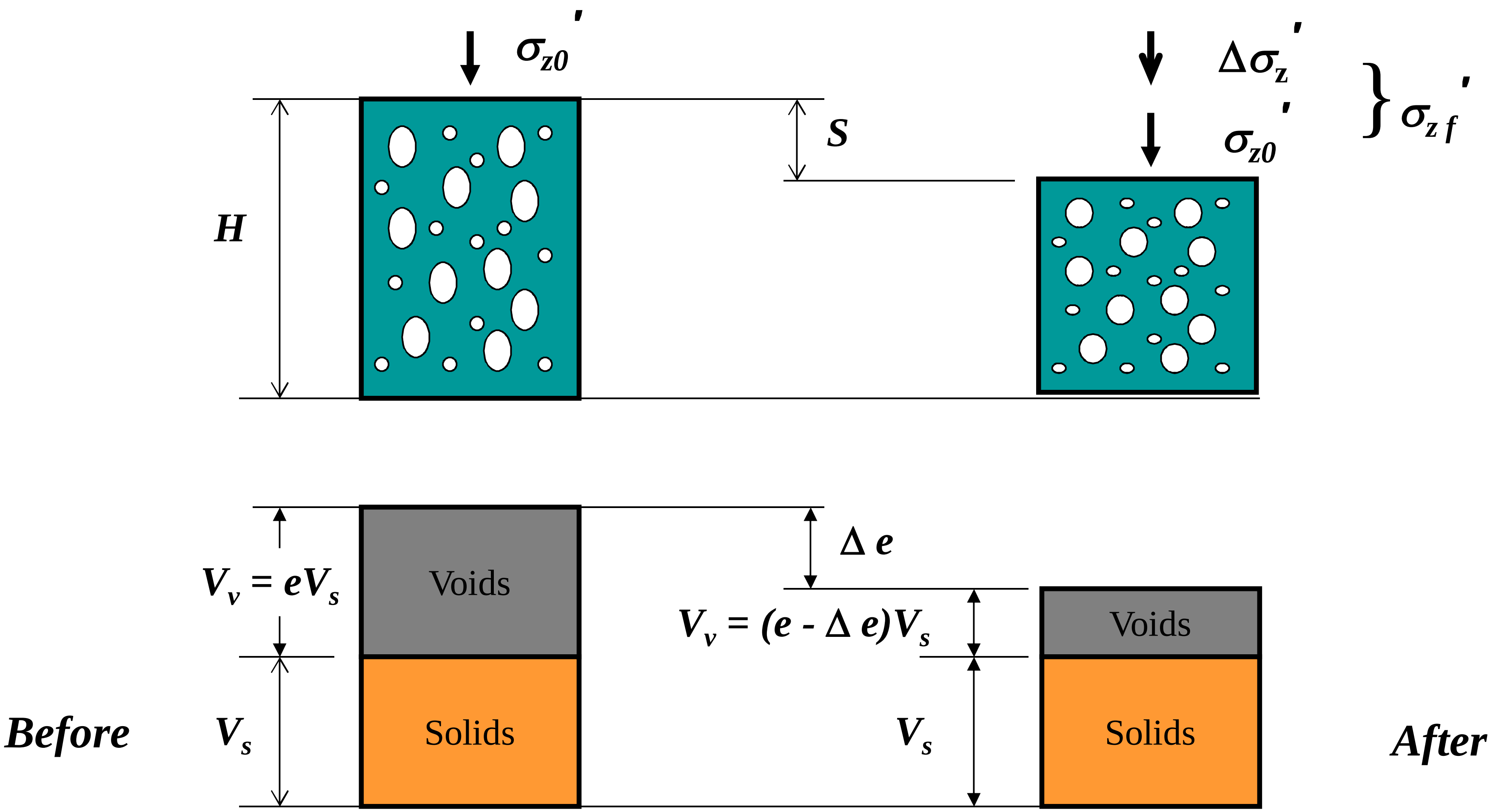
$$\sigma'_{zf} = \sigma'_{z0} + \gamma_{fill} H_{fill}$$

- Due to an external load

$$\sigma'_{zf} = \sigma'_{z0} + \left(\sigma_z\right)_{induced}$$

- Due to change in GWT

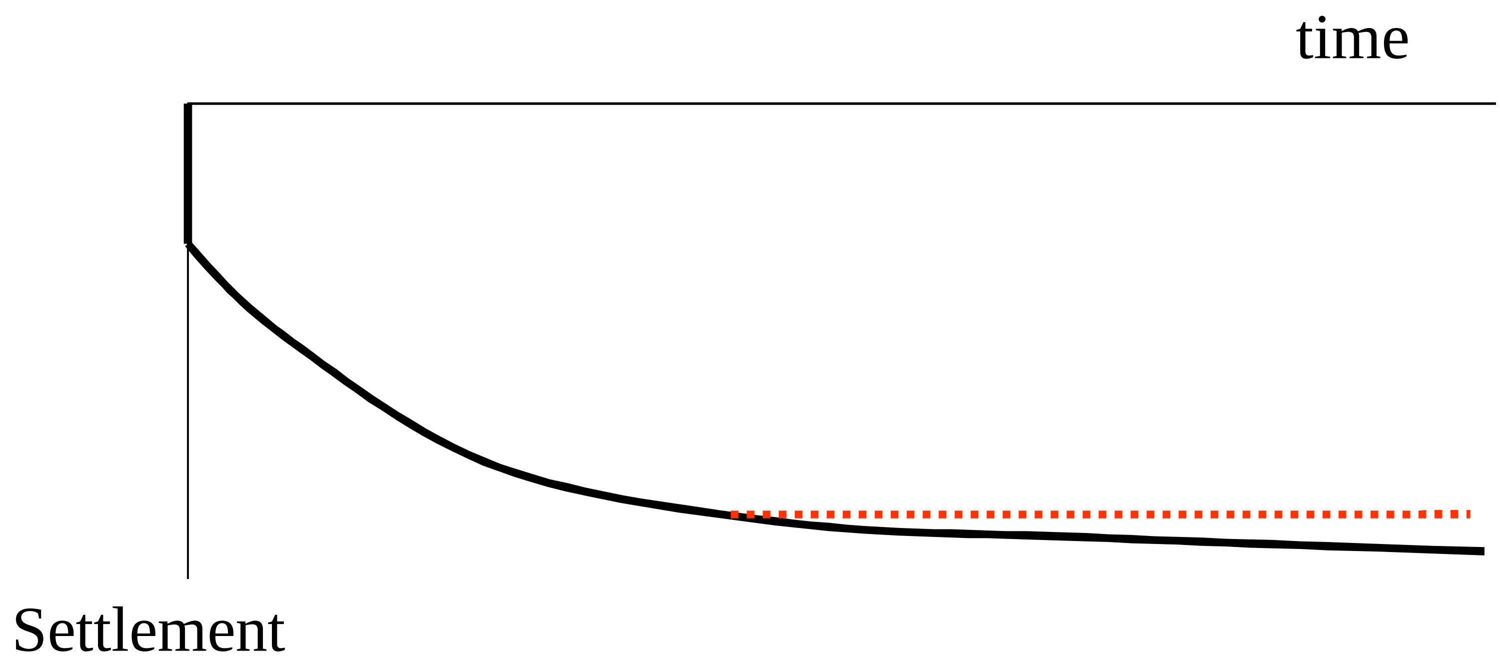
Settlement



σ_{z0}'
$\Delta\sigma_z'$
σ_{z0}'
σ_{zf}'
S
H
Δe
$V_v = eV_s$
Voids
$V_v = (e - \Delta e)V_s$
Voids
Before
V_s
Solids
V_s
Solids
After

Settlement

- Distortion Settlement (Immediate)
- Consolidation (Time Dependent)
- Secondary Compression



Immediate Settlement

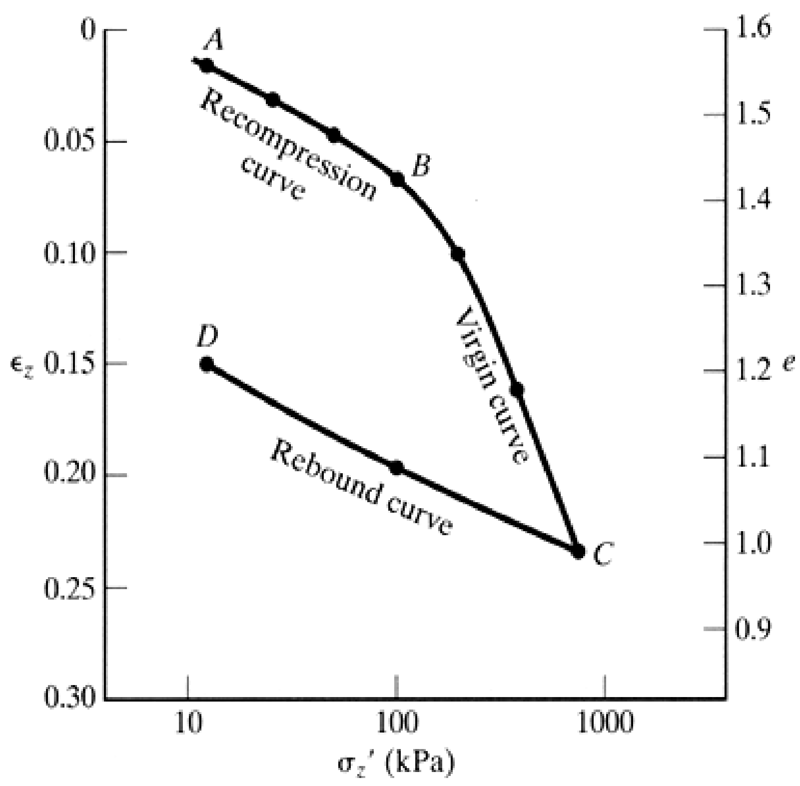
- Depends on elastic modulus, Poisson's ratio and area of the loading

$$S_i = \frac{q.B}{E} I_s$$

Influence Factors

Shape of area	I_s		
	Centre	Corner	Average
Square	1.12	0.56	0.95
Rectangle L/B=2	1.52	0.76	1.30
Rectangle L/B=5	2.10	1.05	1.83
Circle	1.00	0.64	0.85

