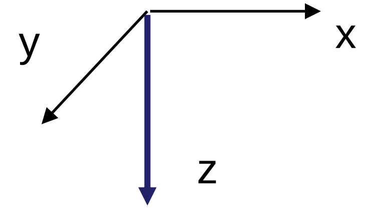
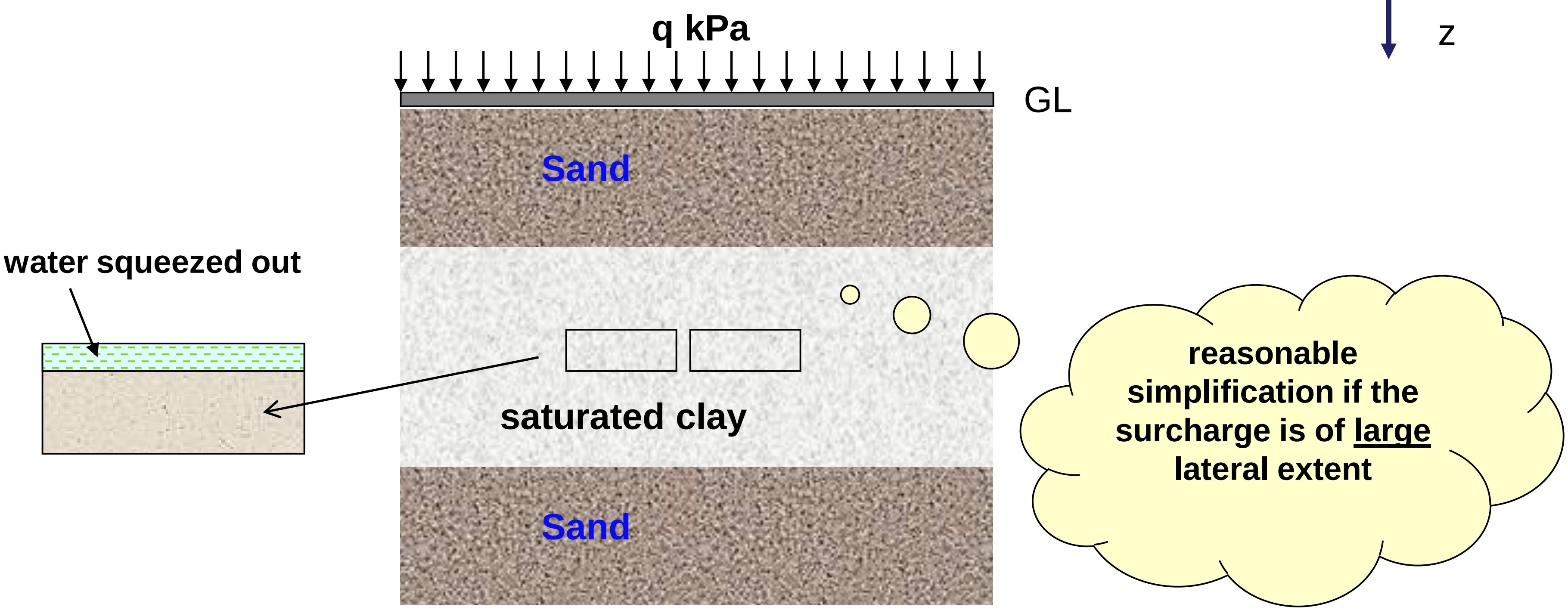
Calculation of 1-D Consolidation Settlement

- A general theory for consolidation, incorporating three-dimensional flow is complicated and only applicable to a very limited range of problems in geotechnical engineering.



- A simplification for solving consolidation problems, drainage and deformations are assumed to be only in the vertical direction.



Calculation of 1-D Consolidation Settlement

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

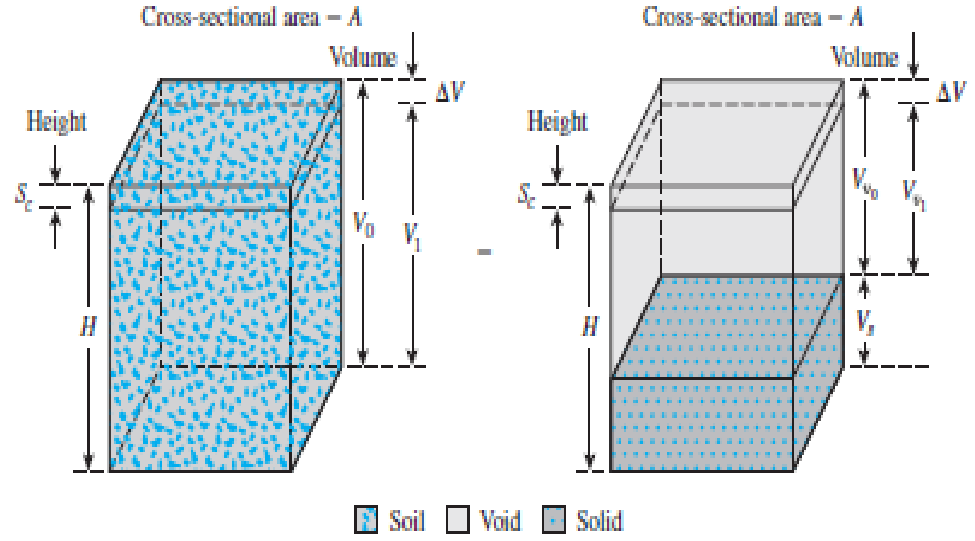
$$\Delta V = S_c A = V_{v_0} - V_{v_1} = \underline{\Delta V_v}$$

$$\Delta V_v = \underline{\Delta e V_s}$$

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

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

$$\boxed{S_c = H \frac{\Delta e}{1 + e_o}}$$



The consolidation settlement can be determined knowing:

- **Initial void ratio e_0.**
- **Thickness of layer H**
- **Change of void ratio Δe**

It only requires the evaluation of Δe

Calculation of 1-D Consolidation Settlement

Settlement Calculation

$$S_c = \Delta H = H_o - H_f = (h_1 - h_2)$$

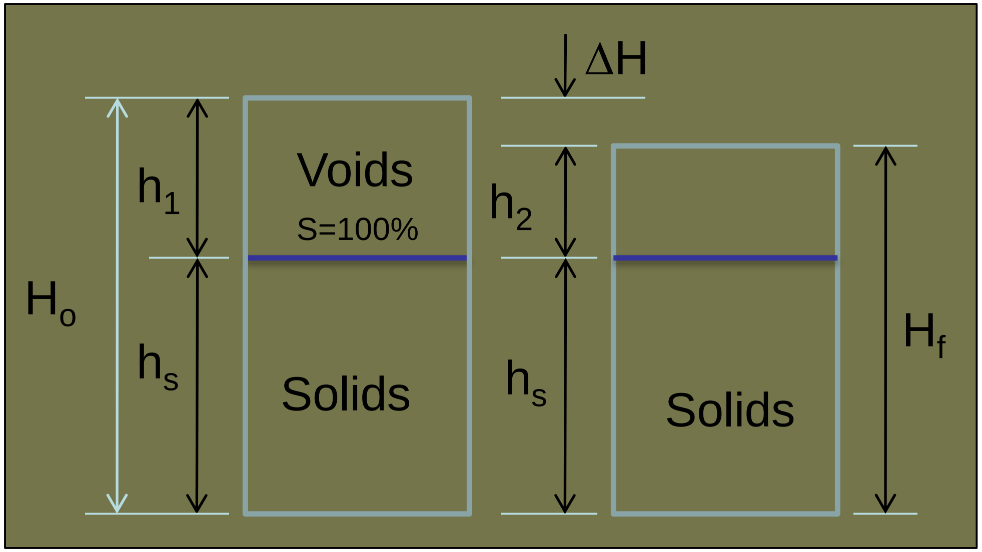
$$S_c = (h_1 - h_2)\frac{H_o}{H_o}$$

$$S_c = (\frac{h_1 - h_2}{h_s + h_1})H$$

$$S_c = (\frac{(h_1 - h_2)/h_s}{(h_s + h_1)/h_s})H$$

$$S_c = (\frac{e_o - e_f}{1 + e_o})H$$

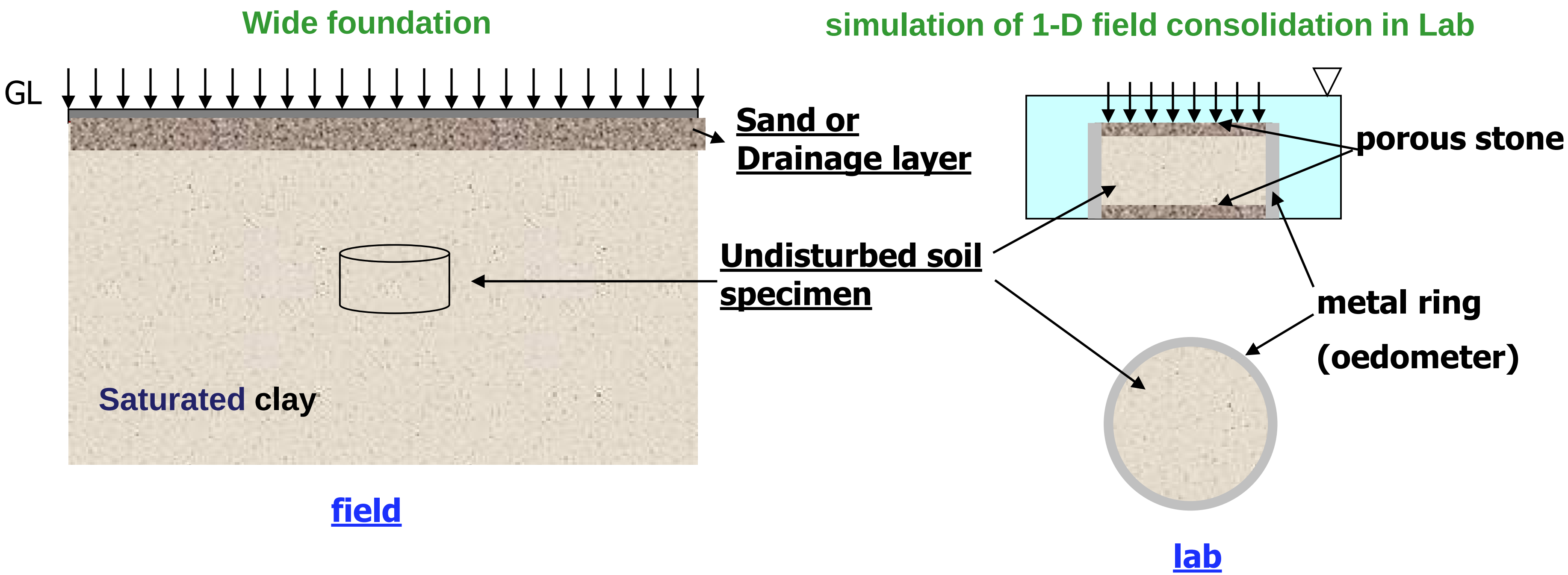
$$S_c = \frac{\Delta e}{1 + e_o}H$$



It only requires the evaluation of Δe

One-dimensional Laboratory Consolidation Test

- **1-D field consolidation can be simulated in laboratory.**
- **Data obtained from laboratory testing can be used to predict magnitude of consolidation settlement reasonably, but rate is often poorly estimated.**

