



INS 3121

SOIL MECHANICS

Compressibility of Soils

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Introduction

- The compression is caused by
 - a. Deformation of soil particles
 - b. Relocations of soil particles
 - c. Expulsion of water or air from the void spaces

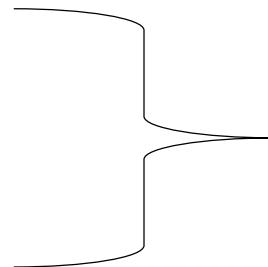
Introduction

- Soil Settlements
 1. Elastic settlement (or Immediate settlement)
elastic deformation of dry soil and of moist and saturated soils without **any change in the moisture content**. Immediate settlement equations derived from the **theory of elasticity**.
 2. Primary consolidation settlement
a volume change in saturated cohesive soils because of **expulsion of the water** that occupies the void spaces.
 3. Secondary consolidation settlement
The result of the **plastic adjustment of soil fabrics**

Introduction

- **Total settlement**

1. Elastic or immediate settlement (S_e)
2. Consolidation settlement (S_c)
3. Secondary settlement (S_s)



$$S_T = S_e + S_c + S_s$$

Elastic Settlement

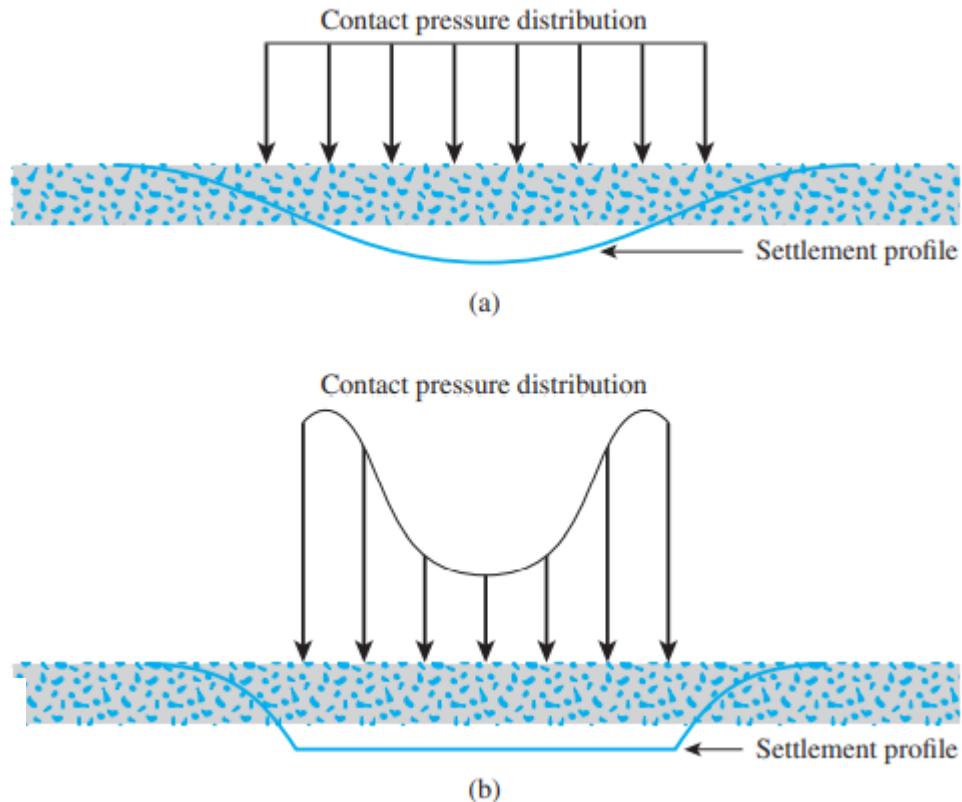


Figure 11.1

Elastic settlement profile and contact pressure in clay:
(a) flexible foundation; (b) rigid foundation

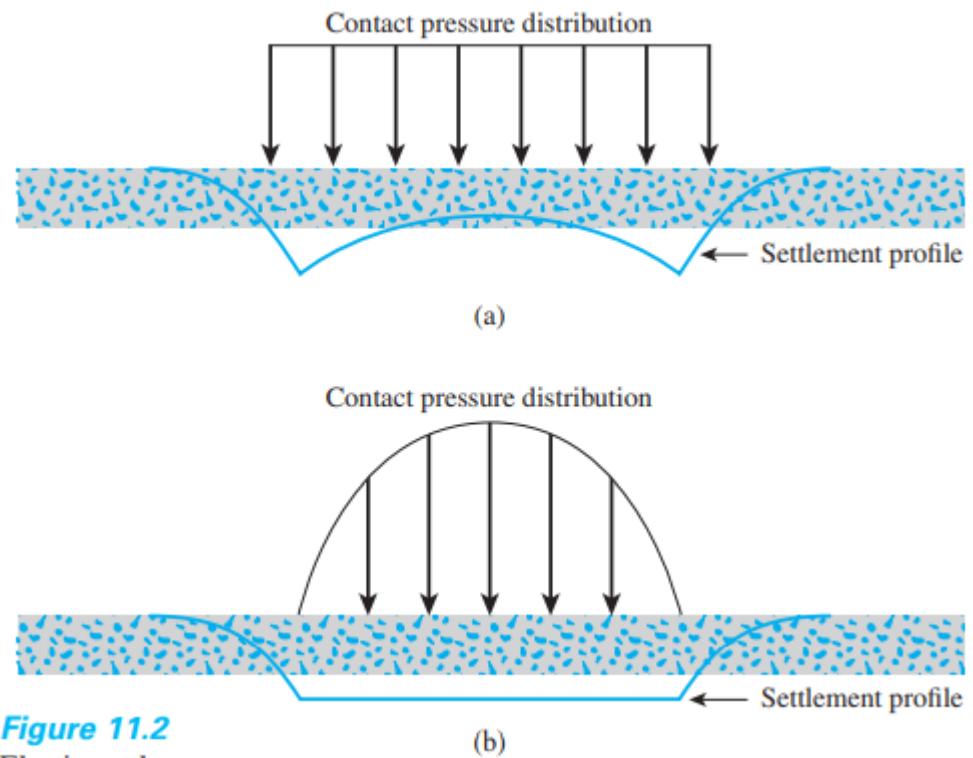


Figure 11.2

Elastic settle-
ment profile and con-
tact pressure in sand:
(a) flexible foun-
dation; (b) rigid
foundation

CONSOLIDATION SETTLEMENT

- When a saturated soil layer is subjected to a stress increase, the pore water pressure is suddenly increased.
- Because of rapid drainage of the pore water in sandy soils, immediate settlement and consolidation take place simultaneously.
- When a saturated compressible clay layer is subjected to a stress increases, elastic settlement occurs immediately.
- The excess pore water pressure generated by loading gradually dissipates over a long period of time.
- The settlement caused by consolidation in clay may be several times greater than the immediate settlement.

CONSOLIDATION SETTLEMENT

t = 0 and valve is closed

- A simple model of the time-dependent deformation of saturated clayey
 - Let the inside area of the cross section of the cylinder be equal to A. The cylinder is filled with water and has a frictionless watertight piston and valve
 - The excess hydrostatic pressure : $\Delta u = \frac{P}{A}$
- $$P = P_s + P_w$$

P_s = load carried by the spring and P_w = load carried by the water.

Valve closed $\rightarrow P_s = 0$ and $P_w = P$

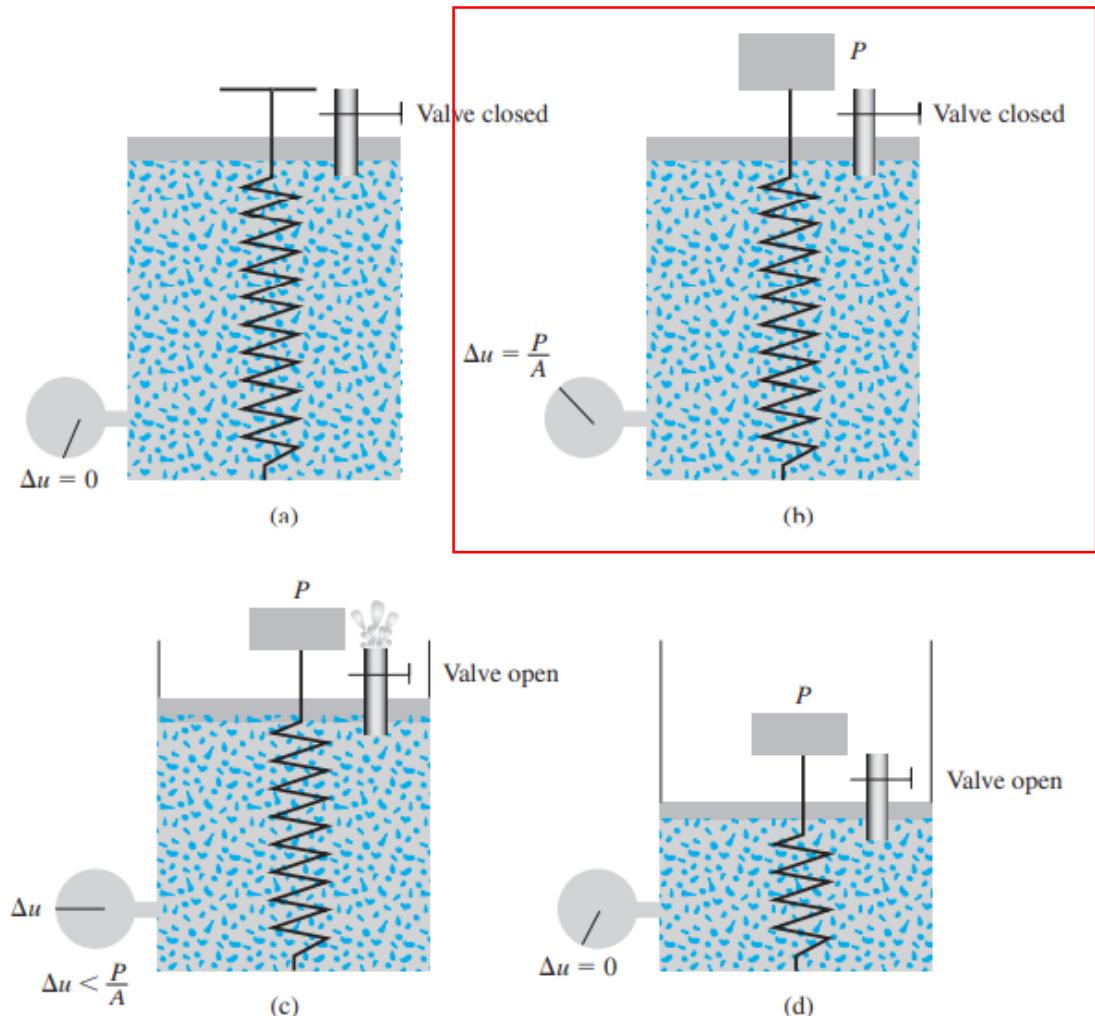


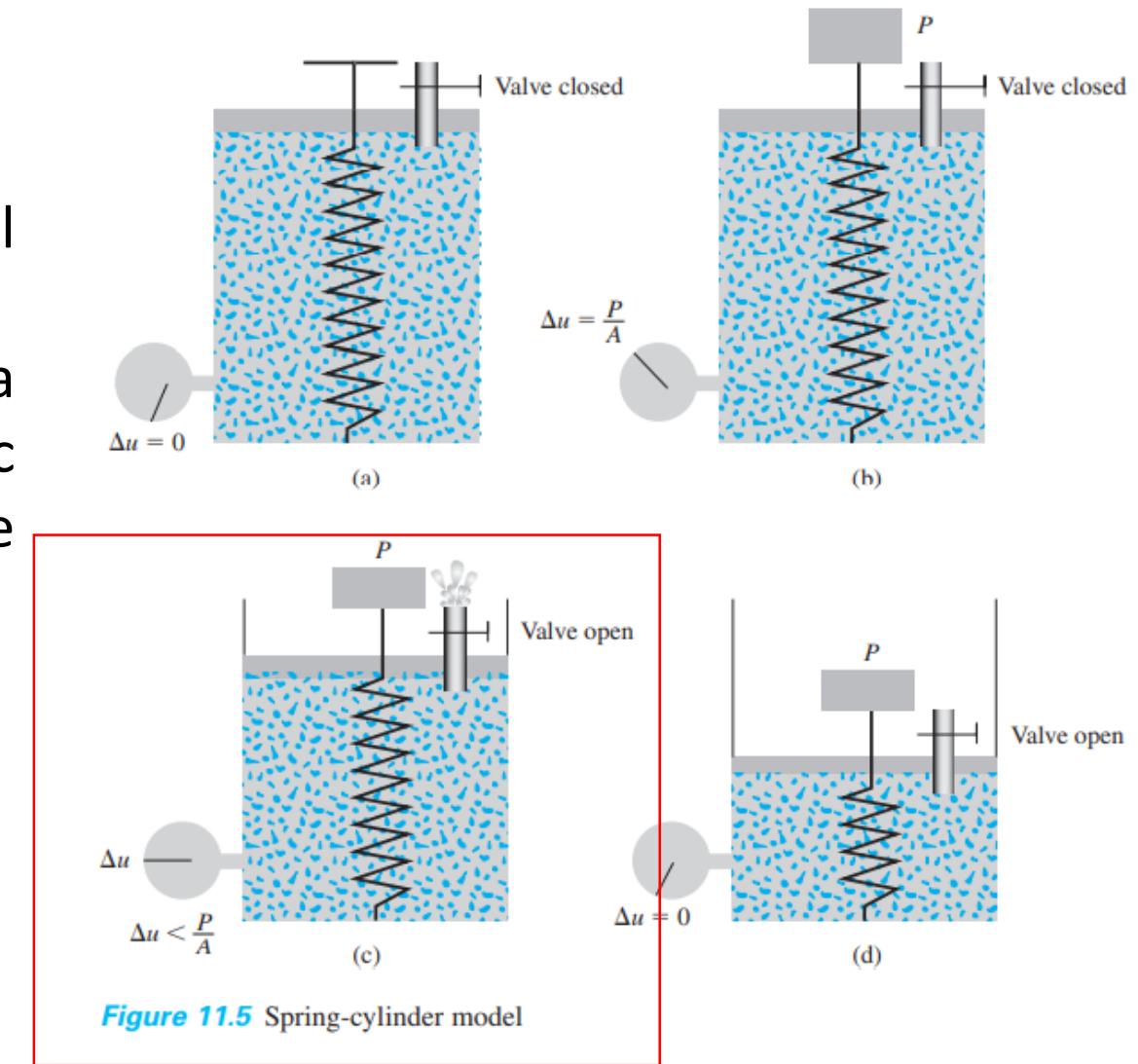
Figure 11.5 Spring-cylinder model

CONSOLIDATION SETTLEMENT

$0 < t < \infty$ AND valve opened

- If the valve is opened, the water will flow outward.
- This flow will be accompanied by a reduction of the excess hydrostatic pressure and an increase in the compression of the spring.

$$P_s > 0 \text{ and } P_w < P \text{ (that is, } \Delta u < \frac{P}{A})$$



CONSOLIDATION SETTLEMENT

t = ∞ AND valve opened

- After some time, the excess hydrostatic pressure will become zero and the system will reach a state of equilibrium

$$P_s = P \text{ and } P_w = P$$

and

$$P = P_s + P_w 0$$

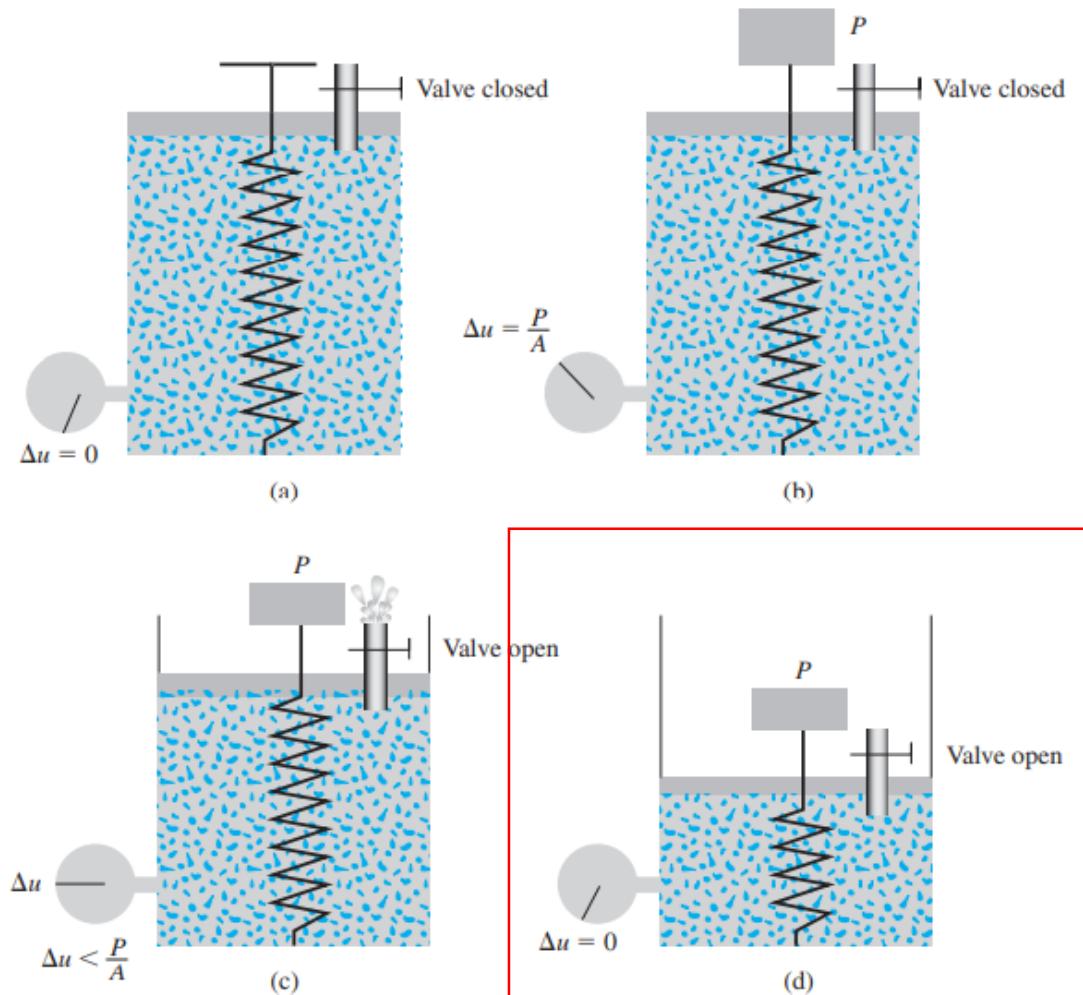
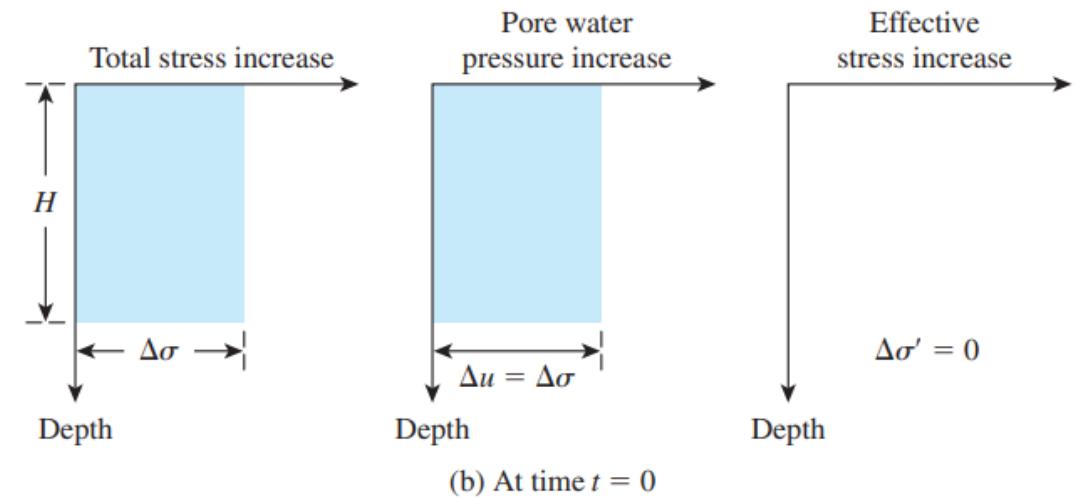
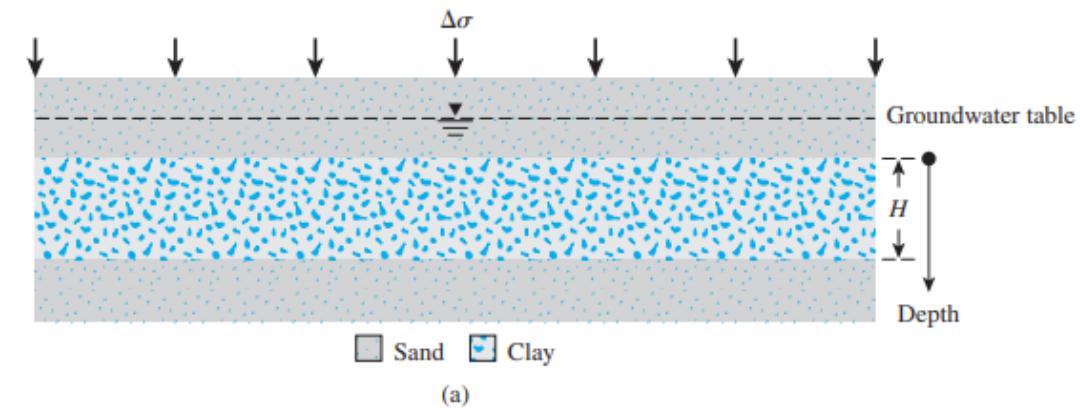