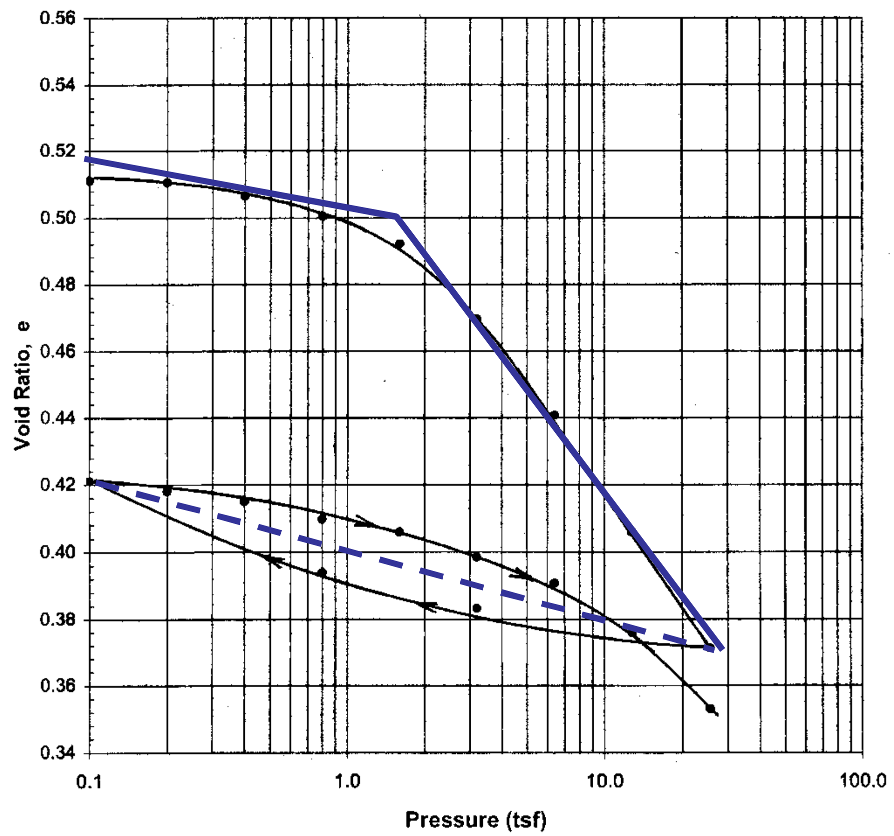
Consolidation



CONSOLIDATION TEST REPORT

PROJECT: Austin Civic Center
SAMPLE: B5, 6.5 - 8.0
LOCATION: Austin, Texas
DESCRIPTION: Lean Clay w/sand

Sample Type:	Undisturbed		
Diameter:	2.5 inches	Classification:	CL
Height:	1.0 inch	Unit Dry Weight (pcf):	109.4
Initial Moisture Content (%):	17.1	Final Moisture Content (%):	13.4
Initial Void Ratio:	0.5118	Final Void Ratio:	0.3531
Initial Saturation (%):	88	Final Saturation (%):	101

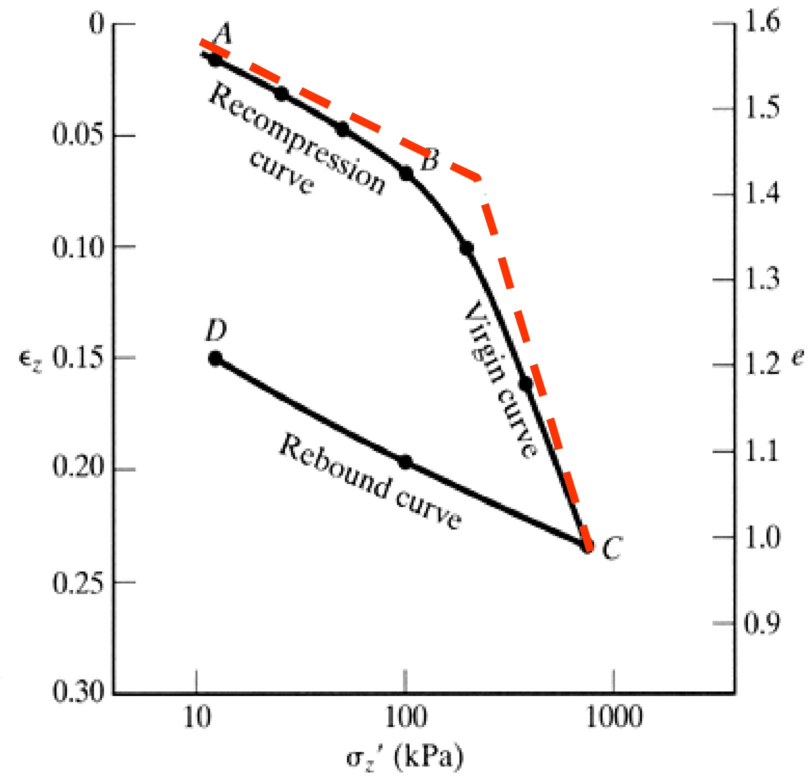


Normally and Over-Consolidated Soils

$$\sigma'_{zo} = \sigma'_p$$ ..... Normally consolidated

$$\sigma'_{zo} < \sigma'_p$$ ..... Over consolidated

$$\sigma'_{zo} > \sigma'_P$$ ..... Under consolidated



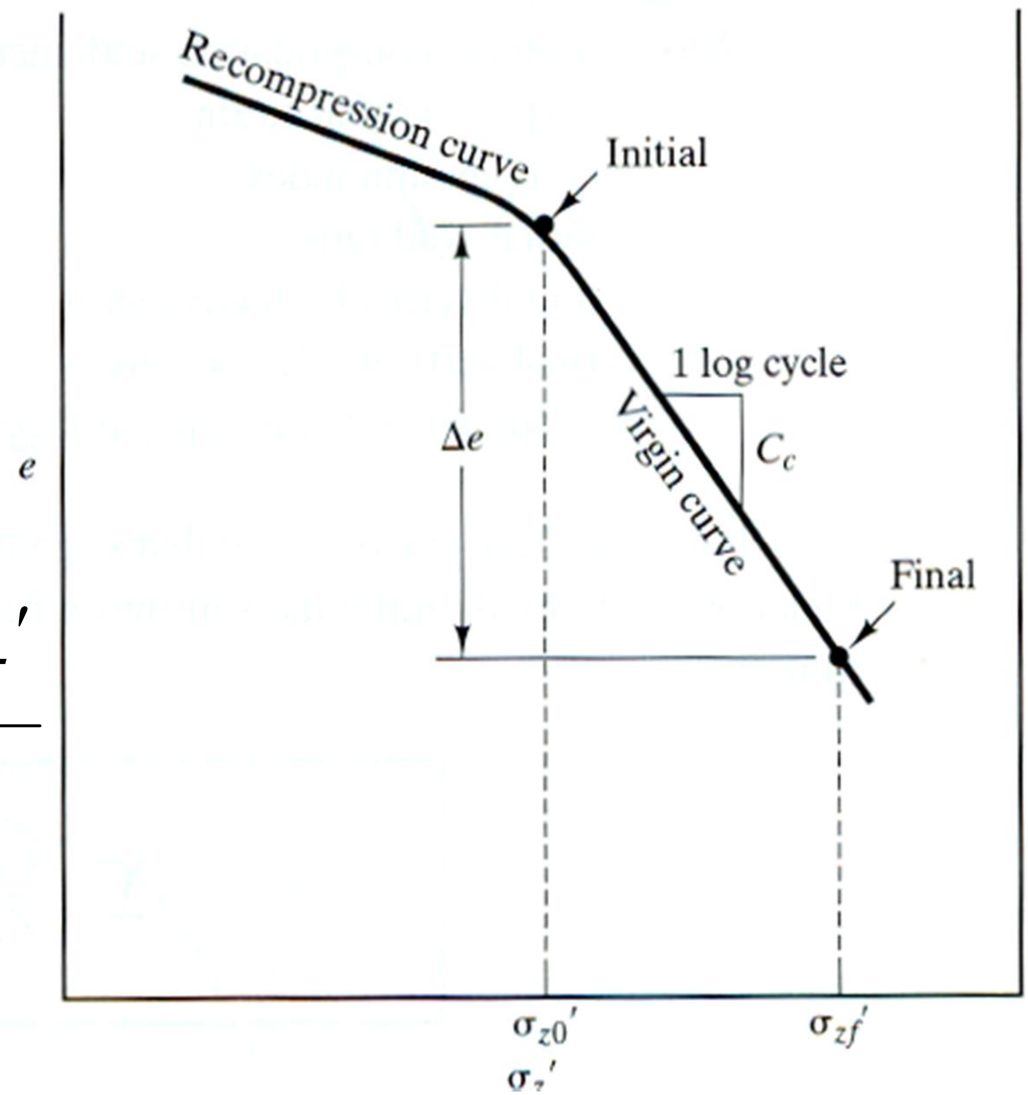
$$OCR = \frac{\sigma'_P}{\sigma'_{zo}}$$ ..... Over consolidation ratio

Settlement Predictions: N.C. Clays

The initial stress is the maximum stress the soil has ever seen (σ_{z0}'=σ_p').

The final stress exceeds the preconsolidation pressure.

Settlement is due to virgin compression only and we use Cc



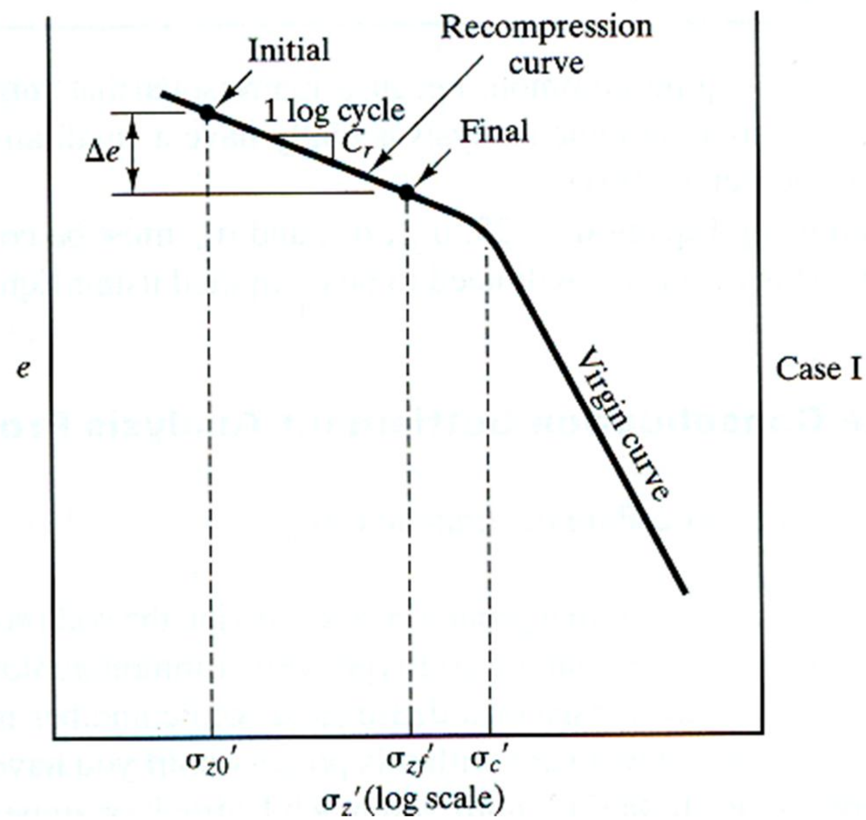
$$s_c = C_c \frac{H_o}{1+e_o} \log \frac{\sigma_{zo}' + \Delta\sigma'}{\sigma_{zo}'}$$

Settlement Predictions: O.C. Clays

The initial stress and final stress are both less than the preconsolidation pressure ($\sigma_{zo}' + \Delta\sigma_z' \leq \sigma_p'$).