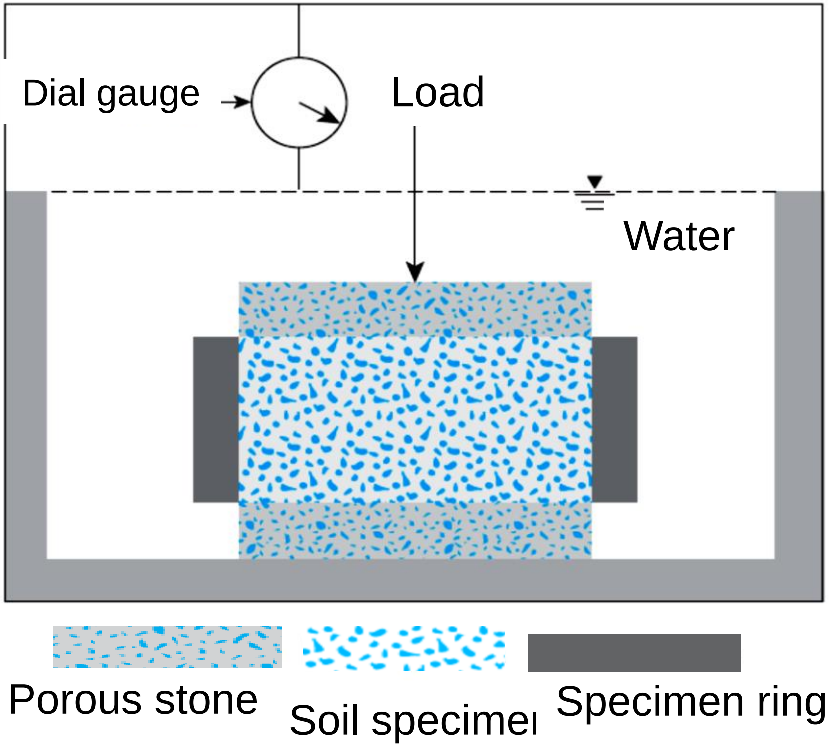


One-dimensional Laboratory Consolidation Test

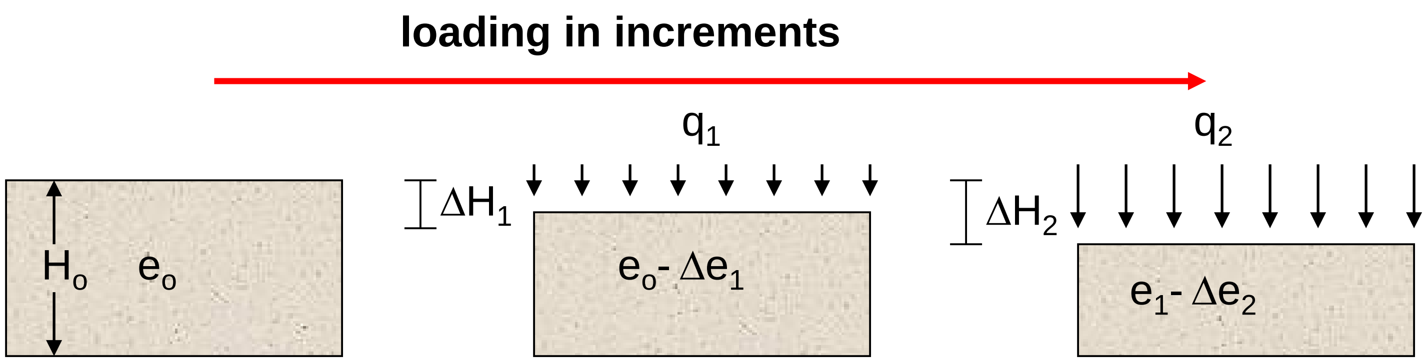
- The one-dimensional consolidation test was first suggested by Terzaghi. It is performed in a consolidometer (sometimes referred to as oedometer). The schematic diagram of a consolidometer is shown below.

- The complete procedures and discussion of the test was presented in CE 380.

Consolidometer or Oedometer



Incremental loading



Load increment ratio (LIR) = $\Delta q/q = 1$ (i.e., double the load)

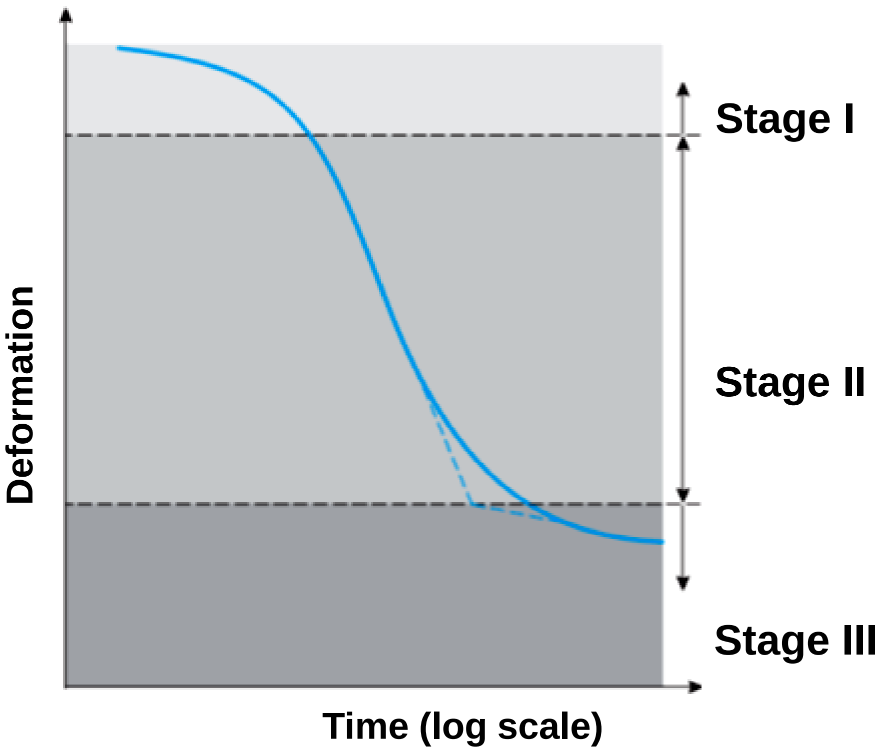
- **Allow full consolidation before next increment (24 hours)**
- **Record compression during and at the end of each increment using dial gauge.**
- **Example of time sequence: (10 sec, 30 sec, 1 min, 2, 4, 8, 15, 30, 1 hr, 2, 4, 8, 16, 24)**
- **The procedure is repeated for additional doublings of applied pressure until the applied pressure is in excess of the total stress to which the clay layer is believed to be subjected to when the proposed structure is built.**
- **The total pressure includes effective overburden pressure and net additional pressure due to the structure.**
- **Example of load sequence (25, 50, 100, 200, 400, 800, 1600, … kPa)**

Presentation of results

- The results of the consolidation tests can be summarized in the following plots:
- Rate of consolidation curves (dial reading vs. log time or dial reading vs. square root time)
- Void ratio-pressure plots (Consolidation curve)

  $e - \sigma_{v'}$ plot or $e - \log \sigma_{v'}$ plot

- The plot of deformation of the specimen against time for a given load increment can observe three distinct stages:



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

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

Stage III: Secondary consolidation, which occurs after complete dissipation of the excess pore water pressure, caused by plastic readjustment of soil fabric.

Presentation of results

After plotting the time-deformation for various loadings are obtained, it is necessary to study the change in the void ratio of the specimen with pressure. See section 11.6 for step-by-step procedure for doing so.

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

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

$$H_v = H - H_s$$

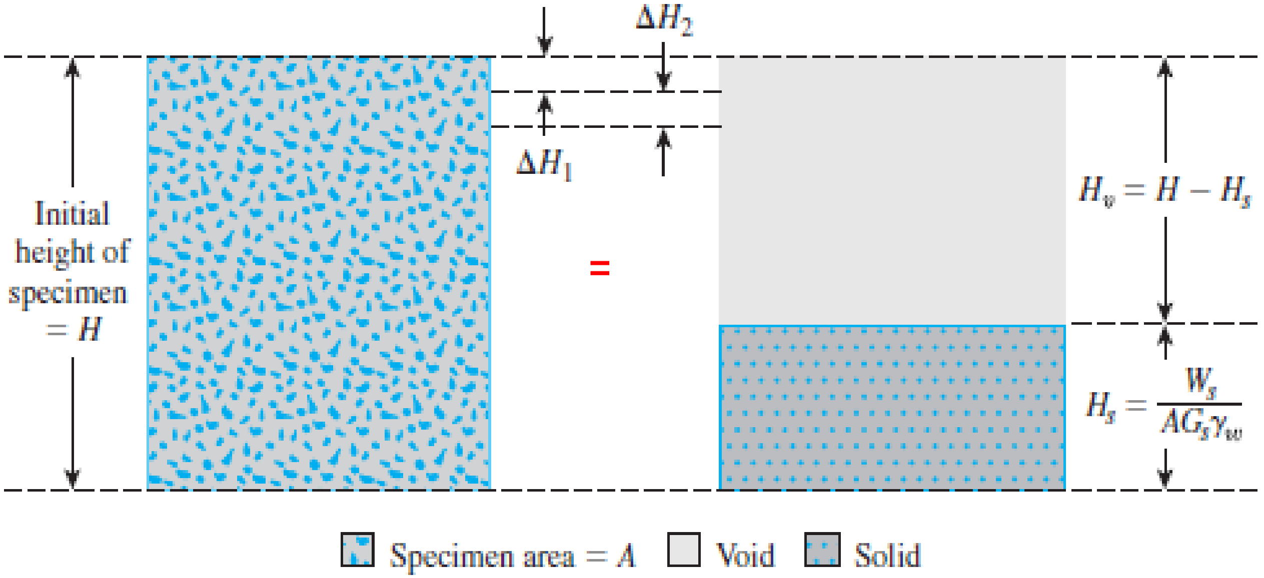


Figure 11.9 Change of height of specimen in one-dimensional consolidation test

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

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

$$e_1 = e_o - \Delta e_1$$

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

Proceeding in a similar manner, one can obtain the void ratios at the end of the consolidation for all load increments. See Example 11.2.

EXAMPLE 11.3

Example 11.3

Following are the results of a laboratory consolidation test on a soil specimen obtained from the field: Dry mass of specimen = 128 g, height of specimen at the beginning of the test = 2.54 cm, $G_s = 2.75$, and area of the specimen = 30.68 cm^2.

Effective pressure, σ' (ton/ft^2)	Final height of specimen at the end of consolidation (cm)
0	2.540
0.5	2.488
1	2.465
2	2.431
4	2.389
8	2.324
16	2.225
32	2.115

Make necessary calculations and draw an e versus log σ' curve.

Solution

From Eq. (11.20),

$$H_s = \frac{W_s}{AG_s\gamma_w} = \frac{M_s}{AG_s\rho_w} = \frac{128 \text{ g}}{(30.68 \text{ cm}^2)(2.75)(1 \text{g/cm}^3)} = 1.52 \text{ cm}$$

Now the following table can be prepared.

Effective pressure, σ' (ton/ft^2)	Height at the end of consolidation, H (cm)	$H_v = H - H_s$ (cm)	$e = H_v/H_s$
0	2.540	1.02	0.671
0.5	2.488	0.968	0.637
1	2.465	0.945	0.622
2	2.431	0.911	0.599
4	2.389	0.869	0.572
8	2.324	0.804	0.529
16	2.225	0.705	0.464
32	2.115	0.595	0.390

The e versus log σ' plot is shown in Figure 11.15.

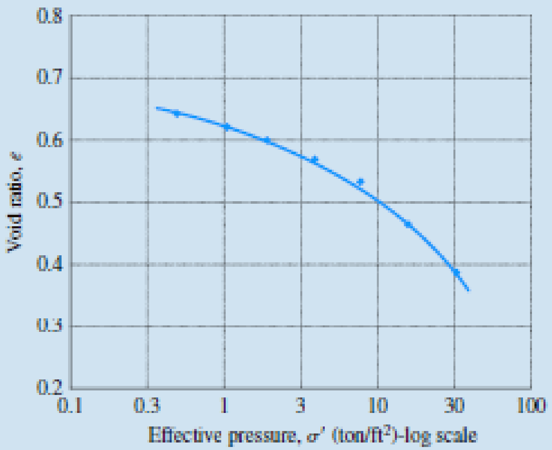
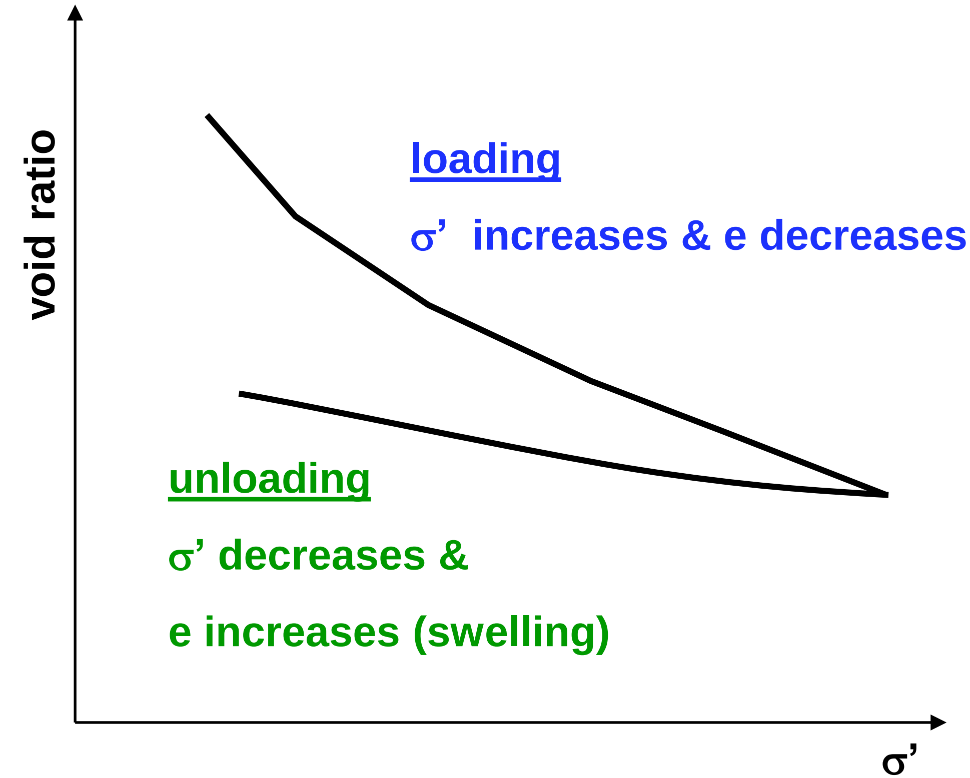


