Fundamentals of Geotechnical Engineering – III

Chapter 1

Soil Compressibility & Settlement Analysis

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General Outline

- Introduction
- Immediate Settlement
- Primary Consolidation
- Secondary Compression
- Rate of Consolidation
- 1D Consolidation Test

1. Introduction

- Terms & Definitions
- Stress-Strain Behaviour of Soils
- Components of Settlement
- Soil Compressibility

Introduction cntd

Terms & Definitions

- **Settlement:** total vertical deformation at soil surface resulting from the load
- **Consolidation** (volume change velocity): rate of decrease in volume with respect to time
- **Compressibility** (volume change flexibility): volume decrease due to a unit load
- **Contraction** (temperature expansion): change in volume of soil due to a change in temperature
- **Swelling:** volume expansion of soil due to increase in water content
- **Shrinkage:** volume contraction of soil due to reduction in water content

Introduction cntd

Stress-Strain Behaviour of Soil

- Soils cannot sustain tension (except for some partially cemented types)
- When loaded, soils will generally undergo a change in volume or an increase in pore fluid pressure
- Saturated soils can only undergo a change in volumes as porewater is squeezed out (or lost by drying); [the rate of water loss (drainage) is controlled by the permeability of the soil]
- Some (hard or stiff) soils will exhibit brittle failure by shearing, while others will simply distort plastically

Introduction cntd

Soils

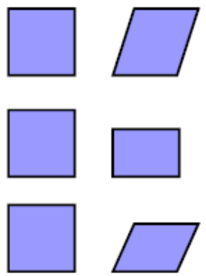
- Considered elastic materials for simplification
- highly nonlinear materials
- have a ‘memory’ - non conservative material

When stressed → soil deform

Stressed released → deformation remains

Soil deformation :

- Distortion (change in shape)
- Compression (change in volume)
- Both



Introduction cntd

Settlement

What? → total vertical deformation at soil surface resulting from the load

Causes of soil movement

- Downward: load increase or lowering water table
- Upward: temporary or permanent excavation

Points of interest:

- How much?
- How fast?

Introduction cntd

Components of Settlement

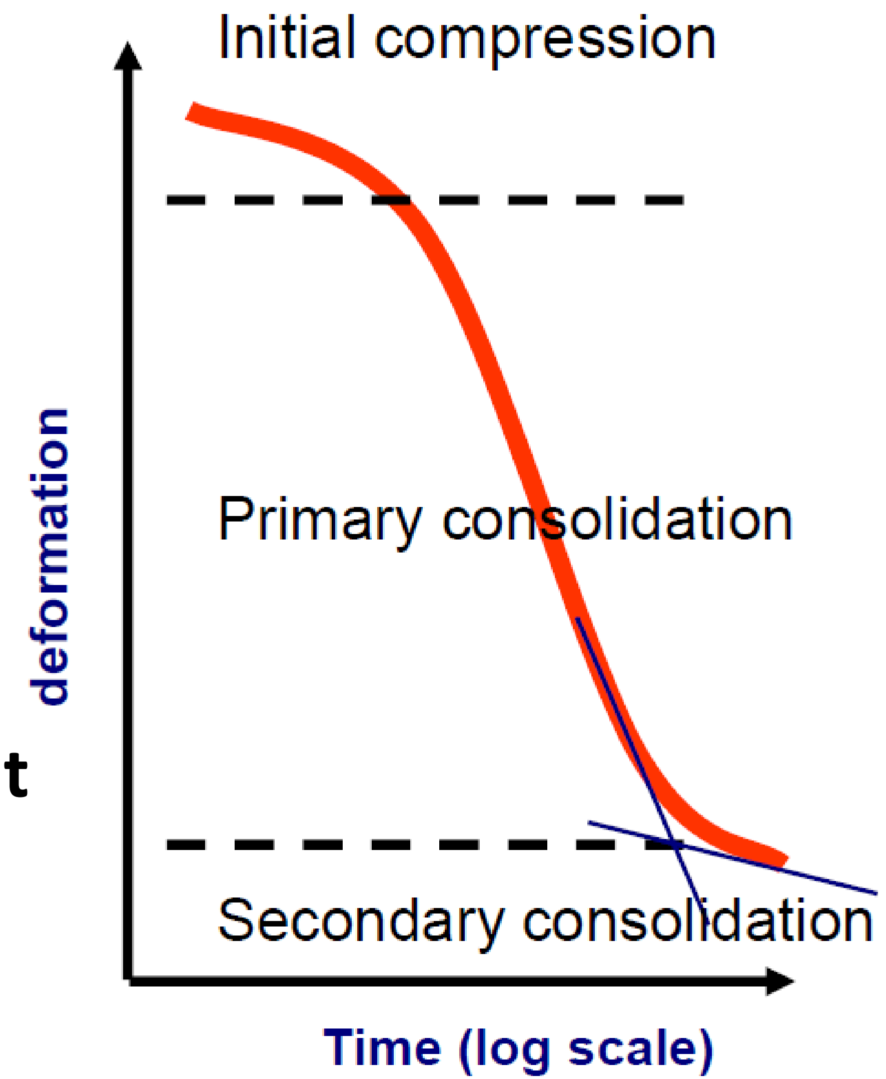
$$S_t = S_i + S_c + S_s$$

S_t = total settlement

S_i = immediate settlement

S_c = consolidation settlement

S_s = secondary compression



Introduction cntd

S_i : immediate / distortion settlement : elastic theory

3D loading → distortion in soil.

compression modulus & volume of stressed soil unknown

design for shallow foundation

S_c : consolidation settlement : time dependent process

occurs in saturated fine-grained soil

low coefficient of permeability

Settlement rate depend on pore water pressure

S_s : secondary compression : time dependent

occurs at constant effective stress

no subsequent changes in pore water pressures

Introduction cntd

Soil Compressibility (volume change flexibility) is the volume decrease due to a unit load.

- Assumption in settlement : 100% saturated and 1D (vertical) soil deformation
- When soil is loaded it will compress because of:
  - Deformation of soil grains (small, can be neglected)
  - Compression of air and water in the voids
  - Squeezing out of water & air from the voids
- Compressible soil mostly found below water table → considered fully saturated

Introduction cntd

As pore fluid squeezed out:

- ✓ Soil grain rearrange themselves → stable & denser configuration
- ✓ Decrease in volume → surface settlement resulted

How fast? → depend on permeability of soil

- ○ Compression of sand occurs instantly
- ○ Consolidation of cohesive soil is very time depend process

How much rearrangement & compression?

→ depend on the rigidity of soil skeleton

2. Immediate Settlement

- Introduction
- Contact Pressure
- Elastic Compression for Cohesive Soils
- Elastic Compression for Cohesionless Soils

Immediate Settlement

Immediate settlement

- caused by the elastic deformation of dry soil and of moist and saturated soils after the application of a load without any change in the moisture content.
- also referred to as the ‘elastic or distortion or contact settlement'
- usually taken to occur immediately on application of the foundation load (within about 7 days).
- calculation generally based on equations derived from the theory of elasticity

Immediate Settlement

Immediate settlement

- The vertical component of the foundation load Its causes a vertical movement of the foundation (immediate settlement) that in the case of a partially saturated soil is mainly due to the expulsion of gases and to the elastic bending reorientation of the soil particles.
- With saturated soils immediate settlement effects are assumed to be the result of vertical soil compression before there is any change in volume.

Immediate Settlement cntd

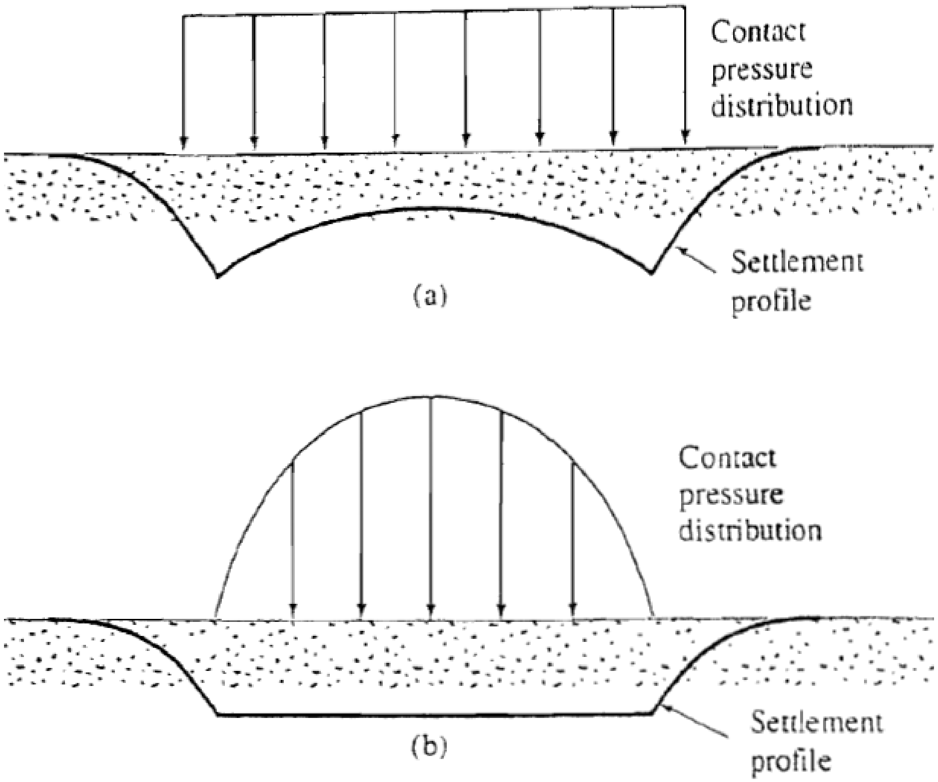
Contact Pressure

- The magnitude of the contact settlement will depend on the flexibility of the foundation and the type of the materials on which it is resting.

Fig. Immediate settlement profile and contact pressure in sand

a) Flexible foundation

b) Rigid foundation



Immediate Settlement cntd

- One can use the theory of elasticity to determine the immediate or elastic settlement of shallow foundations.

- The vertical elastic settlement at the ground surface under a rectangular flexible surface load is

$$S_i = \frac{q_s B(1 - v^2)}{E} I_s$$

where *Is* is a settlement influence factor that is a function of the *L*/*B* ratio (*L* is length and *B* is width).

- However, this equation do not account for the shape of the footing and the depth of embedment, which significantly influence settlement.

Immediate Settlement cntd

- To account for embedment

$$S_i = \frac{q_s B(1-v^2)}{E} I_s \mu'_{emb}$$

where $$\mu'_{emb} = 1 - 0.08\,\frac{D_f}{B_r}\left[1 + \frac{4B_r}{3L_r}\right]$$

where *Br* and *Lr* are the actual width and length, respectively.

An embedded foundation has the following 3 effects in comparison with a surface footing:

1. Soil stiffness generally increases with depth, so the footing loads will be transmitted to a stiffer soil than the surface soil. This can result in smaller settlements.

Immediate Settlement cntd

2. Normal stresses from the soil above the footing level have been shown to reduce the settlement by providing increased confinement on the deforming half-space. This is called the trench effect or embedment effect.

3. Part of the load on the footing may also be transmitted through the side walls depending on the amount of shear resistance mobilized at the soil–wall interface. The accommodation of part of the load by side resistance reduces the vertical settlement. This has been called the side wall–soil contact effect.

Immediate Settlement cntd

For an arbitrarily shaped rigid footing embedded in a deep homogeneous soil

$$S_i = \frac{P}{E_u L}(1 - v_u^2)\mu_s \mu_{emb} \mu_{wall}$$

where P is total vertical load, E_u is the undrained elastic modulus of the soil, L is one-half the length of a circumscribed rectangle, v_u is Poisson's ratio for the undrained condition, and μ_s , μ_{emb} , $and\ \mu_{wall}$ are shape, embedment (trench), and side wall factors given as

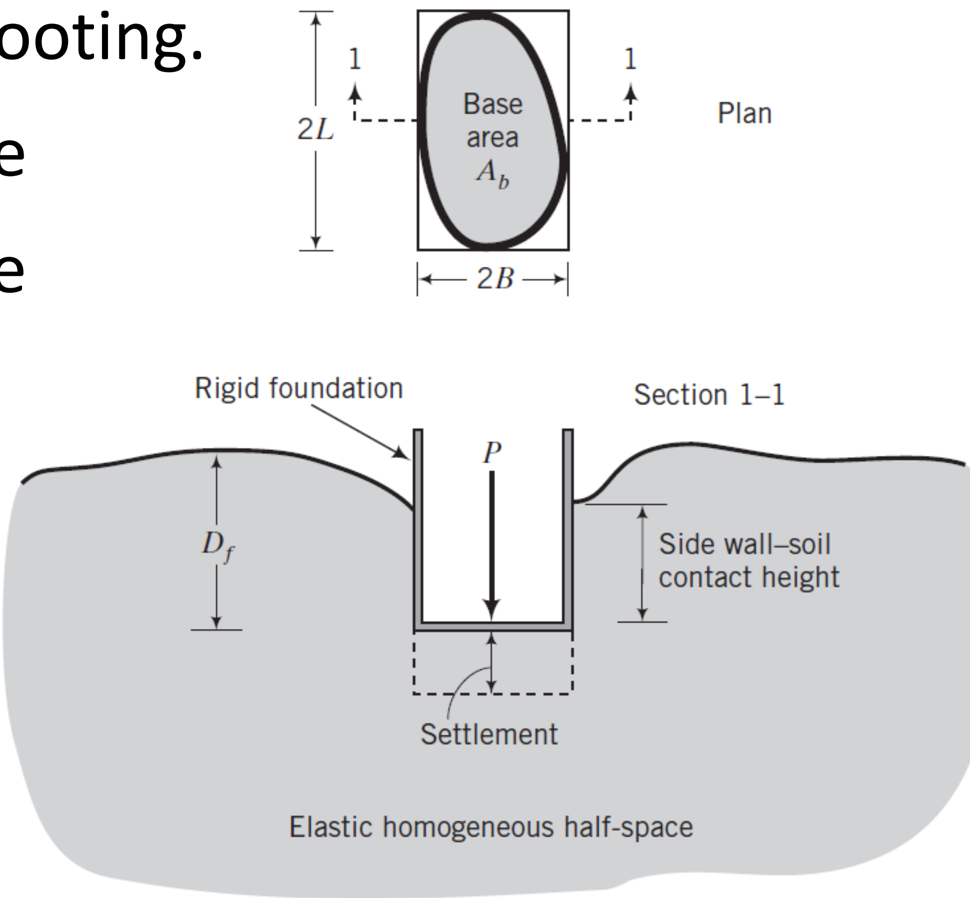
$$\mu_s = 0.45\left(\frac{A_b}{4L^2}\right)^{-0.38} ; \ \mu_{wall} = 1 - 0.16\left(\frac{A_w}{A_b}\right)^{0.54}$$

$$\mu_{emb} = 1 - 0.04\frac{D_f}{B}\left[1 + \frac{4}{3}\left(\frac{A_b}{4L^2}\right)\right]$$

Immediate Settlement cntd

A_b is the actual area of the base of the foundation and A_w is the actual area of the wall in contact with the embedded portion of the footing.

The length and width of the circumscribed rectangle are 2L and 2B, respectively.