Settlement is due to recompression only and we use Cr.

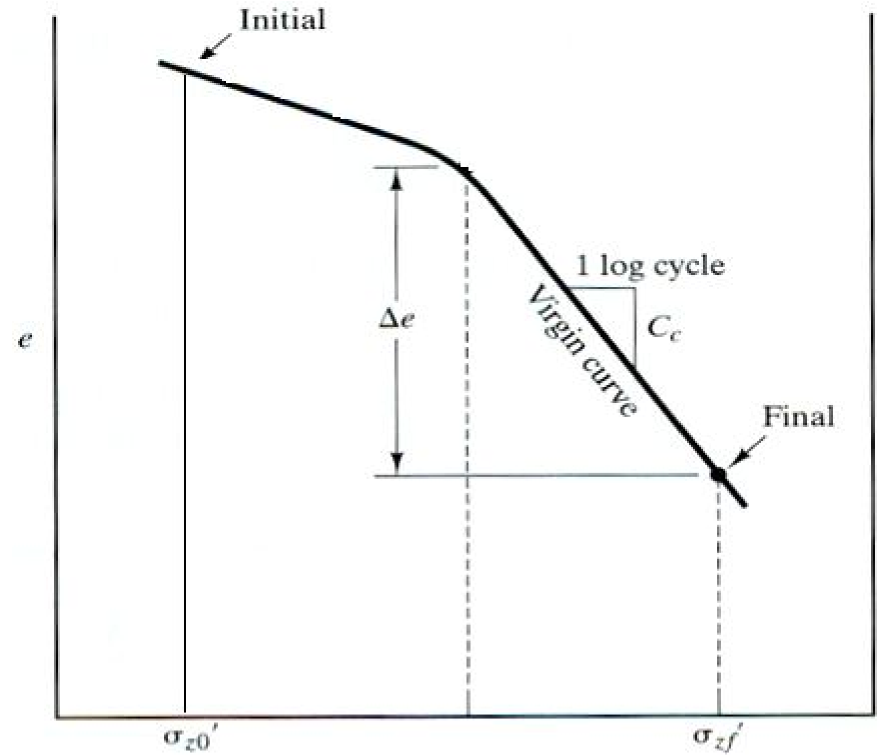
$$s_c = C_r \frac{H_o}{1+e_o} \log \frac{\sigma_{zo}' + \Delta\sigma'}{\sigma_{zo}'}$$



Settlement Predictions: Case III

$$s_c = C_r \frac{H_o}{1+e_o} \log \frac{\sigma_p'}{\sigma_{zo}'} + C_c \frac{H_o}{1+e_o} \log \frac{\sigma_{zo}' + \Delta\sigma'}{\sigma_p'}$$

If initially $\sigma_{zo}' \leq \sigma_p'$ and after consolidation $\sigma_{zo}' + \Delta\sigma_z' > \sigma_p'$ then combine the normally consolidated and overconsolidated settlement equations



Example 4

- A 4 m thick clay layer (w=36%, Gs=2.75) is located under a sand with a thickness of 4 m (e=0.6, Gs=2.65). GWT is 2 m below the surface. A laboratory consolidation test on a sample of this clay has given the following results:

Pressure (kPa)	Void ratio
100	0.9
200	0.6

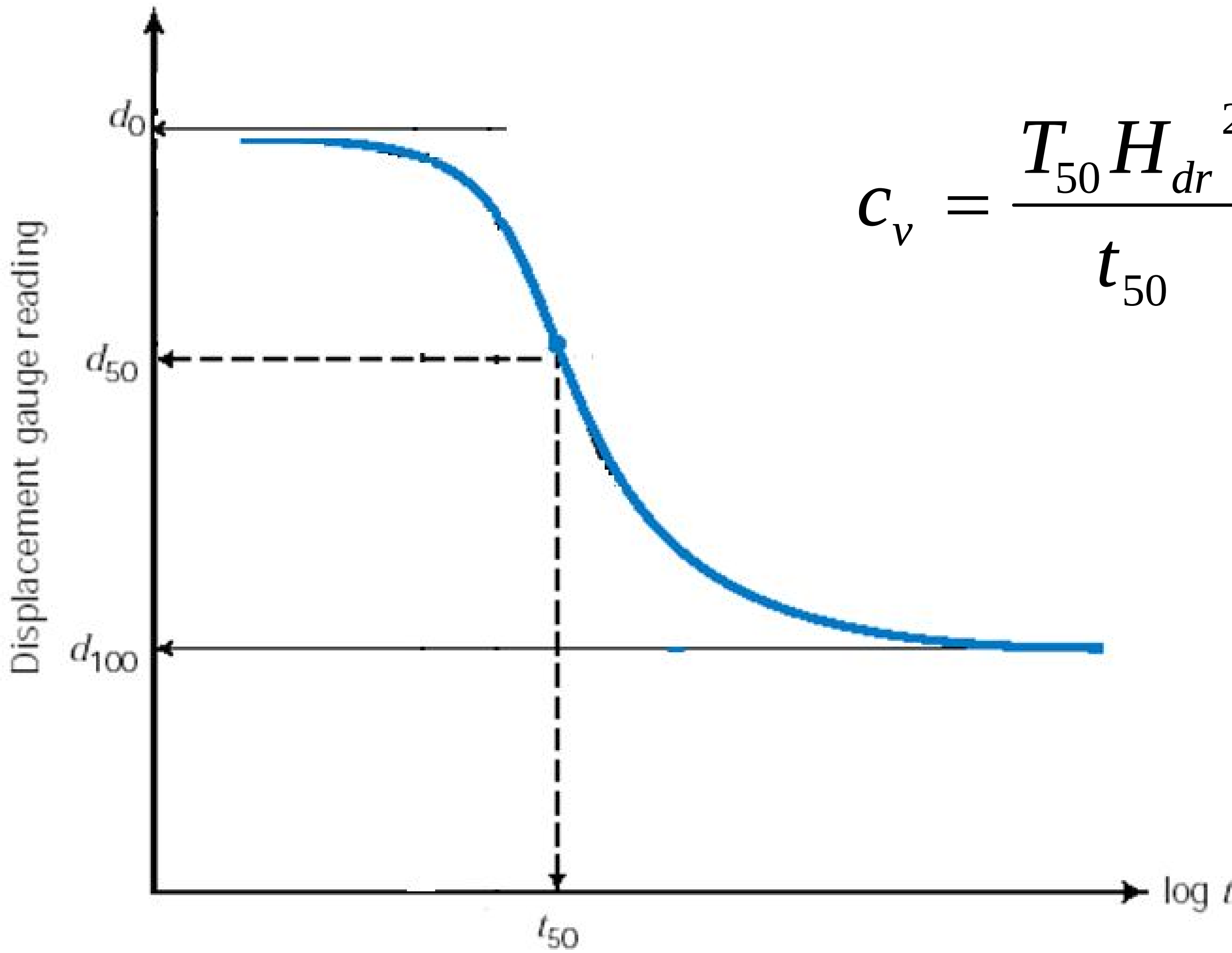
  - Determine the compression index of the soil (NC).
  - If the final effective stress at the middle of this clay is increased to 150 kPA, what would be the total consolidation settlement?

Rate of Consolidation

- When static loads are applied to a structure element like beams or columns, the resulting deformations occur as fast as the loads are applied.
- However, deformations in soil occur much more slowly, especially in saturated clays.
- For a low permeability clay layer, the total consolidation settlement may take several years to reach equilibrium conditions
- It is important to estimate the rate of consolidation and predict the time required for final settlement

Determination of the Coefficient of Consolidation (Cv)

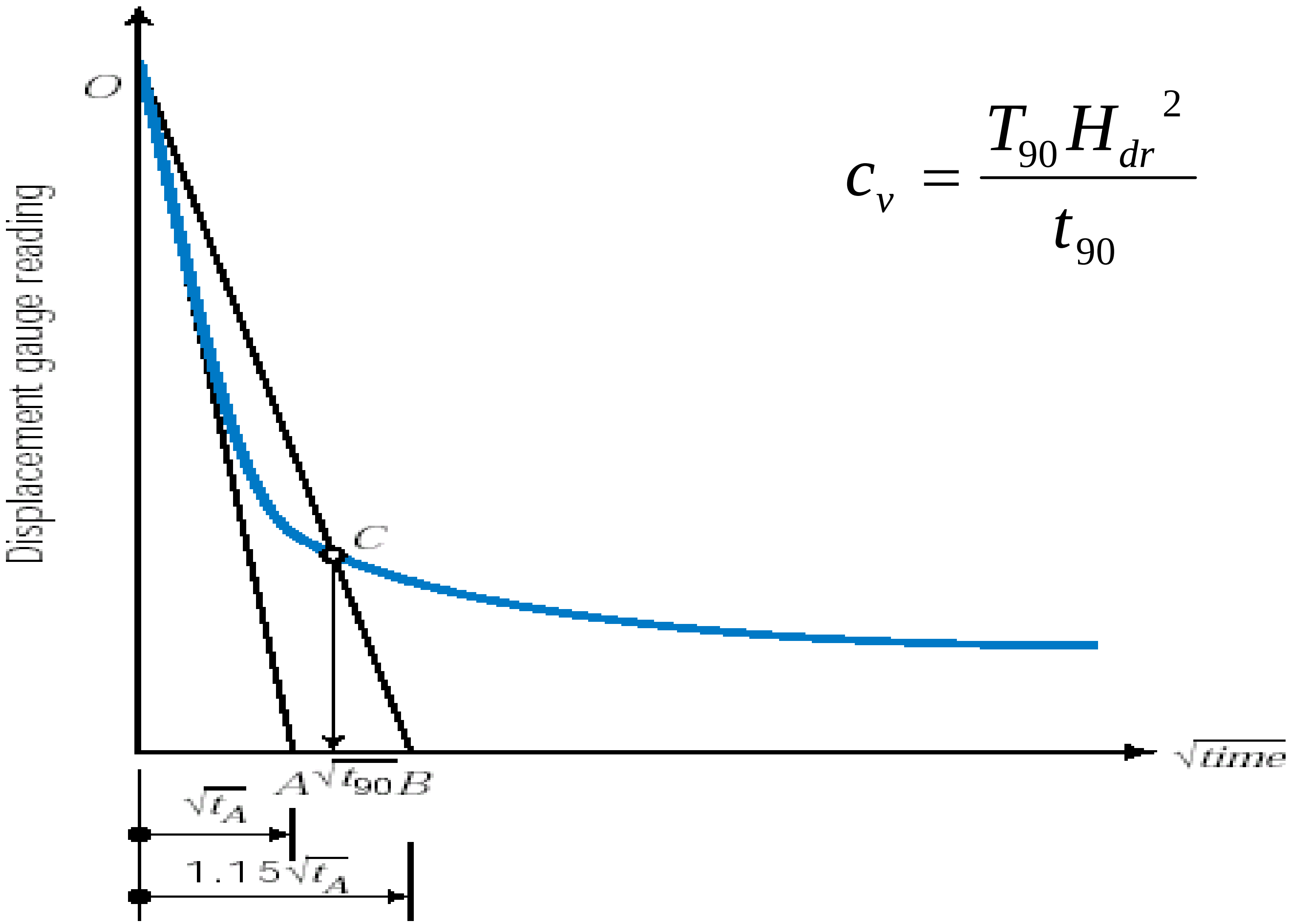
➢ Casagrande's Logarithm of Time Fitting Method