Figure 11.5 Spring-cylinder model

CONSOLIDATION SETTLEMENT

t = 0

- Since clay has a very low coefficient of permeability and water is incompressible as compared to the soil skeleton, at time $t = 0$, the entire incremental stress, $\Delta\sigma$, will be carried by water ($\Delta\sigma = \Delta u$) at all depths.



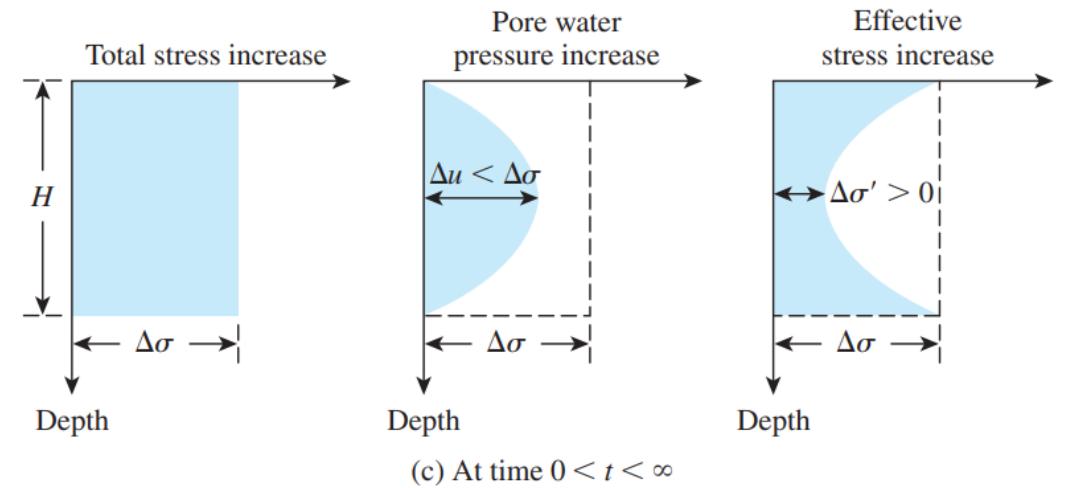
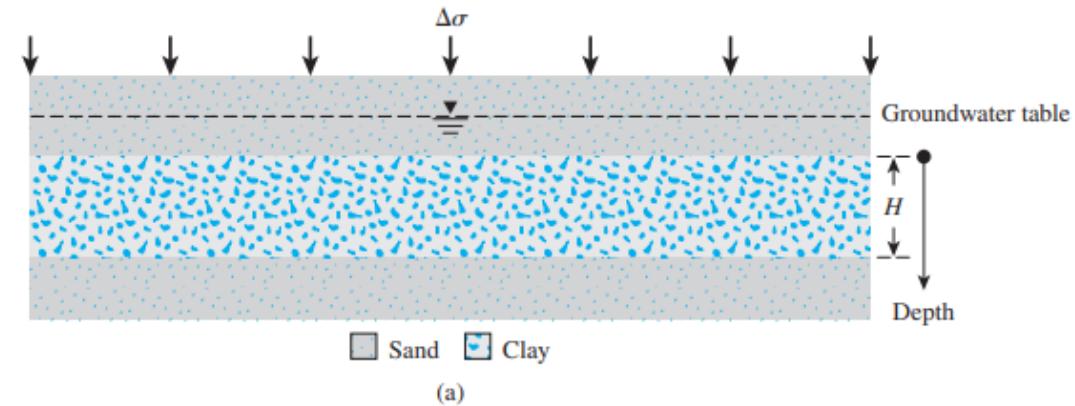
CONSOLIDATION SETTLEMENT

$0 < t < \infty$

- After the application of incremental stress, Δu , to the clay layer, the water in the void spaces will start to be squeezed out and will drain in both directions into the sand layers.

$$\Delta\sigma = \Delta\sigma' + \Delta u \quad (\Delta\sigma' > 0 \text{ and } \Delta u < \Delta\sigma)$$

- However, the magnitudes of $\Delta\sigma'$ and Δu at various depths will change, depending on the minimum distance of the drainage path to either the top or bottom sand layer.



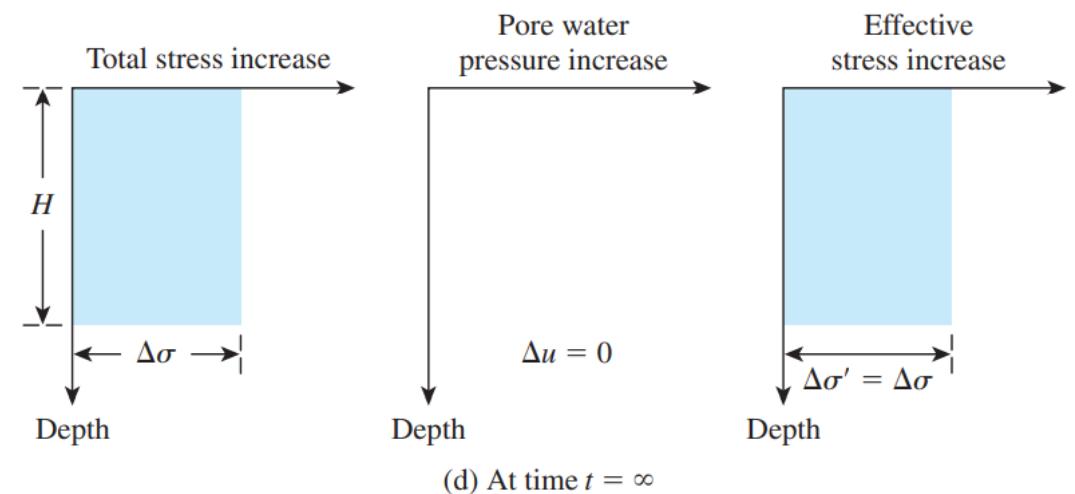
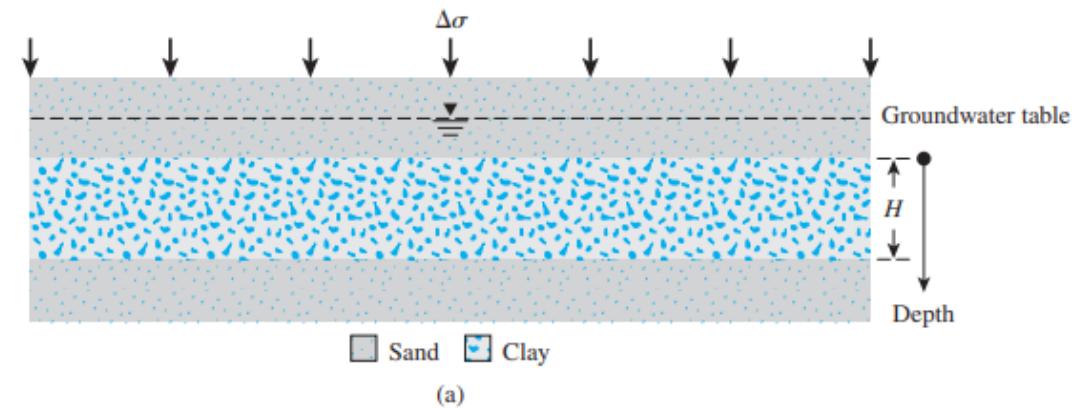
CONSOLIDATION SETTLEMENT

$t = \infty$, theoretically

- the entire excess pore water pressure would be dissipated by drainage from all points of the clay layer, thus giving $\Delta u = 0$.

$$\Delta\sigma = \Delta\sigma'$$

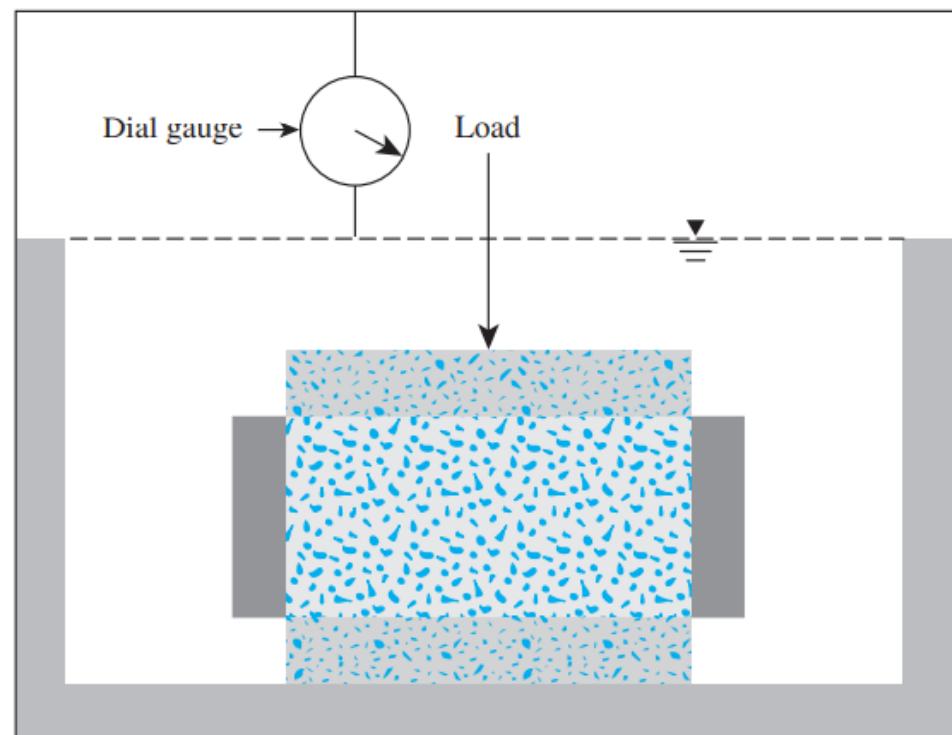
Transfer of excess pore water pressure to effective stress cause the time-dependent settlement in the clay soil layer.



One-Dimensional Laboratory Consolidation Test

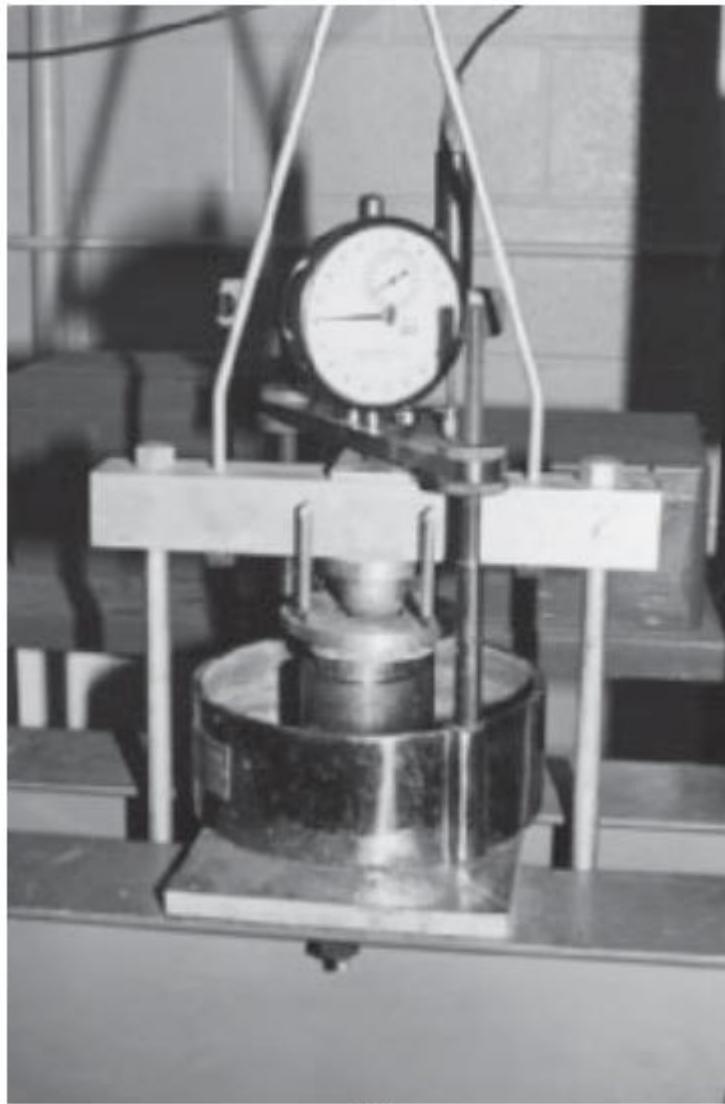
- The one-dimensional consolidation testing procedure was first suggested by Terzaghi
- consolidometer (sometimes referred to as an oedometer)
- The specimens are usually 63.5 mm in diameter and 25.4 mm thick
- Each load is usually kept for **24 hours**.
- After that, the load is usually **doubled**.
- At the end of the test, the dry weight of the test specimen is determined.

One-Dimensional Laboratory Consolidation Test

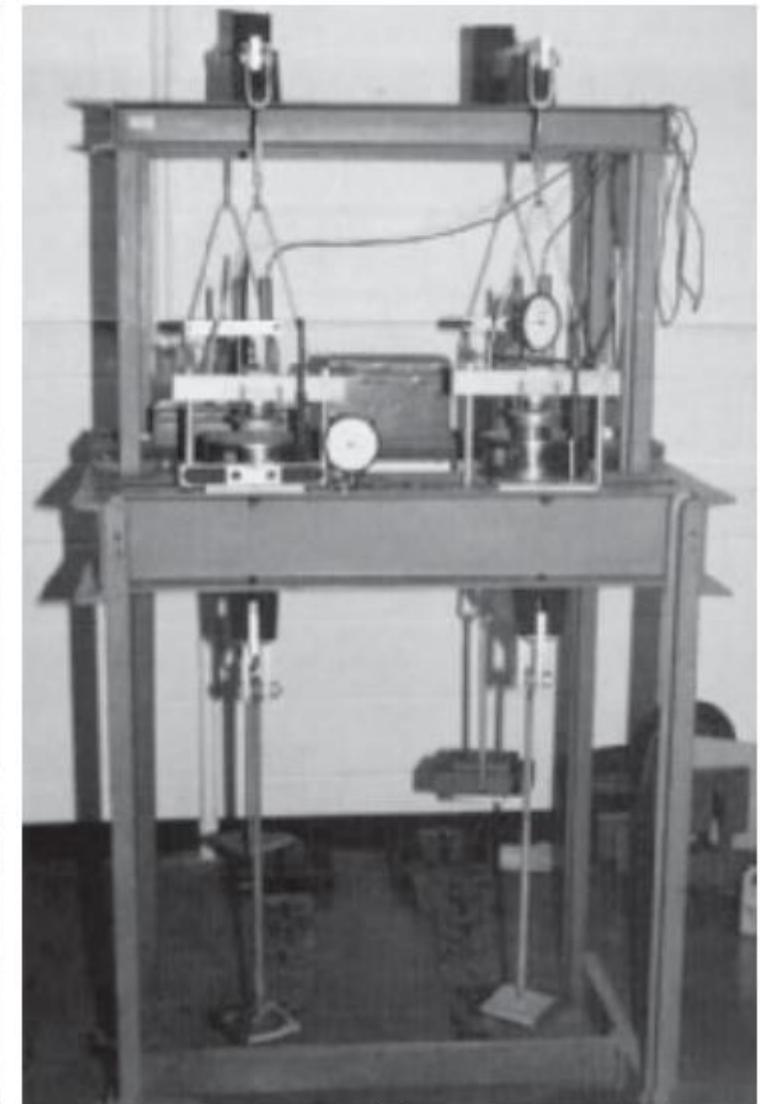


■ Porous stone ■ Soil specimen ■ Specimen ring

(a)



(b)



(c)

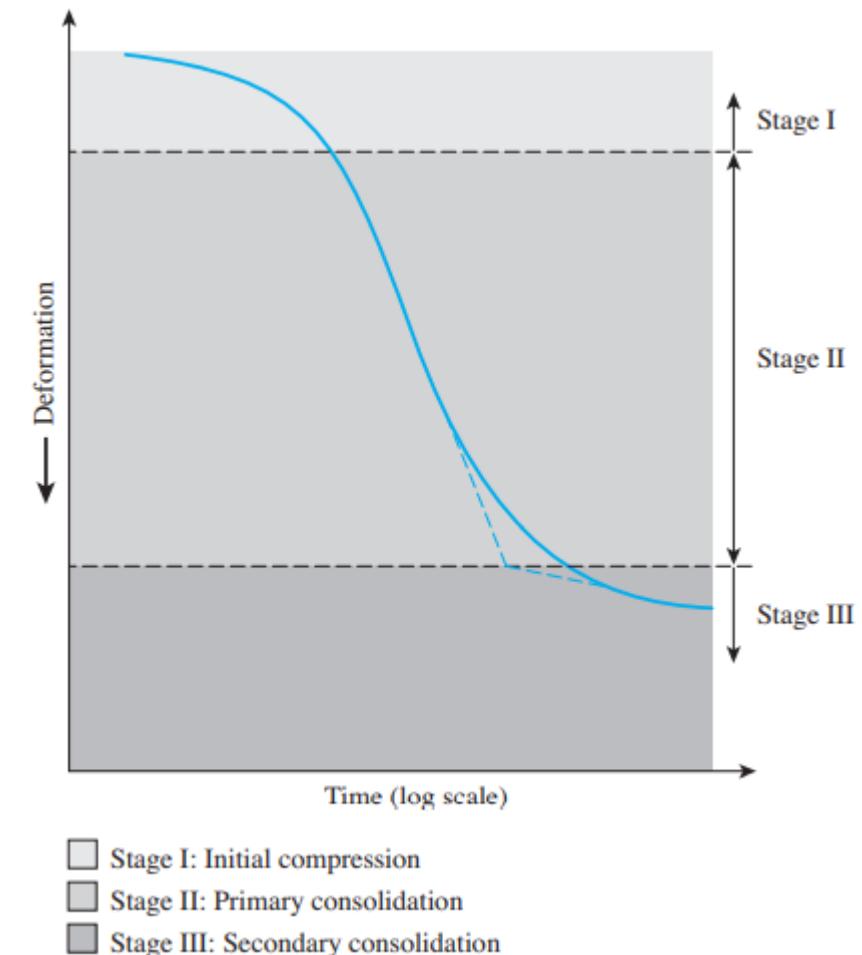
One-Dimensional Laboratory Consolidation Test

- Three distinct stages

Stage 1: **Initial compression**,
which is mostly caused by preloading.