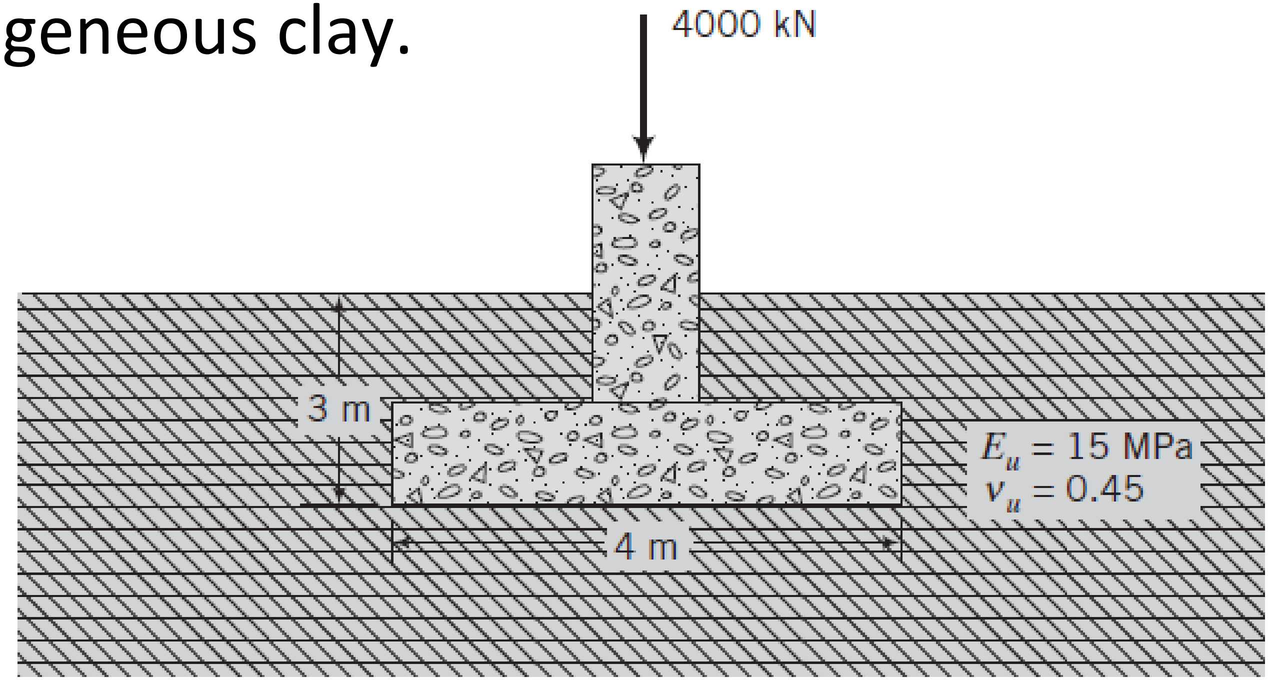


Footing shape	$\frac{A_b}{4L^2}$
Square	1
Rectangle	B/L
Circle	0.785
Strip	0

Immediate Settlement cntd

EXERCISE 1.2.1 - Elastic (Immediate) Settlement of a Footing on a Clay Soil

Determine the immediate settlement of a rectangular footing 4 m wide X 6 m long embedded in a deep deposit of homogeneous clay.

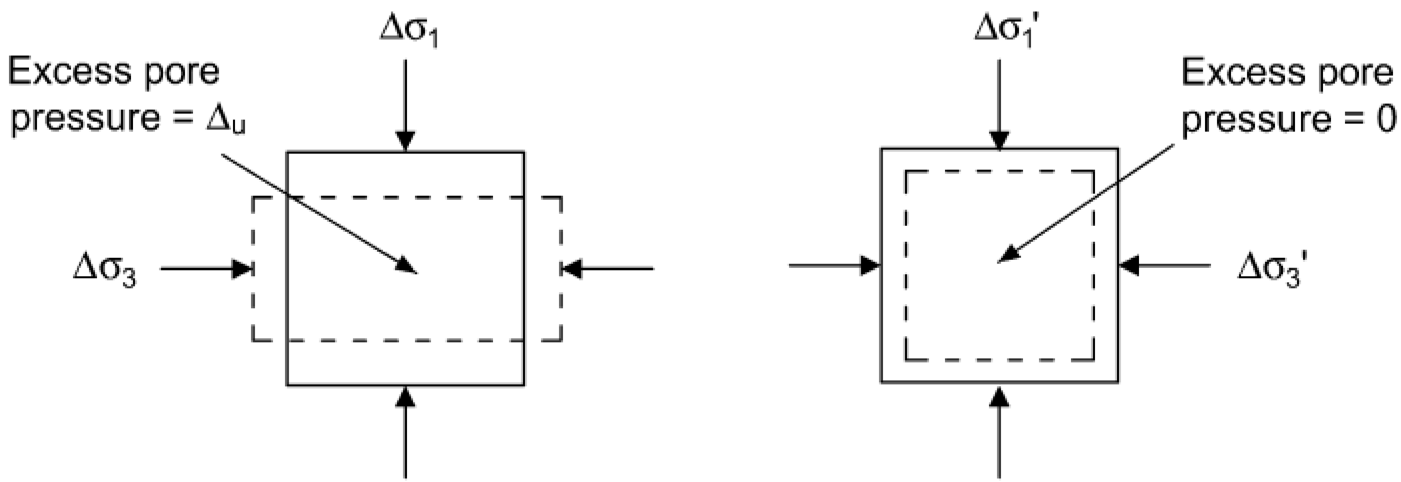


Immediate Settlement cntd

Elastic Compression of Cohesive Soils

- If a saturated clay is loaded rapidly, the soil will be deformed during the load application and excess hydrostatic pore pressures are set up.
- This deformation occurs with virtually no volume change, and due to the low permeability of the clay, little water is squeezed out of the voids.
- Vertical deformation due to the change in shape is the immediate settlement.

Immediate Settlement cntd



(a) Immediate settlement (b) Consolidation settlement

This change in shape is illustrated above, where an element of soil is subjected to a vertical major principal stress increase $\Delta\sigma_1$, which induces an excess pore water pressure, Δu. The lateral expansion causes an increase in the minor principal stress, $\Delta\sigma_3$.

Immediate Settlement cntd

Elastic Compression of Cohesive Soils under Flexible Foundations

The formula was provided by Terzaghi (1943) and is

$$\rho_i = \frac{pB(1-\upsilon^2)N_p}{E}$$

L/B	N_p
1.0	0.56
2.0	0.76
3.0	0.88
4.0	0.96
5.0	1.00

Where

p = uniform contact pressure

B = width of foundation

E = Young's modulus of elasticity for the soil

υ = Poisson's ratio for the soil (=0.5 in saturated soil)

N_p = an influence factor depending upon the dimensions of the flexible foundation.

Immediate Settlement cntd

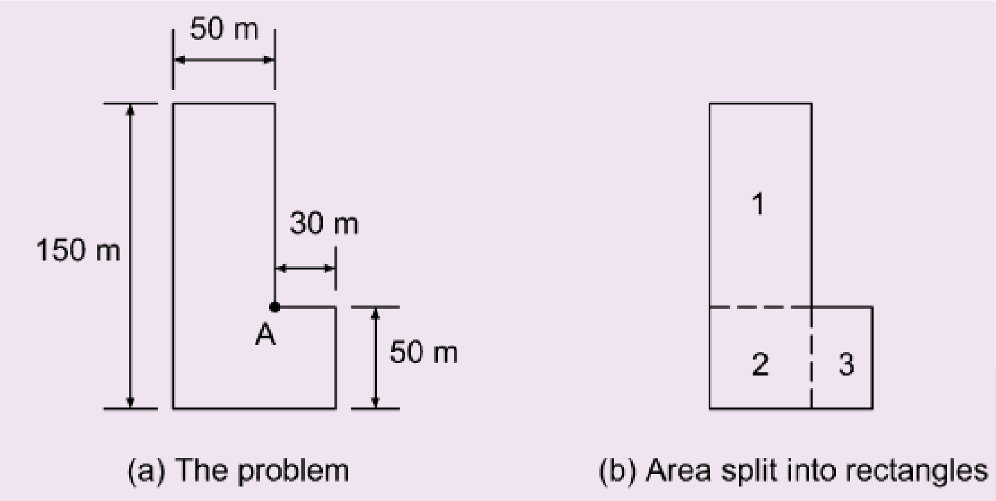
- This rn gives the immediate settlement at the corners of a rectangular footing, length L and width B.
- In the case of a uniformly loaded, perfectly flexible square footing, the immediate settlement under its centre is twice that at its corners.
- By the principle of superposition it is possible to determine the immediate settlement under any point of the base of a foundation.

NB. A spoil heap or earth embankment can be taken as flexible and to determine the immediate settlement of deposits below such a construction N_p should be used.

Immediate Settlement cntd

EXERCISE - Elastic Compression of Cohesive Soils under Flexible Foundations

The plan of a proposed spoil heap is shown below. The tip will be about 23 m high and will sit on a thick, soft alluvial deposit (E = 15 MPa). It is estimated that the eventual uniform bearing pressure on the soil will be about 300 kPa. Estimate the immediate settlement under the point A at the surface of the soil.



Immediate Settlement cntd

Elastic Compression of Cohesive Soils under Rigid Foundations

Foundations are generally more rigid than flexible and tend to impose a uniform settlement which is roughly the same value as the mean value of settlement under a flexible foundation which is given by the expression:

$$\rho_i = \frac{pB(1 - v^2)I_p}{E}$$

[For rectangular foundation on

the surface of a semi-elastic medium]

L/B	I_p
circle	0.73
1	0.82
2	1.00
5	1.22
10	1.26

Where Ip = an influence factor depending upon the dimensions of the foundation.

Immediate Settlement cntd

EXERCISE - Elastic Compression of Cohesive Soils under Rigid Foundations

A reinforced concrete foundation, of dimensions 20 m $\times$ 40 m exerts a uniform pressure of 200 kPa on a semi-infinite saturated soil layer (E = 50 MPa).

Determine the value of immediate settlement under the foundation

Immediate Settlement cntd

Elastic Compression of Cohesionless Soils

- Owing to the high permeabilities of cohesionless soils, both the elastic and the primary effects occur more or less together. The resulting settlement from these factors is termed the immediate settlement.
    - Meyerhof’s method
    - De Beer and Martens’ method
    - Schmertmann’s method

Immediate Settlement cntd

Meyerhof's method

A quick estimate of the settlement, ρ, of a footing on sand was proposed by Meyerhof (1974):

$$\rho = \frac{\Delta p B}{2\overline{C_r}}$$

B = the least dimension of the footing

C_r = the penetration resistance of the cone penetration test,

which is usually expressed in MPa or kPa.

$\overline{C_r}$ = the average value of Cr, over a depth below the footing equal to B

Δp = the net foundation pressure increase, which is simply the foundation loading less the value of vertical effective stress at foundation level, σ'_{vo}

Immediate Settlement cntd

De Beer and Martens' method

- From the results of the in situ tests carried out, a plot of Cr values against depth is prepared.
- With the aid of this plot the profile of the compressible soil beneath the proposed foundation can be divided into a suitable number of layers, preferably of the same thickness, although this is not essential.
- In the case of a deep soil deposit the depth of soil considered as affected by the foundation should not be less than 2.0B, ideally 4.0B; B = foundation width.

Immediate Settlement cntd

De Beer and Martens' method

The method proposes the use of a constant of compressibility, C_S, where $C_S = 1.5\frac{C_r}{p_{o1}}$

Where C_r = static cone resistance (kPa)

p_{o1} = effective overburden pressure at the point tested

$$\rho_i = \frac{H}{C_S}\ln\frac{p_{o2} + \Delta\sigma_z}{p_{o2}}$$

$\Delta\sigma_z$ = vertical stress increase at the centre of the consolidating layer of thickness H

p_{o2} = effective overburden pressure at the centre of the layer before any excavation or load application.

Immediate Settlement cntd

Schmertmann's method

The method is based on two main assumptions:

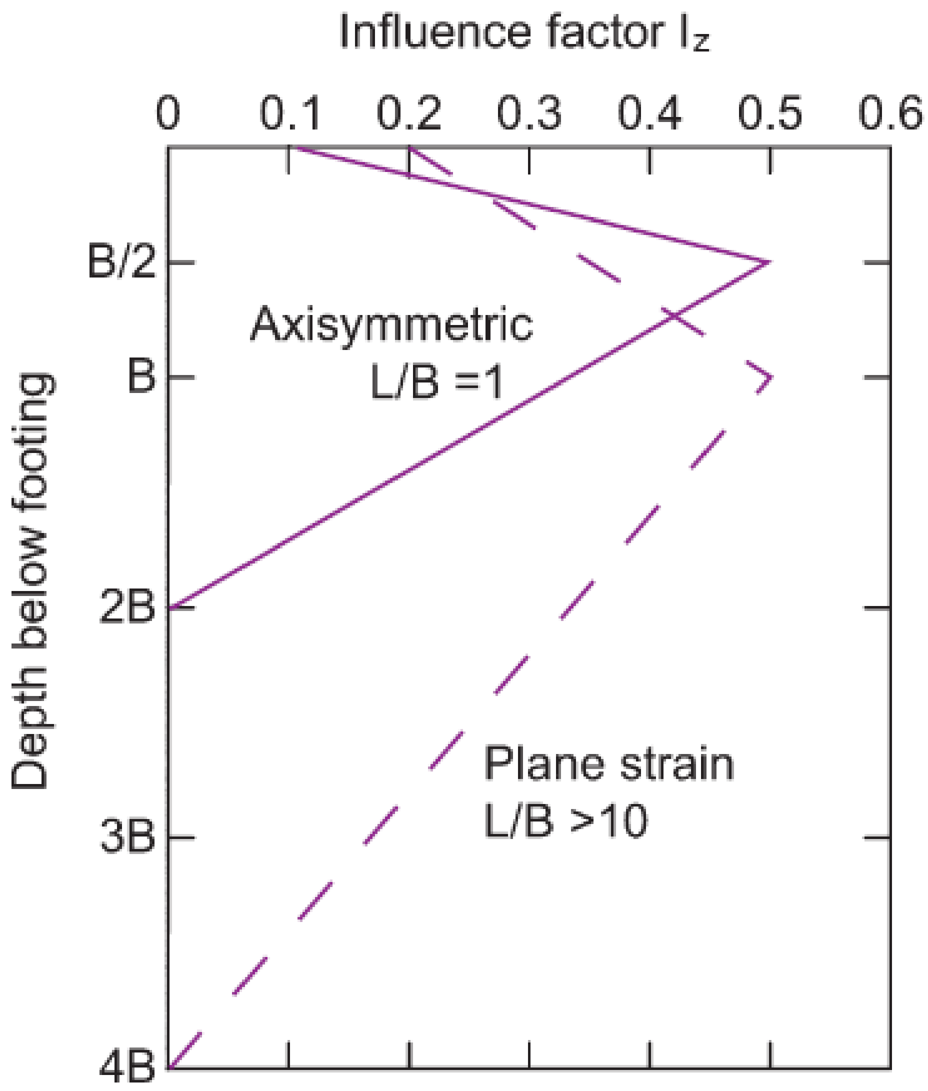
(i) the greatest vertical strain in the soil beneath the centre of a loaded foundation of width B occurs at depth B/2 below a square foundation and at depth of B below a long foundation;

(ii) significant stresses caused by the foundation loading can be regarded as insignificant at depths greater than z=2.0B for a square footing and z=4.0B for a strip footing.

Immediate Settlement cntd

Schmertmann's method

The method involves the use of a vertical strain influence factor, Iz, whose value varies with depth.



Immediate Settlement cntd

Schmertmann's method

The procedure consists of dividing the sand below the footing into n layers, of thicknesses Δz_1, Δz_2, $\Delta z3$ . . . Δz_n. If soil conditions permit it is simpler if the layers can be made of equal thickness, Δz.