$$c_v = \frac{T_{50} H_{dr}^{\ 2}}{t_{50}}$$

Determination of the Coefficient of Consolidation (Cv)

- Taylor's Square Root of Time Fitting Method



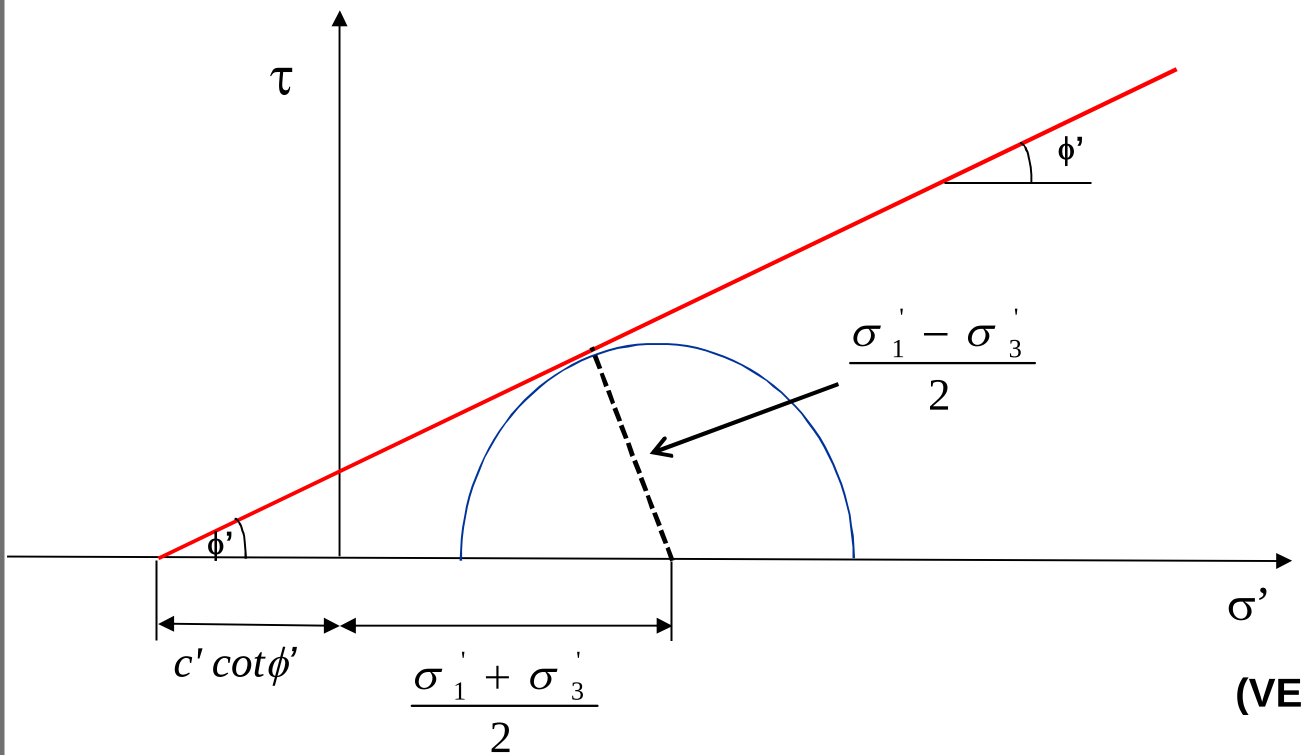
$$c_v = \frac{T_{90} H_{dr}^{\;2}}{t_{90}}$$

Shear Strength of Soils: Cohesion and Friction

- Soil derives its shear strength from two sources:
  - Cohesion between particles (stress independent component)
    - Cementation between sand grains
    - Electrostatic attraction between clay particles
  - Frictional resistance between particles (stress dependent component)

$$\tau_f = c' + \sigma' \tan \phi'$$

Mohr Coulomb Failure Criterion with Mohr Circle of Stress



Relationship between the effective principal stresses at failure and the shear strength parameters4

(VERY USEFUL EQN: Bell's)

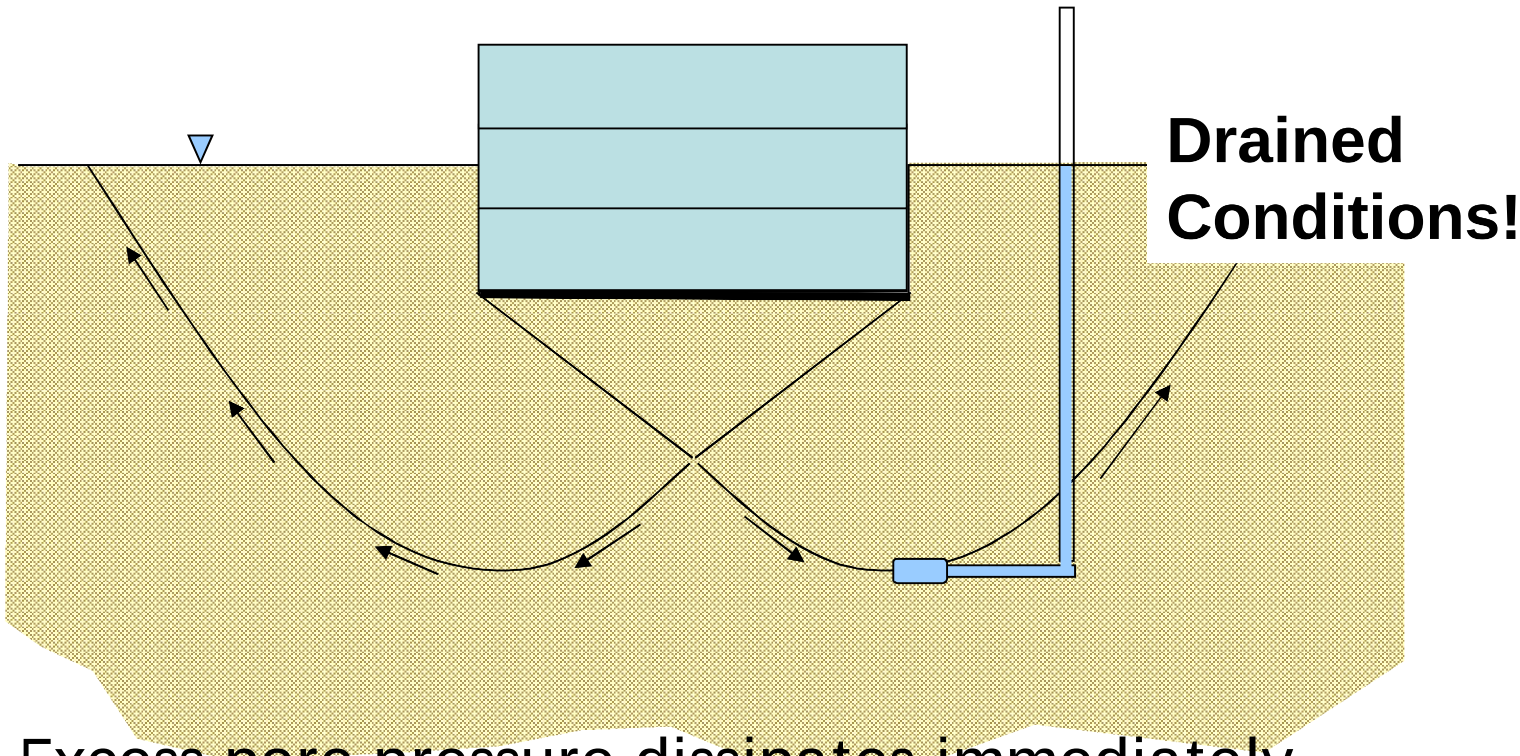
$$\left(\frac{\sigma_1' - \sigma_3'}{2}\right) = \left[c'\cot\phi' + \left(\frac{\sigma_1' + \sigma_3'}{2}\right)\right]\sin\phi$$

$$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c'\tan\left(45 + \frac{\phi'}{2}\right)$$

$$\left(\sigma_1' - \sigma_3'\right) = \left(\sigma_1' + \sigma_3'\right)\sin\phi' + 2c'\cos\phi'$$