Figure 11.15 Variation of void ratio with effective pressure

Presentation of results

e – σ' plot



- **The figure above is usually termed the compressibility curve__, where compressibility is the term applied to 1-D volume change that occurs in cohesive soils that are subjected to compressive loading.**

- **Note: It is more convenient to express the stress-stain relationship for soil in consolidation studies in terms of void ratio and unit pressure instead of unit strain and stress used in the case of most other engineering materials.**

Coefficient of Volume Compressibility [m_v]

- m_v is defined as the volume change per unit volume per unit increase in effective stress

~ denoted by m_v

~ is the volumetric strain in a clay element per unit increase in stress

$$= \frac{\text{change in volume}}{\text{original volume}}$$

i.e.,

$$m_v = \frac{\Delta V / V}{\Delta \sigma}$$

no units

$\Delta V / V = \frac{\Delta e}{1 + e_o}$

m_v: kPa^{-1} or MPa^{-1}

$\Delta \sigma$: kPa or MPa

$$m_v = \frac{\Delta e}{\Delta \sigma' (1 + e_o)}$$

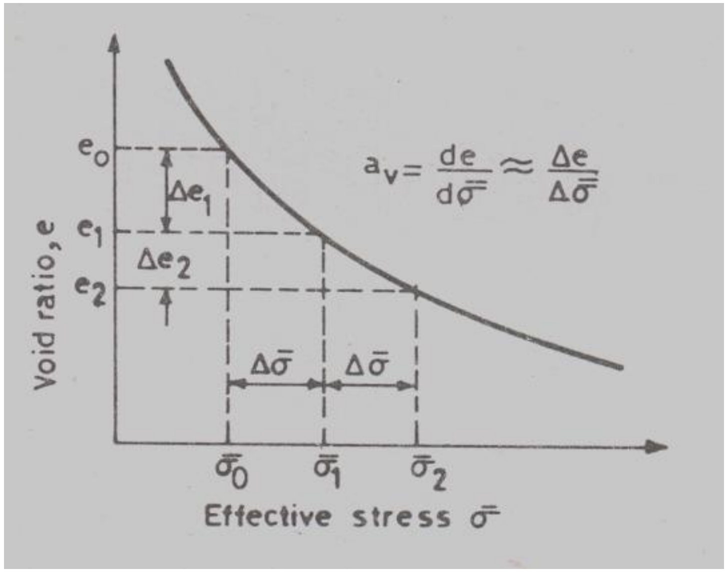
- m_v is also known as **Coefficient of Volume Change**.
- The value of m_v for a particular soil is not constant but depends on the stress range over which it is calculated.

Coefficient of Compressibility a_v

- a_v is the slope of e-σ'plot, or $a_v = -de/d\sigma'$ (m^2/kN)
- Within a narrow range of pressures, there is a linear relationship between the decrease of the voids ratio *e* and the increase in the pressure (stress). Mathematically,

$$a_v = \frac{\Delta e}{\Delta \sigma'}$$

$$m_v = \frac{a_v}{1+e_o}$$

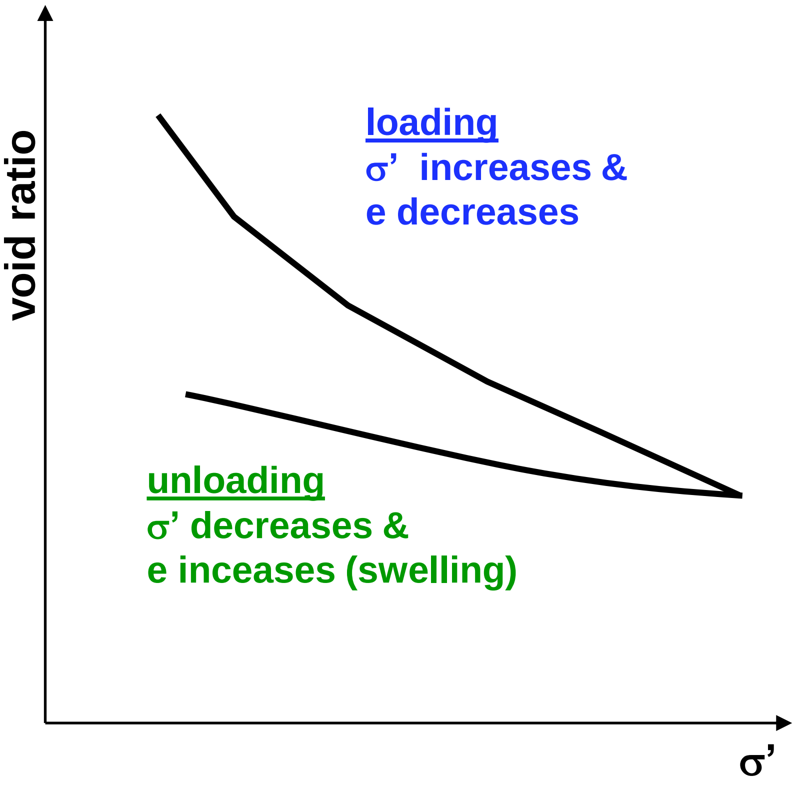


- a_v decreases with increases in effective stress

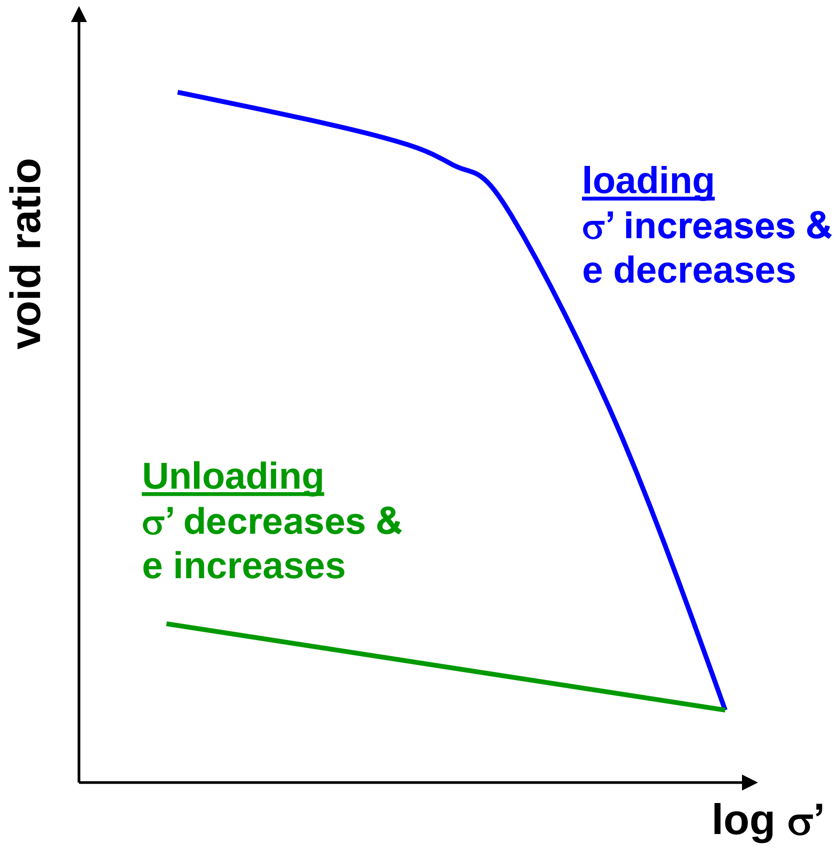
- Because the slope of the curve e-σ' is constantly changing, it is somewhat difficult to use a_v in a mathematical analysis, as is desired in order to make settlement calculations.

Presentation of results

e – σ' plot

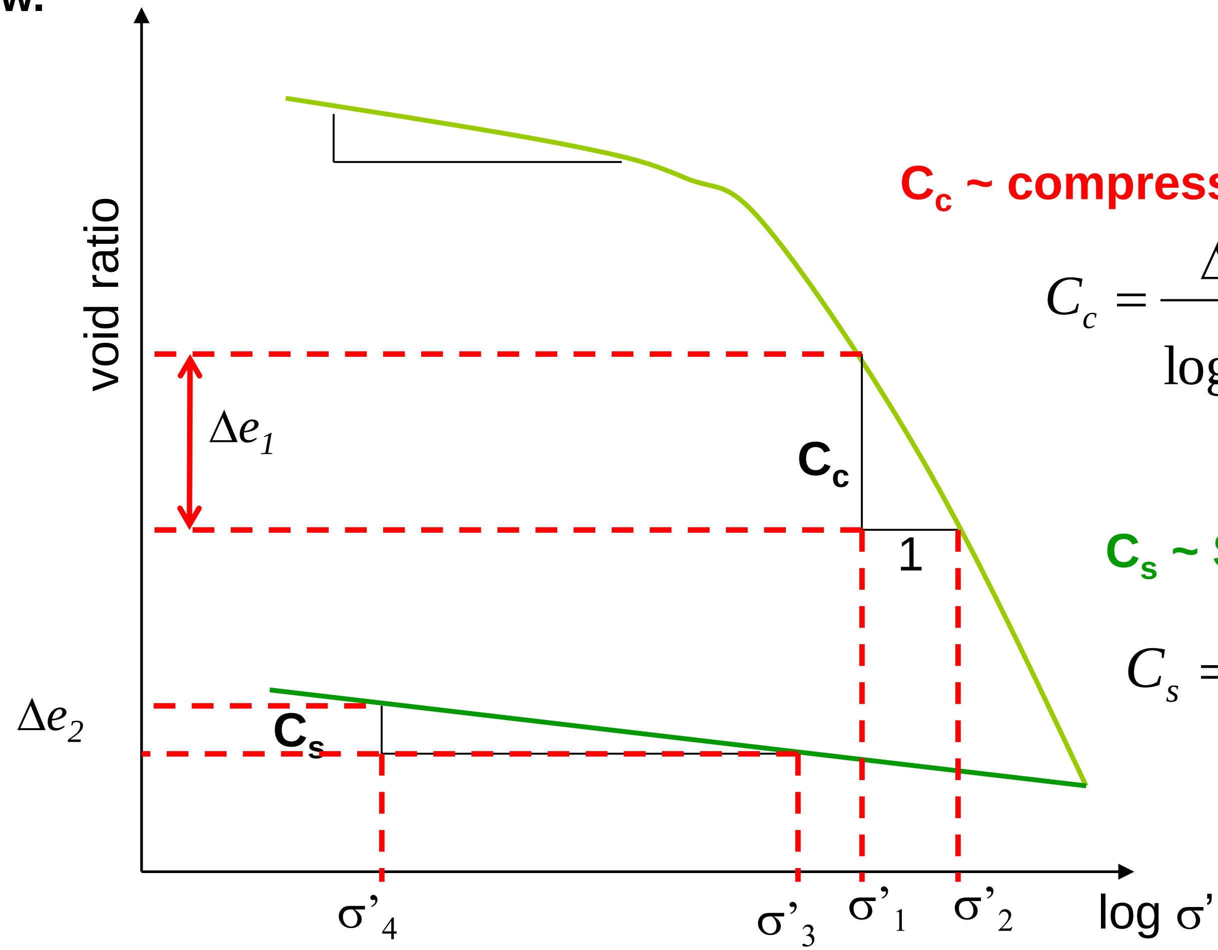


e – log σ' plot



Compression and Swell Indices

As we said earlier, the main limitation of using a_v and m_v in describing soil compressibility is that they are not constant. To overcome this shortcoming the relationship between e and σ_v' is usually plotted in a semi logarithmic plot as shown below.