The vertical strain of a layer is taken as equal to the increase in vertical stress at the centre of the layer, i.e. Δp multiplied by I_z, which is then divided by the product of Cr and a factor x. Hence:

$$\rho = C_1 C_2 \Delta p \sum_{1}^{n} \frac{I_z}{x C_r} \Delta z_1$$

Immediate Settlement cntd

Where x = 2.5 for a square footing and 3.5 for long footing

I_z = the strain influence factor, valued for each layer at its centre, and obtained from a diagram similar to the original diagram for variation of Iz with depth, but redrawn to correspond to the foundation loading

C_1 = a correction factor for the depth of the foundation$= 1.0 - 0.5\frac{\sigma'_v}{\Delta p}$

C_2 = a correction factor for creep$= 1 + 0.2\log_{10} 10t$

(t=time in years after the application of foundation loading for which the settlement values is required).

The variation of Iz with depth graph is redrawn by obtaining a new peak value for Iz from the expression $I_z = 0.5 + 0.1\left(\frac{\Delta p}{\sigma'_{vp}}\right)^{0.5}$ where σ'_{vp} = the effective vertical overburden pressure at a depth of 0.5B for a square foundation and at a depth of 1.0B for a long foundation.

Immediate Settlement cntd

EXERCISE - Settlement on a cohesionless soil

A foundation, 1.5 m square, will carry a load of 300 kPa and will be founded at a depth of 0.75 m in a deep deposit of granular soil. The soil may be regarded as saturated throughout with a unit weight of 20 kN/m^3, and the approximate N to z relationship is shown in Fig.

If the groundwater level occurs at a depth of 1.5 m below the surface of the soil determine a value for the settlement at the centre of the foundation,

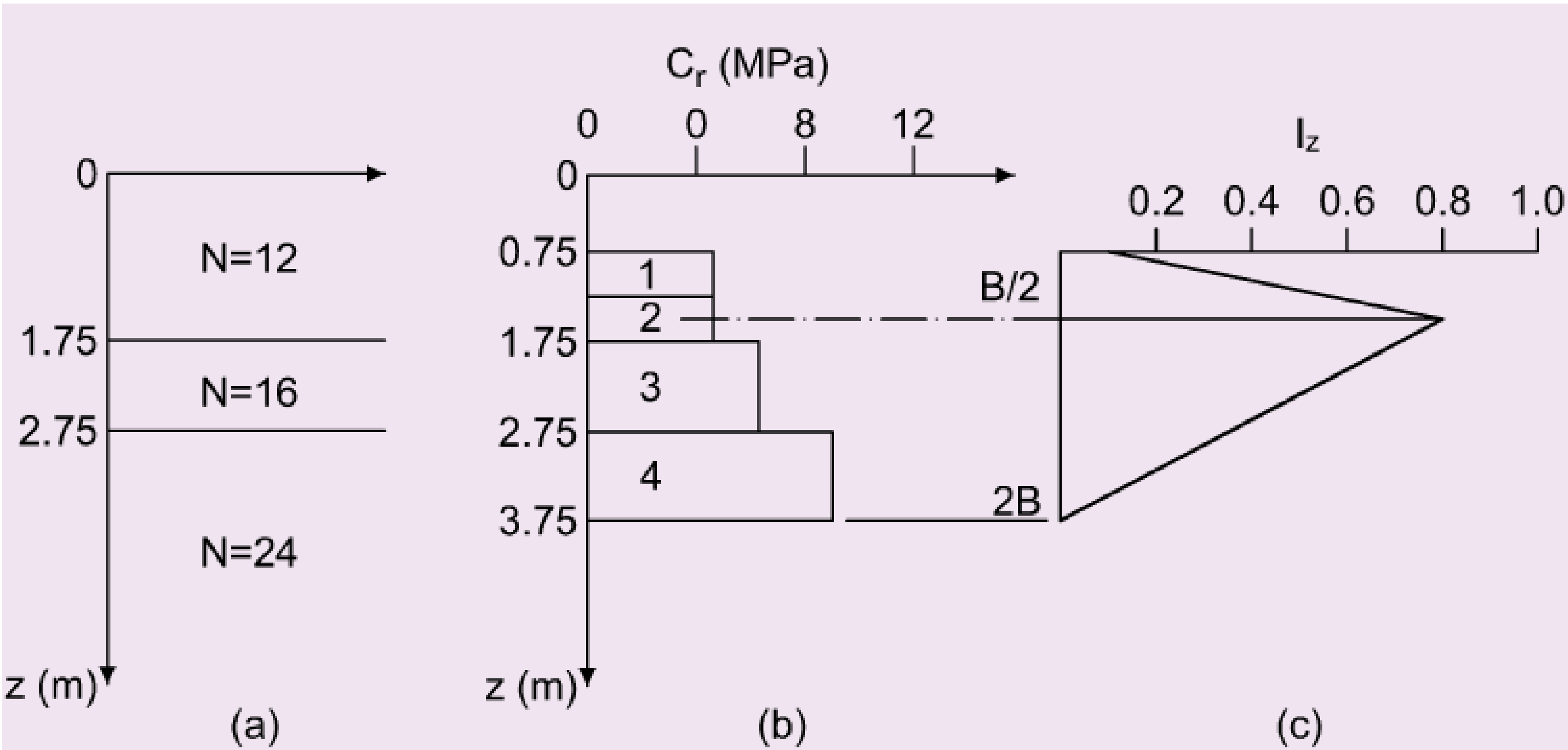
(a) by De Beer and Martens' method,

(b) by Schmertmann's method.

Immediate Settlement cntd

EXERCISE - Settlement on a cohesionless soil

(a) N to z relationship; (b) C_r to z relationship; (c) variation of I_z



Immediate Settlement cntd

READING ASSIGNMENT

- **The effect of depth on immediate settlement calculation**
- **Determination of elastic modulus E and poisson's ratio υ for soils**

3. Primary Consolidation

- Fundamentals of Consolidation
- Effect of Stress History
- Parameters of Primary Consolidation
- Calculation of Primary Consolidation

Consolidation Settlement cntd

Primary consolidation settlement

-result of a volume change in saturated cohesive soil because of expulsion of the water that occupies the void spaces.

-occurs in clays where the value of permeability prevents the initial excess pore water pressures from draining away immediately.

- The sudden application of a foundation load, besides causing elastic compression, creates a state of excess hydrostatic pressure in saturated soil.

Consolidation Settlement cntd

Primary consolidation settlement

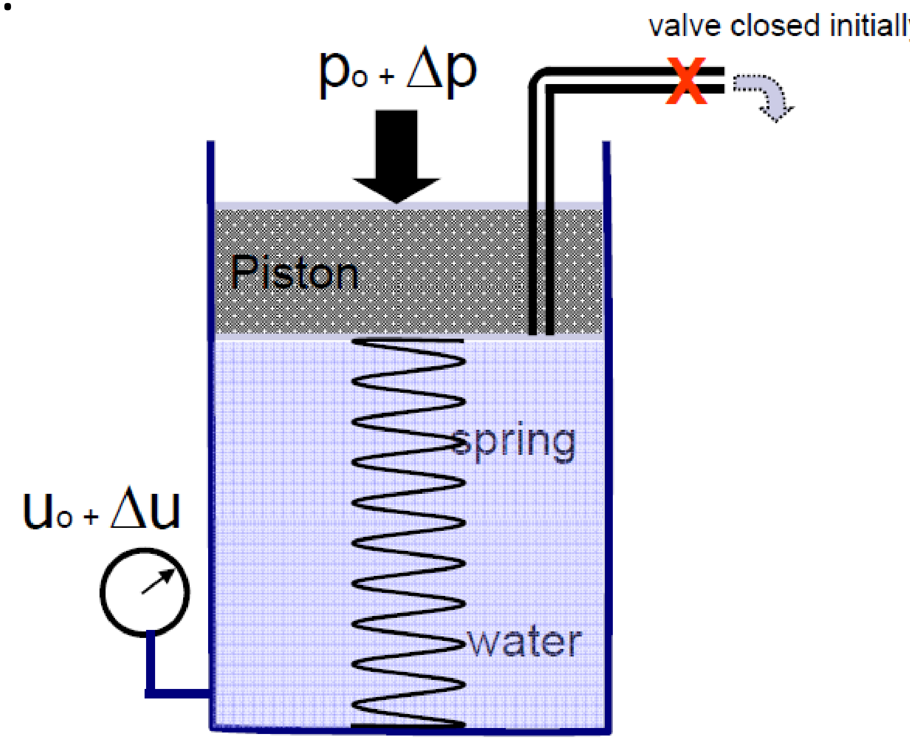
- These excess pore water pressure values can only be dissipated by the gradual expulsion of water through the voids of the soil, which results in a volume change that is time dependent.
- A large wheel load rolling along a roadway resting on a clay will cause an immediate settlement that is in theory completely recoverable once the wheel has passed, but if the same load is applied permanently there will in addition be consolidation.

Consolidation Settlement cntd

Consolidation of Clay under load Δp ($0 < t < \infty$)

- Soil is loaded by increment Δp.
- Valve initially closed.
- Pressure (Δp) is transferred to the water.
- As water is incompressible and valve still closed, no water is out, no deformation of piston.
- Pressure gauge read : $\Delta u = \Delta p$

where Δu is excess hydrostatic pressure.

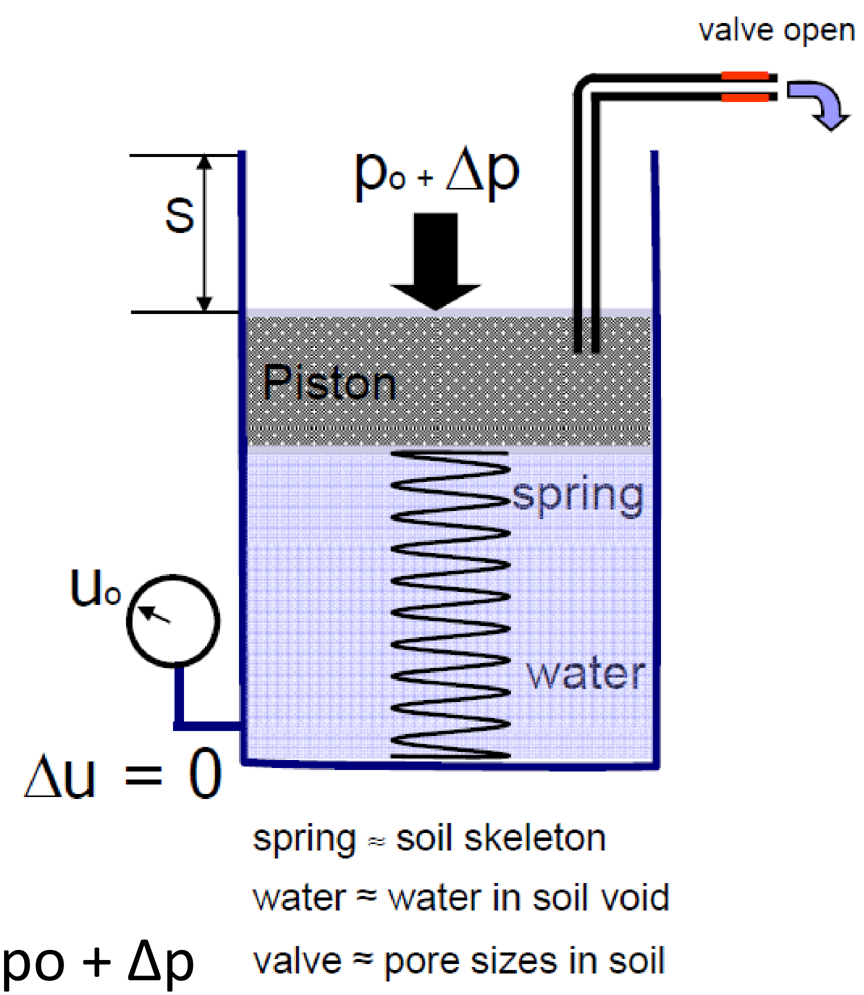


spring ≈ soil skeleton
water ≈ water in soil void
valve ≈ pore sizes in soil

Consolidation Settlement cntd

Consolidation of Clay at Equilibrium ($t = \infty$)

- To simulate a fine grained cohesive soil, where permeability is low valve can be opened
- Water slowly leave chamber
- As water flows out, load (Δp) is transferred to the spring.
- At equilibrium, no further water squeezed out, pore water pressure back to its hydrostatic condition.
- Spring is in equilibrium with load po + Δp



Consolidation Settlement cntd

Settlement Process

- Initially all external load is transferred into excess pore water (excess hydrostatic pressure)
- No change in the effective stress in the soil
- Gradually, as water squeezed out under pressure gradient, the soil skeleton compress, take up the load, and the effective stress increase.
- Eventually, excess hydrostatic pressure becomes zero and the pore water pressure is the same as hydrostatic pressure prior to loading.

Consolidation Settlement cntd

Stress History

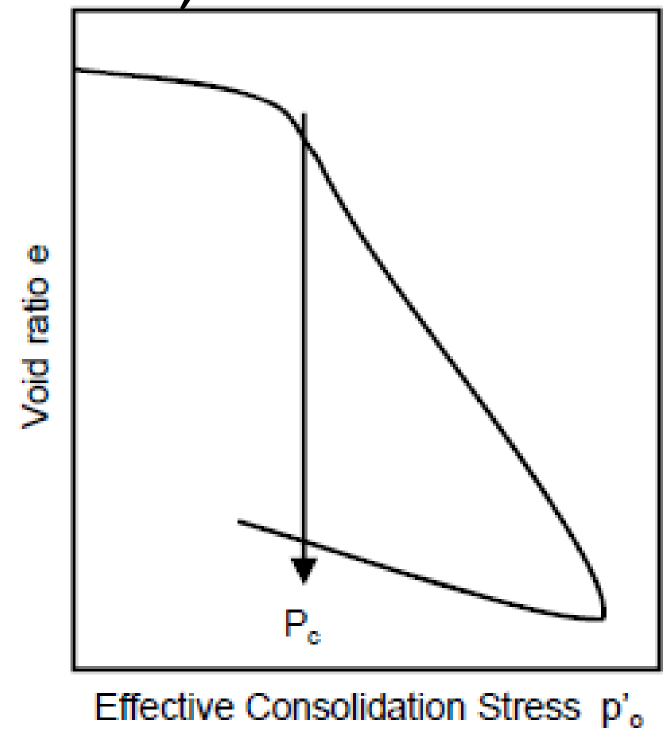
- When soil is loaded to a stress level greater than it ever ‘experienced’ in the past, the soil structure is no longer able to sustain the increased load, and start to breakdown.