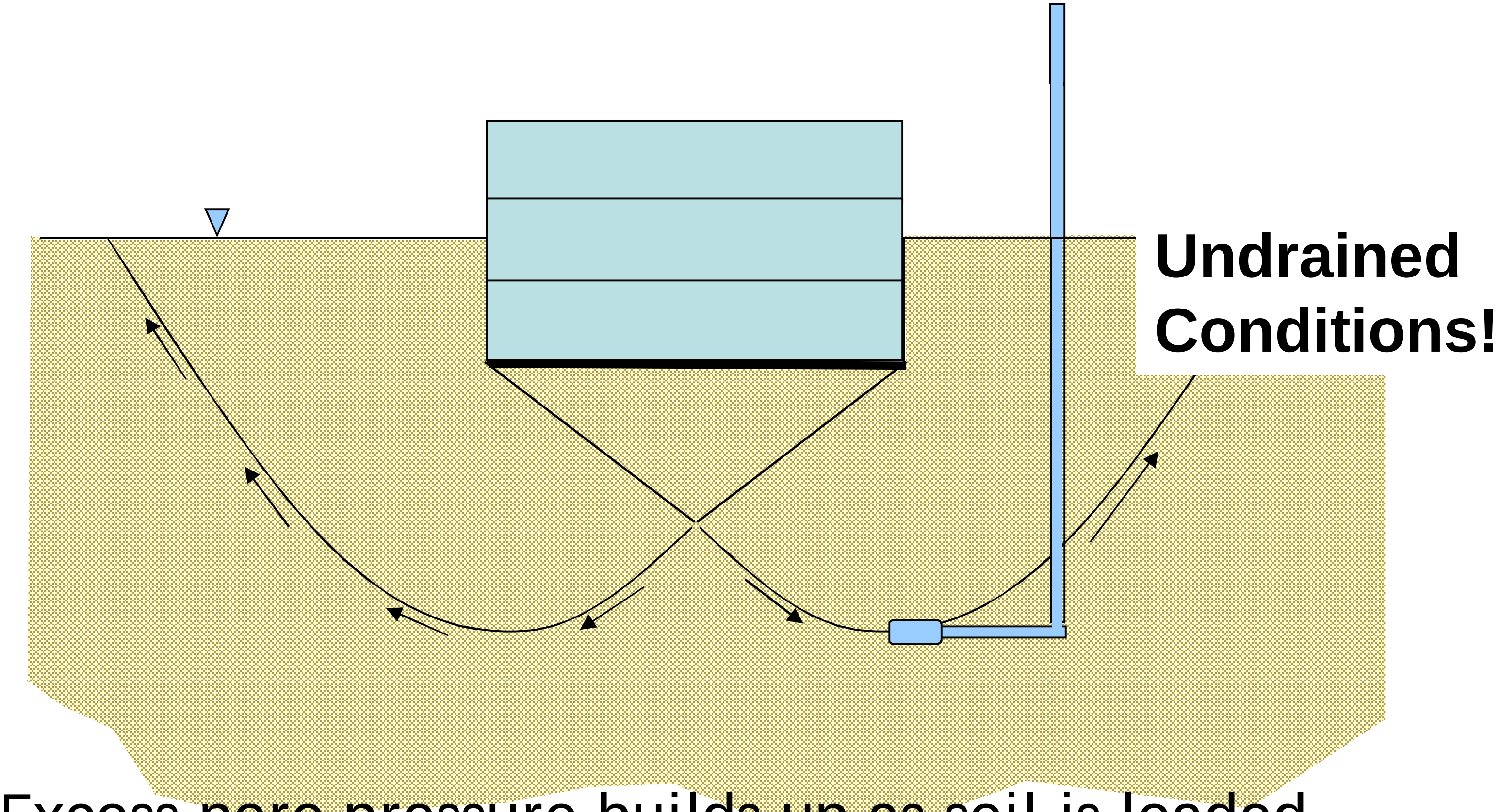
$$\sin\phi' = \frac{\sigma'_1 - \sigma'_3}{2c'\cot\phi'(\sigma'_1 + \sigma'_3)}$$

Soil Response under Loading: Sands and Gravels



- Excess pore pressure dissipates immediately
- Pore pressure remains at hydrostatic value
- σ' changes but can be calculated; $\sigma' = \sigma - u$
- Therefore, use $\tau_f = c' + \sigma' \tan\phi'$

Soil Response under Loading: Clayey Soils



- Excess pore pressure builds up as soil is loaded
- Pore pressure cannot be determined; $u = u_0 + \Delta u$
- σ' remains at initial value, therefore, use $\tau_f = c + \sigma \tan\phi$
- If S=100%, no drainage, use $\tau_f = s_u$; $c = s_u$ and $\phi = 0$

Shear Strength in terms of Total and Effective Stresses

1. Shear Strength in terms of effective stress

$$\sigma' = \sigma - u$$

u at hydrostatic value

$$\tau_f = c' + \sigma' \tan \phi'$$

2. Shear strength in terms of total stress

$$\tau_f = c + \sigma \tan \phi$$

3. For cohesive soils under saturated conditions, ϕ = 0.

$$\tau_f = s_u = c$$

Measuring Shear Strength

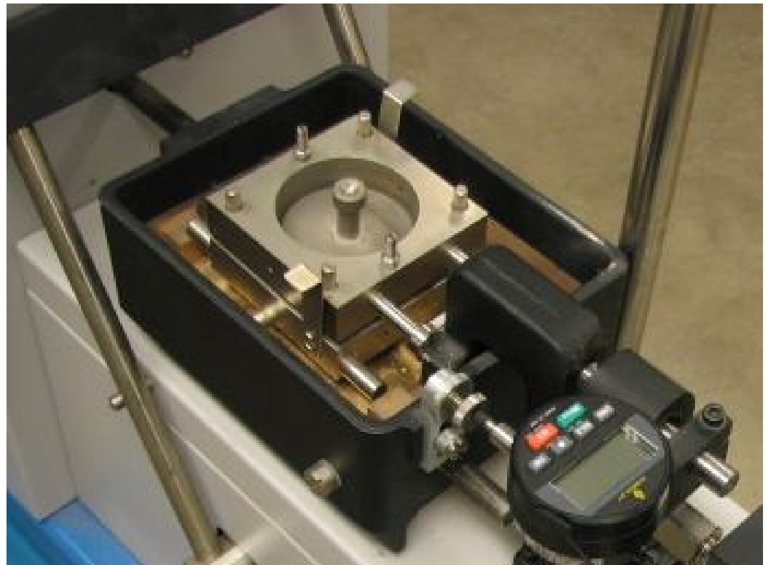
In the Laboratory

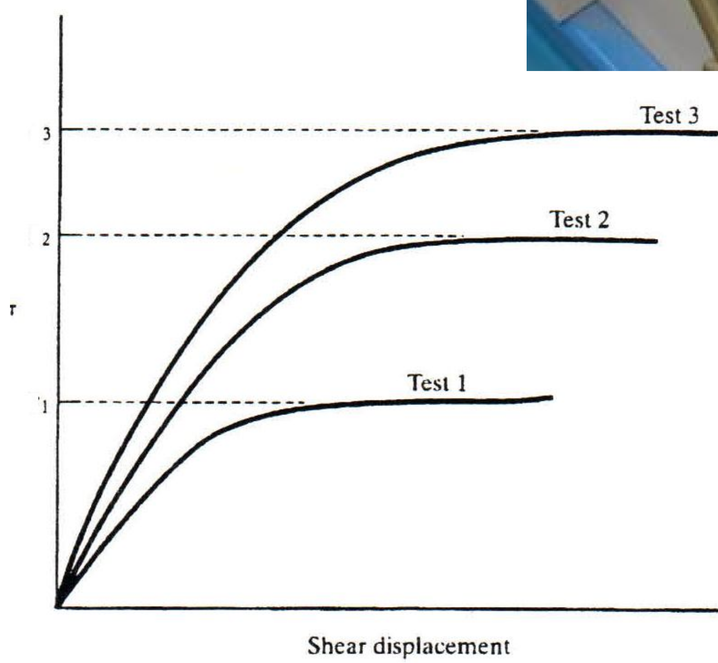
- Direct shear test :sands
- Unconfined compression test: clay
- Triaxial compression test: *used on important projects all soils*

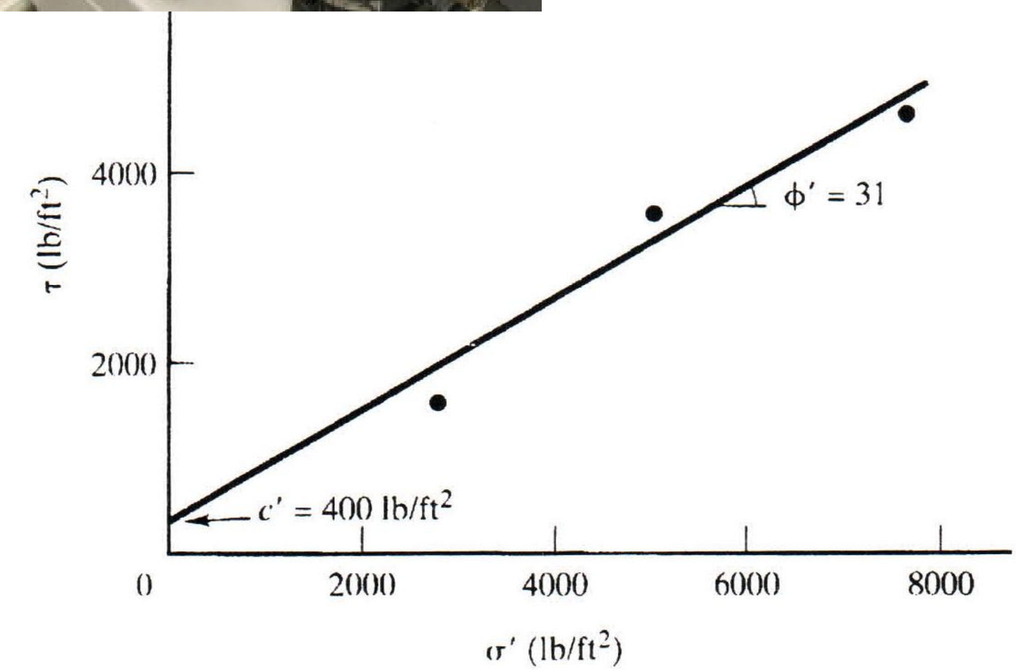
In the Field

- Vane shear test: clay
- Standard Penetration Test (SPT)
- Cone Penetration Test (CPT)