Stage 2: **Primary consolidation**,
during which excess pore water pressure gradually is transferred into effective stress because of the expulsion of pore water.

Stage 3: **Secondary consolidation**,
which occurs after complete dissipation of the excess pore water pressure, when some deformation of the specimen takes place because of the plastic readjustment of soil fabric.



Void Ratio-Pressure Plots

1. Calculate the height of solids, H_s , in the soil specimen

$$H_s = \frac{W_s}{AG_s\gamma_w} = \frac{M_s}{AG_s\rho_w}$$

where W_s = dry weight of the specimen

M_s = dry mass of the specimen

A = area of the specimen

G_s = specific gravity of soil solids

γ_w = unit weight of water

ρ_w = density of water

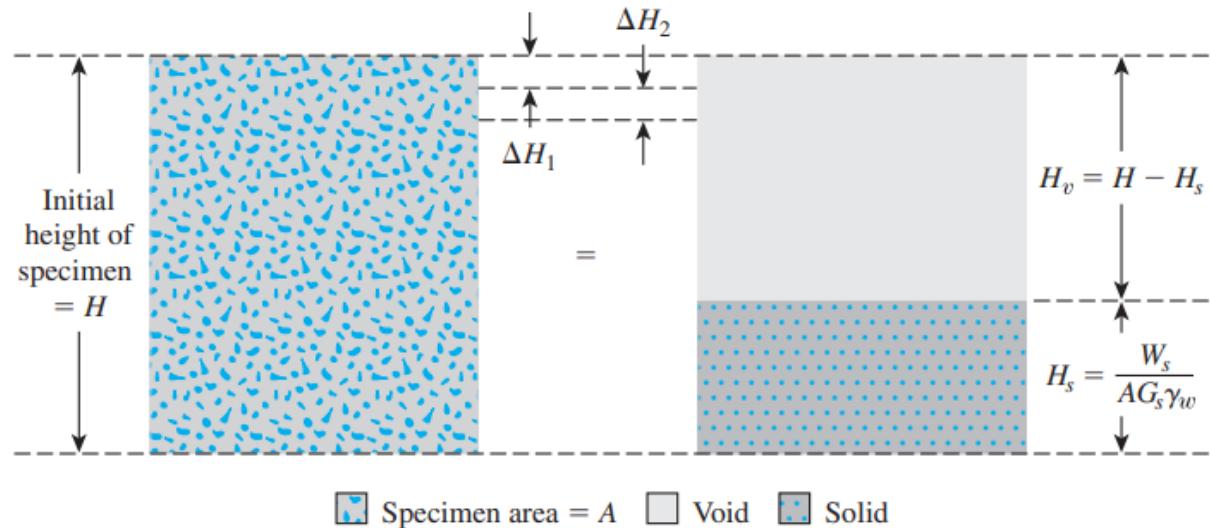


Figure 11.9 Change of height of specimen in one-dimensional consolidation test

Void Ratio-Pressure Plots

2. Calculate the initial height of voids as

$$H_v = H - H_s$$

where H = initial height of the specimen

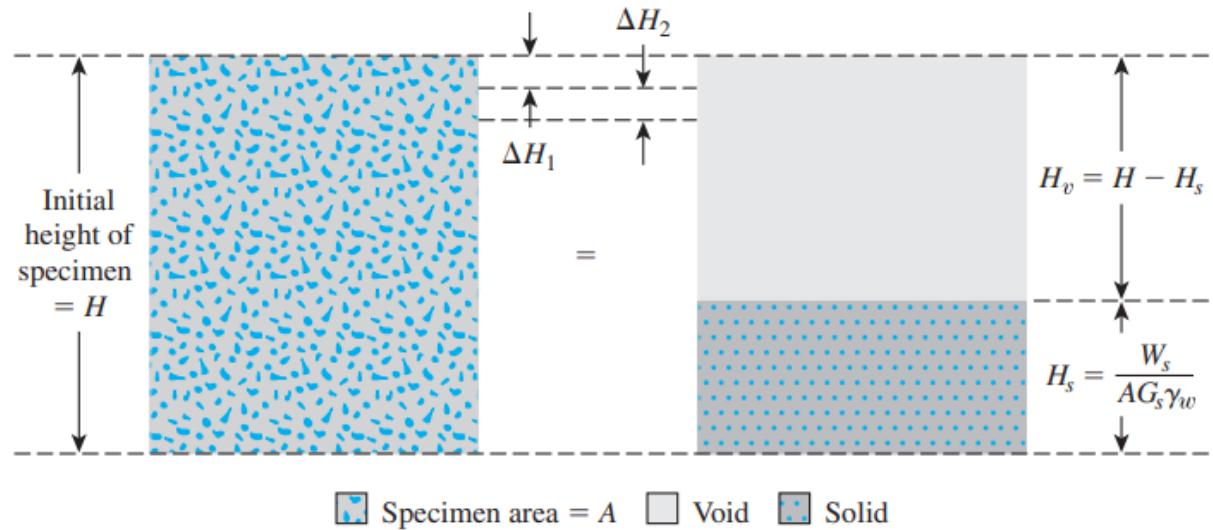


Figure 11.9 Change of height of specimen in one-dimensional consolidation test

Void Ratio-Pressure Plots

3. Calculate the initial void ratio, e_0 , of the specimen, using the equation

$$e_0 = \frac{V_v}{V_s} = \frac{H_v}{H_s} \frac{A}{A} = \frac{H_v}{H_s}$$

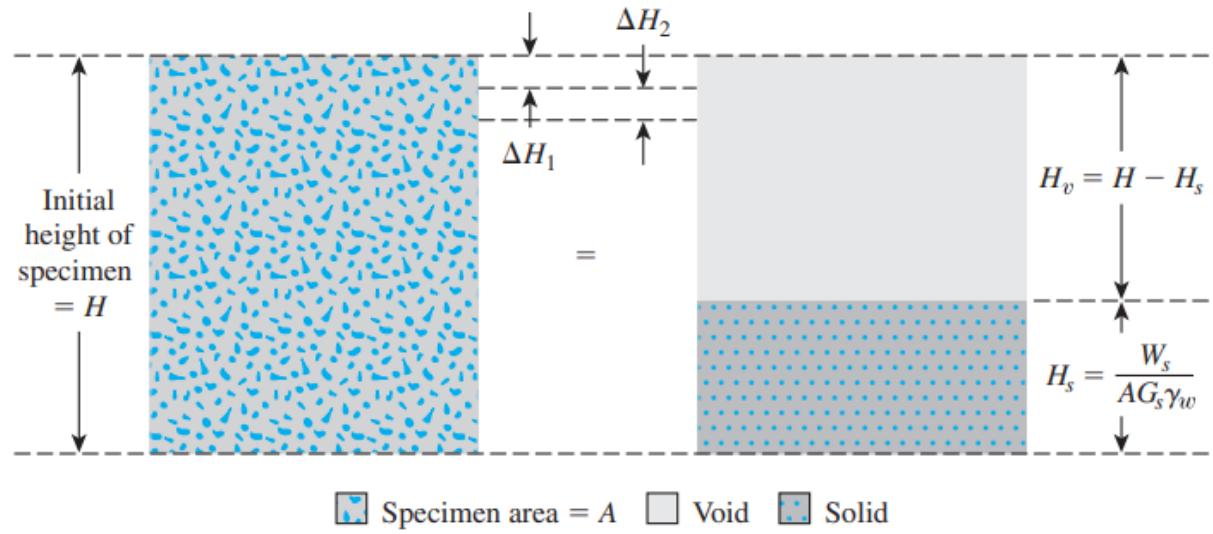


Figure 11.9 Change of height of specimen in one-dimensional consolidation test

Void Ratio-Pressure Plots

4. For the first incremental loading, σ_1 (total load/unit area of sample), which causes a deformation ΔH_1 , calculate the change in the void ratio as

$$\Delta e_1 = \frac{\Delta H_1}{H_s}$$

ΔH_1 is obtained from the initial and the final dial readings for the loading.

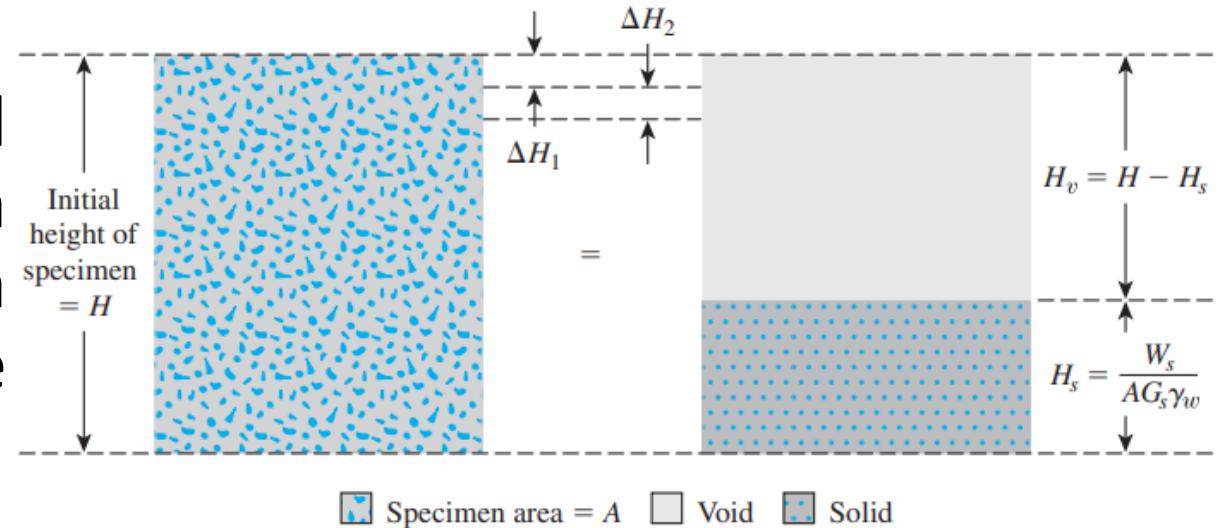


Figure 11.9 Change of height of specimen in one-dimensional consolidation test

Void Ratio-Pressure Plots

5. Calculate the new void ratio, e_1 , after consolidation caused by the pressure increment.

$$e_1 = e_0 - \Delta e_1$$

- For the next loading, σ_2 , which causes additional deformation ΔH_2 . The void ratio e_2 at the end of consolidation can be calculated as

$$e_2 = e_1 - \frac{\Delta H_2}{H_s}$$

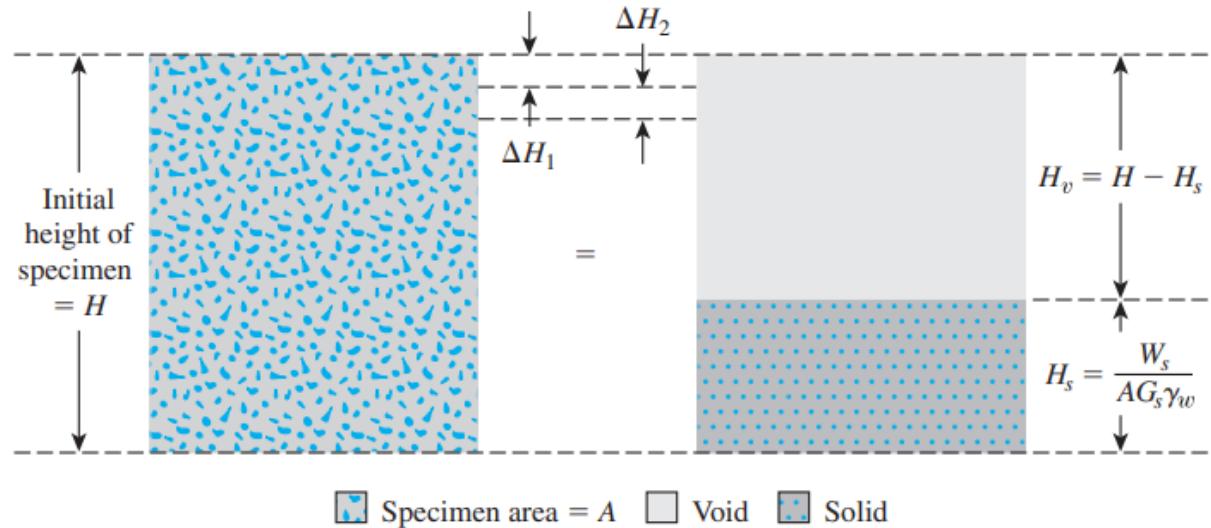


Figure 11.9 Change of height of specimen in one-dimensional consolidation test

Void Ratio-Pressure Plots

- The effective stress σ' and the corresponding void ratios (e) at the end of consolidation are plotted on **semilogarithmic graph paper**

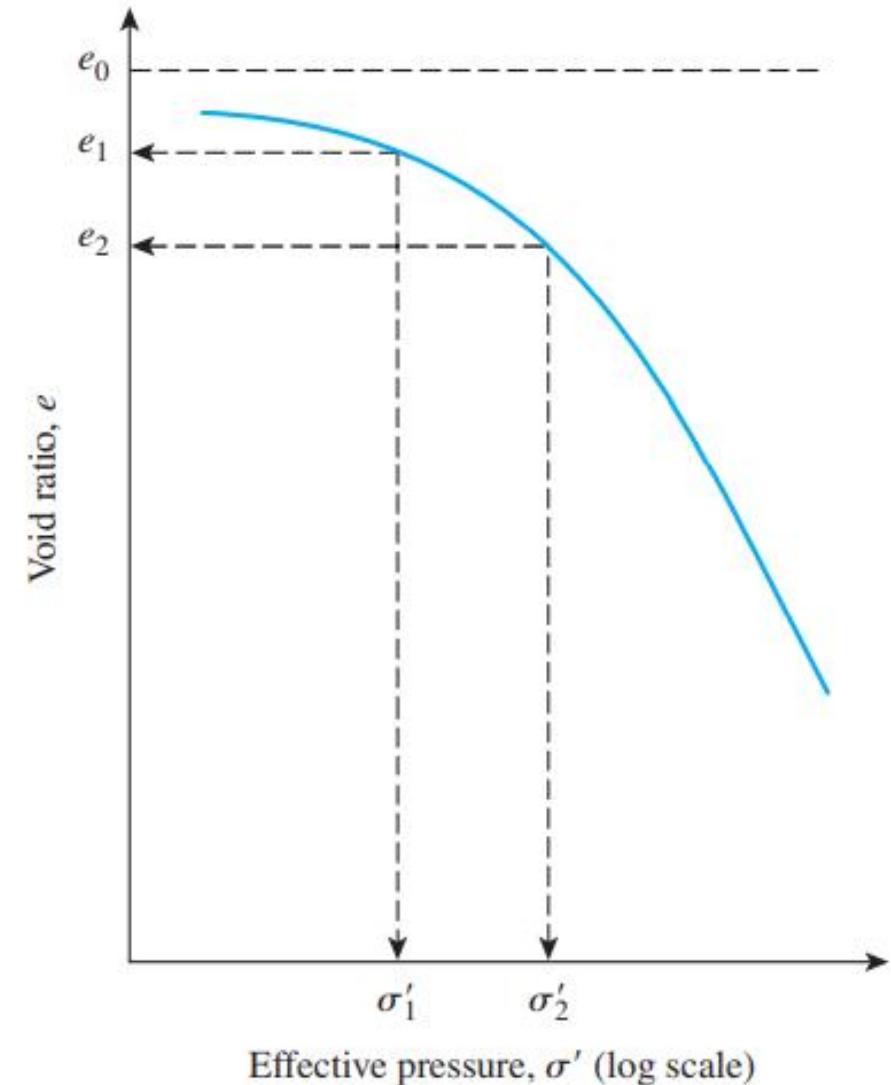


Figure 11.10 Typical plot of e against $\log \sigma'$

Void Ratio-Pressure Plots

- The upper part of the e - $\log \sigma'$ plot is somewhat curved with a flat slope, followed by a linear relationship for the void ratio, with $\log \sigma'$ having a steeper slope.