Pre-consolidation Pressure – Pc:

maximum pressure experienced by soil in the past

Over-consolidation Ratio – OCR:

$$OCR = \frac{P_c}{p'_o}$$

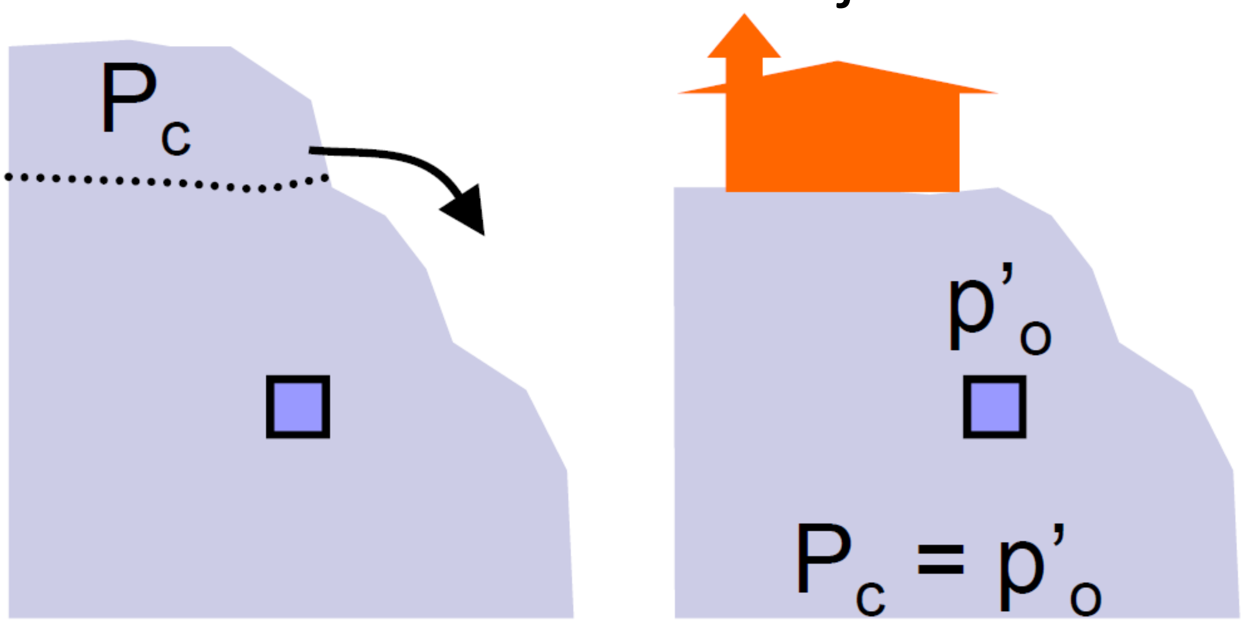


Consolidation Settlement cntd

Stress History

Normal Consolidation: OCR = 1

- when the pre-consolidation pressure is equal to the existing effective vertical overburden pressure Pc=p'o
- present effective overburden pressure is the maximum pressure that soil has been subjected in the past p'_o

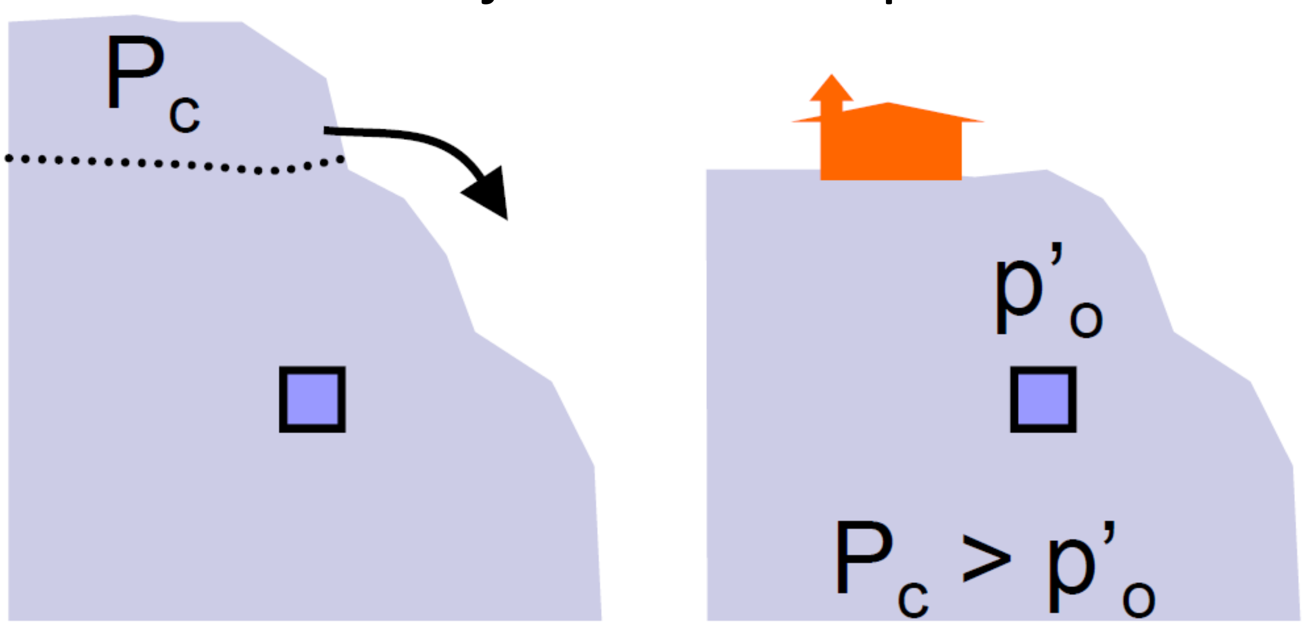


Consolidation Settlement cntd

Stress History

Over Consolidation: OCR > 1

- when pre-consolidation pressure is greater than the existing effective vertical overburden pressure $P_c > p'_o$
- present effective overburden pressure is less than that which soil has been subjected in the past



Consolidation Settlement cntd

Mechanisms causing preconsolidation

- Change in Total Stress
  - due to removal of overburden, past structures, glaciation
- Change in pore water pressure
  - Change in water table elevation / Artesian pressure
  - Deep pumping; flow into tunnel
  - Dessication due to surface drying / plant life
- Environmental changes such as pH, temperature and salt concentration
- Chemical alteration due to ‘weathering’, precipitation, cementing agents, ion exchange

Consolidation Settlement cntd

Effective Stress Changes

- Since the applied vertical stress (total stress) remains constant, then according to the principle of effective stress ($\Delta\boldsymbol{\sigma}'_{z} = \Delta\boldsymbol{\sigma}_{z} - \Delta\boldsymbol{u}$), any reduction of the initial excess porewater pressure must be balanced by a corresponding increase in vertical effective stress.
- Increases in vertical effective stresses lead to soil settlement caused by changes to the soil fabric.
- As time increases, the initial excess porewater continues to dissipate and the soil continues to settle.

Consolidation Settlement cntd

Effective Stress Changes

- After some time, usually within 24 hours for many small soil samples tested in the laboratory, the initial excess porewater pressure in the middle of the soil reduces to approximately zero, and the rate of decrease of the volume of the soil becomes very small.

- Since the initial excess porewater pressure becomes zero, then, from the principle of effective stress, all of the applied vertical stress is transferred to the soil; that is, the vertical effective stress is equal to the vertical total stress.

Consolidation Settlement cntd

Void Ratio Changes

The initial volume (specific volume) of a soil is $V = 1 + e_o$, where e_o is the initial void ratio.

The change in volume of the soil (ΔV) is equal to the change in void ratio (Δe).

The volumetric strain from the change in void ratio is

$$\boldsymbol{\varepsilon_v = \frac{\Delta z}{H_o} = \frac{\Delta e}{1 + e_o}}$$

Where $\boldsymbol{H_o}$ is the initial height of the sample.

Consolidation Settlement cntd

Void Ratio Changes

$$\boldsymbol{\Delta z = H_o \frac{\Delta e}{1 + e_o} = S_c}$$

where $\boldsymbol{S_c}$ represent primary consolidation.

The void ratio at any time under load P is

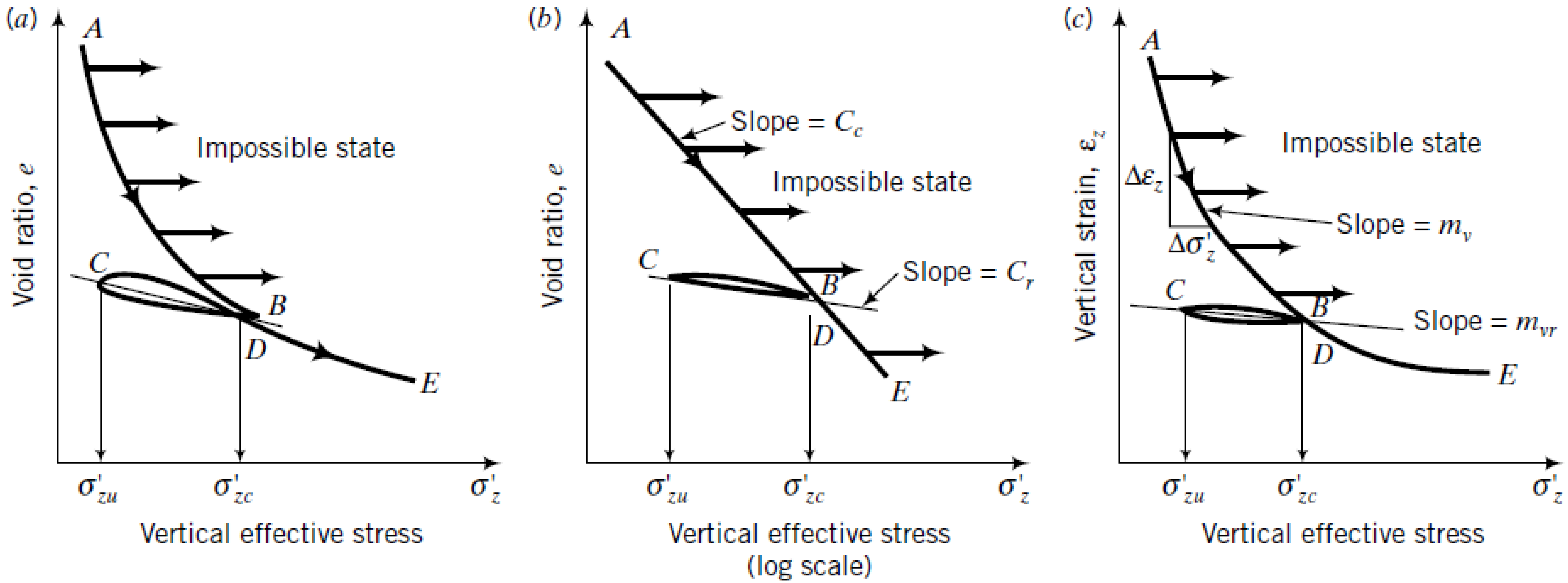
$$e = e_o - \Delta e = e_o - \frac{\Delta z}{H_o}(1 + e_o) = e_o\left(1 - \frac{\Delta z}{H_o}\right) - \frac{\Delta z}{H_o}$$

For saturated soil, $e_o = \omega G_s$. Thus

$$e = \omega G_s\left(1 - \frac{\Delta z}{H_o}\right) - \frac{\Delta z}{H_o}$$

Consolidation Settlement cntd

Parameters of Primary Consolidation



Consolidation Settlement cntd

Parameters of Primary Consolidation

Coefficient of compression or Compression Index (Cc)

$$C_c = -\frac{e_2 - e_1}{\log \frac{(\sigma'_z)_2}{(\sigma'_z)_1}}$$

Modulus of volume compressibility (m_v)

$$m_v = -\frac{(\varepsilon_z)_2 - (\varepsilon_z)_1}{(\sigma'_z)_2 - (\sigma'_z)_1}$$

where subscripts 1 and 2 denote two arbitrarily selected points on NCL.

Consolidation Settlement cntd

Parameters of Primary Consolidation

Recompression Index (Cr)

$$C_r = -\frac{e_2 - e_1}{log\frac{(\sigma'_z)_2}{(\sigma'_z)_1}}$$

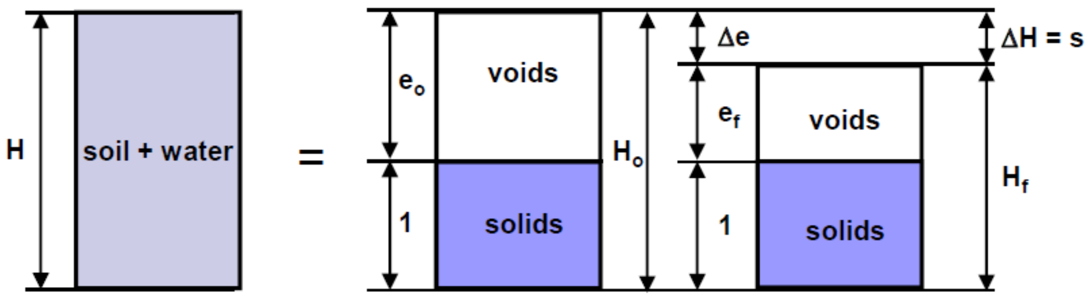
Modulus of volume recompressibility (m_{vr})

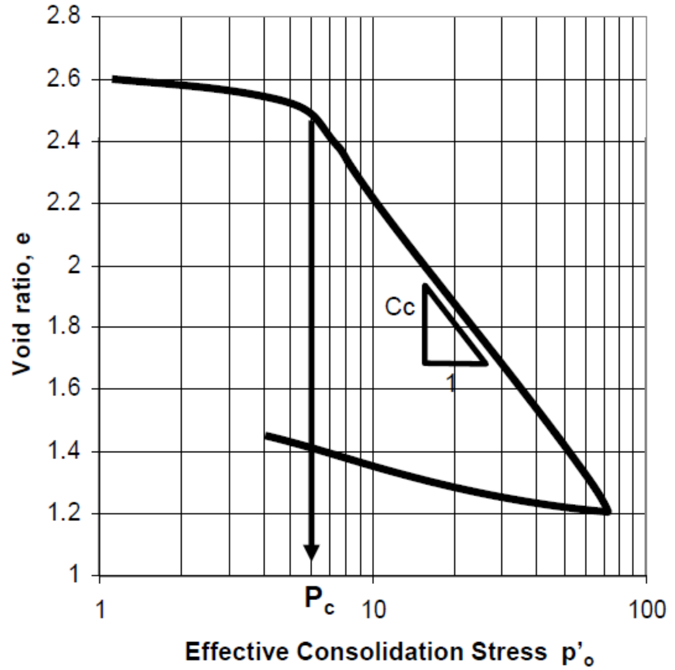
$$m_{vr} = -\frac{(\varepsilon_z)_2 - (\varepsilon_z)_1}{(\sigma'_z)_2 - (\sigma'_z)_1}$$

where subscripts 1 and 2 denote two arbitrarily selected points on URL.

Consolidation Settlement cntd

Calculation of Consolidation Settlement





$$\varepsilon_v = \frac{\Delta L}{L_o} \; or \; \frac{\Delta H}{H_o} = \frac{S}{H} = \frac{\Delta e}{1+e_o}$$

$$S = \frac{\Delta e}{1+e_o} H_o = \varepsilon_v H_o$$

$$C_c = \frac{-\Delta e}{\Delta \log p'_o} = \frac{e_1 - e_2}{\log p'_2 - \log p'_1} = \frac{e_1 - e_2}{\log p'_2 / p'_1}$$

$$S_c = C_c \frac{H_o}{1+e_o} \log \frac{p'_2}{p'_1}$$

Consolidation Settlement cntd

EXERCISE - CONSOLIDATION SETTLEMENT

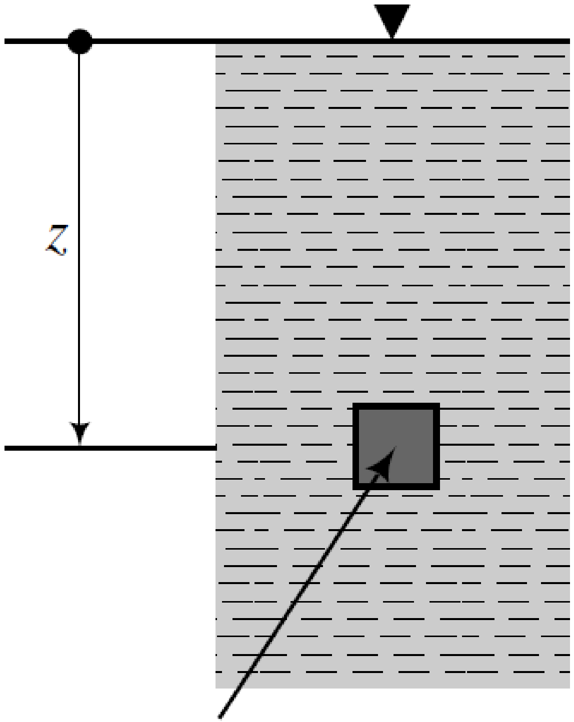
Prior to placement of a fill covering a large area at a site, the thickness of a compressible soil layer was 10m. Its original in situ void ratio was 1.0. Some time after the fill was constructed, measurements indicated that the average void ratio was 0.8.