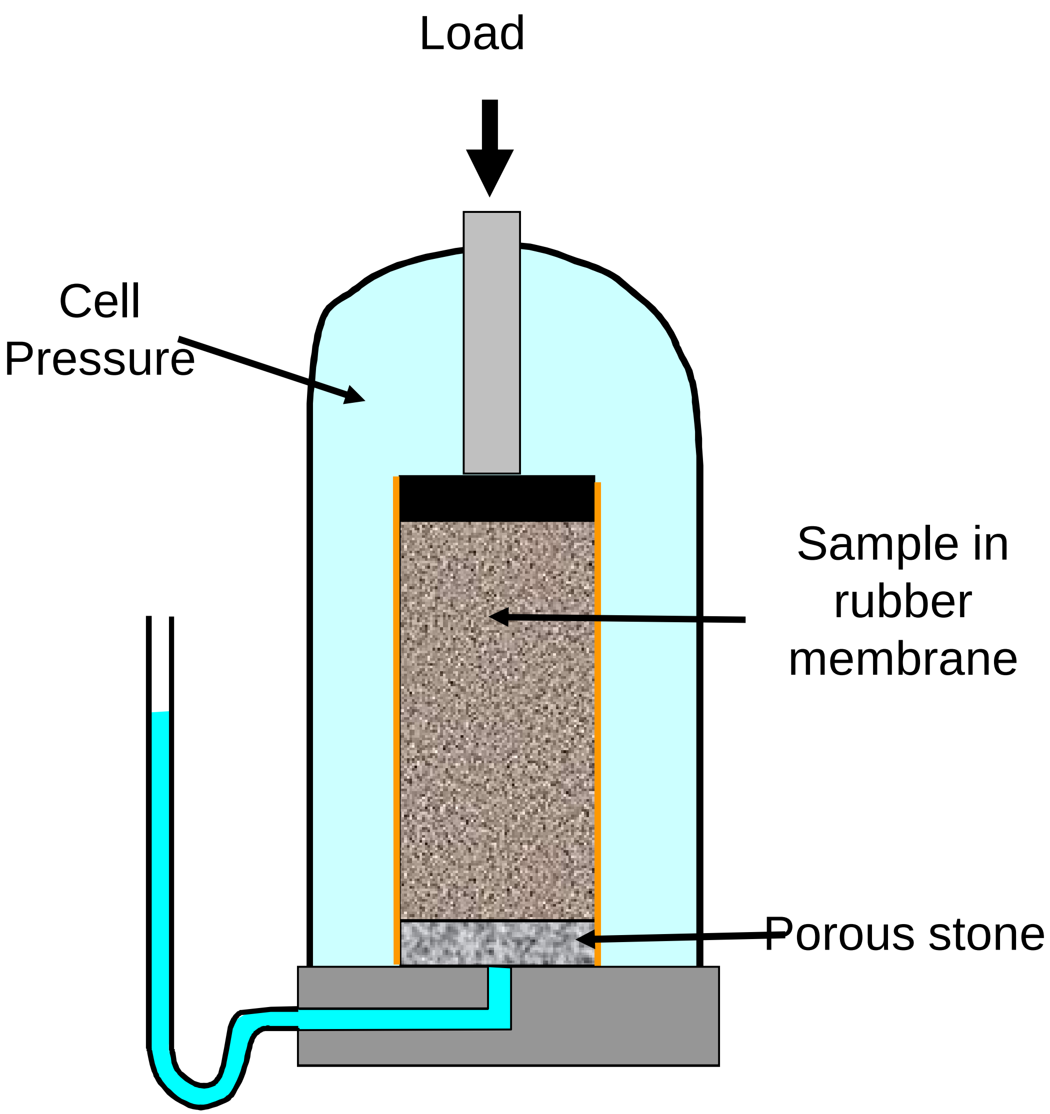
Direct Shear Test







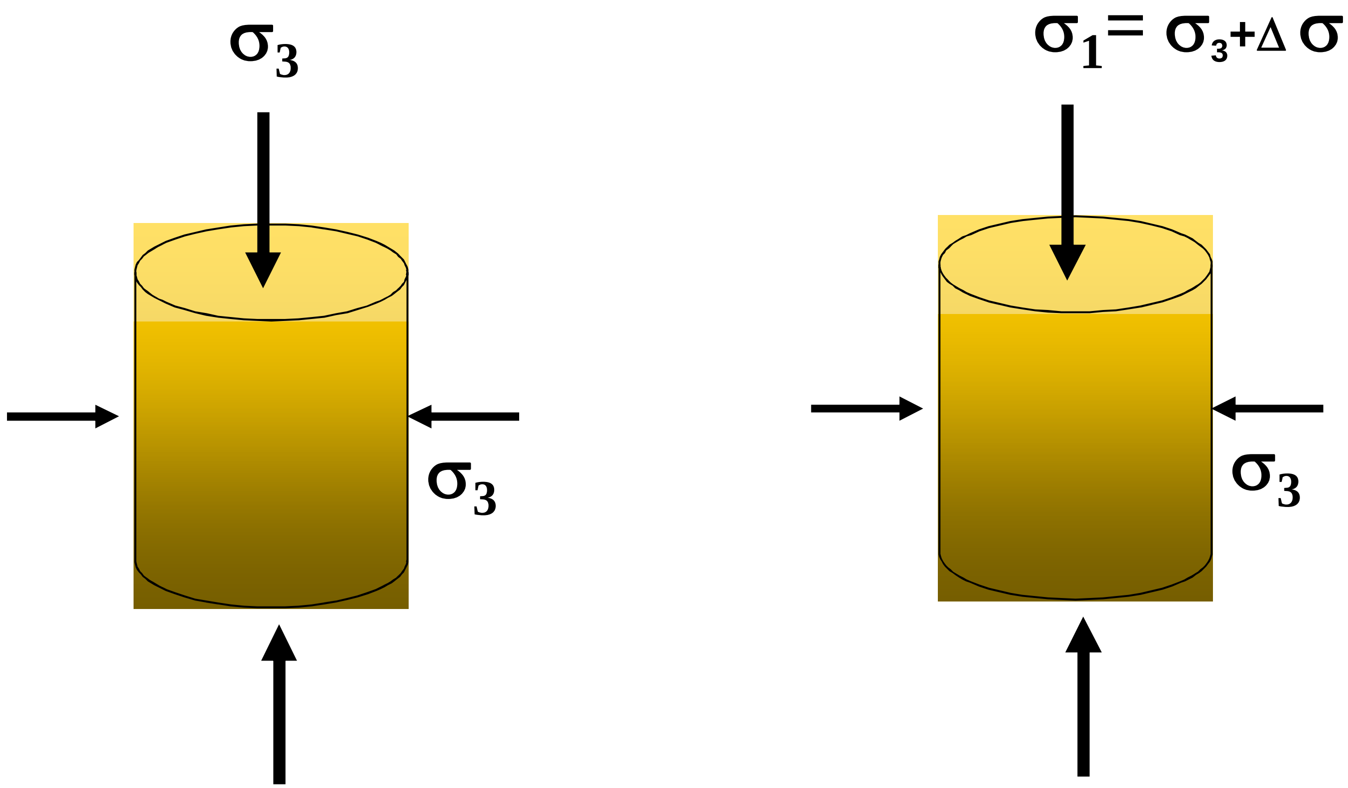
Triaxial Test



Triaxial Test

- UU - Unconsolidated Undrained Tests
  - » Estimate s_u
- CU – Consolidated Undrained Tests
  - » Estimate total stress parameters
- CD - Consolidated Drained Tests
  - » Estimate effective stress parameters