C_c ~ compression index

$$C_c = \frac{\Delta e_1}{\log \frac{\sigma_2'}{\sigma_1'}}$$

C_s ~ Swell index

$$C_s = \frac{\Delta e_2}{\log \frac{\sigma_3'}{\sigma_4'}}$$

Correlations for compression index, c_c

- **This index is best determined by the laboratory test results for void ratio, e, and pressure σ' (as shown above).**

- **Because conducting compression (consolidation) test is relatively time consuming (usually 2 weeks), C_c is usually related to other index properties like:**

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

Empirical expressions for c_c & c_s

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

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

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

$$C_s \simeq \tfrac{1}{5} \text{ to } \tfrac{1}{10} C_c$$

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

G_S: Specific Gravity
e_0 : in situ void ratio
PI: Plasticity Index
LL: Liquid Limit

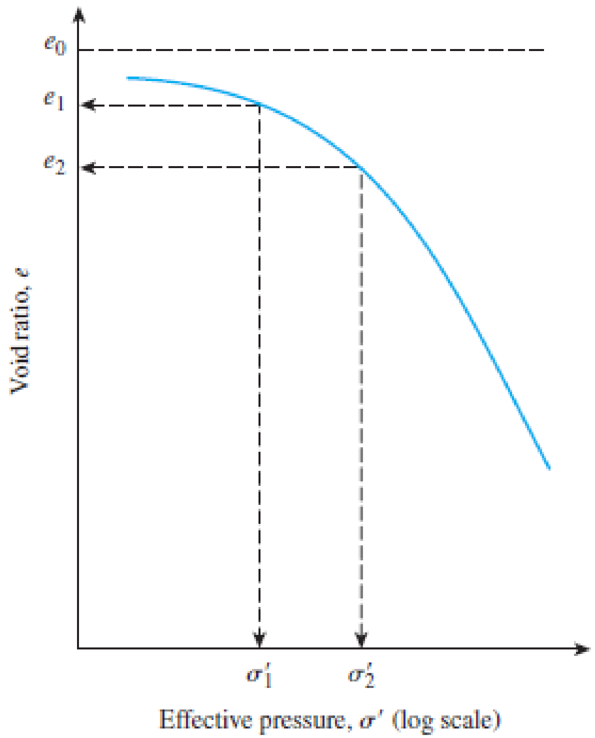
- **Compression and Swell Indices of some Natural Soils**

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

Normally consolidated and overconsolidated clays

The upper part of the e – log σ' plot is as shown below somewhat curved with a flat slope, followed by a linear relationship having a steeper slope.

This can be explained as follows:



- **A soil in the field at some depth has been subjected to a certain maximum effective past pressure in its geologic history.**

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

- **The reduction of effective pressure may be due to natural geological processes or human processes.**

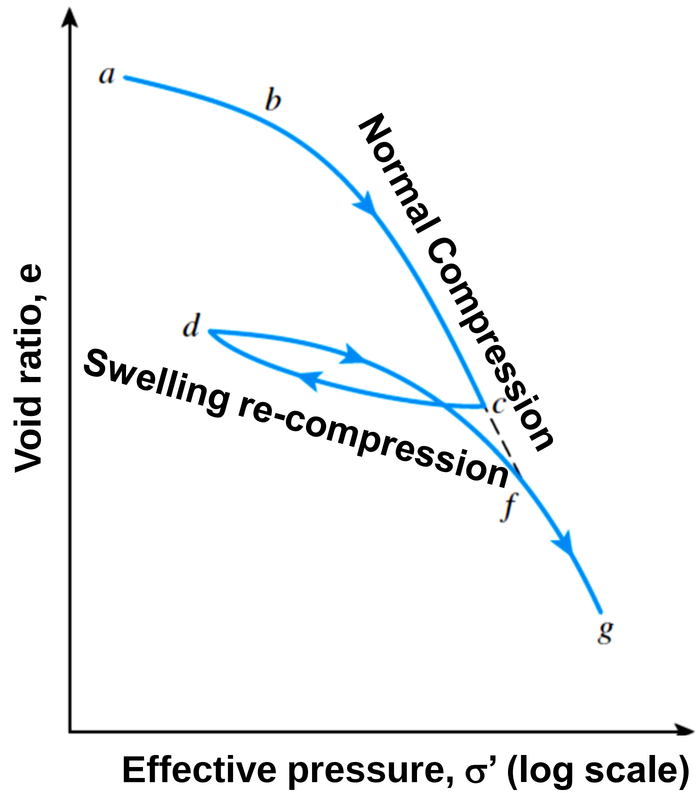
- **During the soil sampling, the existing effective overburden pressure is also released, which results in some expansion.**

Normally consolidated and overconsolidated clays

- The soil will show relatively small decrease of e with load up until the point of the maximum effective stress to which the soil was subjected to in the past.

(Note: this could be the overburden pressure if the soil has not been subjected to any external load other than the weight of soil above that point concerned).

- This can be verified in the laboratory by loading, unloading and reloading a soil sample as shown across.



Normally consolidated and overconsolidated clays

- **Normally Consolidated Clay (N.C. Clay)**

A soil is NC if the present effective pressure to which it is subjected is the maximum pressure the soil has ever been subjected to.

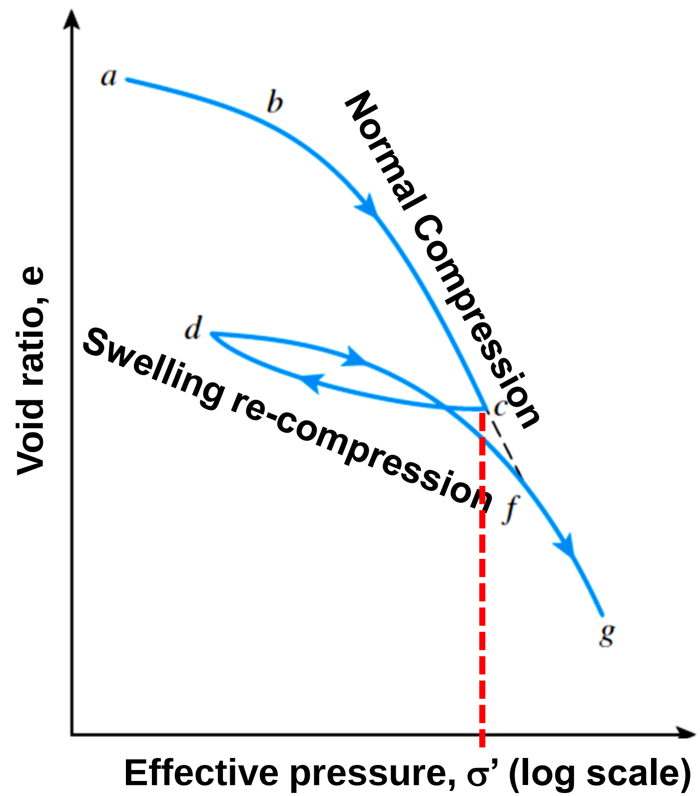
The branches bc and fg are NC state of a soil.

- **Over Consolidated Clays (O.C. Clay)**

A soil is OC if the present effective pressure to which it is subjected to is less than the maximum pressure to which the soil was subjected to in the past

The branches ab, cd, df, are the OC state of a soil.

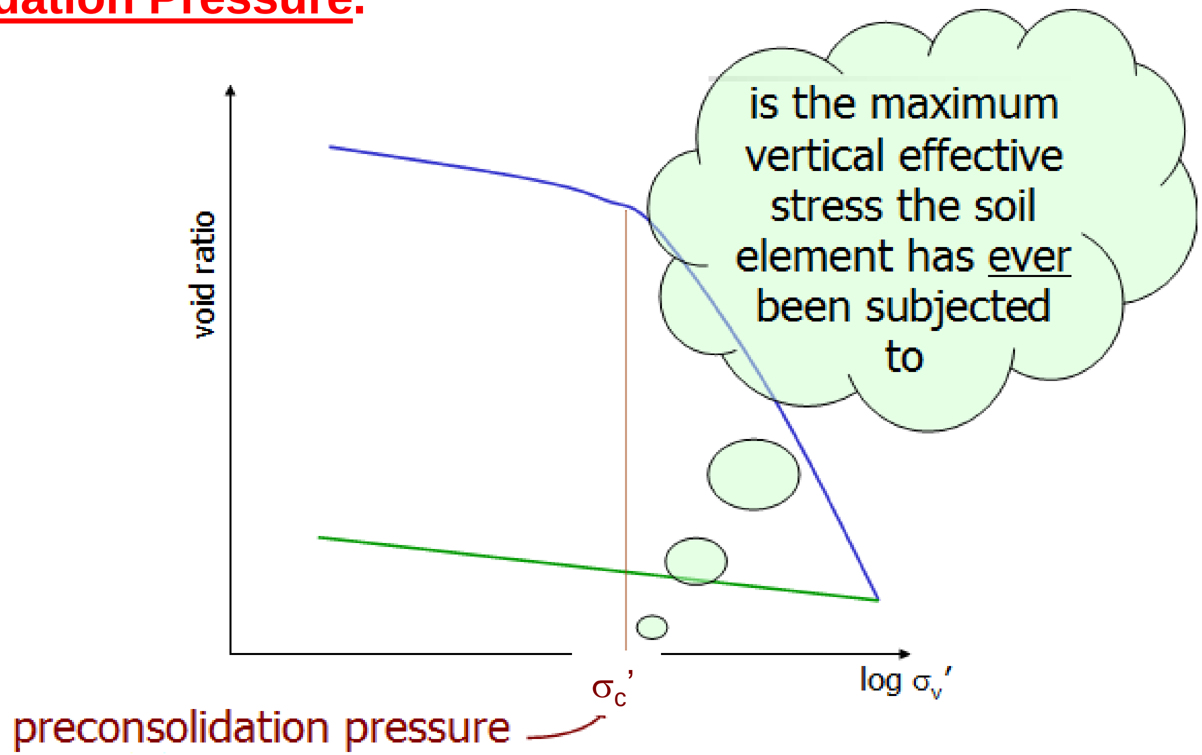
The maximum effective past pressure is called the preconsolidation pressure.



Preconsolidation pressure

- The stress at which the transition or "break" occurs in the curve of e vs. log σ' is an indication of the maximum vertical overburden stress that a particular soil sample has sustained in the past.

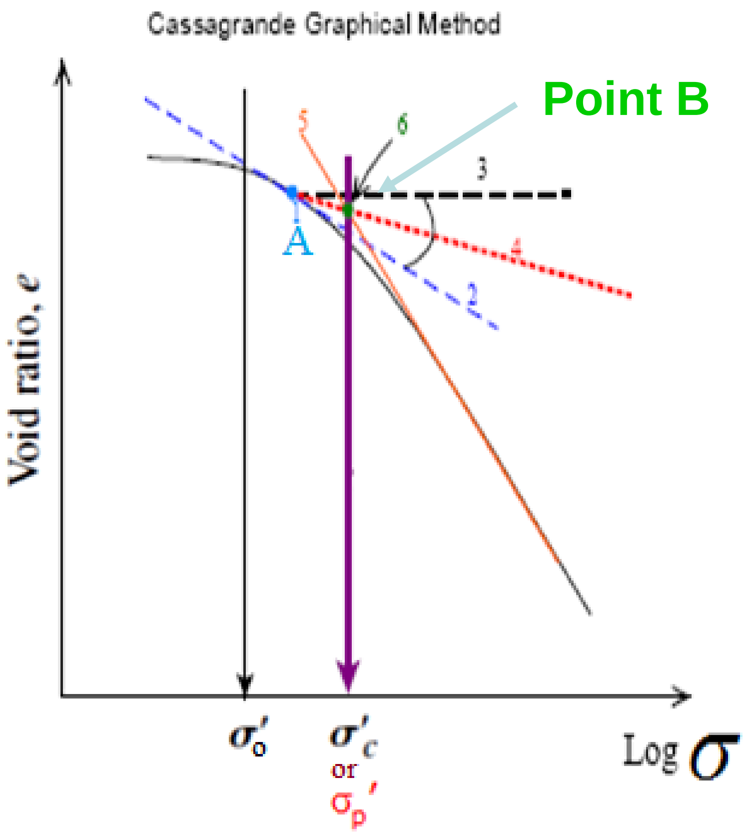
- This stress is very important in geotechnical engineering and is known as Preconsolidation Pressure.