Estimate the settlement of the soil layer.

Consolidation Settlement cntd

Effects of Unloading/Reloading of Soil Samples

Consider a soil sample that we wish to take from the field at a depth z. Assume that the groundwater level is at the surface.



The current vertical effective stress

or overburden effective stress is

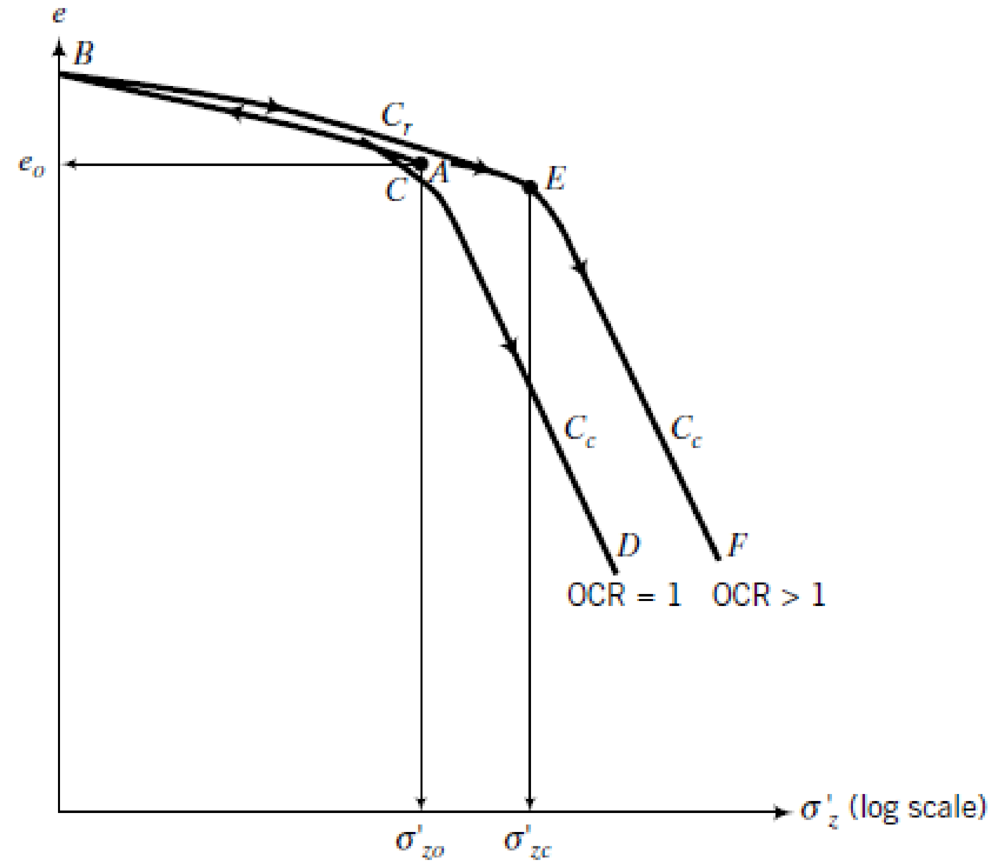
$$\sigma'_{zo} = (\gamma_{sat} - \gamma_w)z = \gamma' z$$

and the current void ratio can be

found from γ_{sat}

Consolidation Settlement cntd

On a plot of σ'_z (log scale) versus e, the current vertical effective stress can be represented as A.



To obtain a sample, we would have to make a borehole and remove the soil above it.

The act of removing the soil and extracting the sample reduces the total stress to zero; that is, we have fully unloaded the soil.

Consolidation Settlement cntd

- From the principle of effective stress, $\sigma'_z = -\Delta u$.

Since σ' cannot be negative—that is, soil solid cannot sustain tension—the porewater pressure must be negative.

- As the porewater pressure dissipates with time, volume changes (swelling) occur.

Using the basic concepts of consolidation, the sample will follow an unloading path AB.

- The point B does not correspond to zero effective stress because we cannot represent zero on a logarithmic scale.

However, the effective stress level at the start of the logarithmic scale is assumed to be small (≈ 0).

Consolidation Settlement cntd

If we were to reload our soil sample, the reloading path followed would depend on the OCR.

- If OCR = 1 (normally consolidated soil), the path followed during reloading would be BCD.

-The average slope of ABC is Cr. Once σ'_{zo} is exceeded, the soil will follow the normal consolidation line, CD, of slope Cc.

- If the soil were overconsolidated, OCR . 1, the reloading path followed would be BEF because we have to reload the soil beyond σ'_{zc} before it behaves like a normally consolidated soil.

-The average slope of ABE is Cr and the slope of EF is Cc. The point E marks the past maximum vertical effective stress.

Consolidation Settlement cntd

Primary Consolidation of NC Clay

Consider a site consisting of a normally consolidated soil on which we wish to construct a building.

Assume that the increase in vertical stress due to the building at depth z, where we took our soil sample, is Δp.

$$\boldsymbol{S_c = C_c \frac{H_o}{1+e_o} log \frac{\sigma'_{zo} + \Delta\sigma_z}{\sigma'_{zo}}}$$

For layered NC clay : $\boldsymbol{S_c = \sum \left[C_c \frac{H_o}{1+e_o} log \frac{\sigma'_{zo}+\Delta\sigma_z}{\sigma'_{zo}} \right]}$

Consolidation Settlement cntd

EXERCISE – S_c of NC Clay

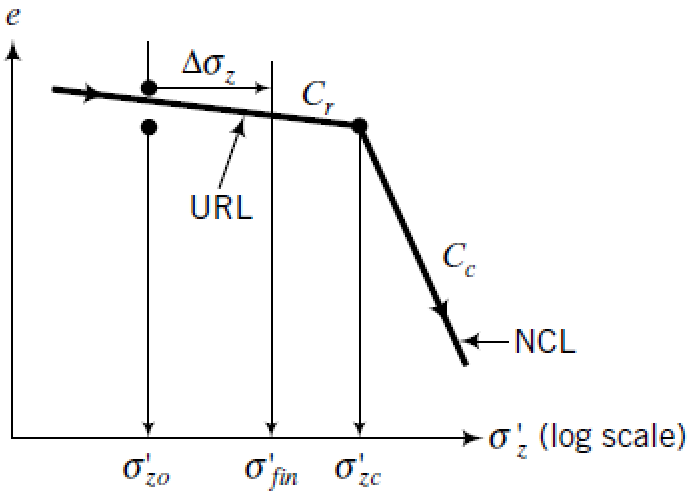
The soil profile at a site for a proposed office building consists of a layer of fine sand 10.4 m thick above a layer of soft, normally consolidated clay 2 m thick. Below the soft clay is a deposit of coarse sand. The groundwater table was observed at 3 m below ground level. The void ratio of the sand is 0.76 and the water content of the clay is 43%. The building will impose a vertical stress increase of 140 kPa at the middle of the clay layer. Estimate the primary consolidation settlement of the clay. Assume the soil above the water table to be saturated, C_c=0.3, and Gs=2.7.

Consolidation Settlement cntd

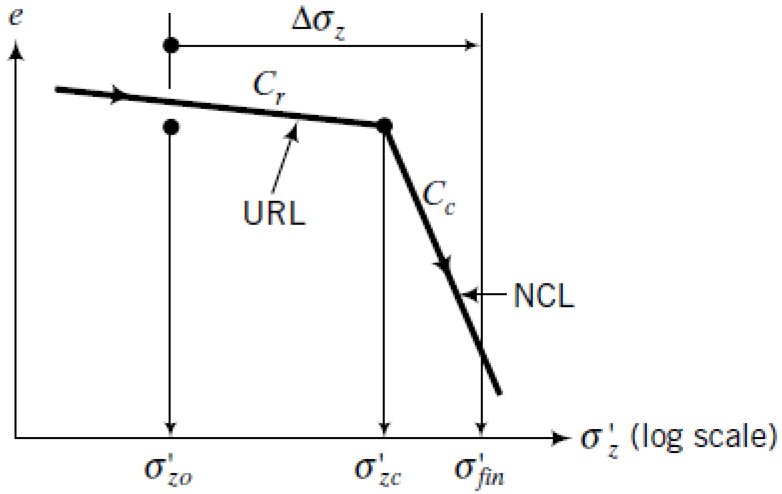
Primary Consolidation of OC Clay

If the soil is overconsolidated, we have to consider two cases depending on the magnitude of $\boldsymbol{\Delta\sigma_z}$.

We will approximate the e-logp curve as two straight lines, as shown in below.



(a) Case 1: $\sigma'_{zo} + \Delta\sigma_z < \sigma'_{zc}$



(b) Case 2: $\sigma'_{zo} + \Delta\sigma_z > \sigma'_{zc}$

Consolidation Settlement cntd

In Case 1, the increase in $\boldsymbol{\Delta\sigma_z}$ is such $\boldsymbol{\sigma'_{zo} + \Delta\sigma_z < \sigma'_{zc}}$. In this case, consolidation occurs along the URL and

$$\boldsymbol{S_c = C_r \frac{H_o}{1 + e_o} log \frac{\sigma'_{zo} + \Delta\sigma_z}{\sigma'_{zo}}}$$

In Case 2, the increase in $\boldsymbol{\Delta\sigma_z}$ is such $\boldsymbol{\sigma'_{zo} + \Delta\sigma_z > \sigma'_{zc}}$. In this case, we have to consider two components of settlement—one along the URL and the other along the NCL.

$$\boldsymbol{S_c = \frac{H_o}{1 + e_o}\left(C_r \, log \frac{\sigma'_{zc}}{\sigma'_{zo}} + C_c \, log \frac{\sigma'_{zo} + \Delta\sigma_z}{\sigma'_{zc}}\right)}$$

EXERCISE 1.3.4 – S_c of OC Clay

Assume the same soil stratigraphy as in EXERCISE 1.3.3. But now the clay is overconsolidated with an OCR=2.5, $\omega = 38\%$, and $C_r = 0.05$.

All other soil values given in EXERCISE 1.3.3 remain unchanged. Determine the primary consolidation settlement of the clay.

EXERCISE 1.3.5 – S_c of Slightly OC Clay

Assume the same soil stratigraphy as in EXERCISE 1.3.4 except that the clay has an overconsolidation ratio of 1.5. Determine the primary consolidation settlement of the clay.

Consolidation Settlement cntd

Primary Consolidation Calculation using $\boldsymbol{m_v}$

Unlike $\boldsymbol{C_c}$, which is constant, $\boldsymbol{m_v}$ varies with stress levels. One should compute an average value of $\boldsymbol{m_v}$ over the stress range $\boldsymbol{\sigma'_{zo}}$ to $(\boldsymbol{\sigma'_{zo}} + \boldsymbol{\Delta\sigma_z})$.

To reduce the effects of nonlinearity, the vertical effective stress difference should not exceed 100 kPa in calculating $\boldsymbol{m_v}$ or $\boldsymbol{m_{vr}}$.

The primary consolidation settlement, using $\boldsymbol{m_v}$, is

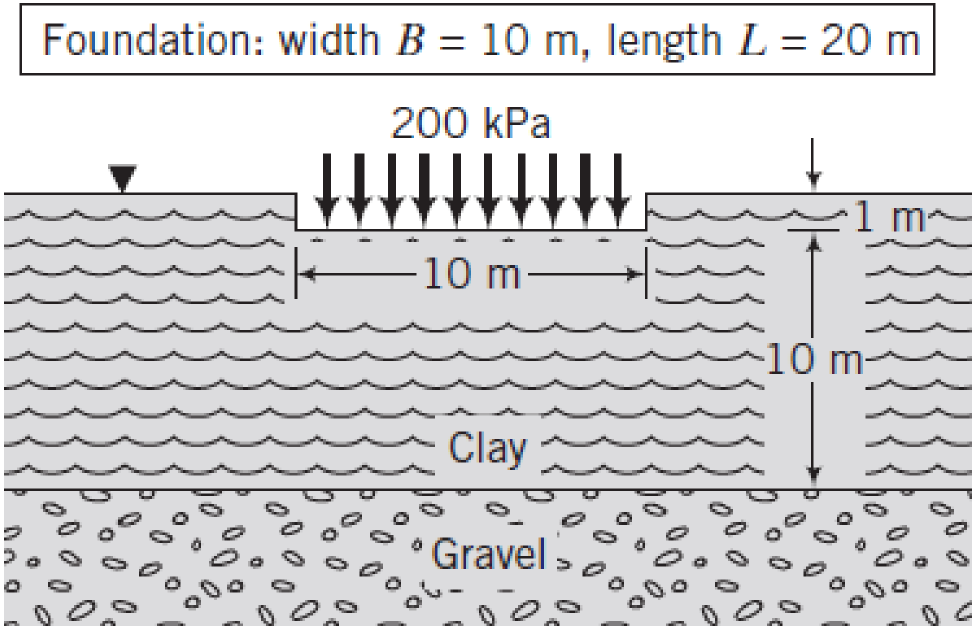
$$\boldsymbol{S_c = H_o m_v \Delta\sigma_z}$$

NB. $\boldsymbol{m_v}$ is readily determined from displacement data in consolidation tests; one do not have to calculate void ratio changes from the test data as required to determine $\boldsymbol{C_c}$

Consolidation Settlement cntd

EXERCISE 1.3.5 – S_c using m_v

A vertical section through a building foundation at a site is shown in the figure below. The average modulus of volume compressibility of the clay is $m_v = 5 \times 10^{-5}\ m^2/kN$. Determine the primary consolidation settlement.



4. Secondary Compression

- Introduction
- Calculation of Secondary Consolidation
- Hypotheses on Creep

Secondary Compression

-observed in saturated cohesive soils and is the result of the plastic adjustment of soil fabrics.

-an additional form of compression that occurs at constant effective stress

- Volume changes that are more or less independent of the excess pore water pressure values cause secondary compression.
- The nature of these changes is not fully understood but they are apparently due to a form of plastic flow resulting in a displacement of the soil particles.

Secondary Compression cntd

- Secondary compression effects can continue over long periods of time and, in the consolidation test, become apparent towards the end of the primary compression stage: due to the thinness of the sample, the excess pore water pressures are soon dissipated and it may appear that the main part of secondary compression occurs after primary compression is completed.
- This effect is absent in the case of an *in situ* clay layer because the large dimensions involved mean that a considerable time is required before the excess pore pressures drain away.

Secondary Compression cntd

- During this time the effects of secondary compression are also taking place so that, when primary compression is complete, little, if any, secondary effect is noticeable.
- The terms 'primary' and 'secondary' are therefore seen to be rather arbitrary divisions of the single, continuous consolidation process.
- The time relationships of these two factors will be entirely different if they are obtained from two test samples of different thicknesses.