Triaxial Test: 2 Stages of Loading

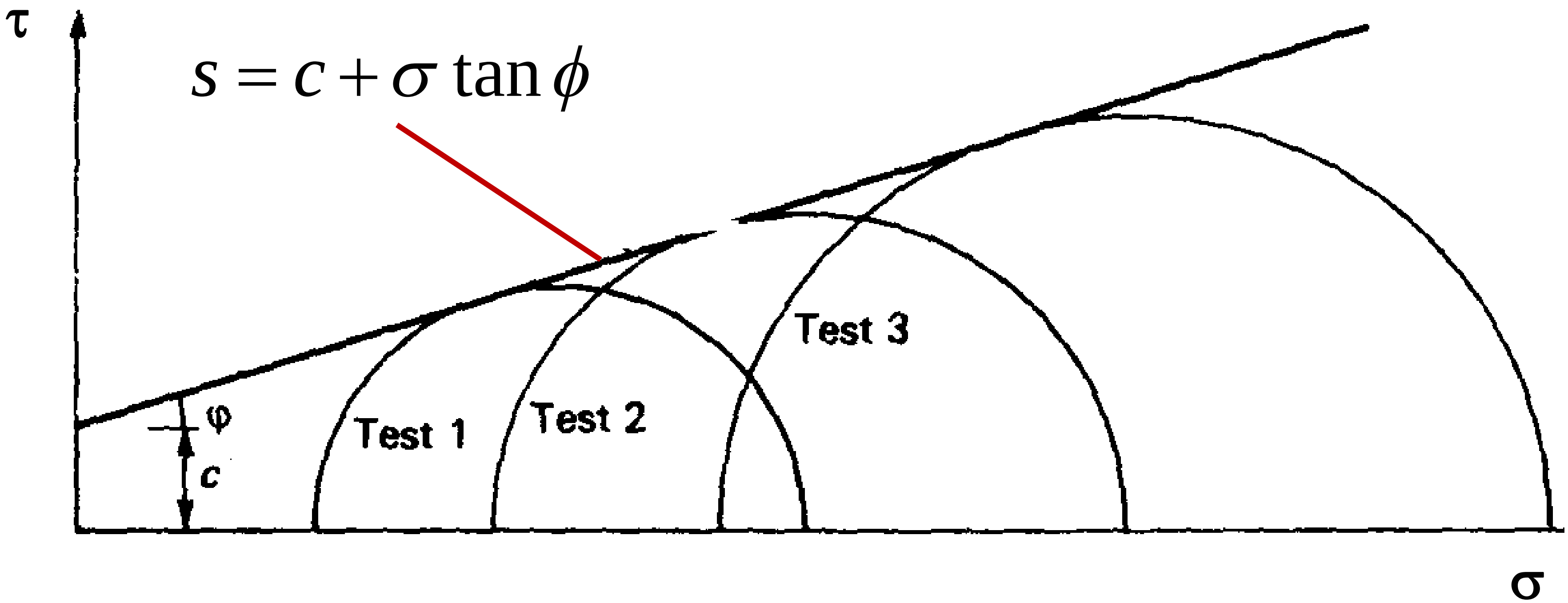


Apply Confining Pressure

Apply Axial Load

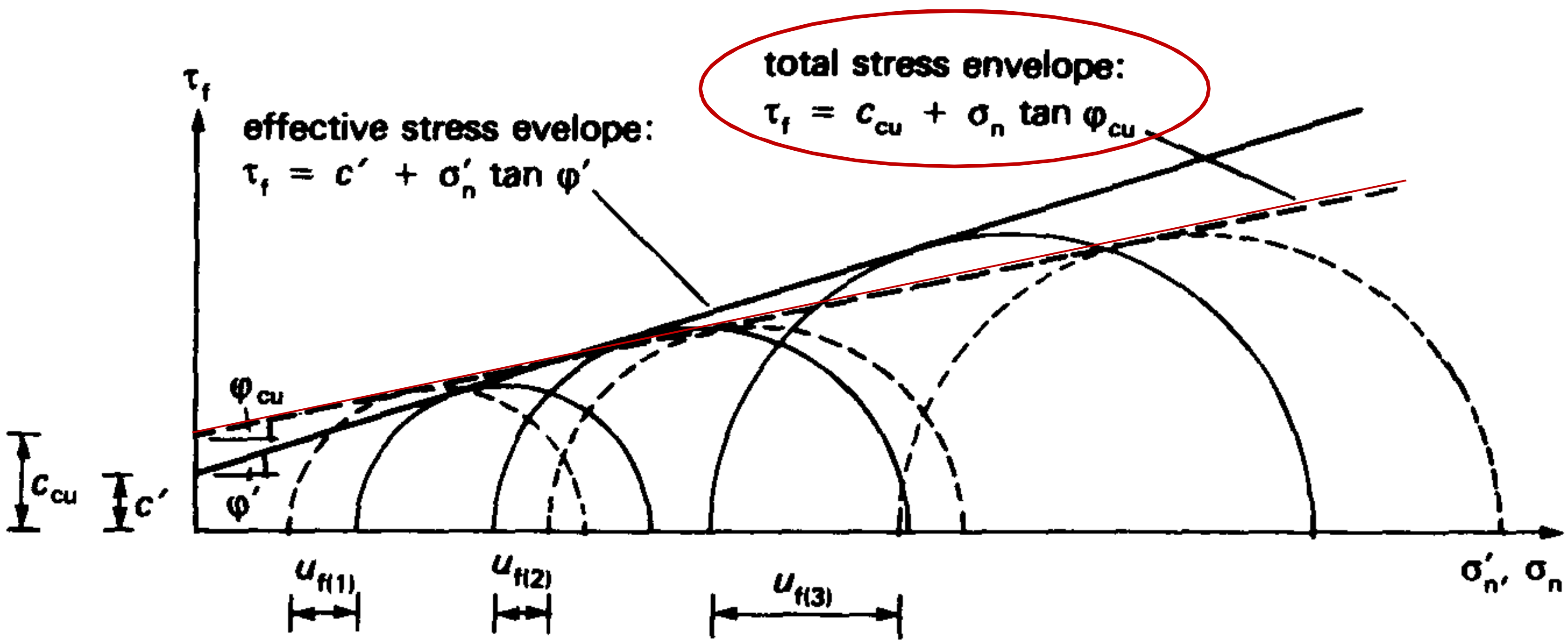
Triaxial Compression Test: Determining C and ϕ

CU – Consolidated Undrained Tests



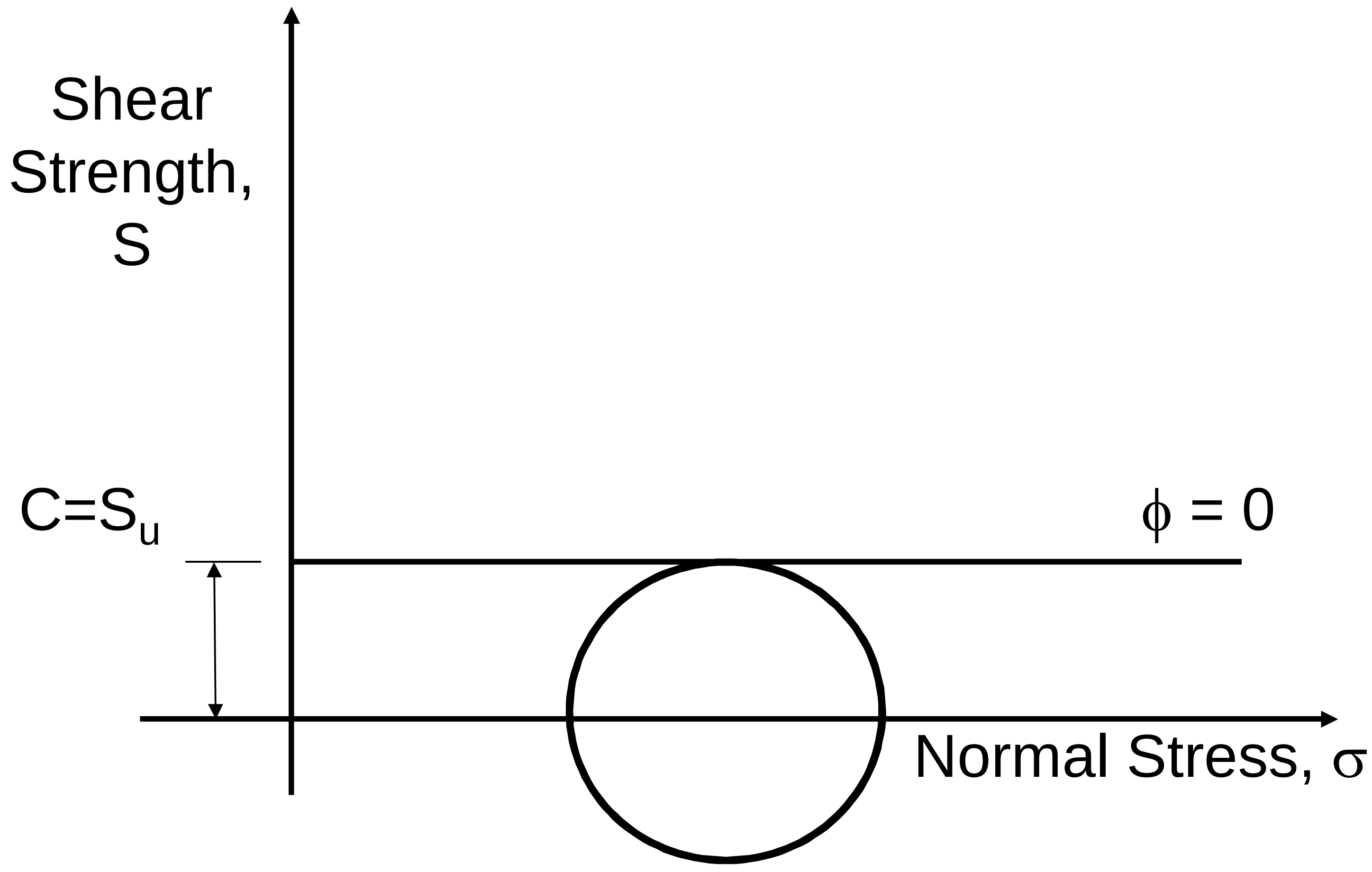
Triaxial Compression Test: Determining C′ and ϕ′

- Consolidated Undrained Test (CU- Test)
- Consolidated Drained (CD-Test); Also called "Drained Test"