Casagrande procedure of determination preconsolidation stress σ_c'

Casagrande (1936) suggested a simple graphic construction to determine the preconsolidation pressure σ'_c from the laboratory e –log σ' plot.

1. Choose by eye the point of minimum radius (or maximum curvature) on the consolidation curve (point A).
2. Draw a line tangent to the curve at point A
3. Draw a horizontal line from point A.
4. Bisect the angle made by steps 2 and 3.
5. Extend the straight line portion of the virgin compression curve up to where it meets the bisector line obtained in step4.
6. The point of intersection of these two lines is the preconsolidation stress (point B)

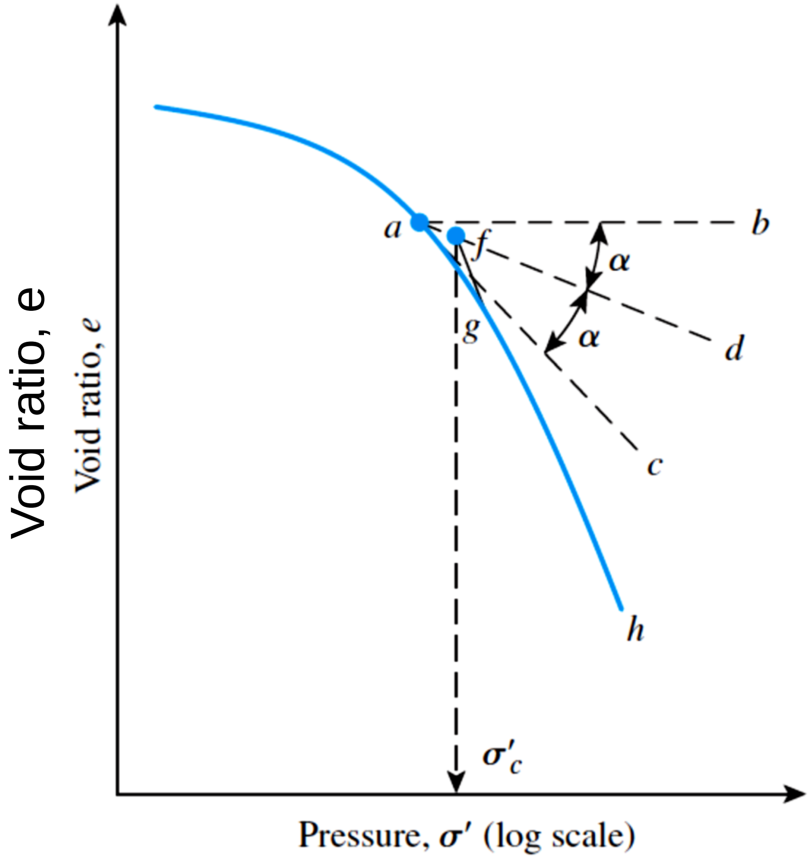


Overconsolidation ratio (OCR)

- In general the **overconsolidation ratio (OCR)** for a soil can be defined as:

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

where σ' is the present effective vertical pressure.



- **From the definition of NC soils, they always have OCR=1.**

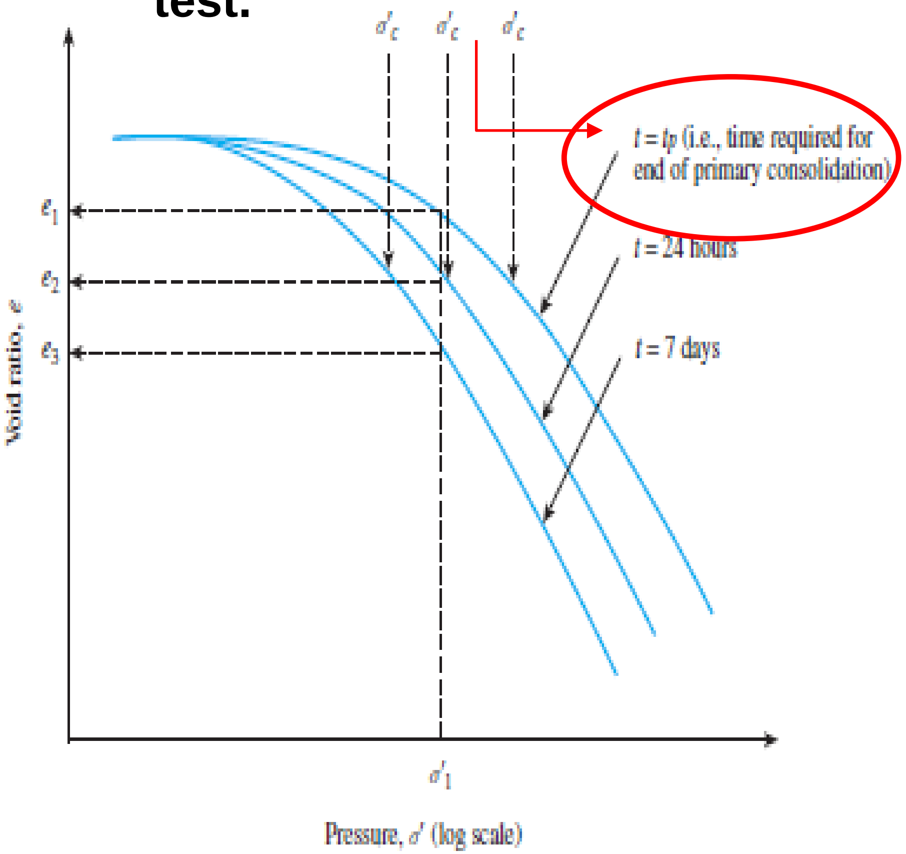
- **To calculate OCR the preconsolidation pressure σ_c' should be known from the consolidation test and σ' is the effective stress in the field.**

Factors affecting the determination of σ_c'

1. Duration of load increment

- When the duration of load maintained on a sample is increased the e vs. log σ' gradually moves to the left.
- The reason for this is that as time increased the amount of secondary consolidation of the sample is also increased. This will tend to reduce the void ratio e.
- The value of σ_c' will increase with the decrease of t.

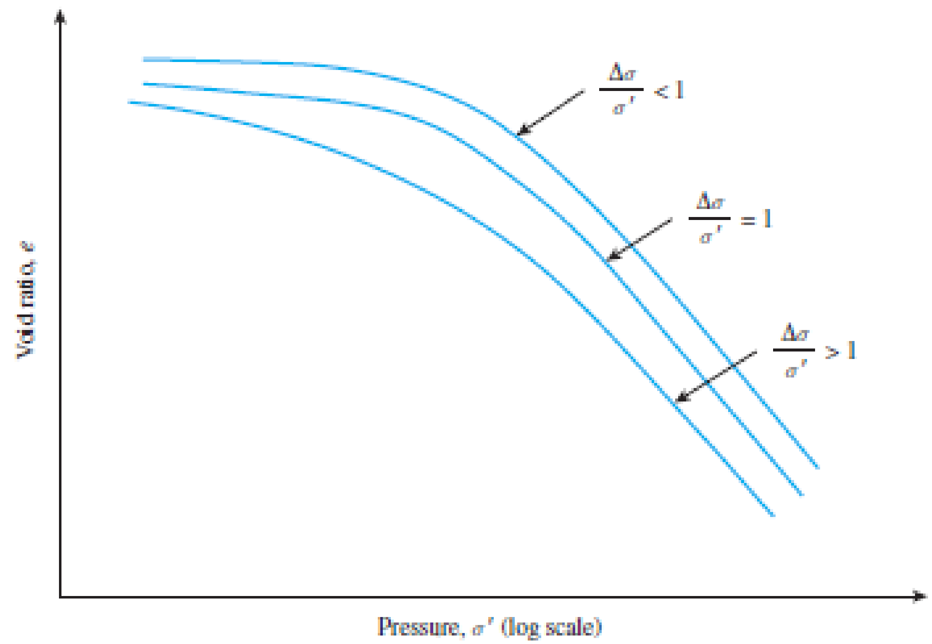
t_p is to be known from either plotting of deformation vs. time or excess p.w.p. if it is being monitored during the test.



Factors affecting the determination of σ_c'

2. Load Increment Ratio (LIR)

- **LIR is defined as the change in pressure of the pressure increment divided by the initial pressure before the load is applied.**
- **LIR =1, means the load is doubled each time, this results in evenly spaced data points on e vs. log σ' curve**
- **When LIR is gradually increased, the e vs. log σ' curve gradually moves to the left.**



Field Compression Curve

- **Due to soil disturbance, even with high-quality sampling and testing the actual compression curve has a SLOPE which is somewhat LESS than the slope of the field VIRGIN COMPRESSION CURVE. The "break" in the curve becomes less sharp with increasing disturbance.**

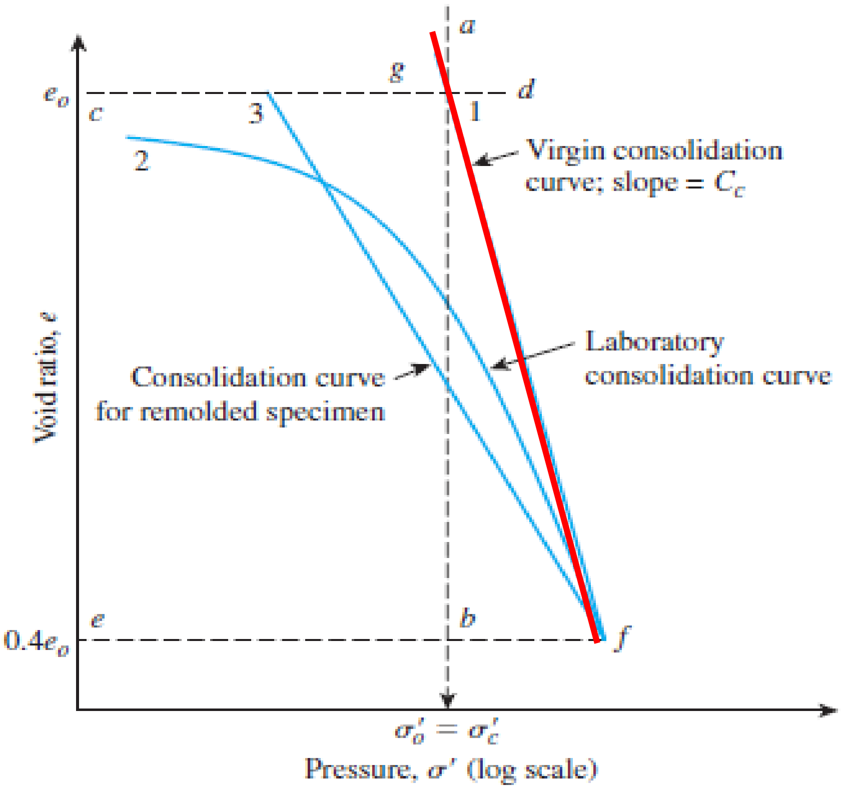
Sources of disturbance:

- **Sampling**
- **Transportation**
- **Storage**
- **Preparation of the specimen (like trimming)**

Normally consolidated and overconsolidated clays

- **We know the present effective overburden σ'_0 and void ratio e_0.**
- **We should know from the beginning whether the soil is NC or OC by comparing σ'_0 and σ'_C . $\sigma'_0 = \gamma z$, σ'_C we find it through the procedures presented in a previous slides.**

Graphical procedures to evaluate the slope of the field compression curve



- **Determine from Curve 2 (Laboratory test) the preconsolidation pressure $\sigma'_C = \sigma'_o$**
- **Draw a vertical line *ab***
- **Calculate the void ratio in the field e_o**

$$e_o = \frac{V_v}{V_s} = \frac{H_v}{H_s}\frac{A}{A} = \frac{H_v}{H_s} \qquad H_s = \frac{W_s}{AG_s\gamma_w} \qquad H_v = H - H_s$$

- **Draw a horizontal line *cd***
- **Calculate $0.4e_o$ Draw a horizontal line *ef***
- **Join Points *f* and *g***