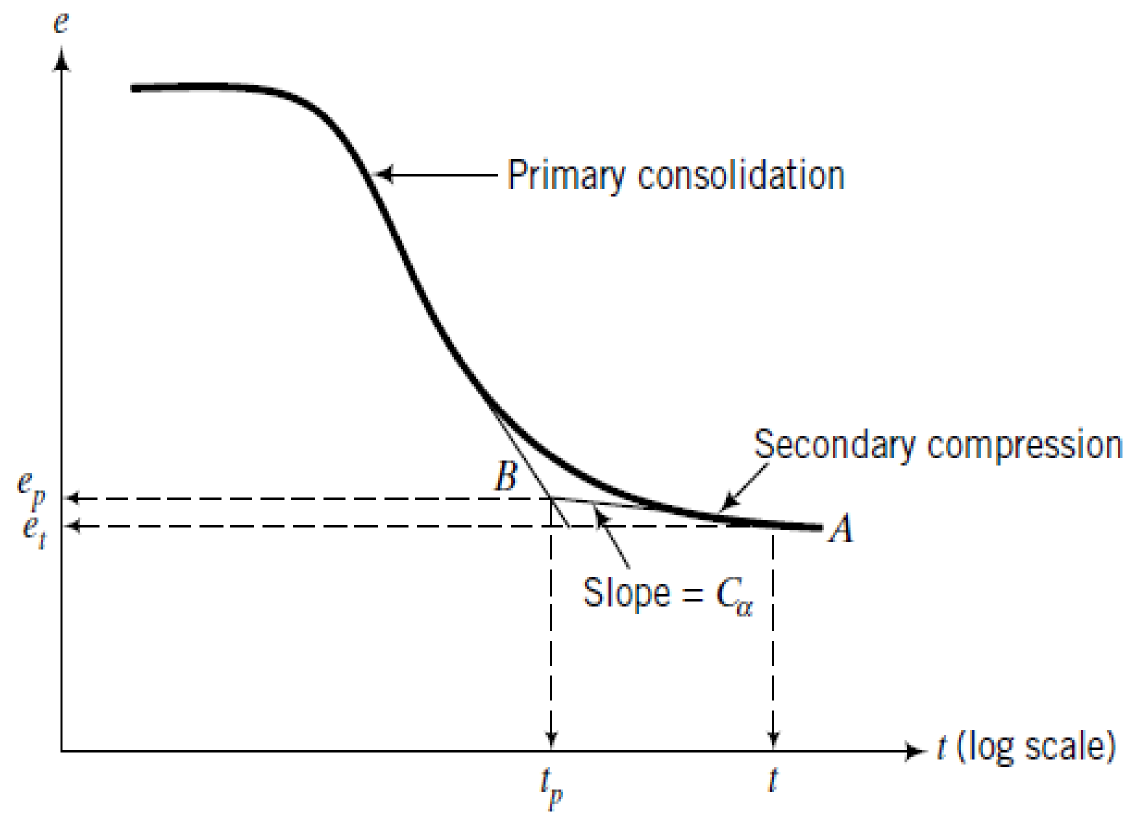
Secondary Compression cntd

- Consolidation settlement consists of two parts.
  - The first part is primary consolidation, which occurs at early times.
  - The second part is secondary compression, or creep, which takes place under a constant vertical effective stress.
- The physical reasons for secondary compression in soils are not fully understood.
  - One plausible explanation is the expulsion of water from micropores; another is viscous deformation of the soil structure.

Secondary Compression cntd

One can make a plot of void ratio versus the logarithm of time from oedometer test as shown here.

Primary consolidation is assumed to end at the intersection of the projection of the two straight parts of the curve.



Secondary Compression cntd

The secondary compression index is

$$C_\alpha = -\frac{(e_t - e_p)}{log(t/t_p)} = \frac{|\Delta e|}{log(t/t_p)}; t > t_p$$

where (t_p, e_p) is the coordinate of the intersection of the tangents to the primary consolidation and secondary compression parts of the logarithm of time versus void ratio curve, and (t, e_t) is the coordinate of any point on the secondary compression curve.

The secondary consolidation settlement is

$$S_s = \frac{H_o}{1 + e_o} C_\alpha \log\left(\frac{t}{t_p}\right)$$

Secondary Compression cntd

EXERCISE 1.4.1 – Creep

A borehole at a site for a proposed building reveals the following soil profile:

0–5 m	Dense sand, $\gamma = 18$ kN/m^3, $\gamma_{sat} = 19$ kN/m^3
At 4 m	Groundwater level
5–10 m	Soft, normally consolidated clay, $\gamma_{sat} = 17.5$ kN/m^3
Below 10 m	Impervious rock

A building is to be constructed on this site with its foundation at 2 m below ground level. The building load is 30 MN and the foundation is rectangular with a width of 10 m and length of 15 m.

Secondary Compression cntd

EXERCISE 1.4.1 – Creep

A sample of the clay was tested in an oedometer, and the following results were obtained:

Vertical stress (kPa)	50	100	200	400	800
Void ratio	0.945	0.895	0.815	0.750	0.705

Calculate the primary consolidation settlement.

Assuming that the primary consolidation took 5 years to achieve in the field, calculate the secondary compression for a period of 10 years beyond primary consolidation.

->The secondary compression index is Cc/6.

Secondary Compression cntd

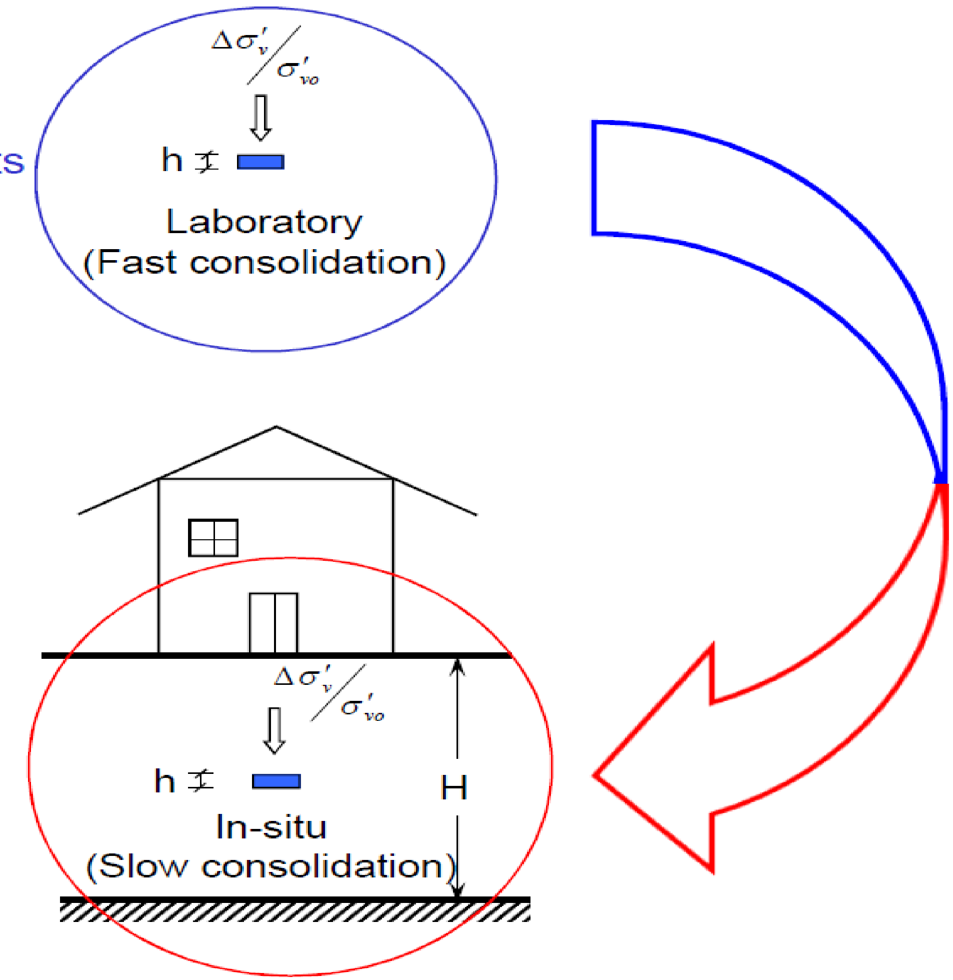
Hyphotheses on Creep

General

Measurements

Significantly different consolidation times

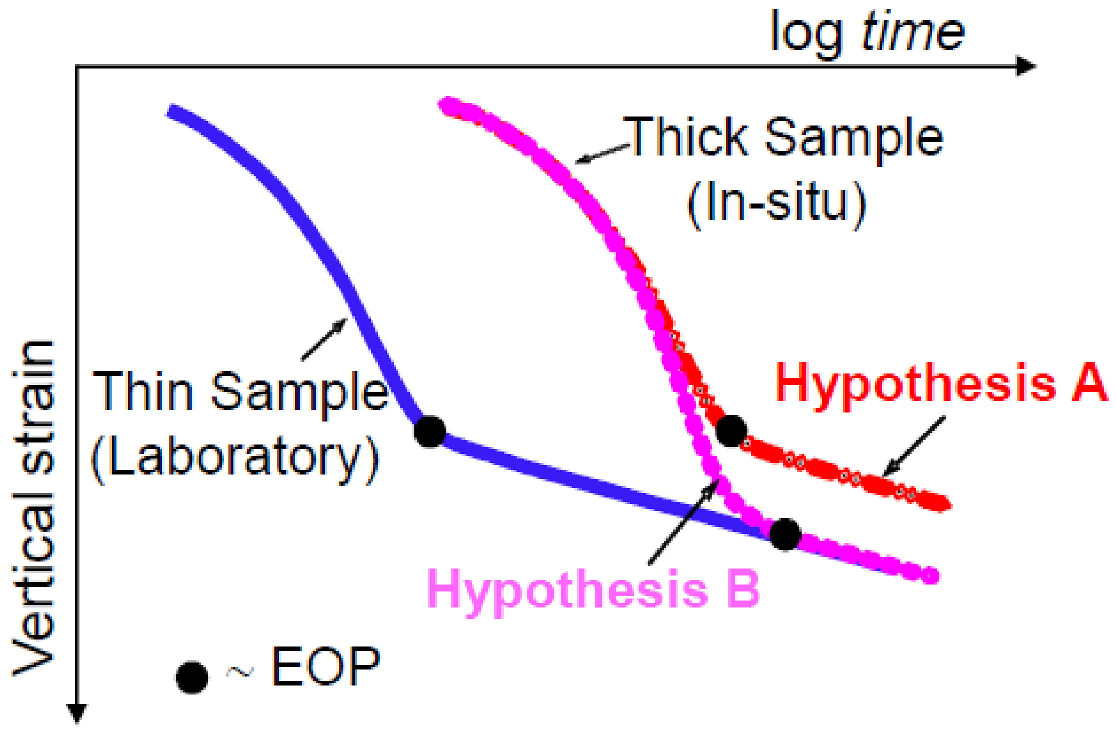
Prediction



Secondary Compression cntd

Hyphotheses - The Question

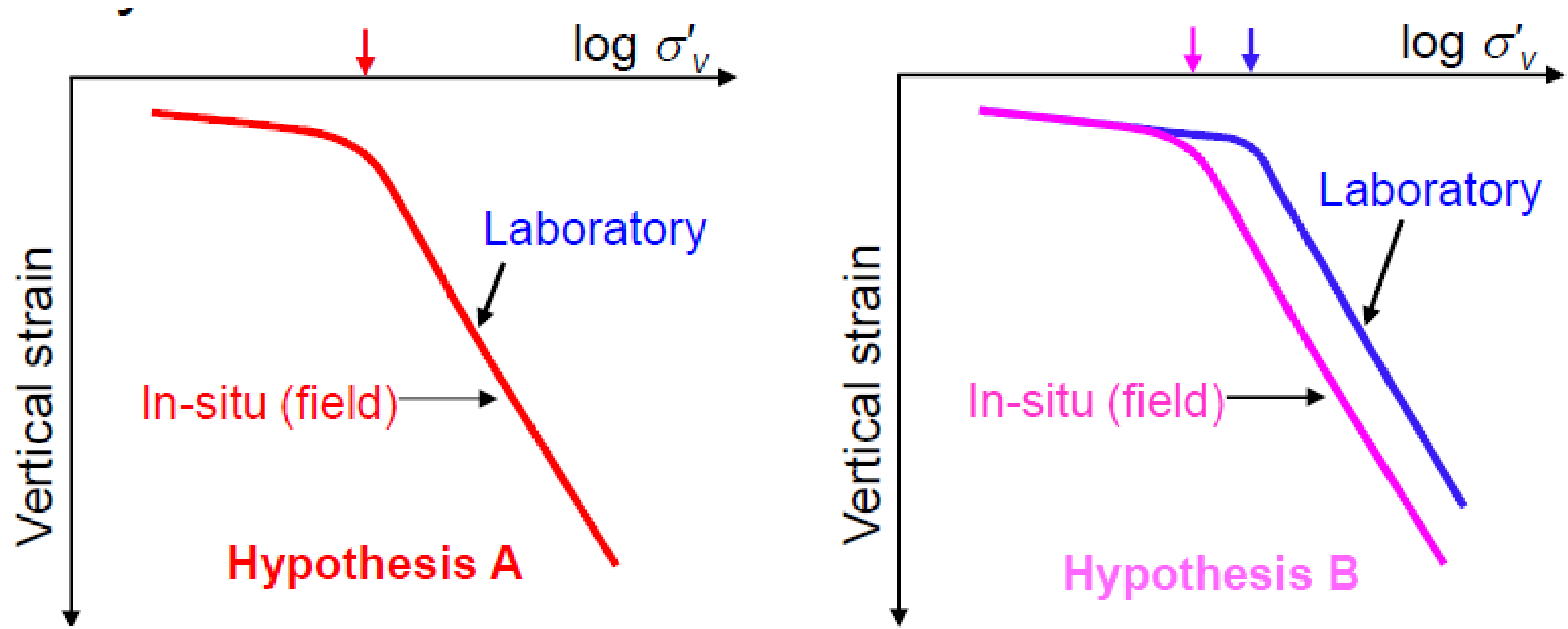
"Does creep act as a separate phenomenon while excess pore pressures dissipate during primary consolidation?"



Secondary Compression cntd

Hyphotheses - The Answer

“Does creep act as a separate phenomenon while excess pore pressures dissipate during primary consolidation?”



5. Rate of Consolidation

- Some Terms
- Terzaghi & Froelich's Theory of Consolidation
- Rate of Consolidation Parameters

Rate of Consolidation

Drainage Path

- The distance of the longest vertical path taken by a particle to exit the soil is called the length of the drainage path.

- If we allow the soil to drain on the top and bottom faces (double drainage), the length of the drainage path, Hdr, is

$$H_{dr} = \frac{H_{av}}{2} = \frac{H_o + H_f}{4}$$

where H_{av} is the average height and H_o and H_f are the initial and final heights, respectively, under the current loading.

Rate of Consolidation cntd

Drainage Path

- If drainage is permitted from only one face of the soil, then $H_{dr} = H_{av}$.
- Shorter drainage paths will cause the soil to complete its settlement in a shorter time than a longer drainage path.
- One can see later that, for single drainage, a soil sample will take four times longer to reach a particular settlement than for double drainage.