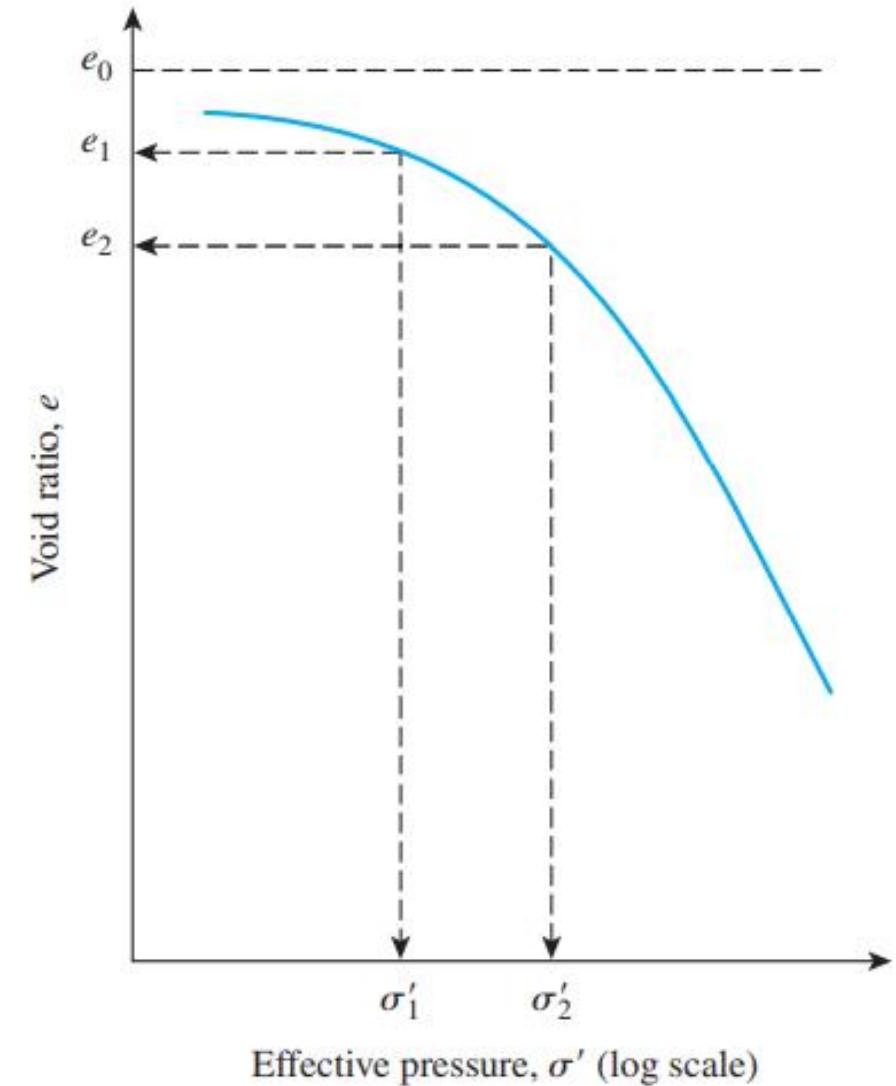


Figure 11.10 Typical plot of e against $\log \sigma'$

Normally Consolidated and Overconsolidated Clays

- Maximum effective past pressure in its geologic history.

This maximum effective past pressure may be equal to or greater than the existing overburden pressure at the time of sampling.

- When this specimen is subjected to a consolidation test, **a small amount of compression** (that is, a small change in void ratio) will occur **when the total pressure applied is less than the maximum effective overburden pressure** in the field to which the soil has been subjected in the past.
- When the total applied pressure on the specimen is **greater than the maximum effective past pressure**, **the change in the void ratio is much larger**, and the $e - \log \sigma'$ relationship is practically linear with a steeper slope.

Normally Consolidated and Overconsolidated Clays

1. Normally consolidated

present effective overburden pressure is the maximum pressure that the soil has been subjected to in the past.

2. Overconsolidated

present effective overburden pressure is less than that which the soil has experienced in the past. The maximum effective past pressure is called the preconsolidation pressure

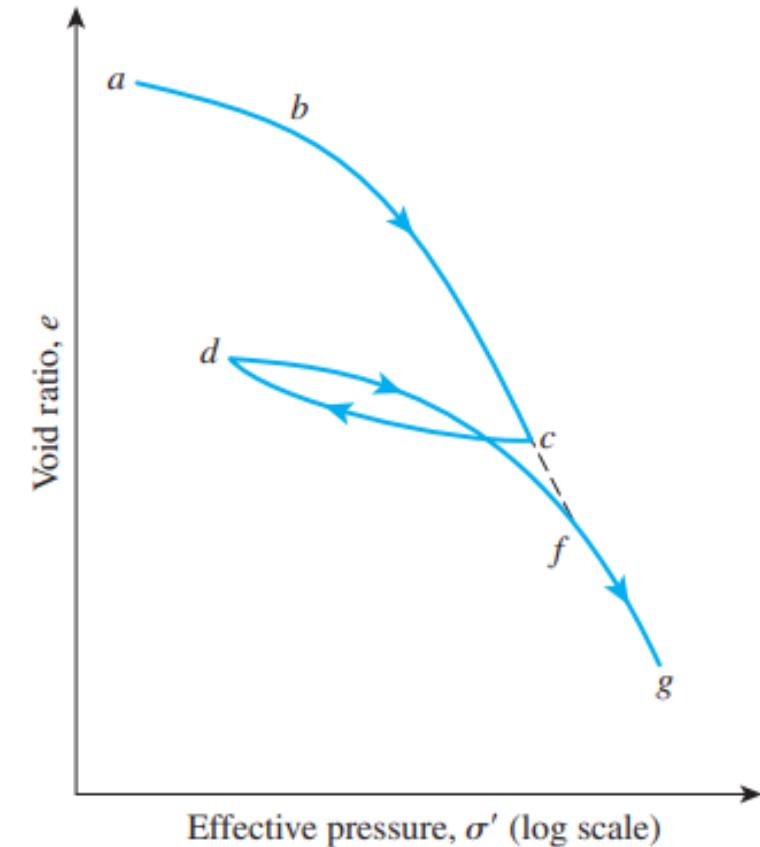


Figure 11.12 Plot of e against $\log \sigma'$ showing loading, unloading, and reloading branches

Normally Consolidated and Overconsolidated Clays

- **Casagrande (1936)** method to determine the preconsolidation pressure,
 1. By visual observation, establish **point a** at which the $e - \log \sigma'$ plot has a minimum radius of curvature.
 2. Draw a horizontal line ab.
 3. Draw the line ac tangent at a.
 4. Draw the line ad, which is the bisector of the angle back.

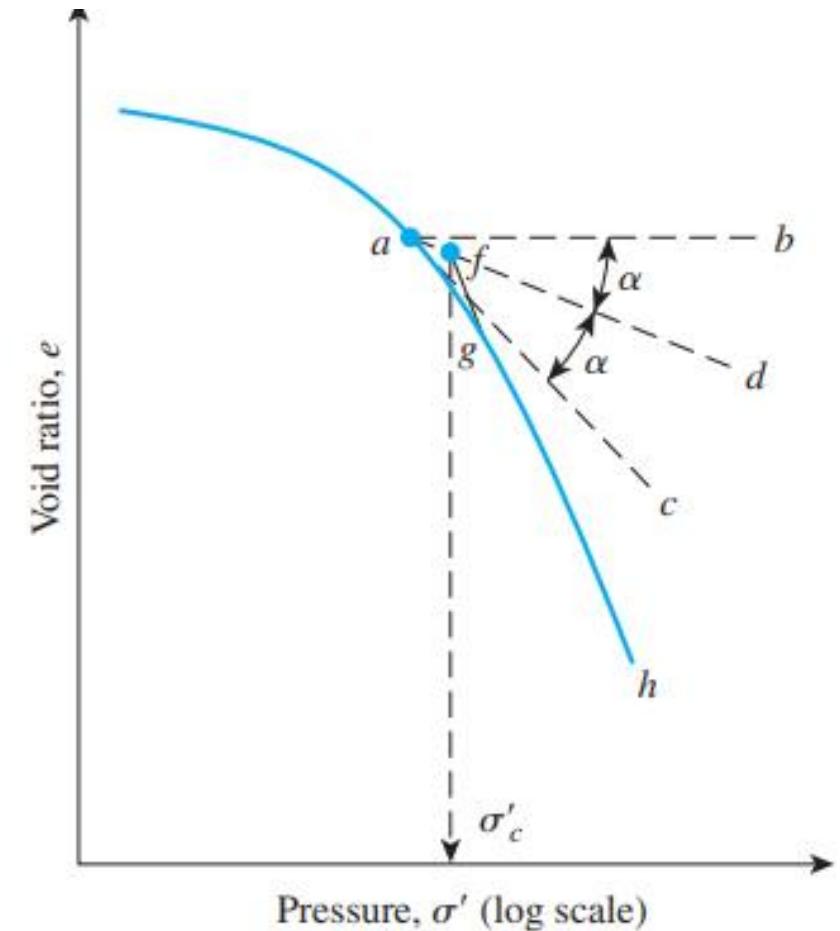


Figure 11.13 Graphic procedure for determining preconsolidation pressure

Normally Consolidated and Overconsolidated Clays

- Casagrande (1936) method to determine the preconsolidation pressure,

5. Project the straight-line portion gh of the $e - \log \sigma'$ plot back to intersect ad at f .

The abscissa of point f is the preconsolidation pressure, σ'_c

- overconsolidation ratio (OCR)

$$OCR = \frac{\sigma'_c}{\sigma'}$$

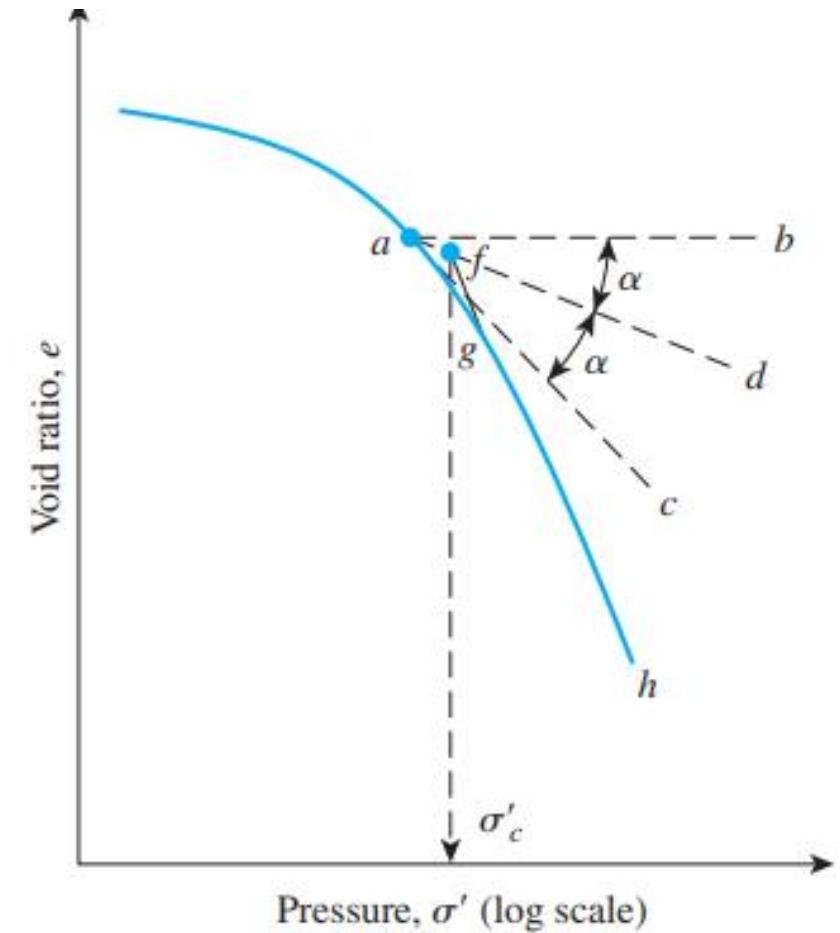


Figure 11.13 Graphic procedure for determining preconsolidation pressure

Effect of Disturbance on Void Ratio-Pressure Relationship

- A soil specimen will be remolded when it is subjected to some degree of disturbance.

Normally Consolidated Clay of Low to Medium Plasticity

1. In Figure 11.14, curve 2 is the laboratory e-log σ' plot. From this plot, determine the preconsolidation pressure (σ'_c) = (σ'_{o_0}) (that is, the present effective overburden pressure). Knowing where $\sigma'_c = \sigma'_{o_0}$, draw vertical line ab.
2. Calculate the void ratio in the field, e_0 . Draw horizontal line cd.
3. Calculate $0.4e_0$ and draw line ef
4. Join points f and g, g is the point of intersection of lines ab and cd. This is the virgin compression curve.

If a soil is remolded completely, the general position of the e-log σ' plot will be as represented by curve 3.

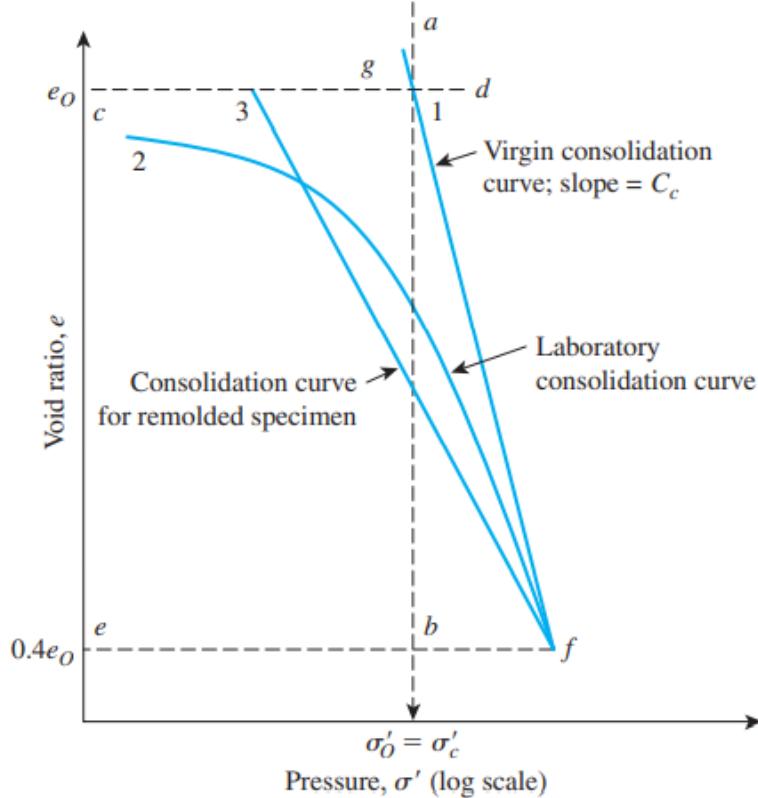


Figure 11.14 Consolidation characteristics of normally consolidated clay of low to medium sensitivity

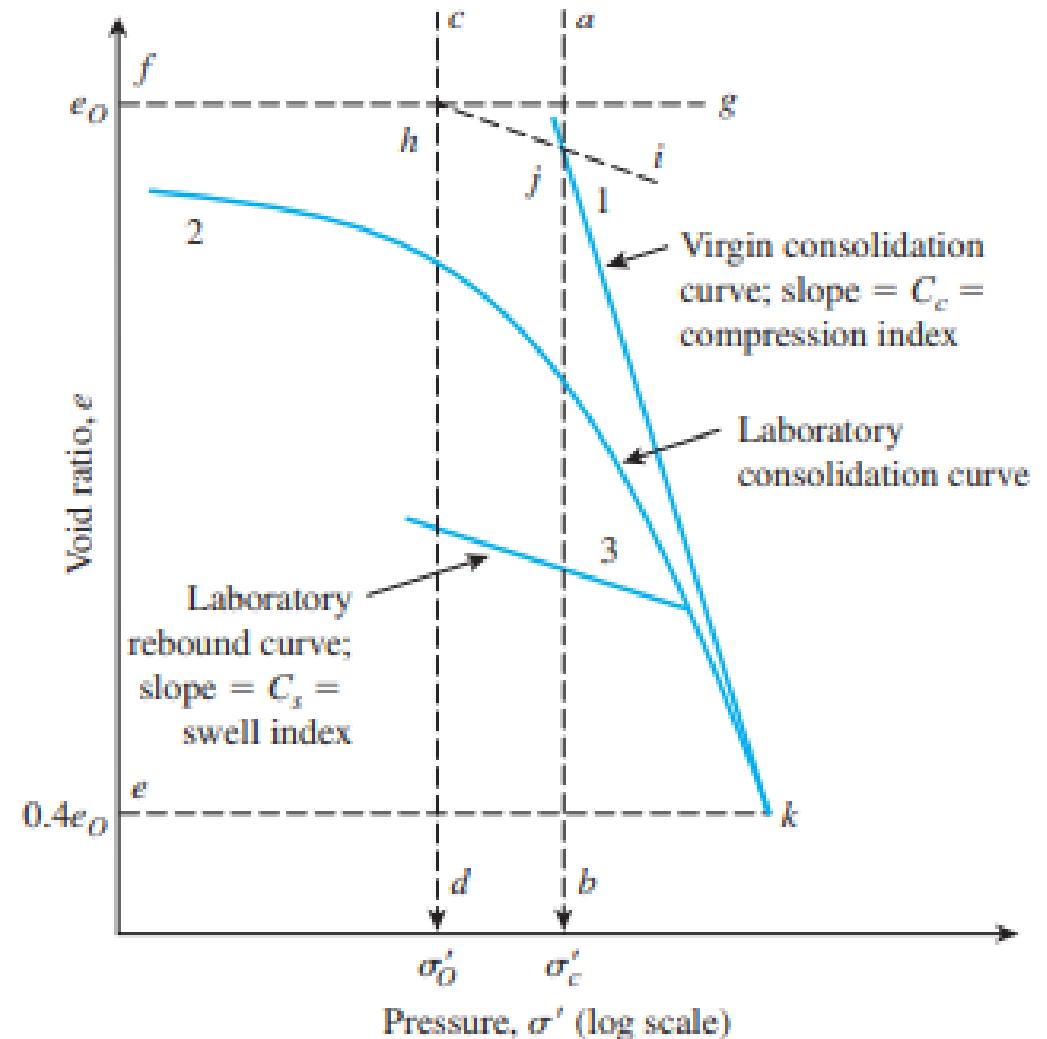
Effect of Disturbance on Void Ratio-Pressure Relationship

OverConsolidated Clay of Low to Medium Plasticity

1. In Figure 11.15, curve 2 is the laboratory e-log σ' plot/loading), and curve 3 is the laboratory unloading, or rebound, curve. From curve 2, determine the preconsolidation pressure σ'_c . Draw the vertical line ab.
2. Determine the field effective overburden pressure σ'_o . Draw vertical line cd.
3. Determine the void ratio in the field, e_0 . Draw the horizontal line fg. The point of intersection of lines fg and cd is h
4. Draw a line hi, which is parallel to curve 3. The point of intersection of line hi and ab is j.
5. Join points j and k. Point k is on curve 2, and its ordinate is $0.4e_0$.
6. The field consolidation plot will take a path hjk. The recompression path in the field is hj and is parallel to the laboratory rebound curve(Schmertmann, 1953)