This is the virgin compression curve

Normally consolidated clays

Graphical procedures to evaluate the slope of the field compression curve

- **Determine from Curve 2 (Laboratory test) the preconsolidation pressure σ'_C**
- **Draw a vertical line *ab***
- **Determine the field effective overburden pressure σ'_o Draw a vertical line *cd***
- **Calculate the void ratio in the field e_o**

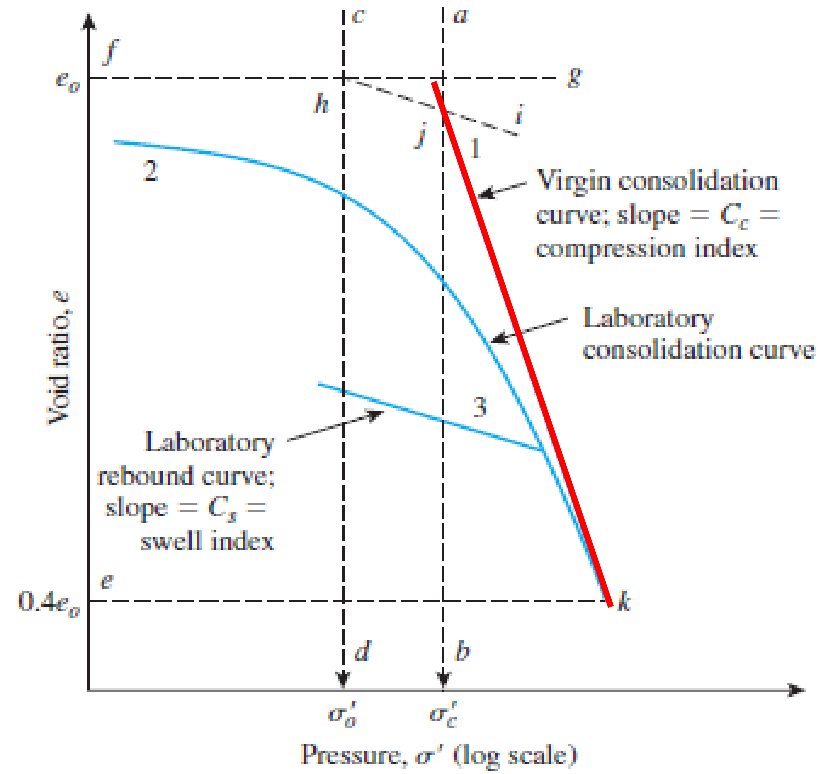
$$e_o = \frac{V_v}{V_s} = \frac{H_v A}{H_s A} = \frac{H_v}{H_s}$$

$$H_s = \frac{W_s}{A G_s \gamma_w}$$

$$H_v = H - H_s$$

- **Draw a horizontal line *fg***
- **Calculate $0.4e_o$ Draw a horizontal line *ek***
- **Draw a line *hi* parallel to curve 3**
- **Join Points *k* and *j***

This is the virgin compression curve



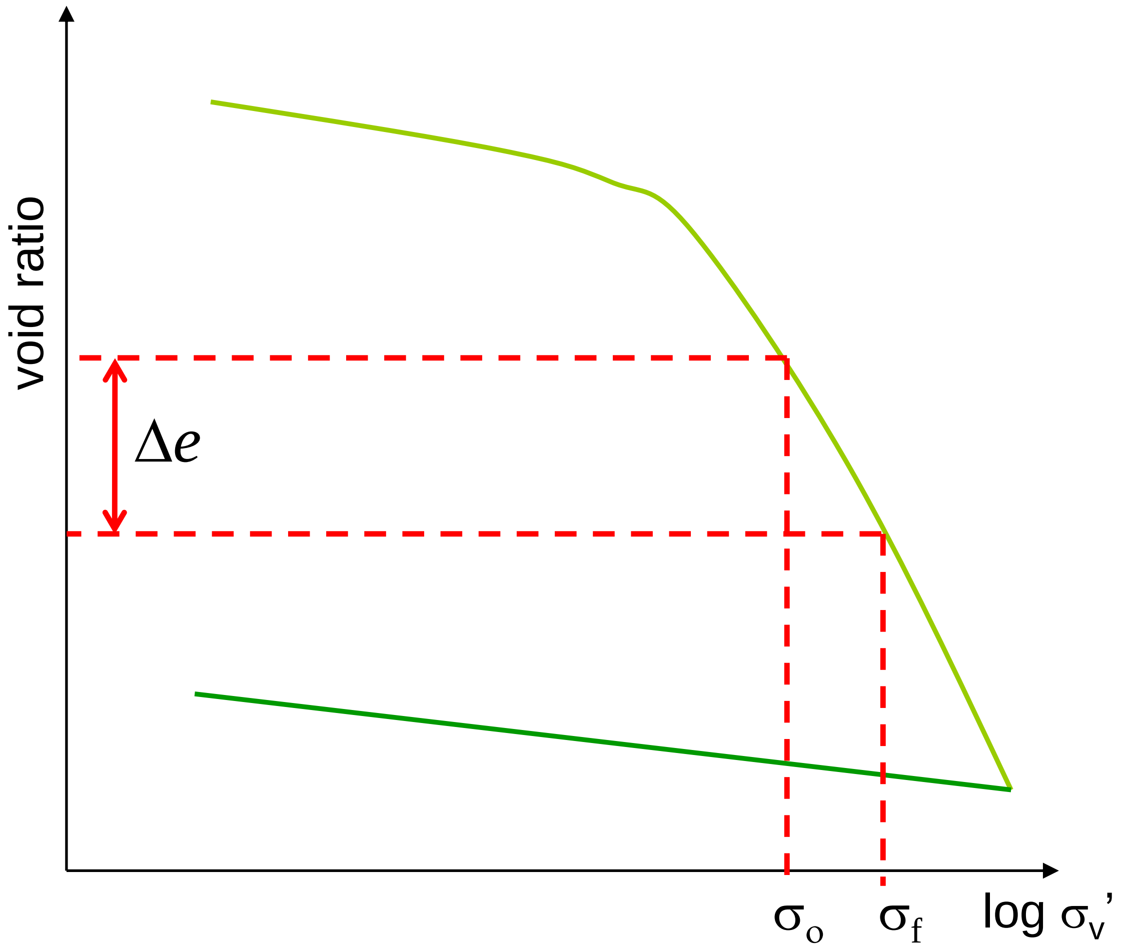
Overconsolidated clays

Calculation of Primary Consolidation Settlement

I) Using e - log σ_v

If the e-log σ' curve is given, Δe can simply be picked off the plot for the appropriate range and pressures.

$$S_c = \frac{\Delta e}{1+e_o} H$$



Calculation of Primary Consolidation Settlement

II) Using m_v

$$S_C = m_v . H . \Delta\sigma$$

$$m_v = \frac{\Delta e}{\Delta\sigma(1+e_0)}$$

Disadvantage

m_v is obtained from e vs. $\Delta\sigma$ which is nonlinear and m_v is stress level dependent. This is on contrast to C_c which is constant for a wide range of stress level.

Calculation of Primary Consolidation Settlement

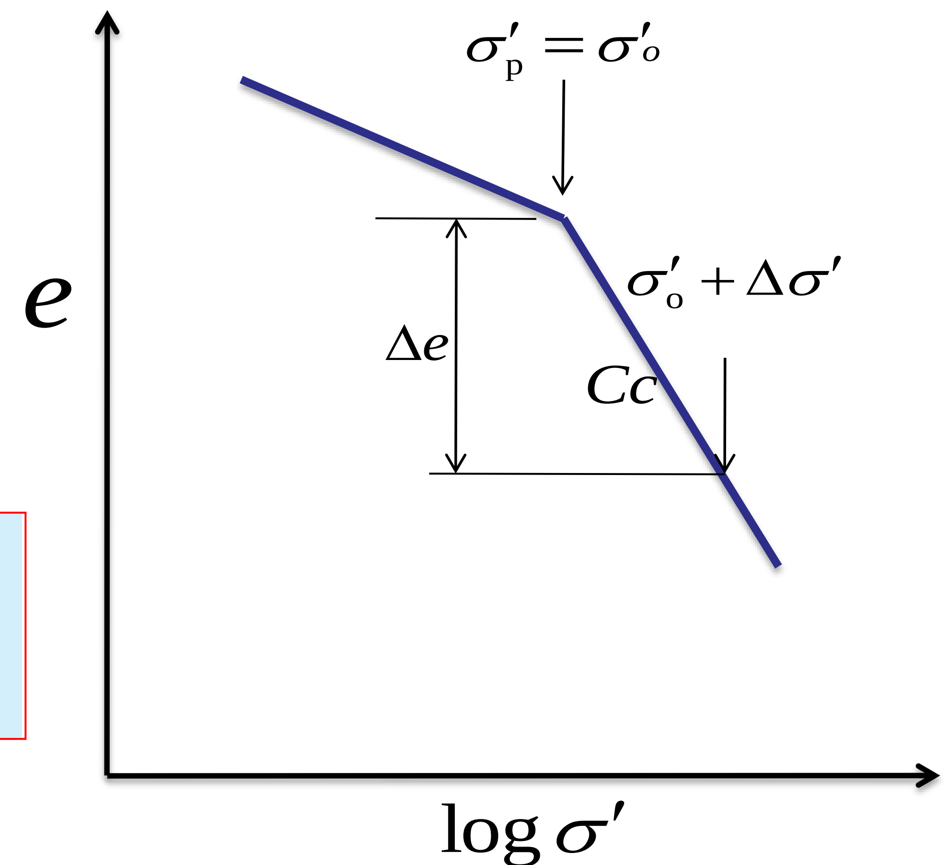
III) Using Compression and Swelling Indices

a) Normally Consolidated Clay ($\sigma'_0 = \sigma_c'$)

$$S_c = \frac{\Delta e}{1+e_o} H$$

$$\Delta e = C_c \log\left(\frac{\sigma'_o + \Delta\sigma}{\sigma'_o}\right)$$

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



Calculation of Primary Consolidation Settlement

b) Overconsolidated Clays

$$S_c = \frac{\Delta e}{1+e_o} H$$

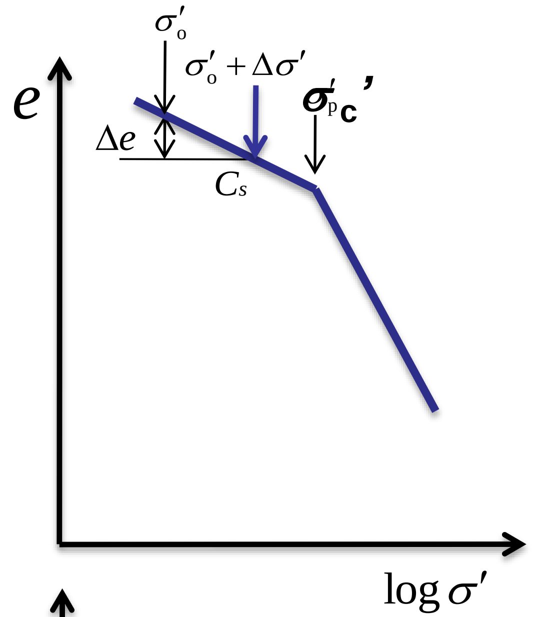
Case I: $\sigma'_0 + \Delta\sigma' \le \sigma_c'$

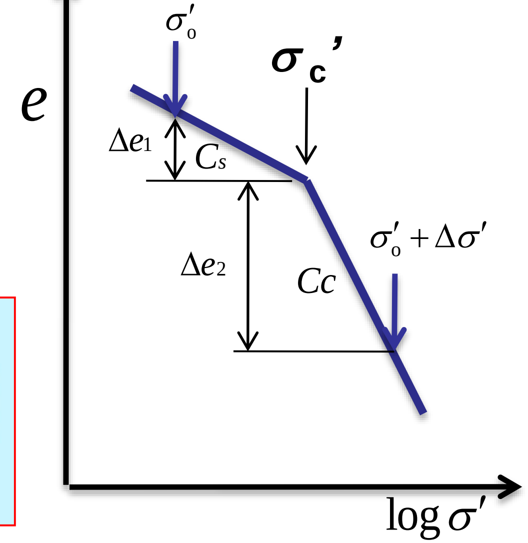
$$\Delta e = C_s[\log(\sigma'_o + \Delta\sigma') - \log \sigma'_o]$$

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

Case II: $\sigma'_0 + \Delta\sigma' > \sigma_c'$

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





Summary of calculation procedure

1. **Calculate σ'$_o$ at the middle of the clay layer**
2. **Determine σ'$_c$ from the e-log σ' plot (if not given)**
3. **Determine whether the clay is N.C. or O.C.**
4. **Calculate Δσ**
5. **Use the appropriate equation**