Rate of Consolidation cntd

Rate of Consolidation

- The rate of consolidation for a homogeneous soil depends on the soil's hydraulic conductivity (permeability), the thickness, and the length of the drainage path.
- A soil with a hydraulic conductivity lower than that of our current soil will take longer to drain the initial excess porewater, and settlement will proceed at a slower rate.

Rate of Consolidation cntd

- The rate at which the water can escape depends upon the permeability of the soil. In clay soils where permeability is low, there is a time lag between the application of load and the change in volume.

- The change in volume corresponds to a change in the amount of pores in the soil. Important to measure the rate at which the void ratio changes and the final amount of consolidation.

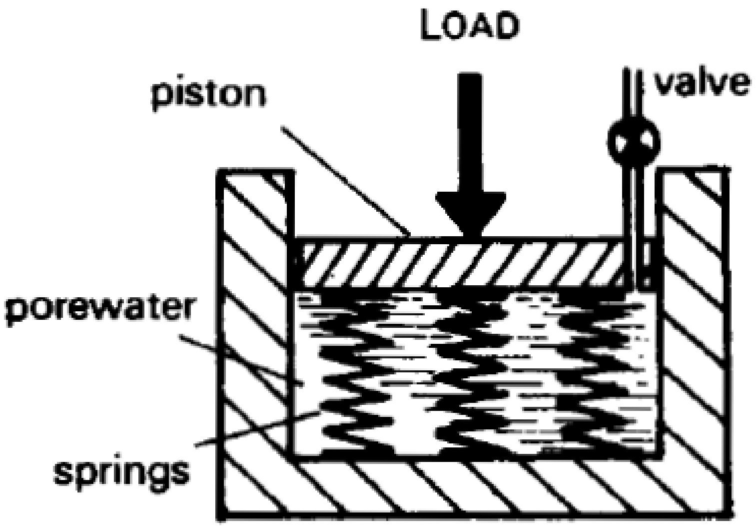
Rate of Consolidation cntd

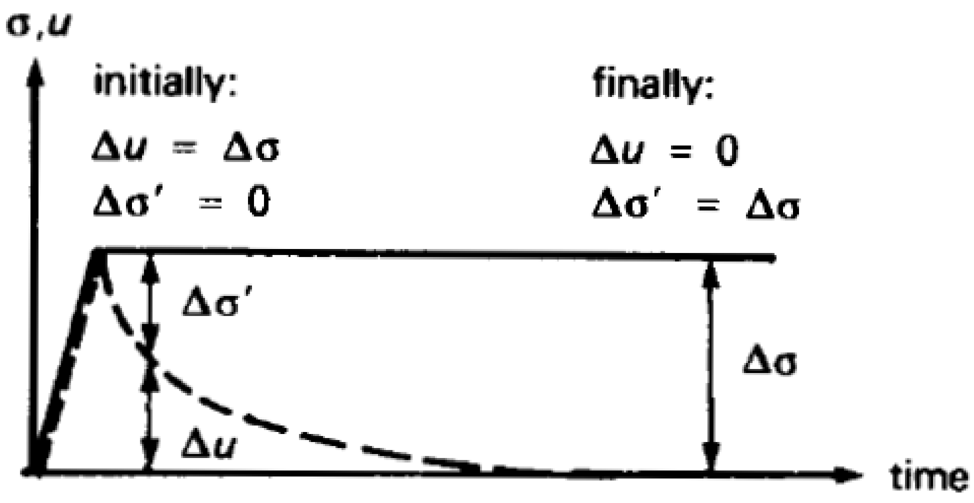
Terzaghi & Froelich's Theory of Consolidation

-A model to explian the mechanics of compression of soils

-Assumed that both the mineral grains and the pore water are incompressible.

-Steel springs represent the soil solid. The frictionless piston is supported by springs and the cyllinder filled with water.





Rate of Consolidation cntd

Soil	Spring Model
• The initial increase in total stress (upon loading) is fully attributed to an increase in porewater pressure	• Load applied to piston with valves closed, length of springs remain unchanged, induced increase in total stress taken wholly by an equal increase in the water.
• As time progresses, the porewater seeps out of the soil, the increase in porewater pressure is dissipated. When the whole excess pore water pressure has been dissipated the soil is fully consolidated.	• Valve opened; excess water pressure cause the water to flow out; water pressure decreases & piston sinks as springs are compressed; load gradually transferred to the springs causing them to shorten until all the load is carried by the springs.
• The rate of compression depends on the permeability of the soil.	• The rate of compression depends on the extent to which the valve is opened.

Rate of Consolidation cntd

Derivation of Governing Equation

- **Purpose:** to derive the theory for time rate of settlement using an element of the soil sample of thickness dz and cross-sectional area $dA = dx\ dy$.
- **Assumptions:**
  - Soil is saturated, isotropic, homogeneous.
  - Darcy's law is valid.
  - Flow only occurs vertically.
  - The strains are small.

Rate of Consolidation cntd

Some important observations from experiments

- The change in volume of the soil (ΔV) is equal to the change in volume of porewater expelled (ΔV_w), which is equal to the change in the volume of the voids (ΔV_w). Since the area of the soil is constant (the soil is laterally constrained), the change in volume is directly proportional to the change in height.
- At any depth, the change in vertical effective stress is equal to the change in excess porewater pressure at that depth. That is, $\boldsymbol{\partial \sigma'_z = \partial u}$

Rate of Consolidation cntd

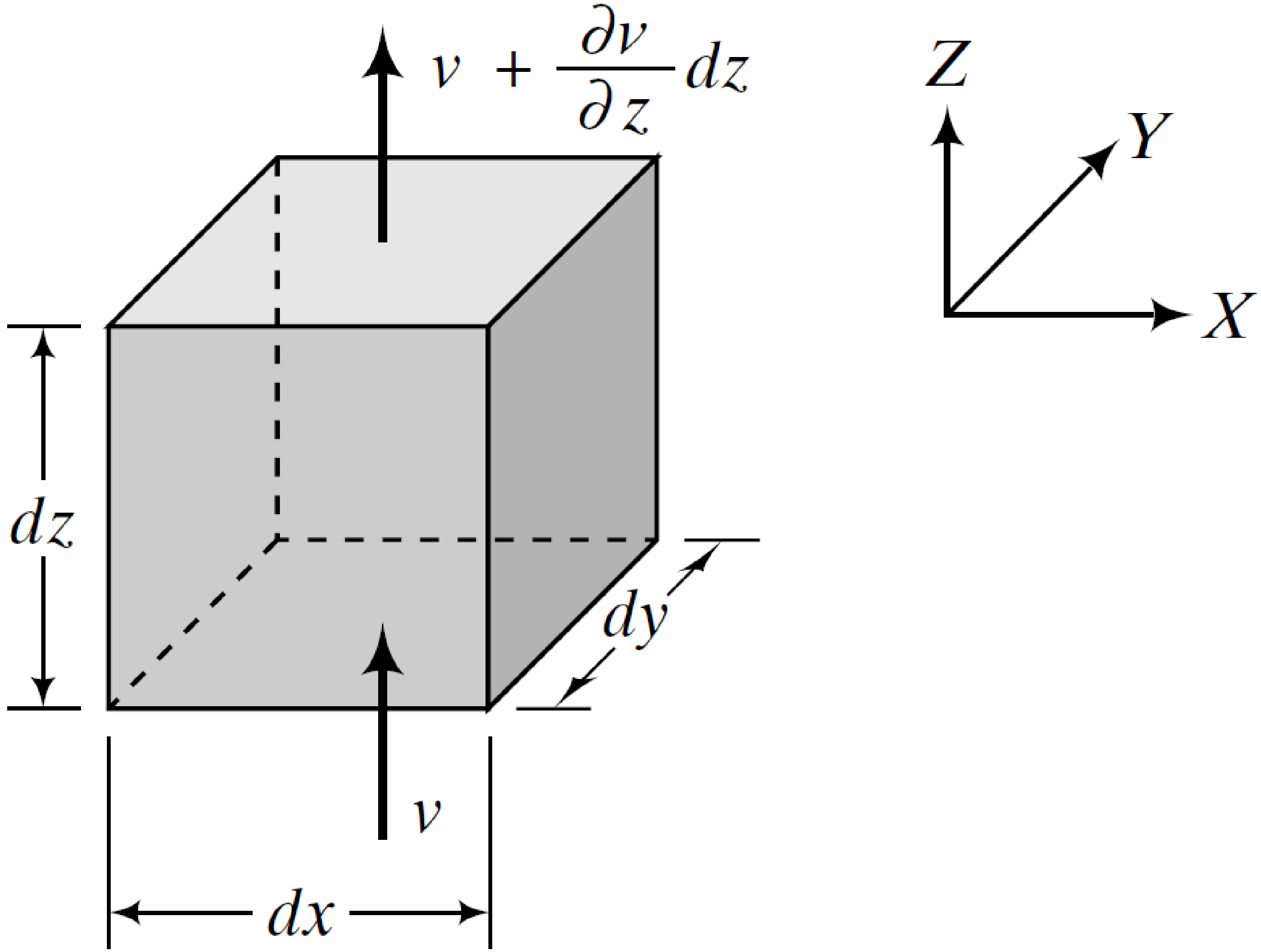


Figure: One-dimensional flow through two-dimensional soil element

Rate of Consolidation cntd

Consider the soil element in previous slide

- The inflow of water is $v\ dA$ and the outflow over the elemental thickness dz is $[v + (\partial v/\partial z)]dA$.
- The flow rate is the product of the velocity and the cross-sectional area normal to its direction. The change in flow is then $(\partial v/\partial z)dzdA$.
- The rate of change in volume of water expelled, which is equal to the rate of change of volume of the soil, must equal the change in flow.

That is, $\frac{\partial V}{\partial t} = \frac{\partial v}{\partial z} dzdA$

Rate of Consolidation cntd

- The volumetric strain is $\varepsilon_p = \partial V/V = \partial e/(1+e_o)$

$$\partial V = \frac{\partial e}{1+e_o} dzdA = m_v \partial \sigma'_z dzdA = m_v \partial u dzdA$$

$$\frac{\partial V}{\partial z} = \frac{\partial u}{\partial t} m_v$$

From Darcy's law, the 1D flow of water is

$$v = k_z t = k_z \frac{\partial h}{\partial z}$$

where k_z is the hydraulic conductivity in the vertical direction.

Rate of Consolidation cntd

- Partial differential equation wrt z

$$\frac{\partial v}{\partial z} = k_z \frac{\partial^2 h}{\partial z^2}$$

The pore water pressure at any time in our experiment is

$$u = h\gamma_w$$

where h is the height of water in the burette.

- Partial differential equation wrt z

$$\frac{\partial^2 h}{\partial z^2} = \frac{1}{\gamma_w} \frac{\partial^2 u}{\partial z^2}$$

Rate of Consolidation cntd

- Through substitution

$$\frac{\partial v}{\partial z} = \frac{k_z}{\gamma_w}\frac{\partial^2 u}{\partial z^2}$$

Equating

$$\frac{\partial u}{\partial z} = \frac{k_z}{m_v \gamma_w}\frac{\partial^2 u}{\partial z^2}$$

Let $\frac{k_z}{m_v \gamma_w} = C_v$ with units like cm^2/min

$$\frac{\partial u}{\partial z} = C_v \frac{\partial^2 u}{\partial z^2}$$

Terzaghi-Froelich 1D Consolidation Equation

Rate of Consolidation cntd

- In the derivation of the equation we tacitly assumed that k_z and m_v are constants.
- This is usually not the case because as the soil consolidates, the void spaces are reduced and k_z decreases. Also m_v is not linearly related to σ'_z.

k_z and m_v are constants → C_v is not constant.

- In practice, C_v is assumed to be constant, and this assumption is reasonable only if the stress changes are small enough such that k_z and m_v do not change significantly.

Rate of Consolidation cntd

Solution to the Governing Equation

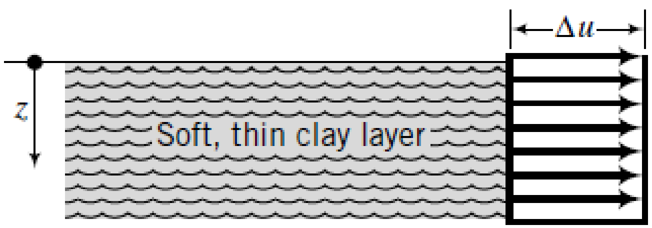
- requires knowdledge of the boundary condition
- can be obtained using Fourier Series or Finite Diffrence Method
- can be found by specification of the initial distribution of excess porewater pressures at the boundaries

i.e we can obtain solutions for the spatial variation of excess porewater pressure with time and depth.

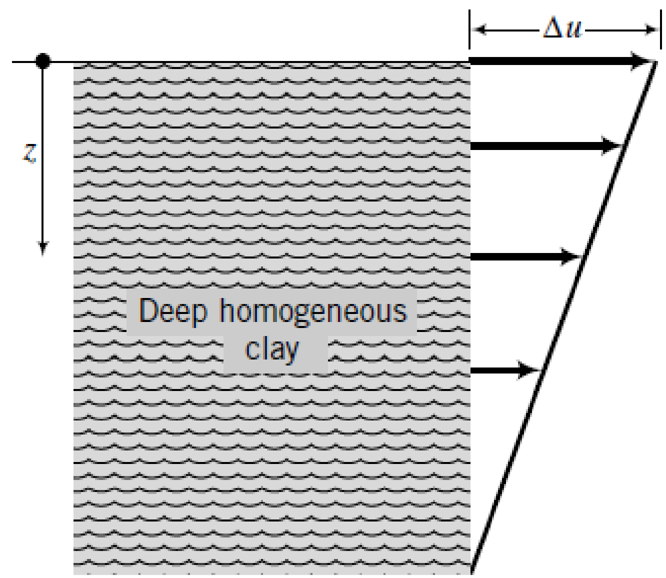
Rate of Consolidation cntd

Various distributions of porewater pressures within a soil layer are possible.

- uniform distribution - occur in a thin layer of fine-grained soils.
- triangular distribution - occur in a thick layer of fine-grained soils.



(*a*) Uniform distribution



(*b*) Triangular distribution

Rate of Consolidation cntd

$$\Delta u(z,t) = \sum_{m=0}^{\infty} \frac{2\Delta u_o}{M} \sin(\frac{M_z}{H_{dr}}) \exp(-M^2 T_v)$$

where $M = (\pi/2)(2m+1)$ and m is a positive integer with values from 0 to ∞ and

$$T_v = \frac{C_v t}{H_{dr}^2}$$

where T_v is known as the time factor; it is a dimensionless term.

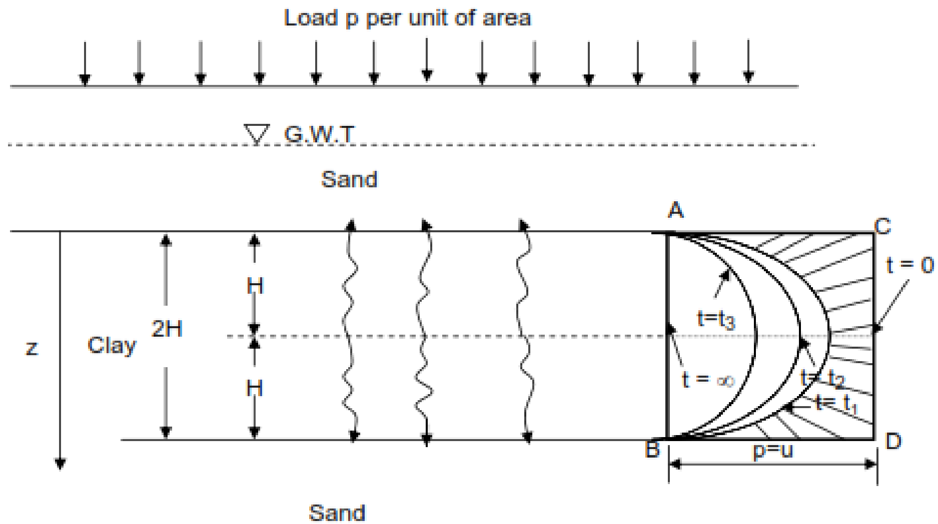
$$T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 \; for\; U < 60\%$$

$$T_v = 1.781 - 0.933 \log(100 - U) \;\; for\; U \geq 60\%$$

Rate of Consolidation cntd

Thickness of the clay layer	Influences the distances through which the water in the soil voids must travel.
Number of drainage face	Influences the distances through which the water in the soil voids must travel.
Permeability of the soil	Controls the rate at which the water can escape
Magnitude of the consolidating pressure acting on the layer	Influences the hydrostatic excess pressure, which causes the out flow of water.