Effect of Disturbance on Void Ratio-Pressure Relationship

OverConsolidated Clay of Low to Medium Plasticity



Calculation of Settlement from One-Dimensional Primary Consolidation

Effective overburden pr. σ'_0 , increased effective pr. $\Delta\sigma'$ → primary settlement S_c

$$\Delta V = V_0 - V_1 = HA - (H - S_c)A = S_c A$$

$$\Delta V = S_c A = V_{v0} - V_{v1} = \Delta V_v$$

$$\Delta V_v = \Delta e V_s$$

$$V_s = \frac{V_0}{1 + e_0} = \frac{AH}{1 + e_0}$$

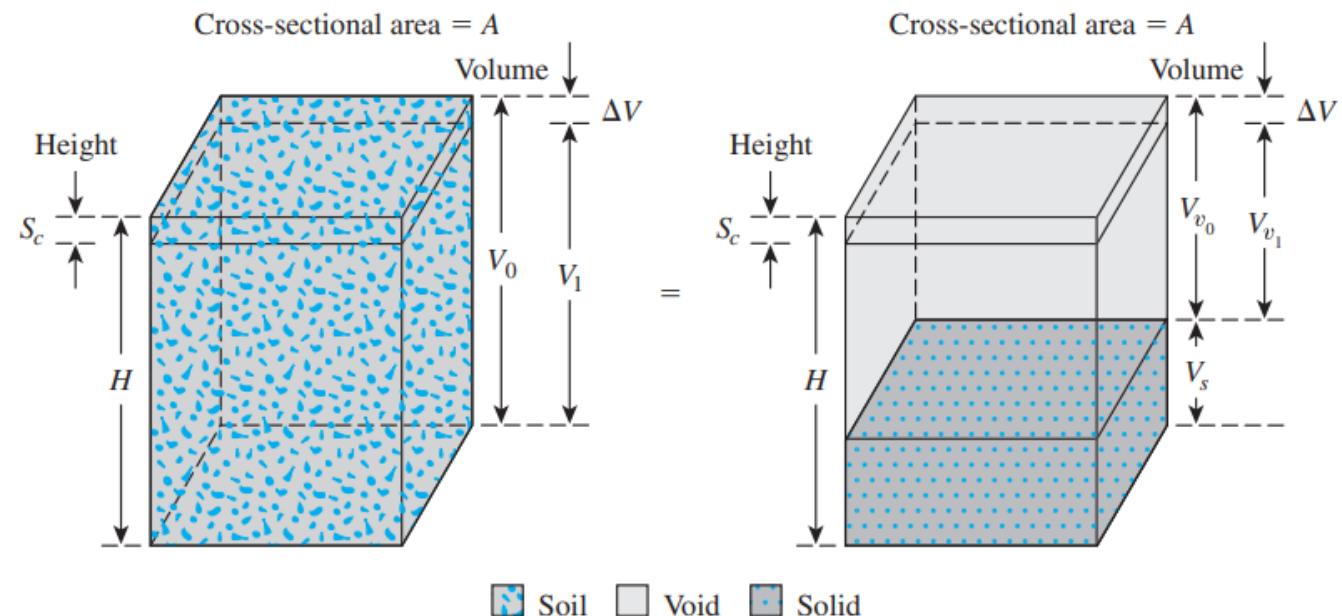


Figure 11.16 Settlement caused by one-dimensional consolidation

Calculation of Settlement from One-Dimensional Primary Consolidation

Effective overburden pr. σ'_0 , increased effective pr. $\Delta\sigma'$ → primary settlement S_c

$$V_s = \frac{V_0}{1 + e_0} = \frac{AH}{1 + e_0}$$

$$\Delta V = S_c A = \Delta e V_s = \frac{AH}{1 + e_0} \Delta e$$

$$S_c = H \left(\frac{\Delta e}{1 + e_0} \right)$$

- For normally consolidated clays

$$\Delta e = C_c [\log(\sigma'_0 + \Delta\sigma') - \log\sigma'_0]$$

C_c = Compression index

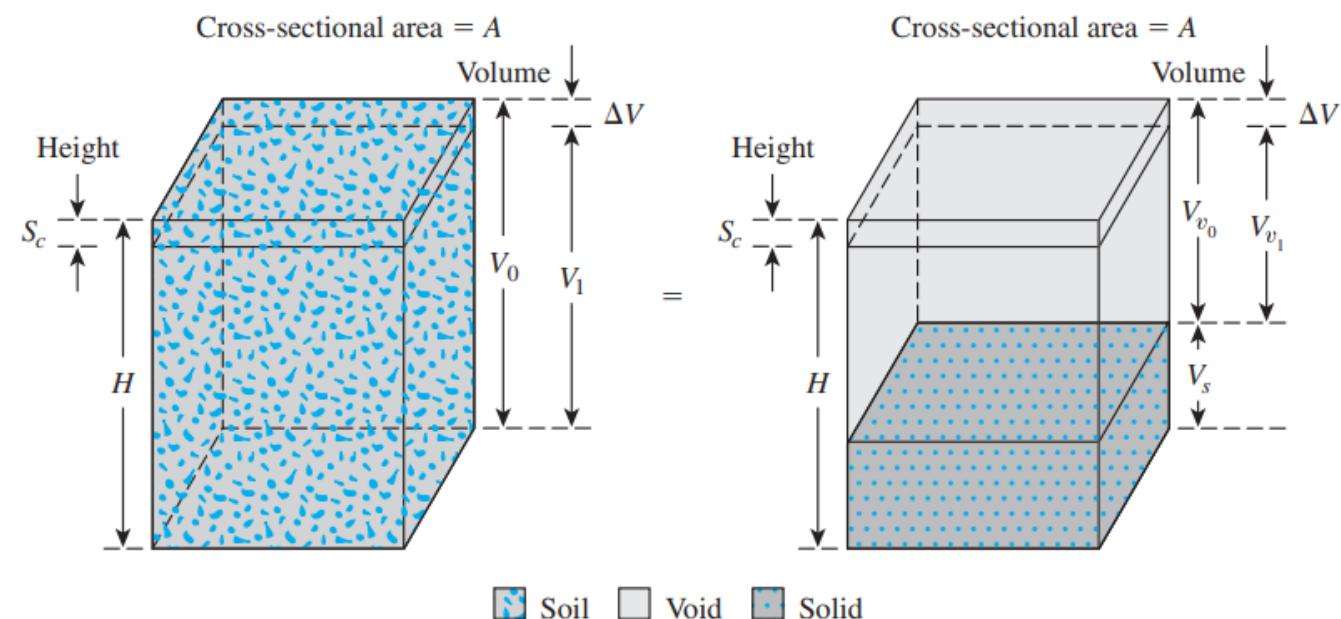


Figure 11.16 Settlement caused by one-dimensional consolidation

Calculation of Settlement from One-Dimensional Primary Consolidation

Effective overburden pr. σ'_0 , increased effective pr. $\Delta\sigma'$ → primary settlement S_c

$$S_c = \frac{C_c H}{(1 + e_0)} \log \left(\frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0} \right)$$

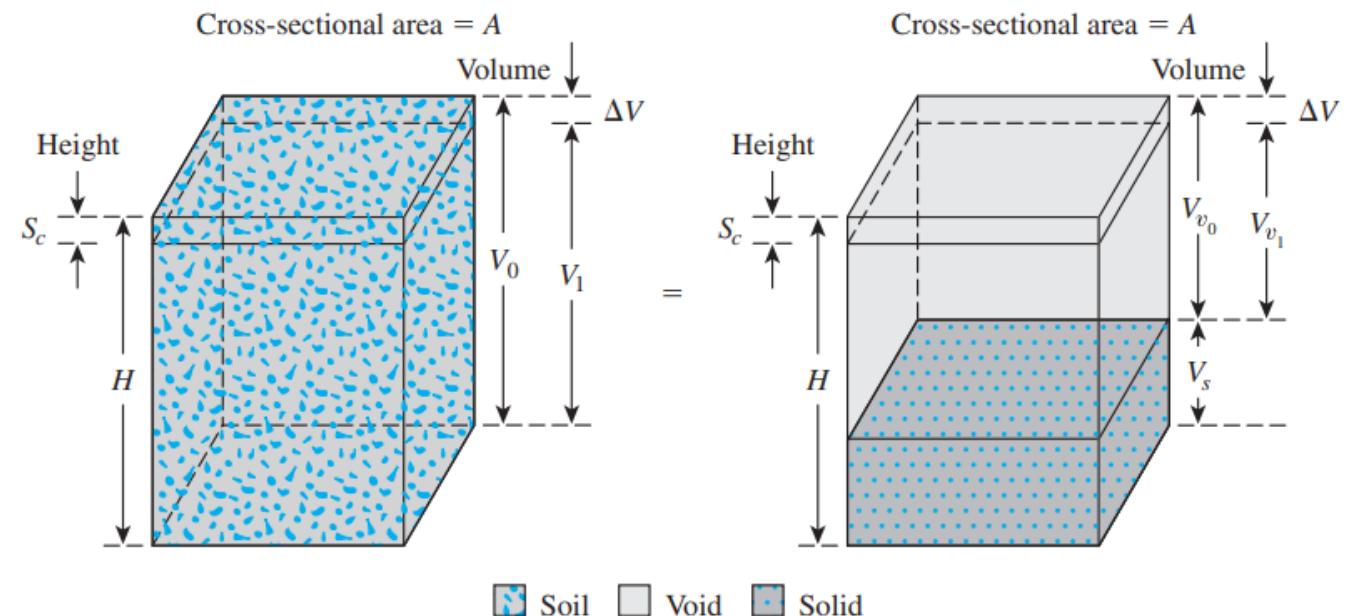


Figure 11.16 Settlement caused by one-dimensional consolidation

Calculation of Settlement from One-Dimensional Primary Consolidation

Effective overburden pr. σ'_0 , increased effective pr. $\Delta\sigma'$ → primary settlement S_c

- In overconsolidated clays, for

$$\sigma'_0 + \Delta\sigma' \leq \sigma'_c$$

$$\Delta e = C_s [\log(\sigma'_0 + \Delta\sigma') - \log\sigma'_0]$$

$$S_c = \frac{C_s H}{(1 + e_0)} \log \left(\frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0} \right)$$

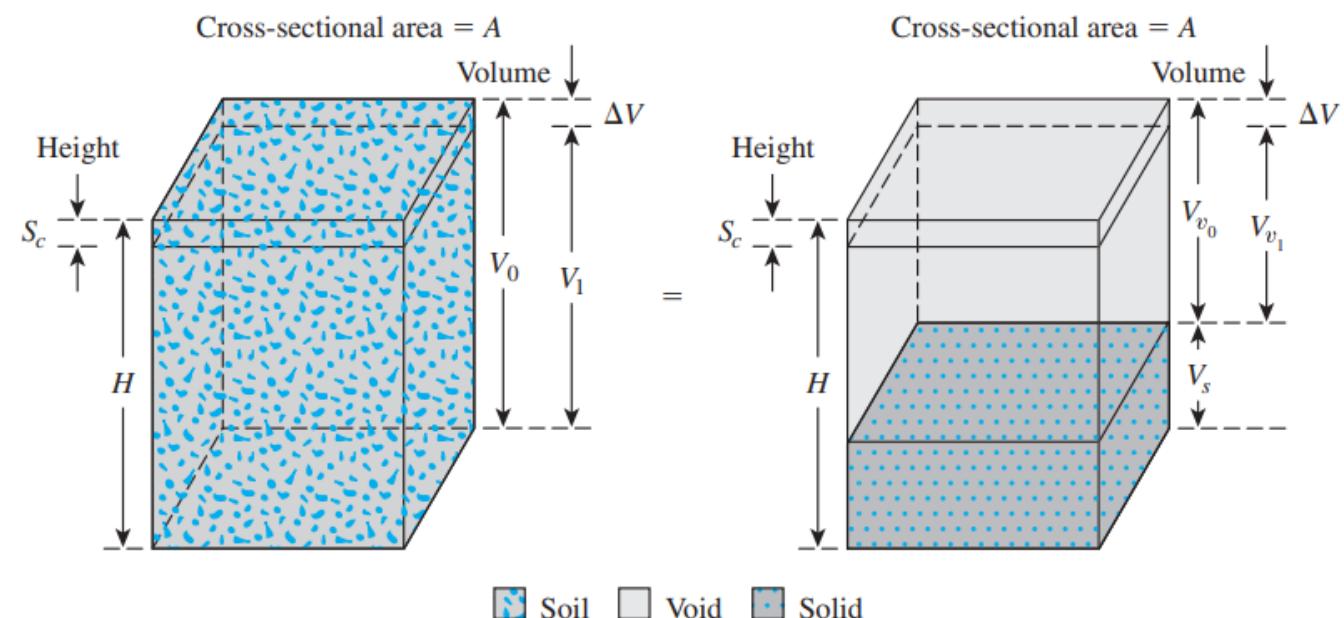


Figure 11.16 Settlement caused by one-dimensional consolidation

Calculation of Settlement from One-Dimensional Primary Consolidation

Effective overburden pr. σ'_0 , increased effective pr. $\Delta\sigma'$ → primary settlement S_c

- In overconsolidated clays, for

$$\sigma'_0 + \Delta\sigma' \geq \sigma'_c$$

$$S_c = \frac{C_s H}{(1 + e_0)} \log \left(\frac{\sigma'_c}{\sigma'_0} \right) + \frac{C_c H}{(1 + e_0)} \log \left(\frac{\sigma'_0 + \Delta\sigma'}{\sigma'_c} \right)$$

- However, if the e-log p curve is given, it is possible to simply to pick Δe of the plot for the appropriate range of pressures

Compression Index (C_c)

- Terzaghi and Peck (1967)

- For undisturbed clays :

$$C_c = 0.009(LL - 10)$$

- For remolded clays

$$C_c = 0.007(LL - 10)$$

- Rendon-Herrero (1983)

$$C_c = 0.141 G_s^{1.2} \left(\frac{1 + e_0}{G_s} \right)^{2.38}$$

- Nagaraj and Murty (1985)

$$C_c = 0.2343 \left[\frac{LL(\%)}{100} \right] G_s$$

Compression Index (C_c)

Table 11.6 Correlations for Compression Index, C_c *

Equation	Reference	Region of applicability
$C_c = 0.007(LL - 7)$	Skempton (1944)	Remolded clays
$C_c = 0.01w_N$		Chicago clays
$C_c = 1.15(e_O - 0.27)$	Nishida (1956)	All clays
$C_c = 0.30(e_O - 0.27)$	Hough (1957)	Inorganic cohesive soil: silt, silty clay, clay
$C_c = 0.0115w_N$		Organic soils, peats, organic silt, and clay
$C_c = 0.0046(LL - 9)$		Brazilian clays
$C_c = 0.75(e_O - 0.5)$		Soils with low plasticity
$C_c = 0.208e_O + 0.0083$		Chicago clays
$C_c = 0.156e_O + 0.0107$		All clays

*After Rendon-Herrero, 1980. With permission from ASCE.

Note: e_O = *in situ* void ratio; w_N = *in situ* water content.

Swelling Index (C_s)

$$C_s \approx \frac{1}{5} \text{ to } \frac{1}{10} C_c$$

- Nagaraj and Murty (1985)

$$C_s = 0.0463 \left[\frac{LL(\%)}{100} \right] G_s$$

Table 11.7 Compression and Swell of Natural Soils

Soil	Liquid limit	Plastic limit	Compression index, C_c	Swell index, C_s
Boston blue clay	41	20	0.35	0.07
Chicago clay	60	20	0.4	0.07
Ft. Gordon clay, Georgia	51	26	0.12	—
New Orleans clay	80	25	0.3	0.05
Montana clay	60	28	0.21	0.05

Time Rate of Consolidation

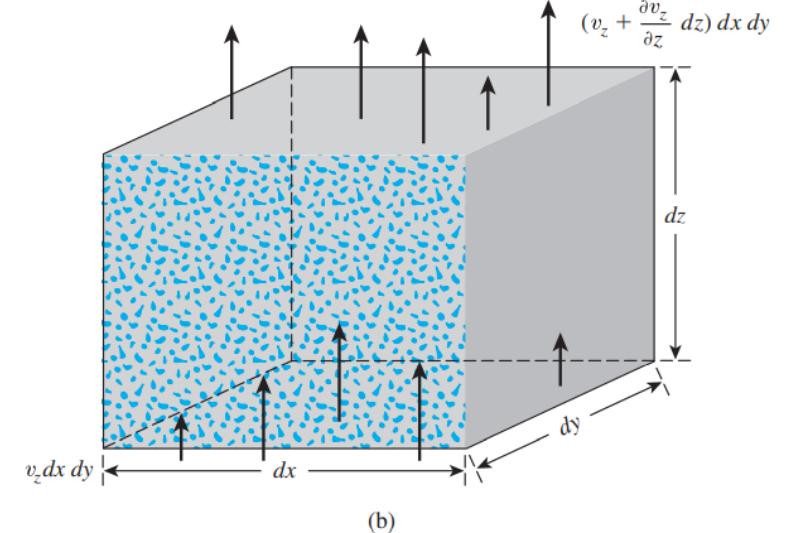
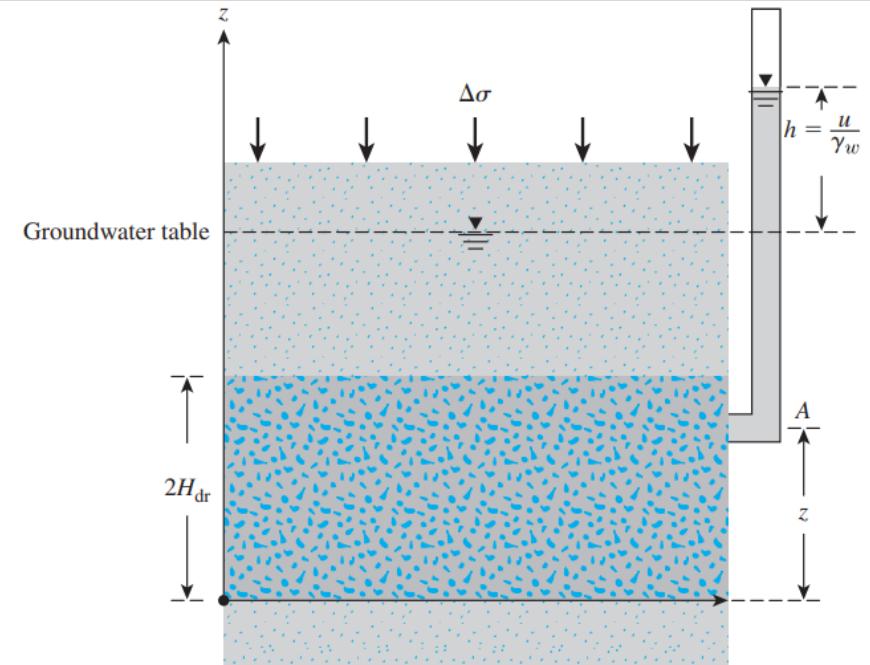
- Terzaghi(1925)
- The mathematical derivations are based on the following six assumptions (also see Taylor, 1948):
 1. The clay-water system is homogeneous.
 2. Saturation is complete.
 3. Compressibility of water is negligible.
 4. Compressibility of soil grains is negligible (but soil grains rearrange).
 5. The flow of water is in one direction only (that is, in the direction of compression.)
 6. Darcy's law is valid.