- **If N.C.**

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

- **If O.C.**

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

$\underline{If\ \ \sigma'_o + \Delta\sigma \le \sigma'_c}$

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

$\underline{If\ \ \sigma'_o + \Delta\sigma > \sigma'_c}$

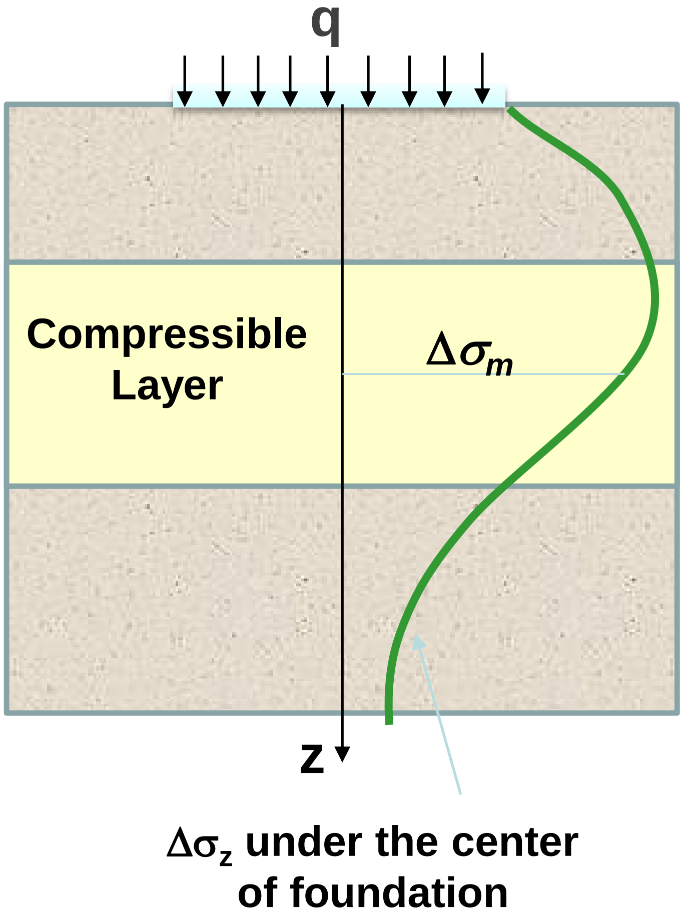
Nonlinear pressure increase

Approach 1: Middle of layer (midpoint rule)

- **For settlement calculation, the pressure increase $\Delta\sigma_z$ can be approximated as :**

$$\Delta\sigma_z = \Delta\sigma_m$$

where $\Delta\sigma_m$ represent the increase in the effective pressure in the middle of the layer.

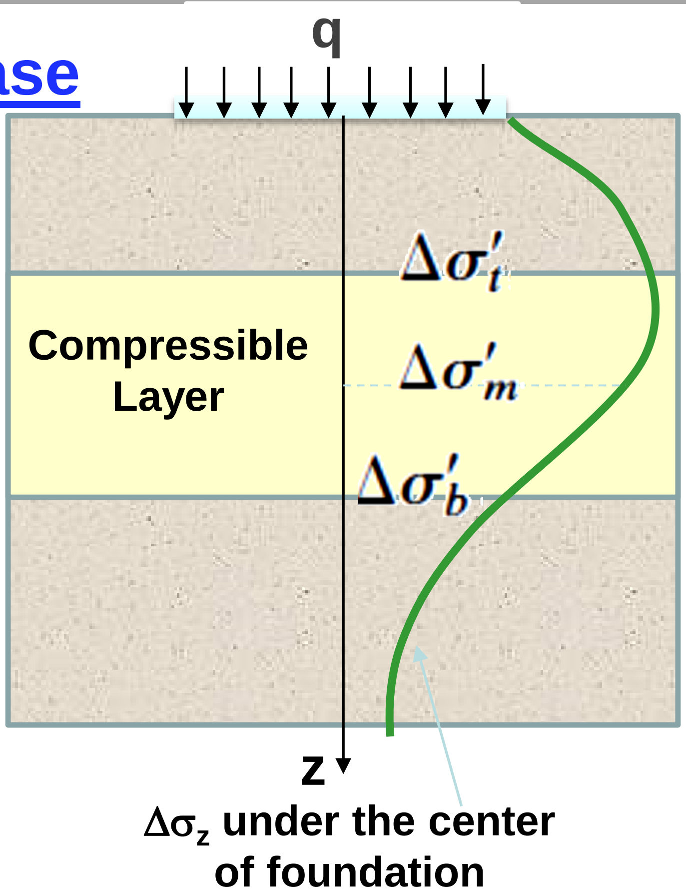


Nonlinear pressure increase

Approach 2: Average pressure increase

- For settlement calculation we will use the average pressure increase $\Delta\sigma_{av}$, using weighted average method (Simpson's rule):

$$\Delta\sigma'_{av} = \frac{\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b}{6}$$



where $\Delta\sigma_t$, $\Delta\sigma_m$ and $\Delta\sigma_b$ represent the increase in the pressure at the top, middle, and bottom of the clay, respectively, under the center of the footing.

EXAMPLE 11.6

Example 11.6

The following are the results of a laboratory consolidation test:

Pressure, σ' (kN/m^2)	Void ratio, e	Remarks	Pressure, σ' (kN/m^2)	Void ratio, e	Remarks
25	0.93	Loading	800	0.61	Loading
50	0.92		1600	0.52	
100	0.88		800	0.535	Unloading
200	0.81		400	0.555	
400	0.69		200	0.57	

a. Calculate the compression index and the ratio of C_s/C_c.
b. On the basis of the average e-log σ' plot, calculate the void ratio at $\sigma'_o = 1000$ kN/m^2.

Solution

Part a

The e versus log σ' plot is shown in Figure 11.23. From the average e-log σ' plot, for the loading and unloading branches, the following values can be determined:

Branch	e	σ'_o (kN/m^2)
Loading	0.8	200
	0.7	400
Unloading	0.57	200
	0.555	400

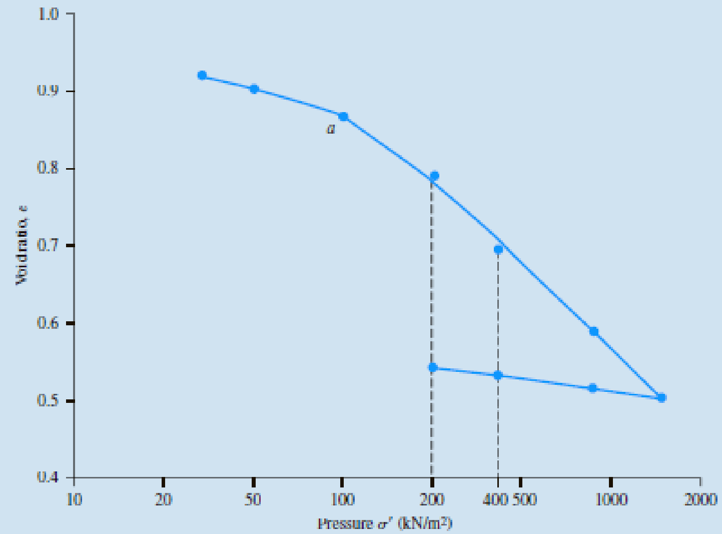


Figure 11.23 Plot of e versus log σ'

From the loading branch,

$$C_c = \frac{e_1 - e_2}{\log \frac{\sigma'_2}{\sigma'_1}} = \frac{0.8 - 0.7}{\log\left(\frac{400}{200}\right)} = \mathbf{0.33}$$

From the unloading branch,

$$C_s = \frac{e_1 - e_2}{\log \frac{\sigma'_2}{\sigma'_1}} = \frac{0.57 - 0.555}{\log\left(\frac{400}{200}\right)} = 0.0498 \approx 0.05$$

$$\frac{C_s}{C_c} = \frac{0.05}{0.33} = \mathbf{0.15}$$

Part b

$$C_c = \frac{e_1 - e_3}{\log \frac{\sigma'_3}{\sigma'_1}}$$

We know that $e_1 = 0.8$ at $\sigma'_1 = 200$ kN/m^2 and that $C_c = 0.33$ [part (a)]. Let $\sigma'_3 = 1000$ kN/m^2. So,

$$0.33 = \frac{0.8 - e_3}{\log\left(\frac{1000}{200}\right)}$$

$$e_3 = 0.8 - 0.33 \log\left(\frac{1000}{200}\right) \approx \mathbf{0.57}$$

EXAMPLE 11.7

Example 11.7

For a given clay soil in the field, given: $G_s = 2.68$, $e_o = 0.75$. Estimate C_c based on Eqs. (11.40) and (11.44).

Solution

From Eq. (11.40),

$$C_c = 0.141 G_s^{1.2}\left(\frac{1 + e_o}{G_s}\right)^{2.38} = (0.141)(2.68)^{1.2}\left(\frac{1 + 0.75}{2.68}\right)^{2.38} \approx \mathbf{0.167}$$

From Eq. (11.44),

$$C_c = \frac{n_o}{371.747 - 4.275 n_o}$$

$$n_o = \frac{e_o}{1 + e_o} = \frac{0.75}{1 + 0.75} = 0.429$$

$$C_c = \frac{(0.429)(100)}{371.747 - (4.275)(0.429 \times 100)} = \mathbf{0.228}$$

Note: It is important to know that the empirical correlations are approximations only and may deviate from one soil to another.

EXAMPLE 11.8

Example 11.8

A soil profile is shown in Figure 11.24. If a uniformly distributed load, $\Delta\sigma$, is applied at the ground surface, what is the settlement of the clay layer caused by primary consolidation if

a. The clay is normally consolidated

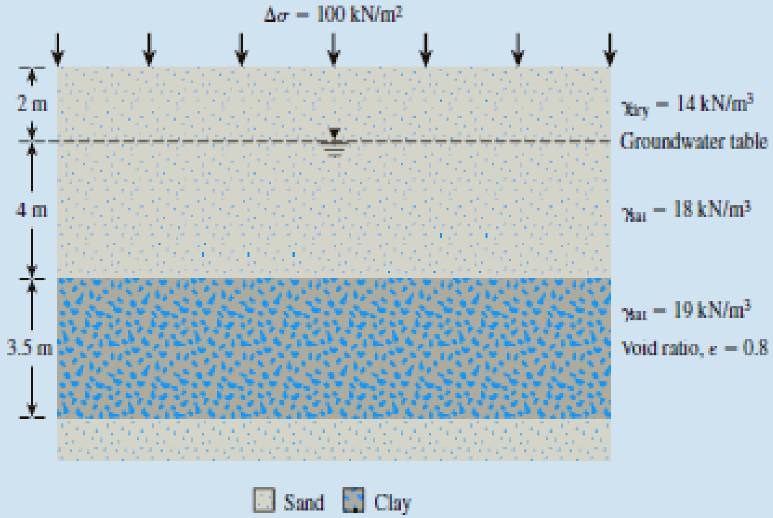


Figure 11.24

b. The preconsolidation pressure, $\sigma'_c = 200$ kN/m²
c. $\sigma'_c = 150$ kN/m²

Use $C_s \approx \frac{1}{5} C_c$; and Eq. (11.40).