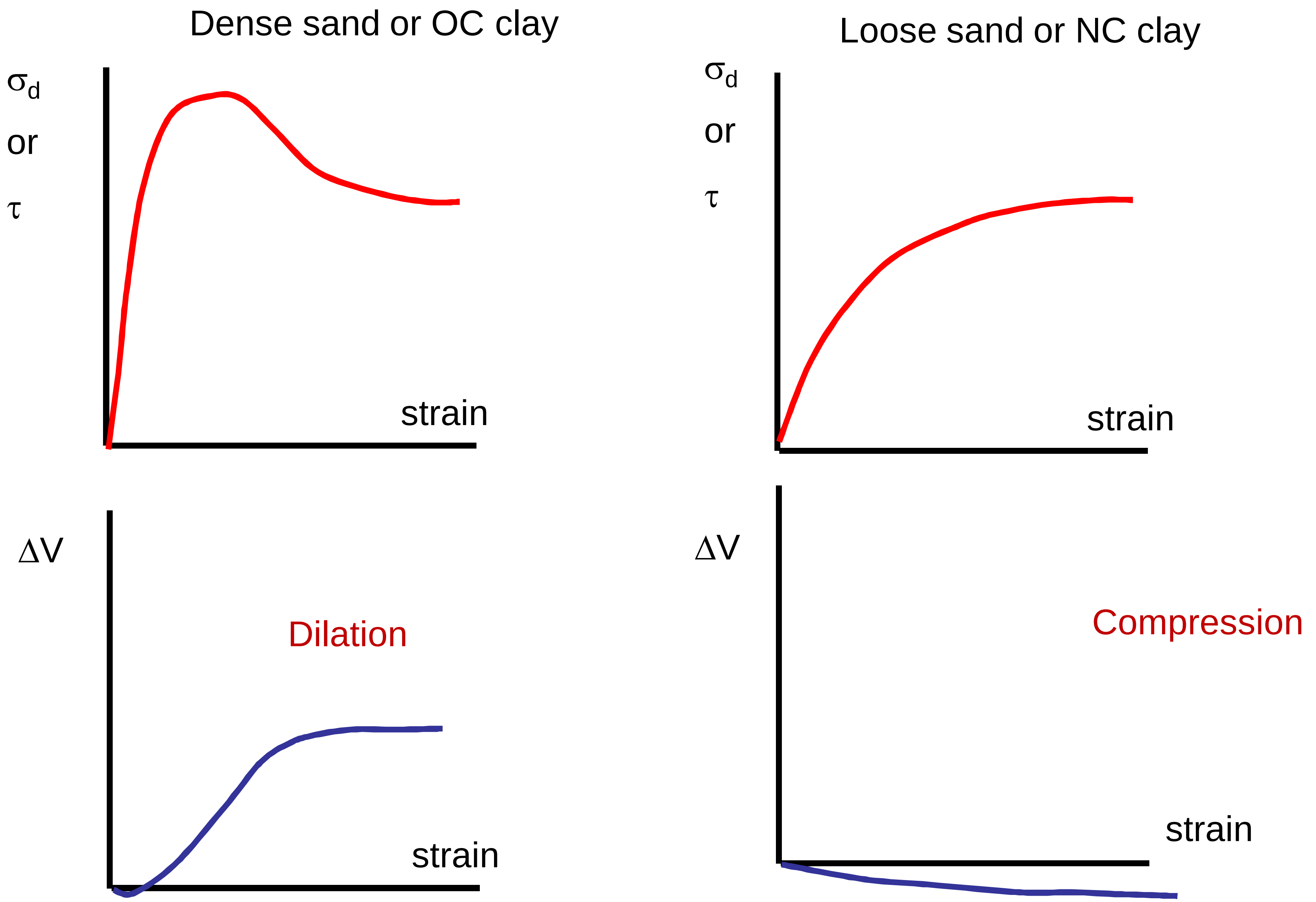


Fully Undrained Conditions: UU test

UU - Unconsolidated Undrained Tests

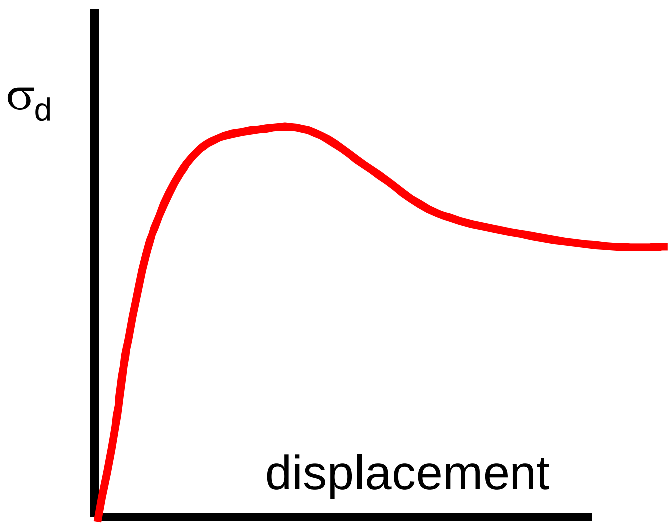


Shear Strength of Soils - Drained

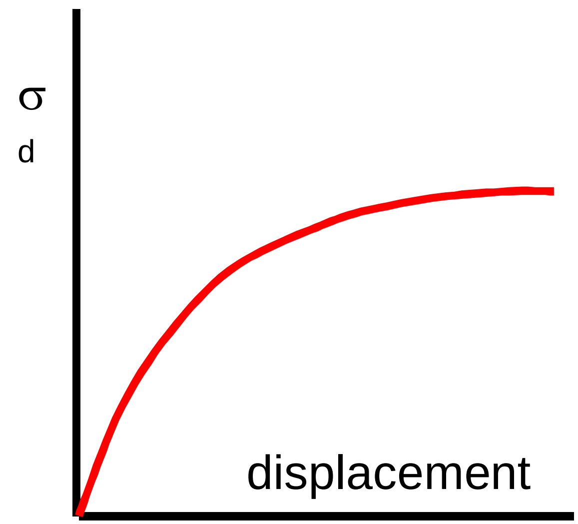


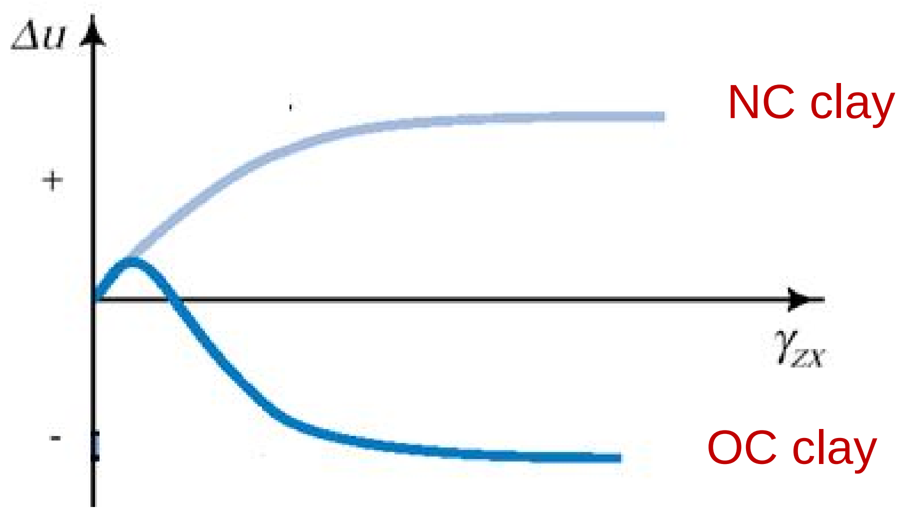
Shear Strength of Soils - Undrained

OC clay



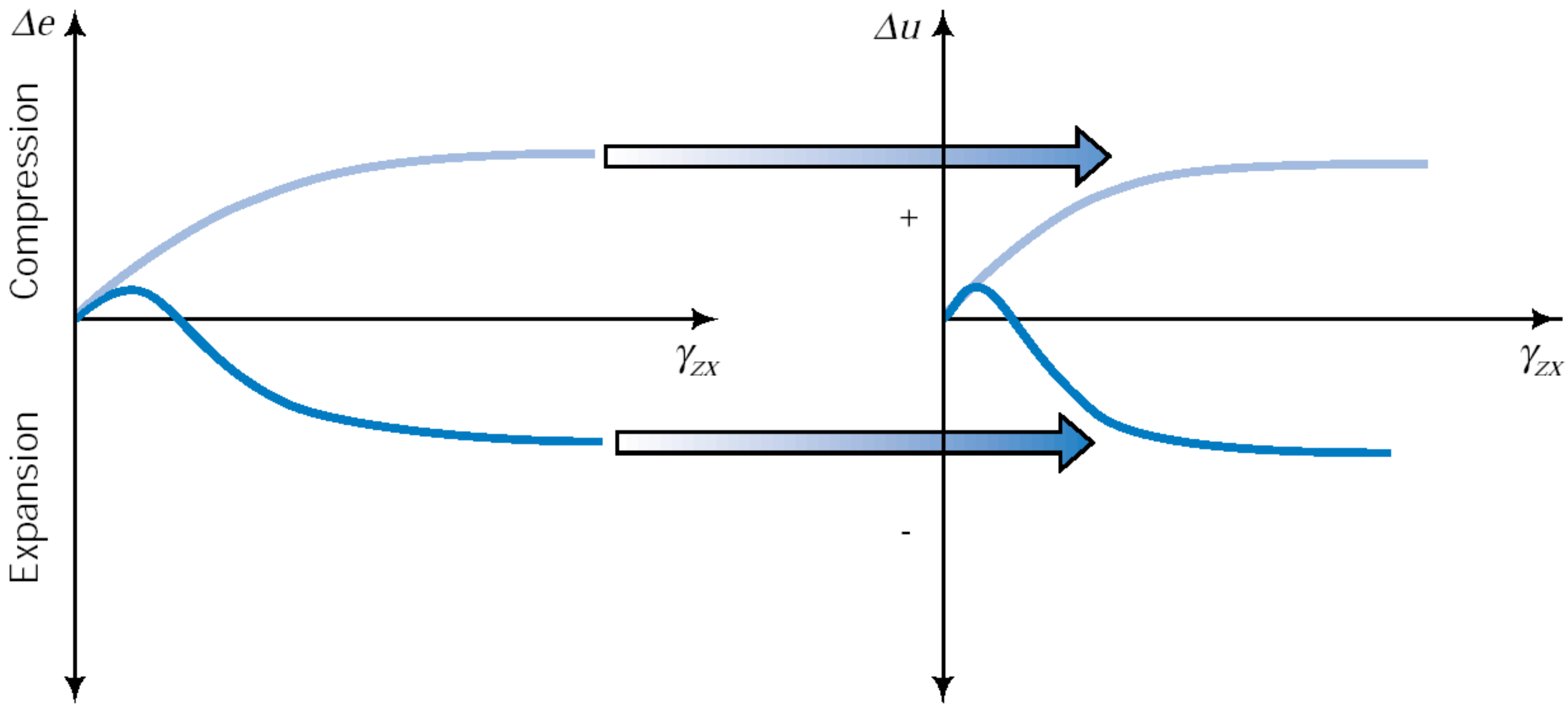
NC clay





Shear Strength of Clays - Undrained

➢ Undrained Response



(a) Drained condition

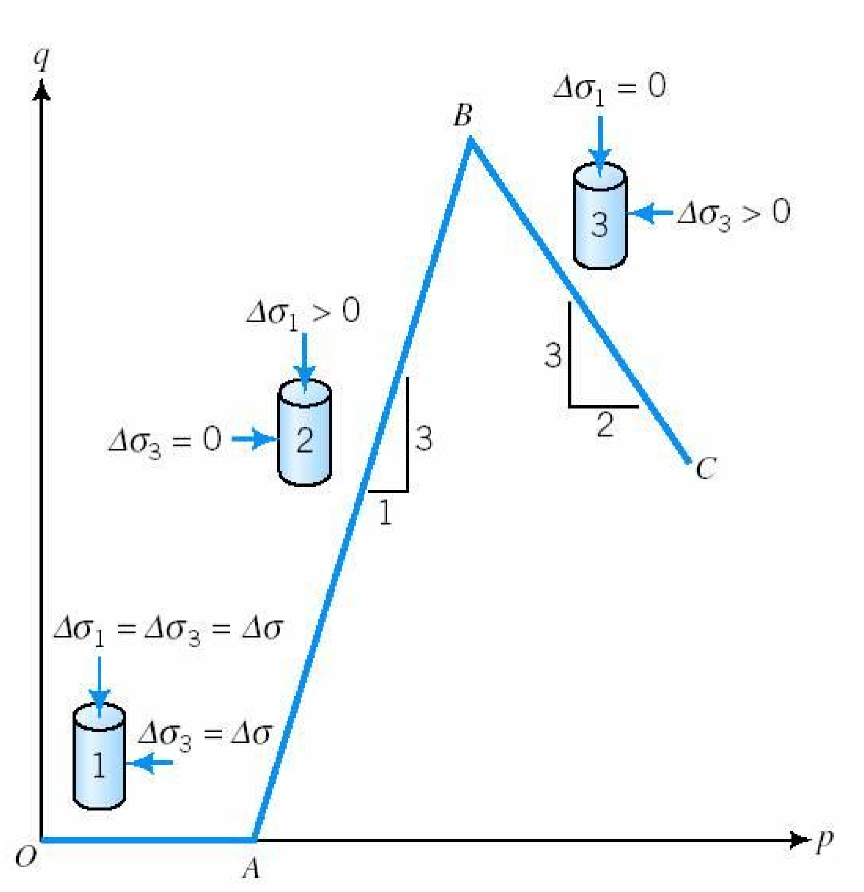
(b) Undrained condition

Example 5

- From a CU triaxial test we have: σ_3=40 kPa, σ_d=30 kPa, u_f=20 kPa and c=0.
  - Define drained and undrained friction angle of the soil.

Stress path for axisymmetric loading

- Phase OA
  - Hydrostatic compression
- Phase AB
  - Axial compression
  - Constant Lateral Pressure
- Phase BC
  - Increasing Lateral Pressure
  - Constant Axial stress



Final Exam

- Similar to mid-term
  - Conceptual 20%
  - Computation: 80%
- Duration: 3 hours
- Authorized Memoranda:
  - Closed book,
  - calculators permitted,
  - Formula sheet will be provided

- **Good Luck in Your Final Exam**