Time Rate of Consolidation

$(\text{Rate of outflow of water}) - (\text{Rate of inflow of water}) =$
 $(\text{Rate of volume change})$

$$\left(v_z + \frac{\partial v_z}{\partial z} dz \right) dx dy - v_z dx dy = \frac{\partial V}{\partial t}$$

$$\frac{\partial v_z}{\partial z} dx dy dz = \frac{\partial V}{\partial t}$$



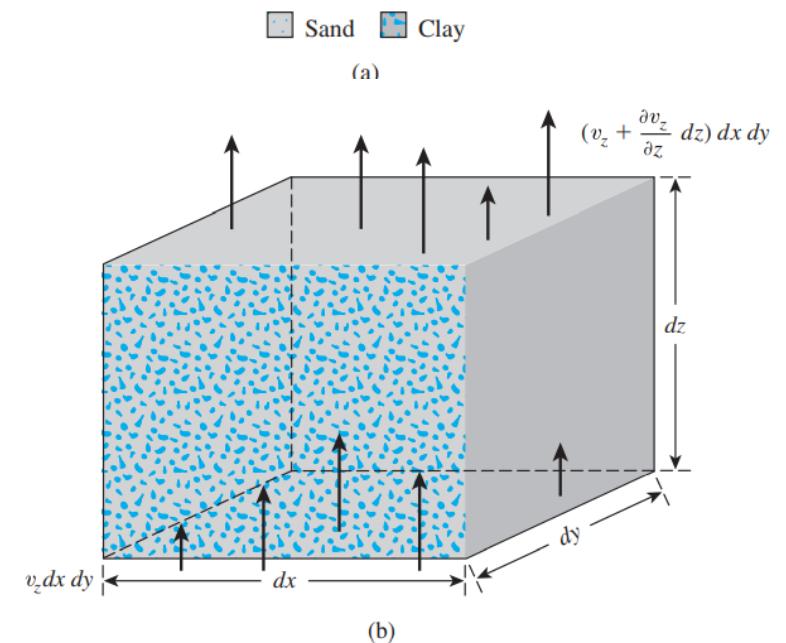
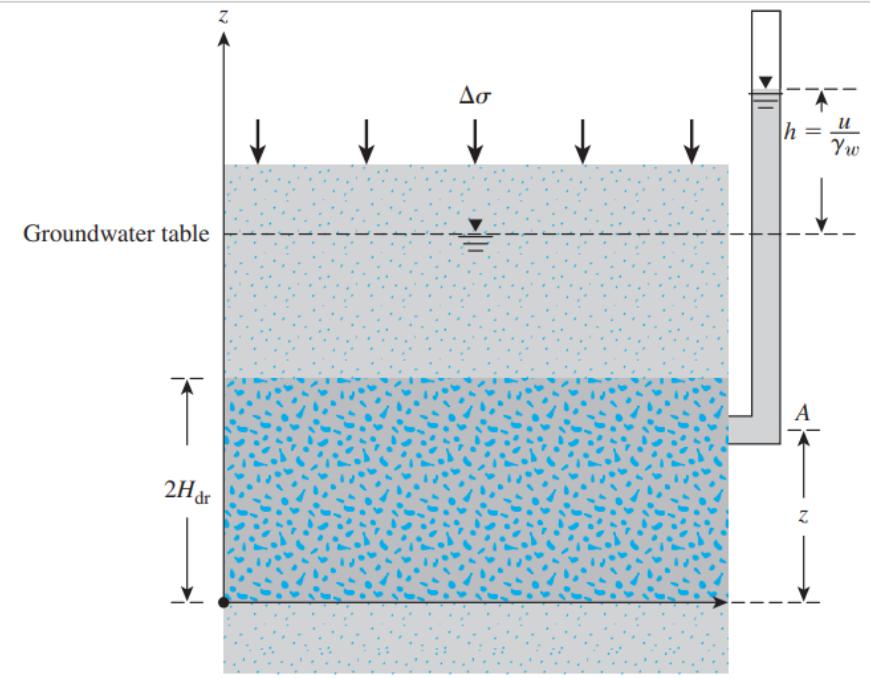
Time Rate of Consolidation

- Using Darcy's law

$$v_z = ki = -k \frac{\partial h}{\partial z} = -\frac{k}{\gamma_w} \frac{\partial u}{\partial z}$$

- Elevation head constant
- u = excess pore water pressure caused by the increase of stress

$$-\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{dx dy dz} \frac{\partial V}{\partial t}$$



Time Rate of Consolidation

- During consolidation

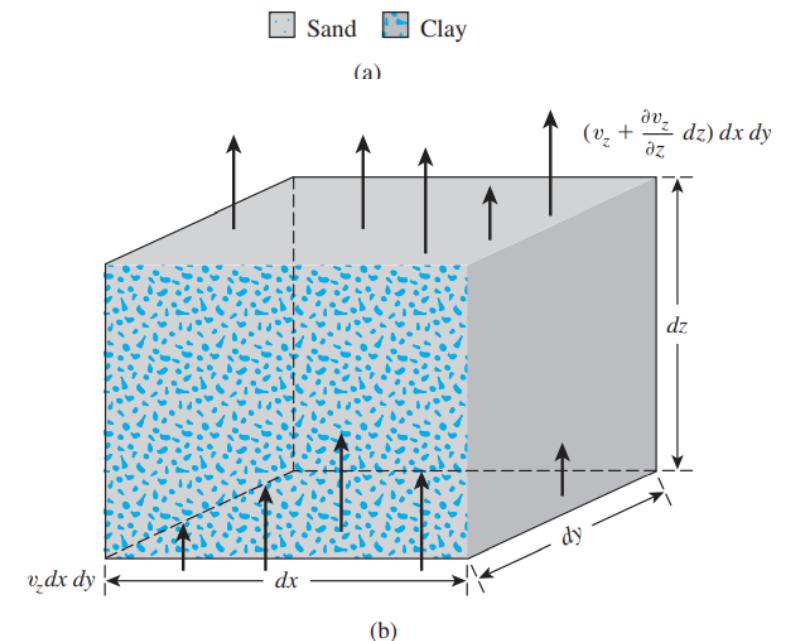
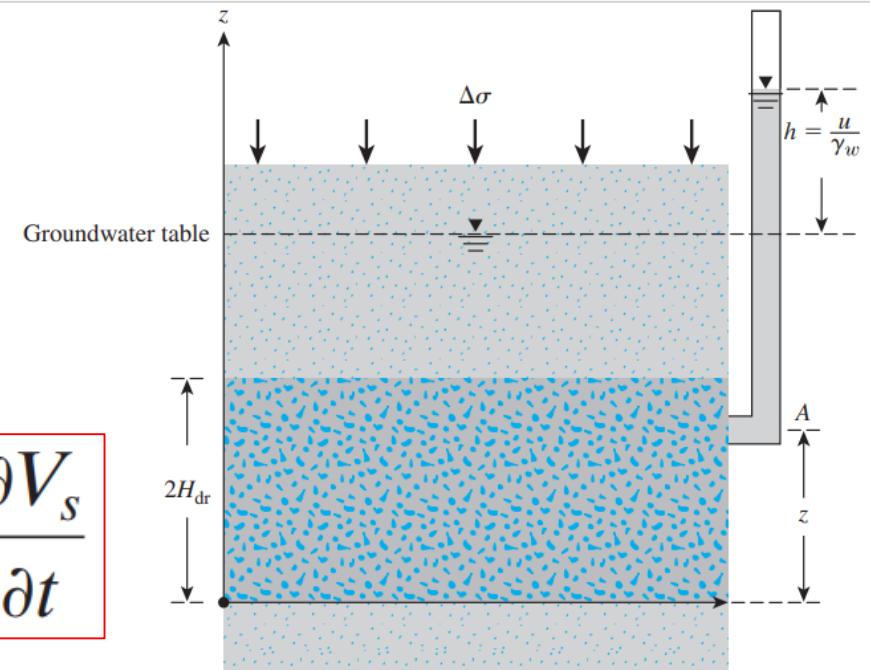
$$\frac{\partial V}{\partial t} = \frac{\partial V_v}{\partial t} = \frac{\partial(V_s + eV_s)}{\partial t} = \frac{\partial V_s}{\partial t} + V_s \frac{\partial e}{\partial t} + e \frac{\partial V_s}{\partial t}$$

- Assuming that soil solids are incompressible

$$V_s = \frac{V}{1 + e_o} = \frac{dx dy dz}{1 + e_o}$$

- Substitution for $\partial V_s / \partial t$ and V_s in yields

$$\frac{\partial V}{\partial t} = \frac{dx dy dz}{1 + e_o} \frac{\partial e}{\partial t}$$

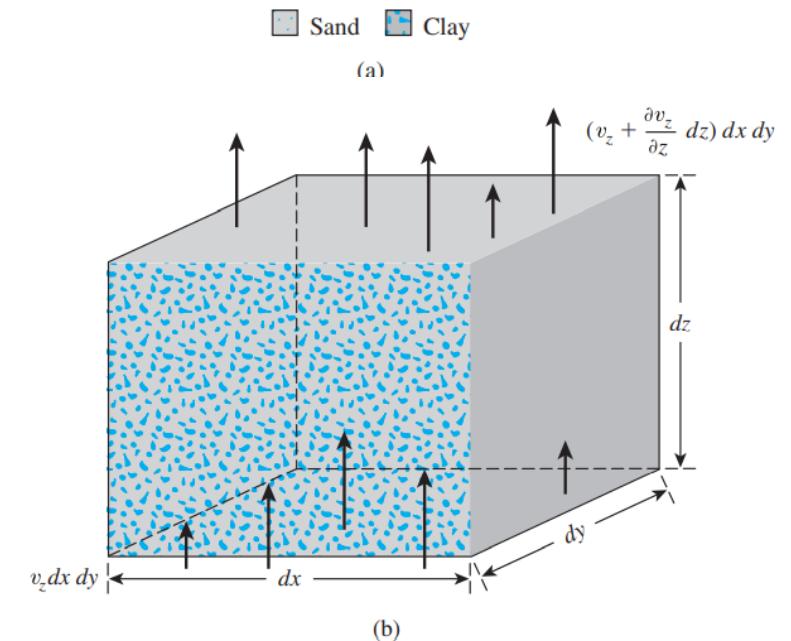
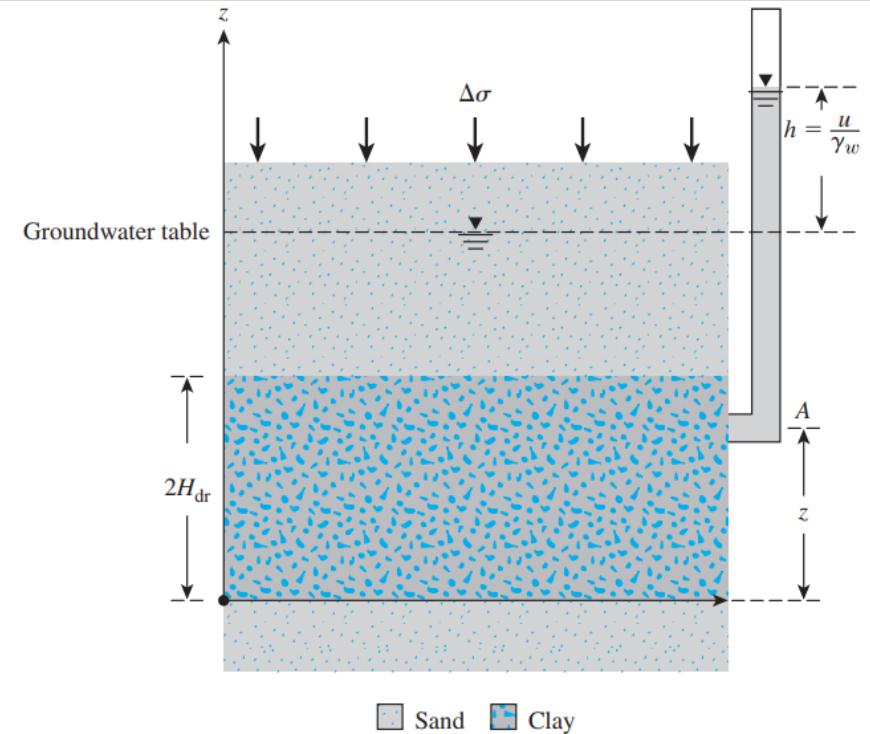


Time Rate of Consolidation

- Combining the Eqs. Gives

$$-\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{1 + e_o} \frac{\partial e}{\partial t}$$

- The change in the void ratio is caused by the increase of effective stress (that is, decrease of excess pore water pressure).

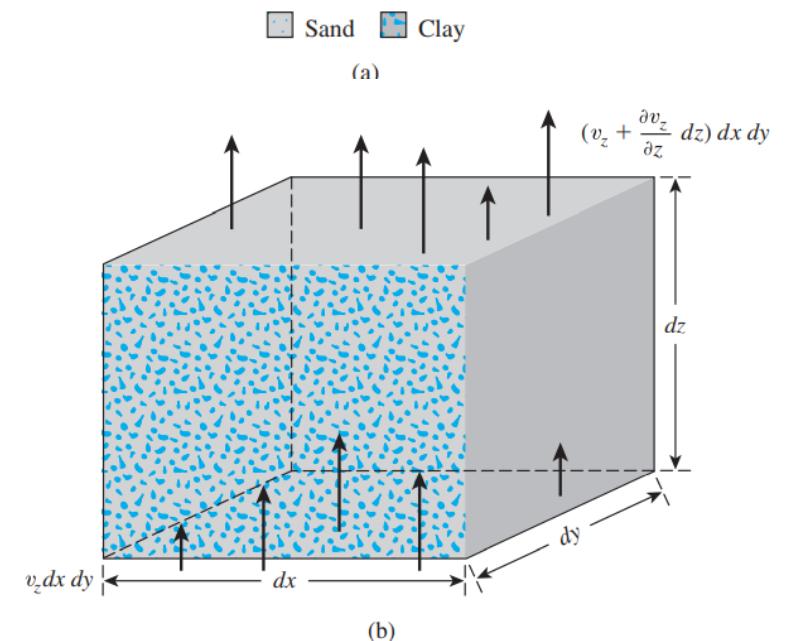
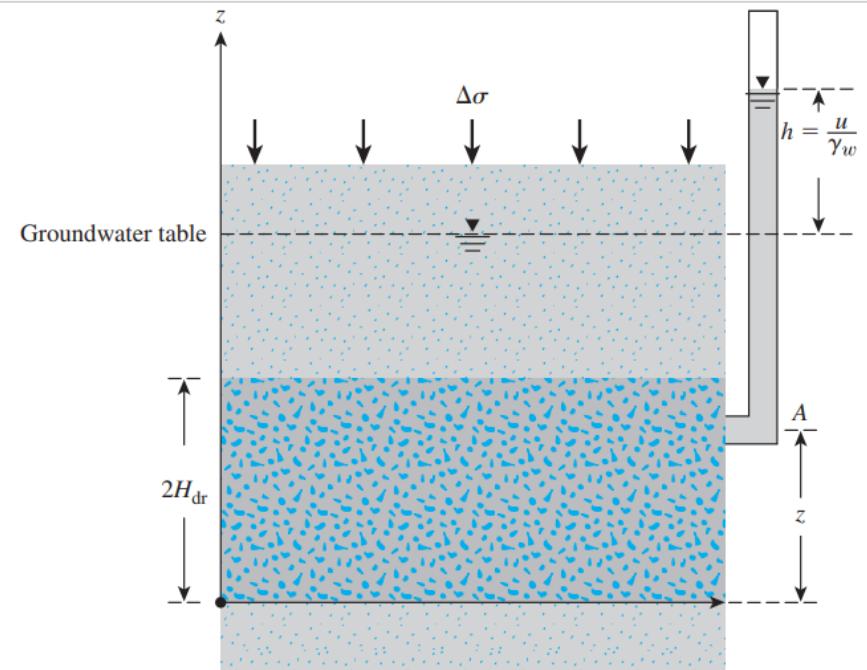


Time Rate of Consolidation

- Assuming that they are linearly related, we have

$$\partial e = a_v \partial(\Delta\sigma') = -a_v \partial u$$

- a_v = coefficient of compressibility (a_v can be considered to be constant for a narrow range of pressure increase)

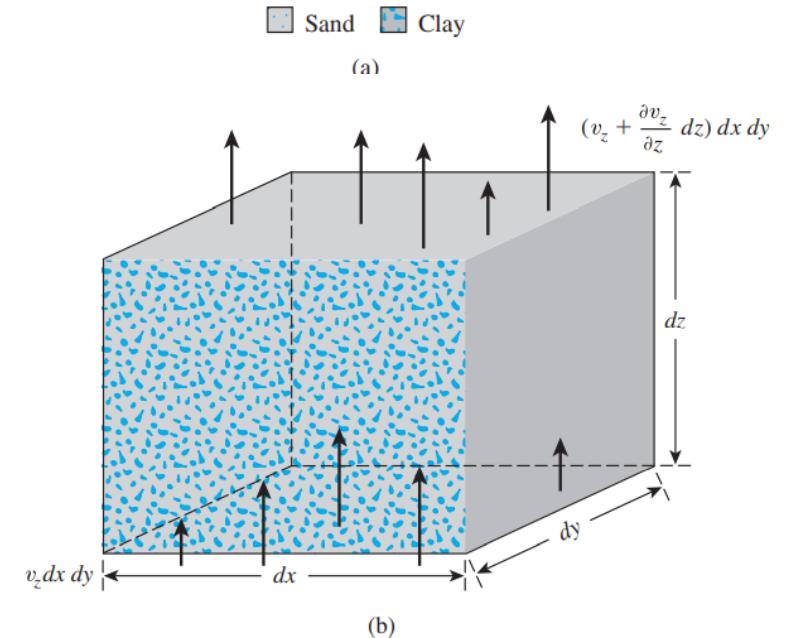
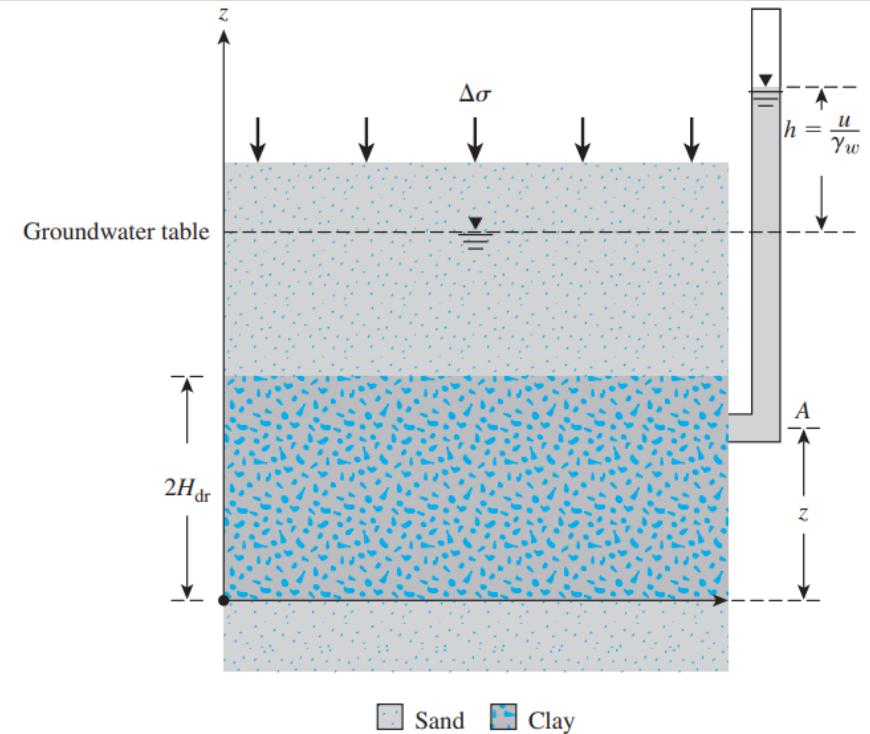