Solution

Part a

The average effective stress at the middle of the clay layer is

$$\sigma'_o = 2\gamma_{dry} + 4[\gamma_{sat(sand)} - \gamma_w] + \frac{3.5}{2}[\gamma_{sat(clay)} - \gamma_w]$$

$$\sigma'_o = (2)(14) + 4(18 - 9.81) + 1.75(19 - 9.81) = 76.08 \text{ kN/m}^2$$

$$\gamma_{sat(clay)} = 19 \text{ kN/m}^3 = \frac{(G_s + e)\gamma_w}{1 + e} = \frac{(G_s + 0.8)(9.81)}{1 + 0.8}; G_s = 2.686$$

From Eq. (11.40),

$$C_c = 0.141 G_s^{1.2}\left(\frac{1 + e_o}{G_s}\right)^{2.38} = (0.141)(2.686)^{1.2}\left(\frac{1 + 0.8}{2.686}\right)^{2.38} = 0.178$$

From Eq. (11.35),

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

So,

$$S_c = \frac{(0.178)(3.5)}{1 + 0.8} \log\left(\frac{76.08 + 100}{76.08}\right) = 0.126 \text{ m} = \mathbf{126\ mm}$$

EXAMPLE 11.8

Part b

$$\sigma'_o + \Delta\sigma' = 76.08 + 100 = 176.08 \text{ kN/m}^2$$

$$\sigma'_c = 200 \text{ kN/m}^2$$

Because $\sigma'_o + \Delta\sigma' < \sigma'_c$, use Eq. (11.37):

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

$$C_s = \frac{C_c}{5} = \frac{0.178}{5} = 0.0356$$

$$S_c = \frac{(0.0356)(3.5)}{1 + 0.8} \log\left(\frac{76.08 + 100}{76.08}\right) = 0.025 \text{ m} = \mathbf{25\ mm}$$

Part c

$$\sigma'_o = 76.08 \text{ kN/m}^2$$

$$\sigma'_o + \Delta\sigma' = 176.08 \text{ kN/m}^2$$

$$\sigma'_c = 150 \text{ kN/m}^2$$

Because $\sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'$, use Eq. (11.38):

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

$$= \frac{(0.0356)(3.5)}{1.8} \log\left(\frac{150}{76.08}\right) + \frac{(0.178)(3.5)}{1.8} \log\left(\frac{176.08}{150}\right)$$

$$\approx 0.0445 \text{ m} = \mathbf{44.5\ mm}$$

EXAMPLE 11.9

Example 11.9

Refer to Example 11.8. For each part, calculate and plot a graph of e vs. σ' at the beginning and end of consolidation.

Solution

For each part, e = **0.8** at the beginning of consolidation. For e at the end of consolidation, the following calculations can be made.

Part a

$$e = 0.8 - C_c \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right) = 0.8 - 0.178 \log\left(\frac{176.08}{76.08}\right) = \mathbf{0.735}$$

Part b

$$e = 0.8 - C_s \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right) = 0.8 - 0.0356 \log\left(\frac{176.08}{76.08}\right) = \mathbf{0.787}$$

Part c

$$e = 0.8 - \left[C_s \log\left(\frac{\sigma'_c}{\sigma'_o}\right) + C_c \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right)\right]$$

$$= 0.8 - \left[0.0356 \log\left(\frac{150}{76.08}\right) + 0.178 \log\left(\frac{176.08}{150}\right)\right]$$

$$= 0.8 - 0.0105 - 0.0124 = \mathbf{0.771}$$

A plot of e versus $\log \sigma'$ is shown in Figure 11.25.

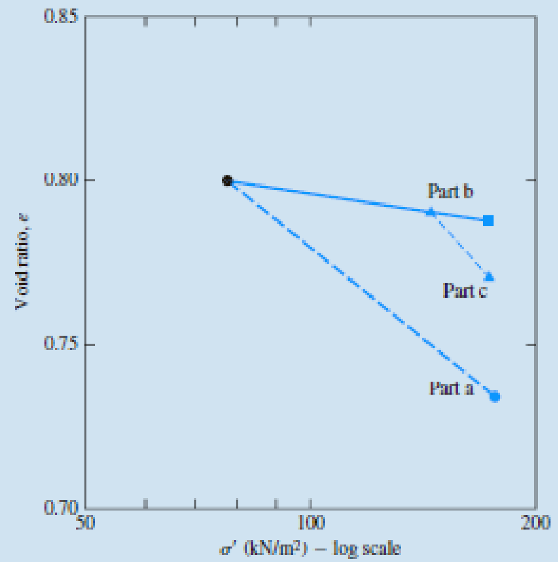


Figure 11.25

EXAMPLE 11.10

Example 11.10

A soil profile is shown in Figure 11.26a. Laboratory consolidation tests were conducted on a specimen collected from the middle of the clay layer. The field consolidation curve interpolated from the laboratory test results is shown in Figure 11.26b. Calculate the settlement in the field caused by primary consolidation for a surcharge of 60 kN/m² applied at the ground surface.

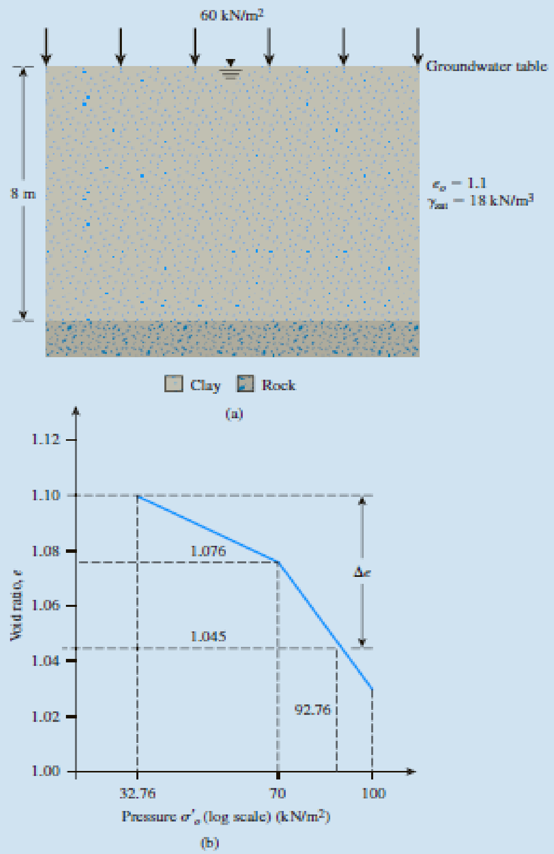


Figure 11.26 (a) Soil profile (b) field consolidation curve

Solution

$$\sigma'_o = (4)(\gamma_{sat} - \gamma_w) = 4(18.0 - 9.81)$$
$$= 32.76 \text{ kN/m}^2$$
$$e_o = 1.1$$
$$\Delta\sigma' = 60 \text{ kN/m}^2$$
$$\sigma_o' + \Delta\sigma' = 32.76 + 60 = 92.76 \text{ kN/m}^2$$

The void ratio corresponding to 92.76 kN/m² (see Figure 11.26b) is 1.045. Hence, $\Delta e = 1.1 - 1.045 = 0.055$. We have

$$\text{Settlement, } S_c = H\frac{\Delta e}{1 + e_o} \qquad \text{[Eq. (11.33)]}$$

So,

$$S_c = 8\frac{(0.055)}{1 + 1.1} = 0.21 \text{ m} = \mathbf{210 \text{ mm}}$$

Secondary Consolidation Settlement

- In some soils (especially recent organic soils) the compression continues under constant loading after all of the excess pore pressure has dissipated, i.e. after primary consolidation has ceased.
- This is called secondary compression or creep, and it is due to plastic adjustment of soil fabrics.
- Secondary compression is different from primary consolidation in that it takes place at a constant effective stress.
- This settlement can be calculated using the secondary compression index, C_α.
- The Log-Time plot (of the consolidation test) can be used to estimate the coefficient of secondary compression C_α as the slope of the straight line portion of e vs. log time curve which occurs after primary consolidation is complete.

Secondary Consolidation Settlement

- **The magnitude of the secondary consolidation can be calculated as:**

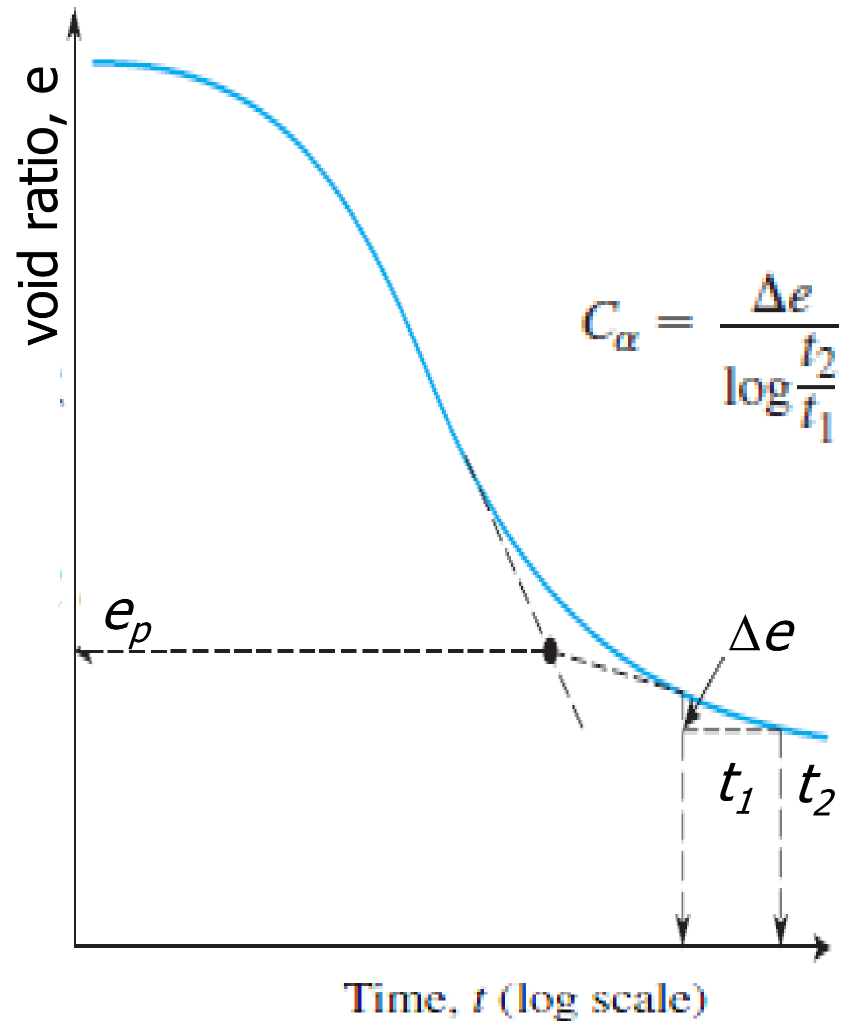
$$S_s = \frac{H}{1+e_p}\Delta e$$

e_p void ratio at the end of primary consolidation,
H thickness of clay layer.

$$\Delta e = C_\alpha \log(t_2/t_1)$$

C_α = coefficient of secondary compression

$$S_s = \frac{C_\alpha H}{1+e_p}\log\left(\frac{t_2}{t_1}\right)$$



Secondary Consolidation Settlement

Remarks

- **Causes of secondary settlement are not fully understood but is attributed to:**
  - **Plastic adjustment of soil fabrics**
  - **Compression of the bonds between individual clay particles and domains**
- **Factors that might affect the magnitude of S_s are not fully understood. In general secondary consolidation is large for:**
  - **Soft soils**
  - **Organic soils**
  - **Smaller ratio of induced stress to effective overburden pressure.**

Example 11.11

Example 11.11

For a normally consolidated clay layer in the field, the following values are given:

Thickness of clay layer = 8.5 ft
Void ratio, $e_o = 0.8$
Compression index, $C_c = 0.28$
Average effective pressure on the clay layer, $\sigma'_o = 2650$ lb/ft^2
$\Delta\sigma' = 970$ lb/ft^2
Secondary compression index, $C_\alpha = 0.02$

What is the total consolidation settlement of the clay layer five years after the completion of primary consolidation settlement? (*Note*: Time for completion of primary settlement = 1.5 years.)

Solution

From Eq. (11.49),

$$C'_\alpha = \frac{C_\alpha}{1 + e_p}$$

The value of e_p can be calculated as

$$e_p = e_o - \Delta e_{\text{primary}}$$

Combining Eqs. (11.33) and (11.34), we find that

$$\Delta e = C_c \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right) = 0.28 \log\left(\frac{2650 + 970}{2650}\right)$$

$$= 0.038$$

Primary consolidation:

$$S_c = \frac{\Delta e H}{1 + e_o} = \frac{(0.038)(8.5 \times 12)}{1 + 0.8} = 2.15 \text{ in.}$$

It is given that $e_o = 0.8$, and thus,

$$e_p = 0.8 - 0.038 = 0.762$$

Hence,

$$C'_\alpha = \frac{0.02}{1 + 0.762} = 0.011$$

From Eq. (11.48),

$$S_s = C'_\alpha H \log\left(\frac{t_2}{t_1}\right) = (0.011)(8.5 \times 12) \log\left(\frac{5}{1.5}\right) \approx 0.59 \text{ in.}$$

Total consolidation settlement = primary consolidation (S_c) + secondary settlement (S_s). So

Total consolidation settlement = 2.15 + 0.59 = **2.74 in.**