Time Rate of Consolidation

- Combining the Eqs. gives

$$-\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = -\frac{a_v}{1 + e_0} \frac{\partial u}{\partial t} = -m_v \frac{\partial u}{\partial t}$$

- m_v = coefficient of volume compressibility
 $= a_v / (1 + e_0)$

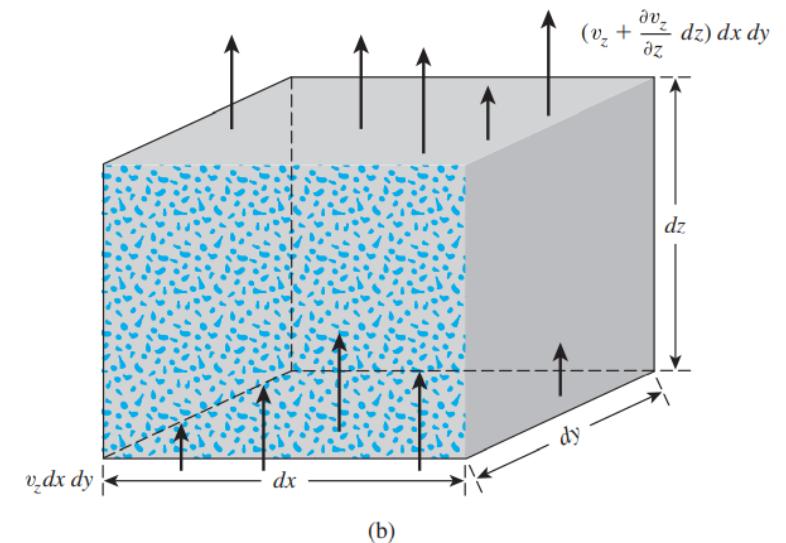
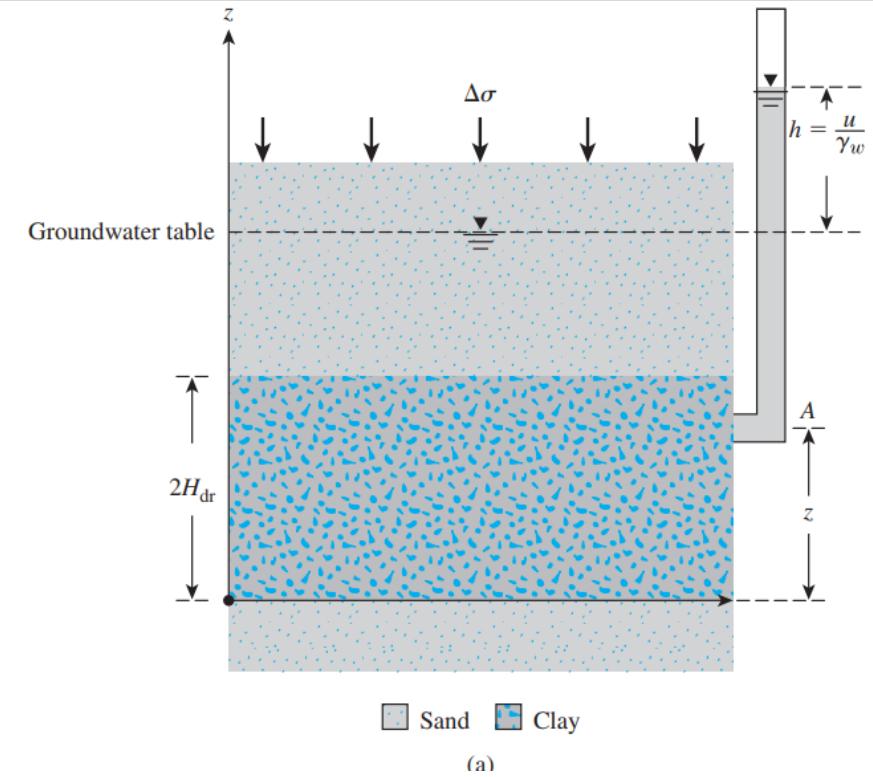


Time Rate of Consolidation

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}$$

- c_v = coefficient of consolidation = $\frac{k}{\gamma_w m_v}$
-

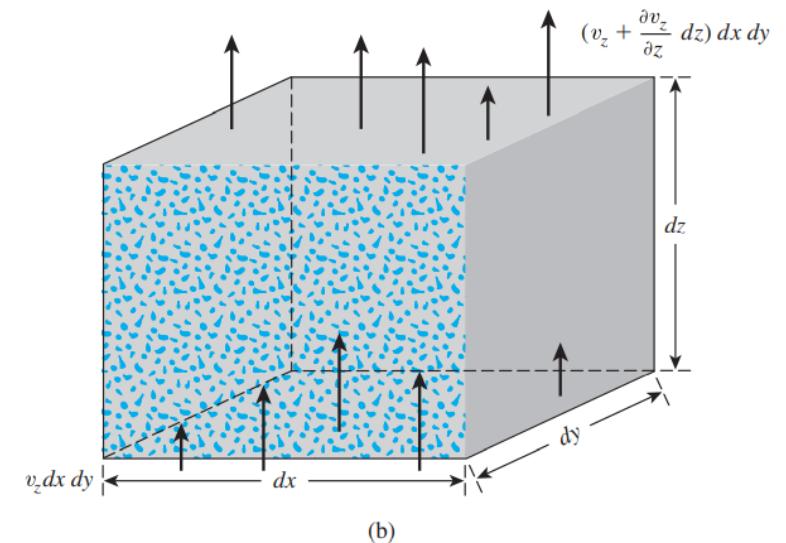
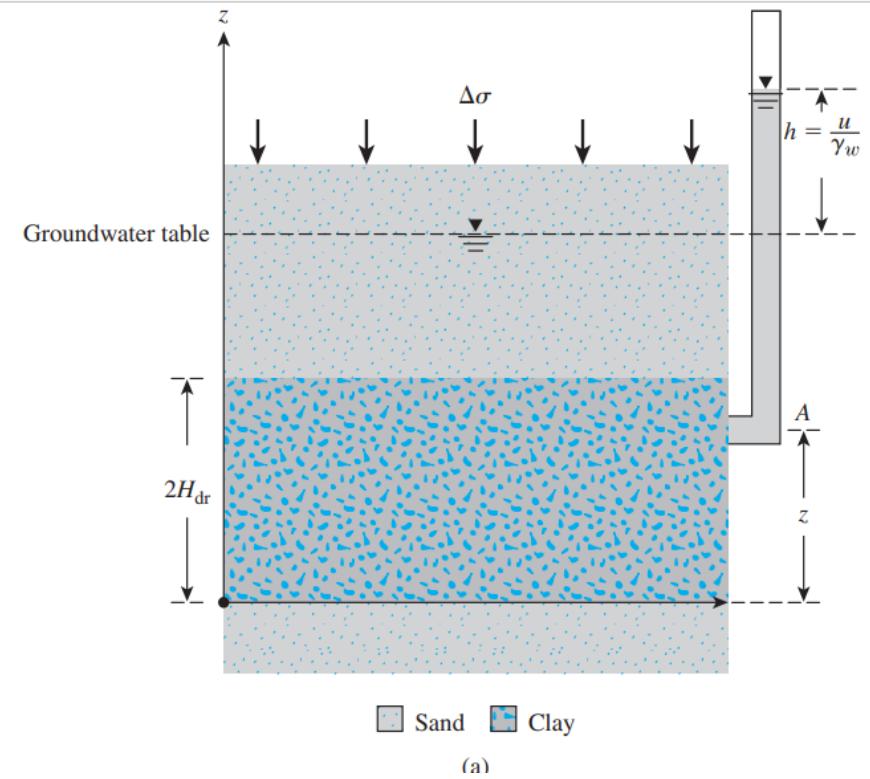
$$c_v = \frac{k}{\gamma_w m_v} = \frac{k}{\gamma_w \left(\frac{a_v}{1 + e_o} \right)}$$



Time Rate of Consolidation

- *Boundary conditions:*
- $Z=0, u=0$
- $Z=2H_{dr}, u=0$
- $t=0, u = u_0$
- The solution yields

$$u = \sum_{m=0}^{m=\infty} \left[\frac{2u_0}{M} \sin\left(\frac{Mz}{H_{dr}}\right) \right] e^{-M^2 T_v}$$



Time Rate of Consolidation

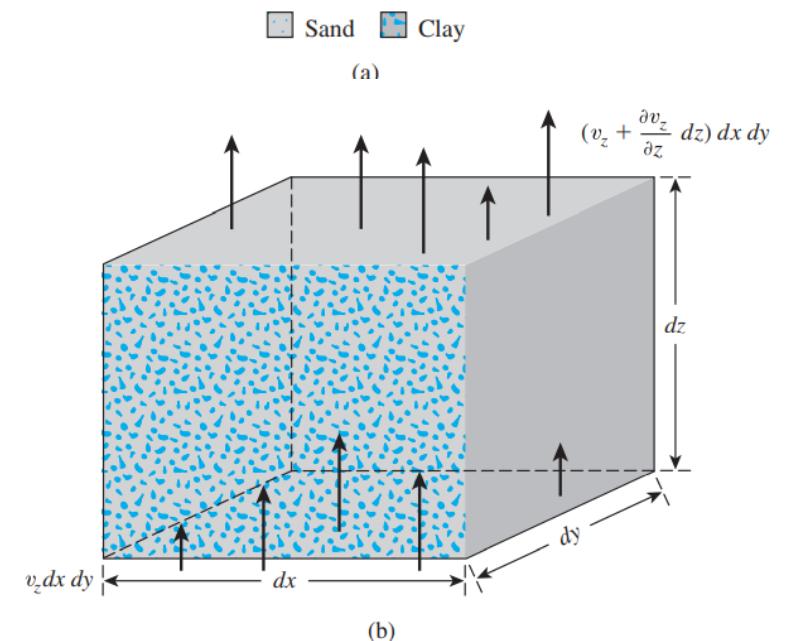
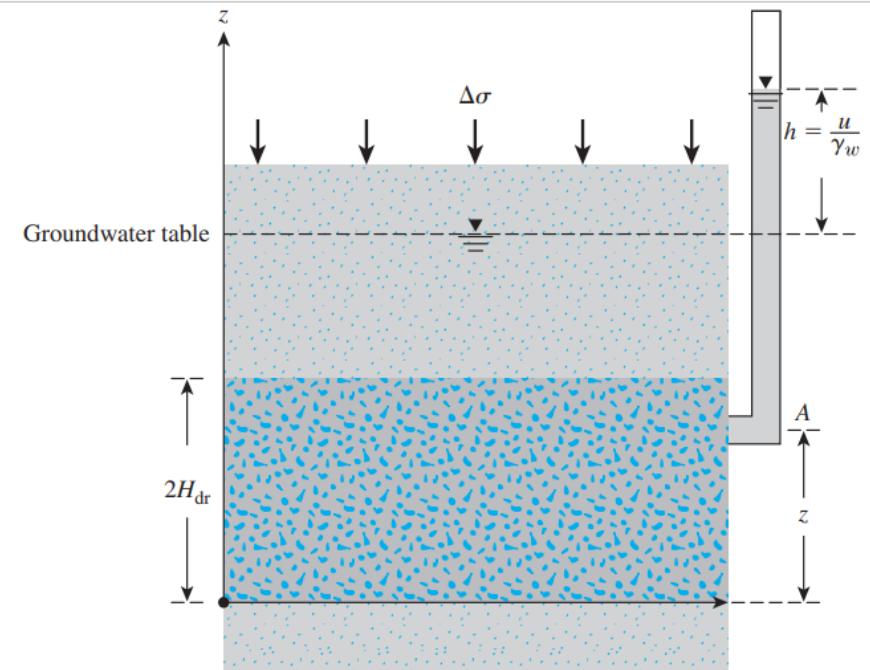
$$u = \sum_{m=0}^{m=\infty} \left[\frac{2u_0}{M} \sin\left(\frac{Mz}{H_{dr}}\right) \right] e^{-M^2 T_v}$$

- Where , m is an integer

$$M = \left(\frac{\pi}{2}\right) (2m + 1)$$

u_0 = initial excess pore water pressure

$$T_v = \frac{c_v t}{H_{dr}^2} = \text{time factor}$$



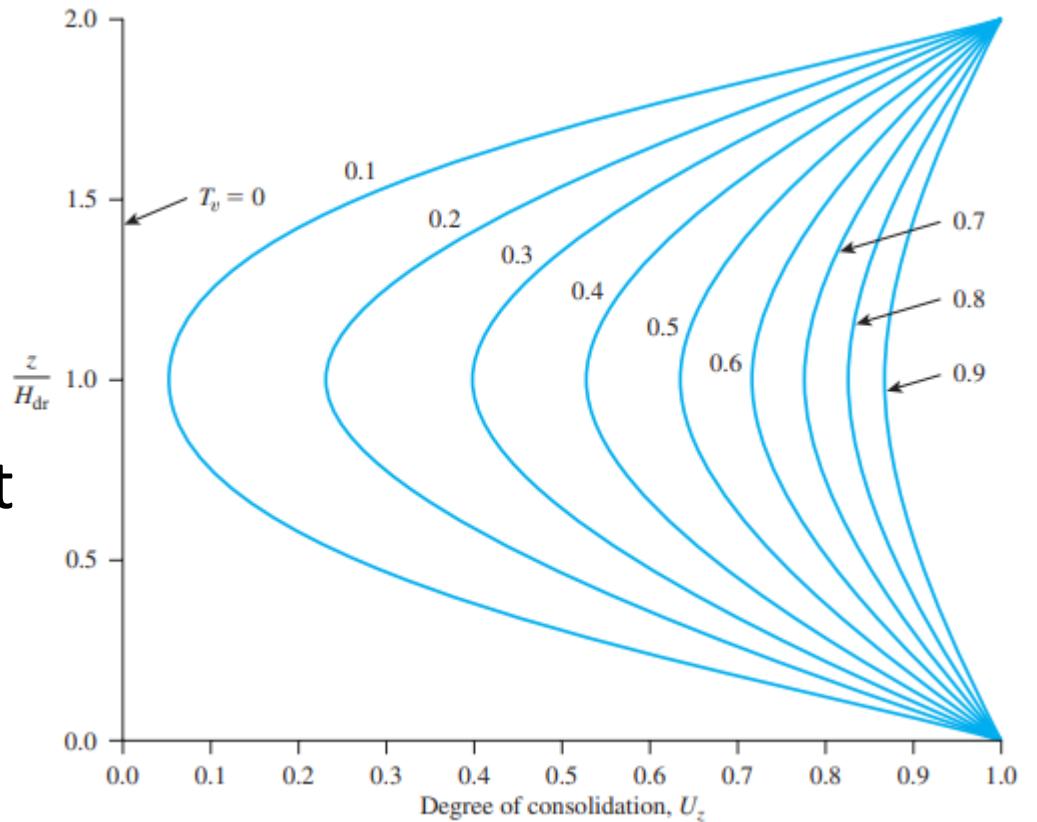
Time Rate of Consolidation

- Degree of consolidation at a distance z at any time t

$$U_z = \frac{u_o - u_z}{u_o} = 1 - \frac{u_z}{u_o}$$

- U_z = excess pore water pressure at time t
- average degree of consolidation for the entire depth of the clay layer

$$U = \frac{S_{c(t)}}{S_c} = 1 - \frac{\left(\frac{1}{2H_{dr}}\right) \int_0^{2H_{dr}} u_z dz}{u_o}$$

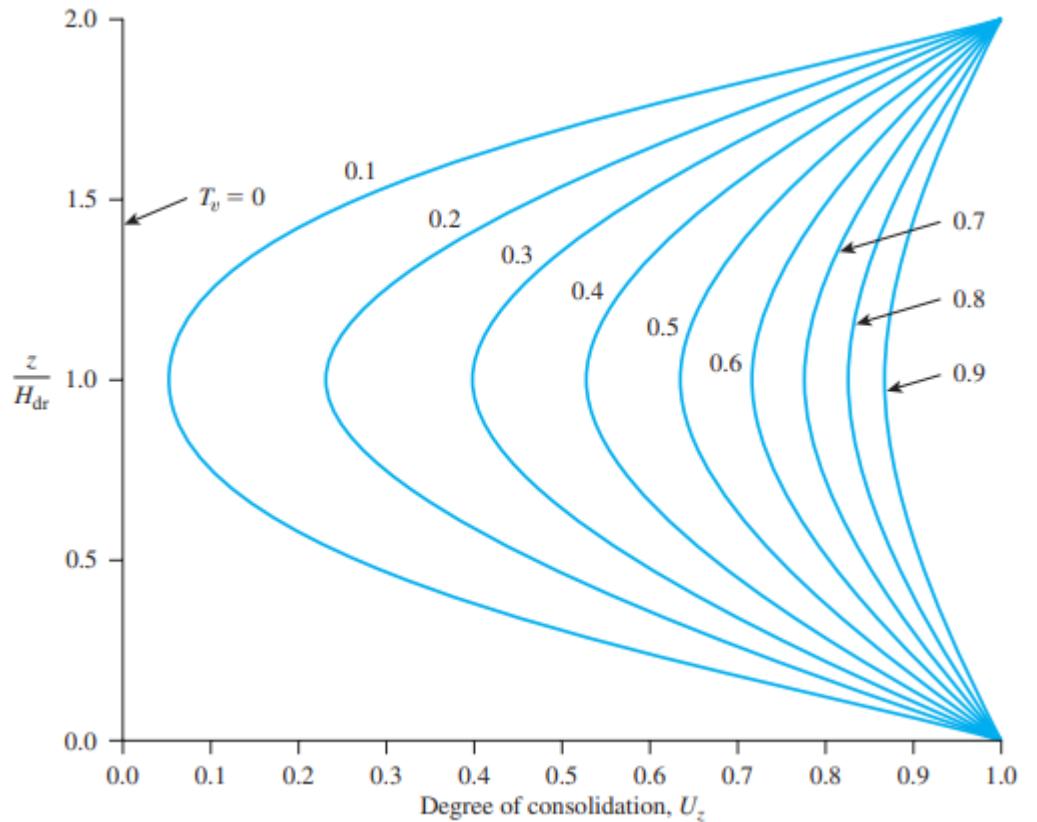


Time Rate of Consolidation

$$U = \frac{S_{c(t)}}{S_c} = 1 - \frac{\left(\frac{1}{2H_{dr}}\right) \int_0^{2H_{dr}} u_z dz}{u_o}$$

- Where U = average degree of consolidation
 $S_{c(t)}$ = settlement of the layer at time t
 S_c = ultimate settlement of the layer from primary consolidation

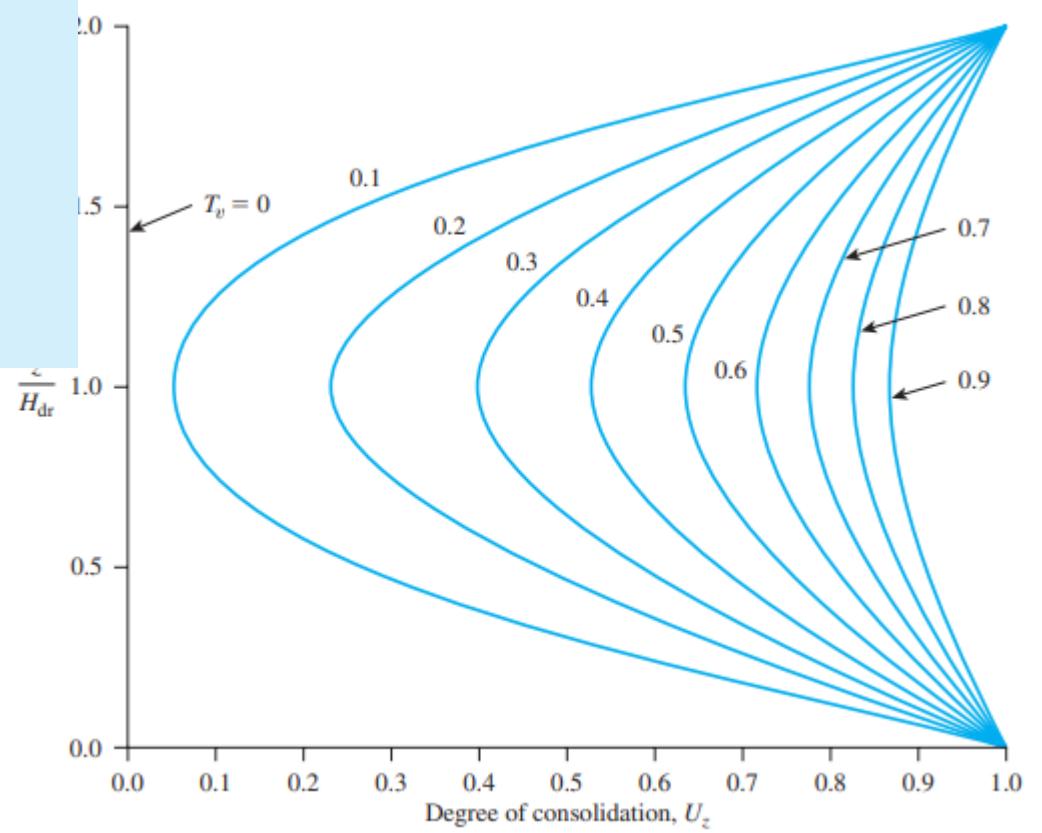
$$U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} e^{-M^2 T_v}$$



Time Rate of Consolidation

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

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



Time Rate of Consolidation

- Variation of average degree of consolidation with time factor, T_v (u_O constant with depth)

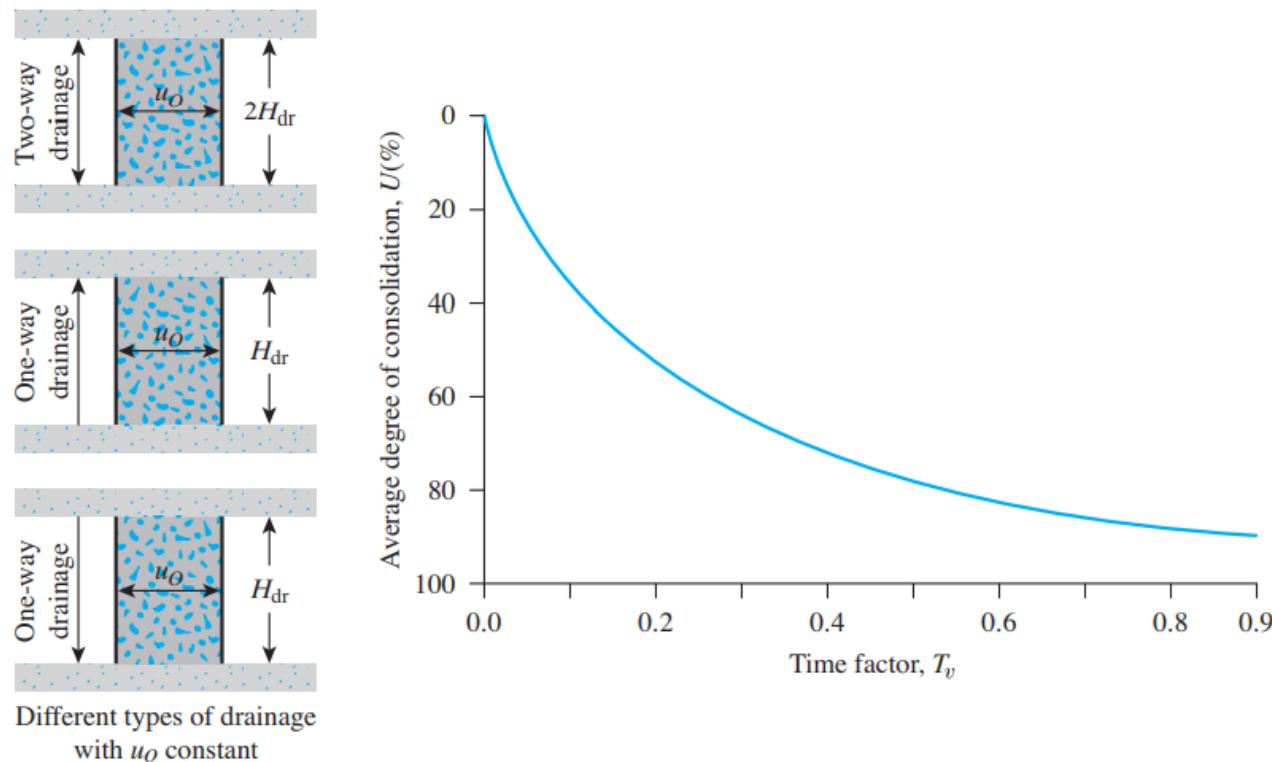


Figure 11.24 Variation of average degree of consolidation with time factor, T_v (u_O constant with depth)

Time Rate of Consolidation

Table 11.8 Variation of T_v with U

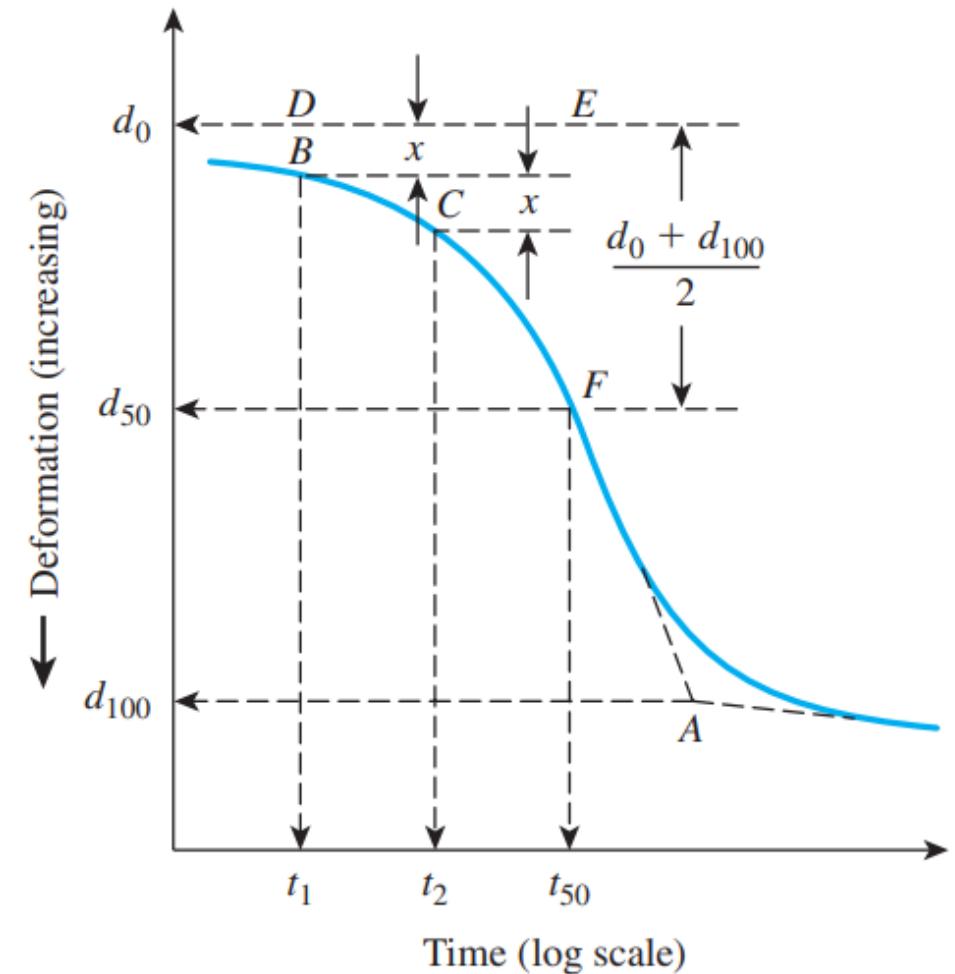
U (%)	T_v	U (%)	T_v	U (%)	T_v	U (%)	T_v
0	0	26	0.0531	52	0.212	78	0.529
1	0.00008	27	0.0572	53	0.221	79	0.547
2	0.0003	28	0.0615	54	0.230	80	0.567
3	0.00071	29	0.0660	55	0.239	81	0.588
4	0.00126	30	0.0707	56	0.248	82	0.610
5	0.00196	31	0.0754	57	0.257	83	0.633
6	0.00283	32	0.0803	58	0.267	84	0.658
7	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
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		

Coefficient of Consolidation

- logarithm-of-time method proposed by Casagrande and Fandum(1940)
- square-root-of-time method suggested by Taylor(1942)

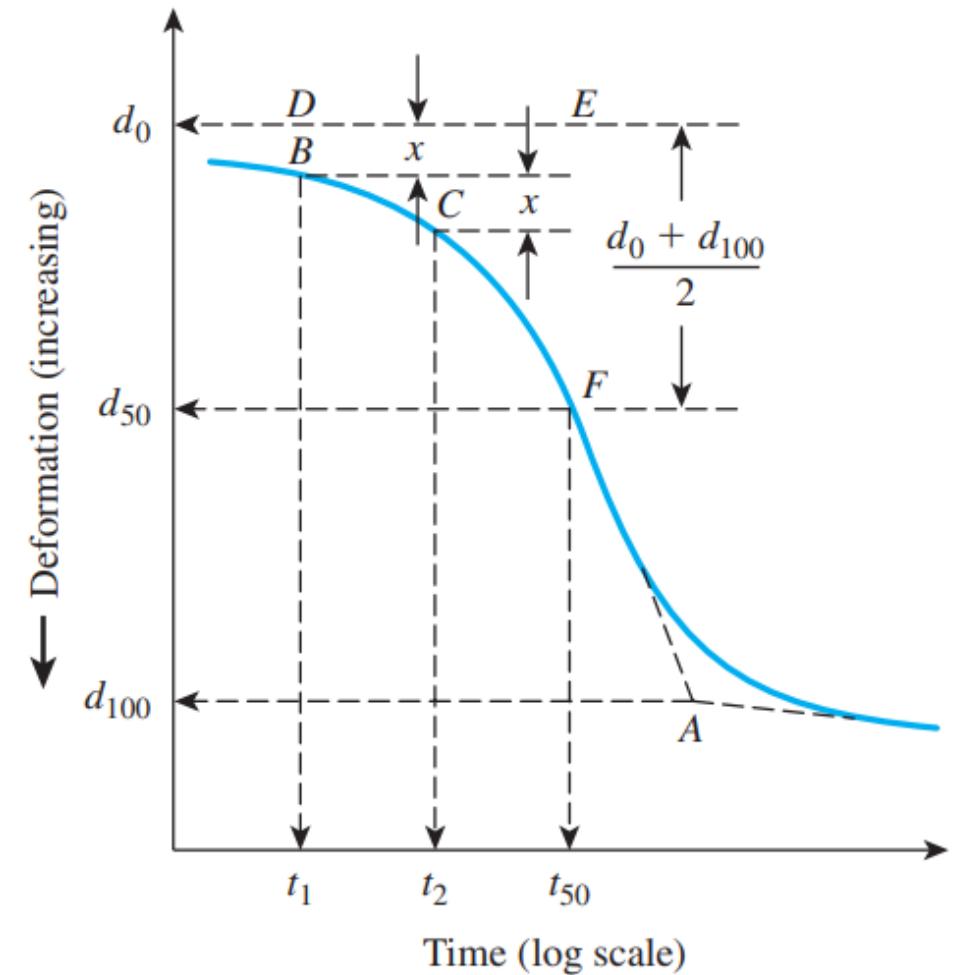
Logarithm-of-time method

- Construction procedure
 1. Extend the straight-line portions of primary and secondary consolidations to intersect to **intersect at A**. The ordinate of **A is represented by d_{100}** - that is, the deformation at the end of **100% primary consolidation**.
 2. The initial curved portion of the plot of deformation versus $\log t$ is approximated to be a parabola on the natural scale. **Select time t_1 and t_2 on the curved portion such that $t_2 = 4t_1$.** Let the difference of sample deformation during time $(t_2 - t_1)$ be equal to x .



Logarithm-of-time method

- Construction procedure
3. Draw a horizontal line DE such that the vertical distance BD is equal to x . The deformation corresponding to the line DE is d_0 (that is deformation at 0% consolidation).
 4. The ordinate of point F on the consolidation curve represents the deformation at 50% primary consolidation, and its abscissa represents the corresponding time(t_{50}).



Logarithm-of-time method

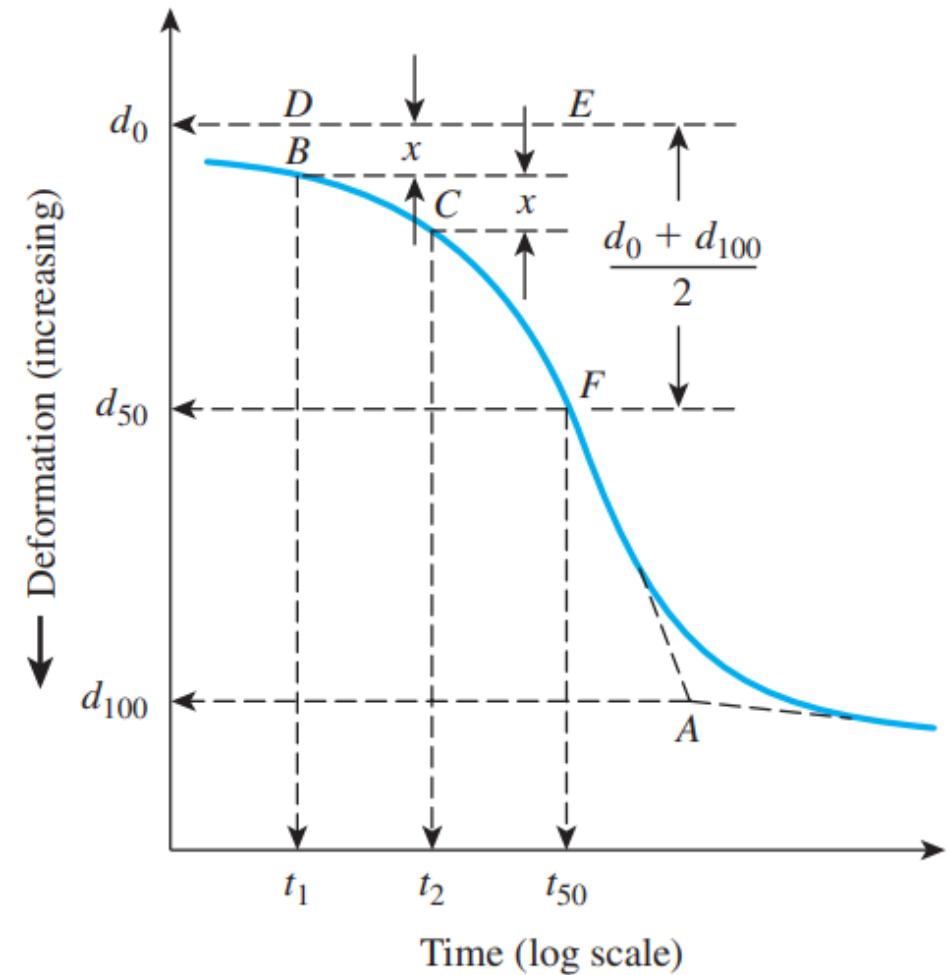
- Construction procedure
- 5. For 50% average degree of consolidation,

$$T_v = 0.197 \text{ or } 0.196$$

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

$$c_v = \frac{0.197 H_{dr}^2}{t_{50}}$$

- For specimens drained **at both top and bottom**, H_{dr} equals one half of the average height of the specimen.
- For specimens drained **on only one side**, H_{dr} equals the average height of the specimen during consolidation.



Square-Root-of-Time Method

- Construction procedure
 1. Draw a line AB through the early portion of the curve.
 2. Draw a line AC through that $\overline{OC} = 1.15\overline{OB}$. The abscissa of point D, which is the intersection of AC and the consolidation curve, gives the square root of time for 90% consolidation $\sqrt{t_{90}}$
 3. For 90% consolidation, $T_{90} = 0.848$, so

$$T_{90} = 0.848 = \frac{c_v t_{90}}{H_{dr}^2}$$

$$c_v = \frac{0.848 H_{dr}^2}{t_{90}}$$