U (%)	T	U (%)	T
0	0	55	0.238
10	0.008	60	0.287
15	0.018	65	0.342
20	0.031	70	0.405
25	0.049	75	0.477
30	0.071	80	0.565
35	0.096	85	0.684
40	0.126	90	0.848
45	0.159	95	1.127
50	0.197	100	∞



- Boundary conditions
  - u = 0 at z = 0
  - u = 0 at z = 2H
  - At t = 0, u = uo = p

6. 1D Consolidation Test

- Basis
- Apparatus
- Procedures
- Outputs
- Interpretations
- Relations with Field Consolidations
- Empirical Relations for Consolidation

1D Consolidation Test

Basis

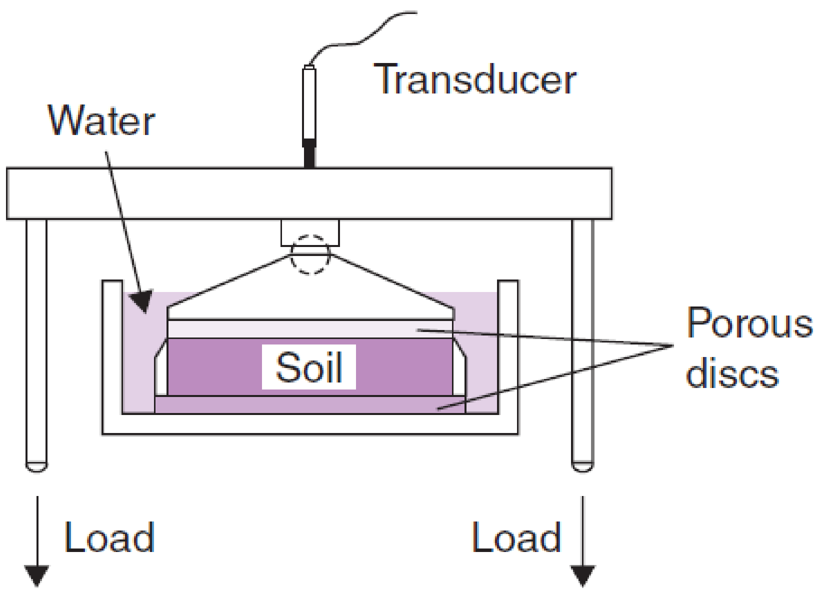
- The pore water in a saturated clay will commence to drain away soon after immediate settlement has taken place: the removal of this water leading to the volume change is known as consolidation.
- The element contracts both horizontally and vertically under the actions of σ_3 and $\Delta\sigma_1$, which gradually increase in magnitude as the excess pore water pressure, Δu, decreases. Eventually, when $\Delta u = 0$, then $\sigma_3 = \sigma'_3$ and $\sigma_1 = \sigma'_1$, and at this stage consolidation ceases, although secondary consolidation may still be apparent.

1D Consolidation Test cntd

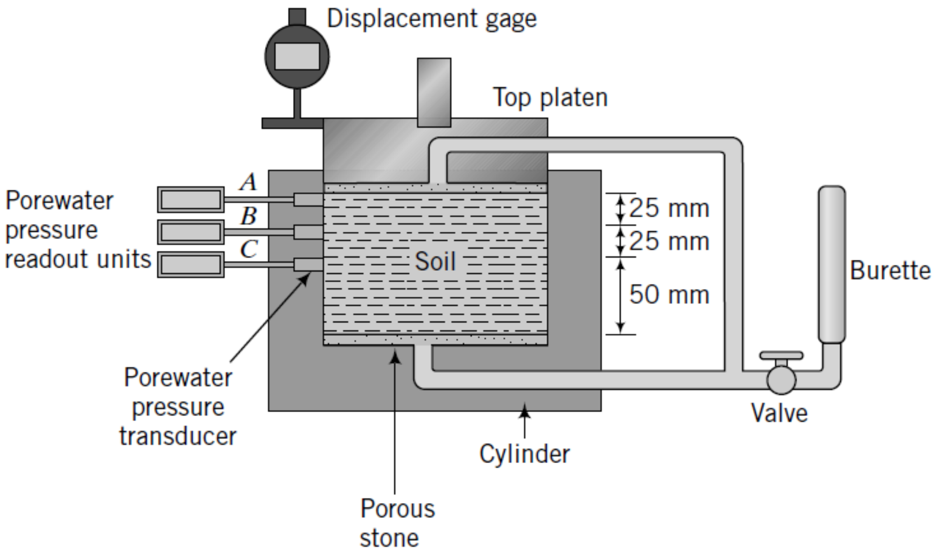
- If it can be arranged for the lateral expansion due to the change in shape to equal the lateral compression consequent upon the change in volume, and for these changes to occur together, then there will be no immediate settlement and the resulting compression will be one-dimensional with all the strain occurring in the vertical direction.
- Settlement by one-dimensional strain is by no means uncommon in practice, and most natural soil deposits have experienced one-dimensional settlement during the process of deposition and consolidation.

Apparatus

- The apparatus generally used in the laboratory to determine the primary compression characteristics of a soil is known as consolidomter (or oedometer)



(a) Consolidation apparatus



(b) Experimental setup for illustrating basic concepts of consolidation.

1D Consolidation Test cntd

Procedures

- The soil sample (generally 75 mm diameter and 20 mm thick) is encased in a steel cutting ring.
- Porous discs, saturated with air-free water, are placed on top of and below the sample, which is then inserted in the oedometer.
- A vertical load is then applied and the resulting compression measured by means of a transducer at intervals of time, readings being logged until the sample has achieved full consolidation (usually for a period of 24 hours).

1D Consolidation Test cntd

Procedures

- Further load increments are then applied and the procedure repeated, until the full stress range expected in situ has been covered by the test.
- The test sample is generally flooded with water soon after the application of the first load increment in order to prevent pore suction.
- After the sample has consolidated under its final load increment the pressure is released in stages at 24 hour intervals and the sample allowed to expand. In this way an expansion to time curve can also be obtained.

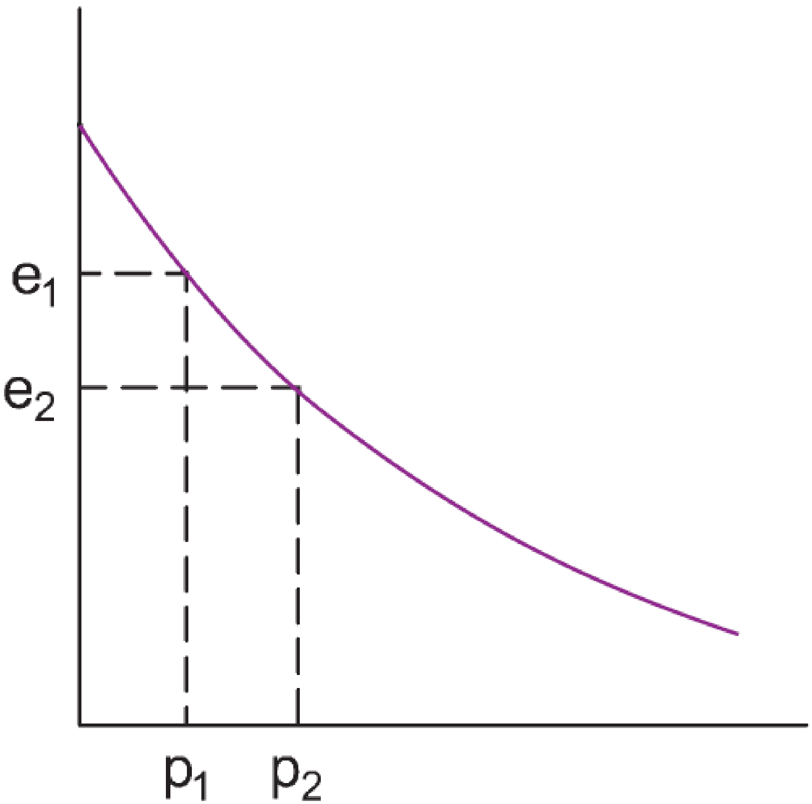
1D Consolidation Test cntd

Procedures

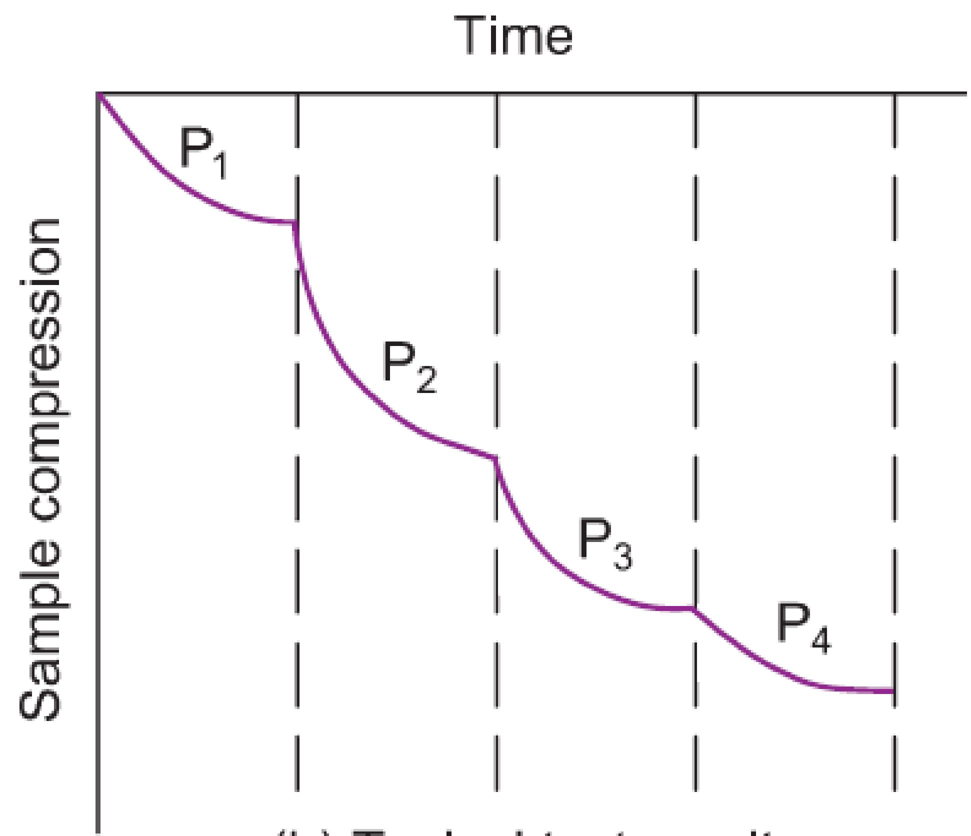
- After the loading has been completely removed the final thickness of the sample can be obtained, from which it is possible to calculate the void ratio of the soil for each stage of consolidation under the load increments. The graph of void ratio to consolidation pressure can then be drawn, such a curve generally being referred to as an e–p curve.
- It should be noted that the values of p refer to effective stress, for after consolidation the excess pore pressures become zero and the applied stress increment is equal to the effective stress increment.

1D Consolidation Test cntd

Outputs



(a) Typical e-p curve



(b) Typical test results

1D Consolidation Test cntd

- The assumption is that the porewater and the soil particles are incompressible, and the initial porewater pressure is zero.
- The volume of excess porewater that drains from the soil is then a measure of the volume change of the soil resulting from the applied loads.
- Since the side wall of the container is rigid, no radial displacement can occur. The lateral and circumferential strains are then equal to zero ($\varepsilon_r = \varepsilon_\theta = 0$).

1D Consolidation Test cntd

- The volumetric strain ($\varepsilon_p = \varepsilon_z + \varepsilon_\theta + \varepsilon_r$) is equal to the vertical strain, $\varepsilon_z = \Delta z / H_o$ where Δz is the change in height or thickness and H_o is the initial height or thickness of the soil.

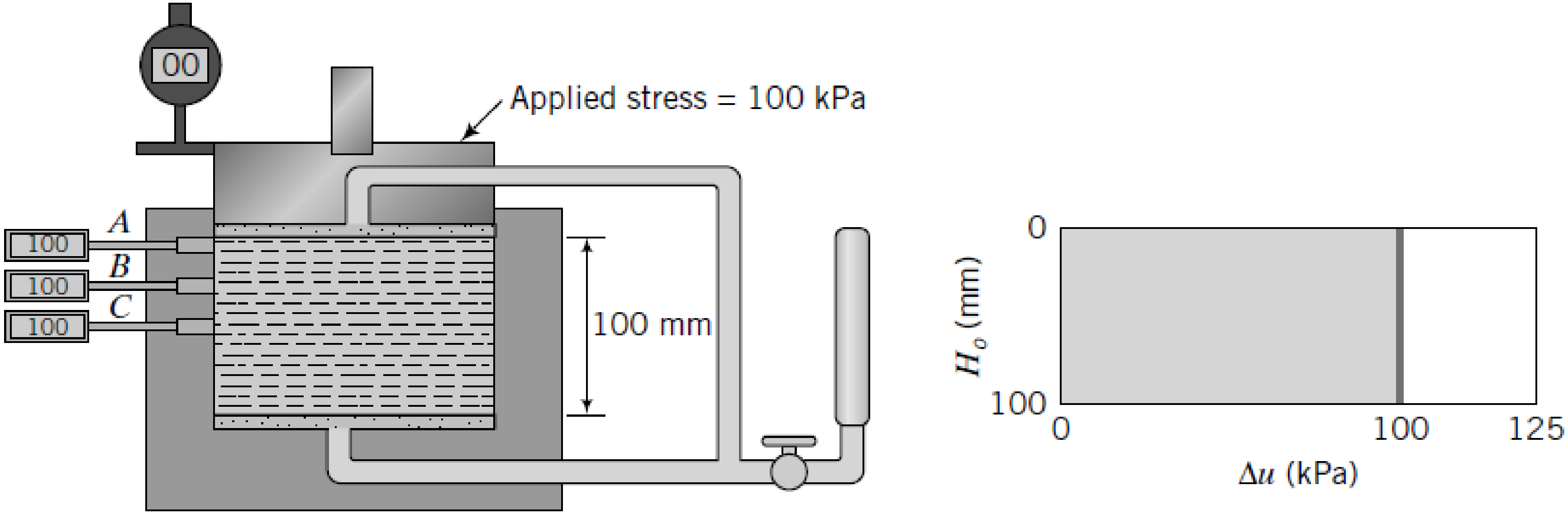


Fig. Instantaneous or initial excess porewater pressure when a vertical load is applied

1D Consolidation Test cntd

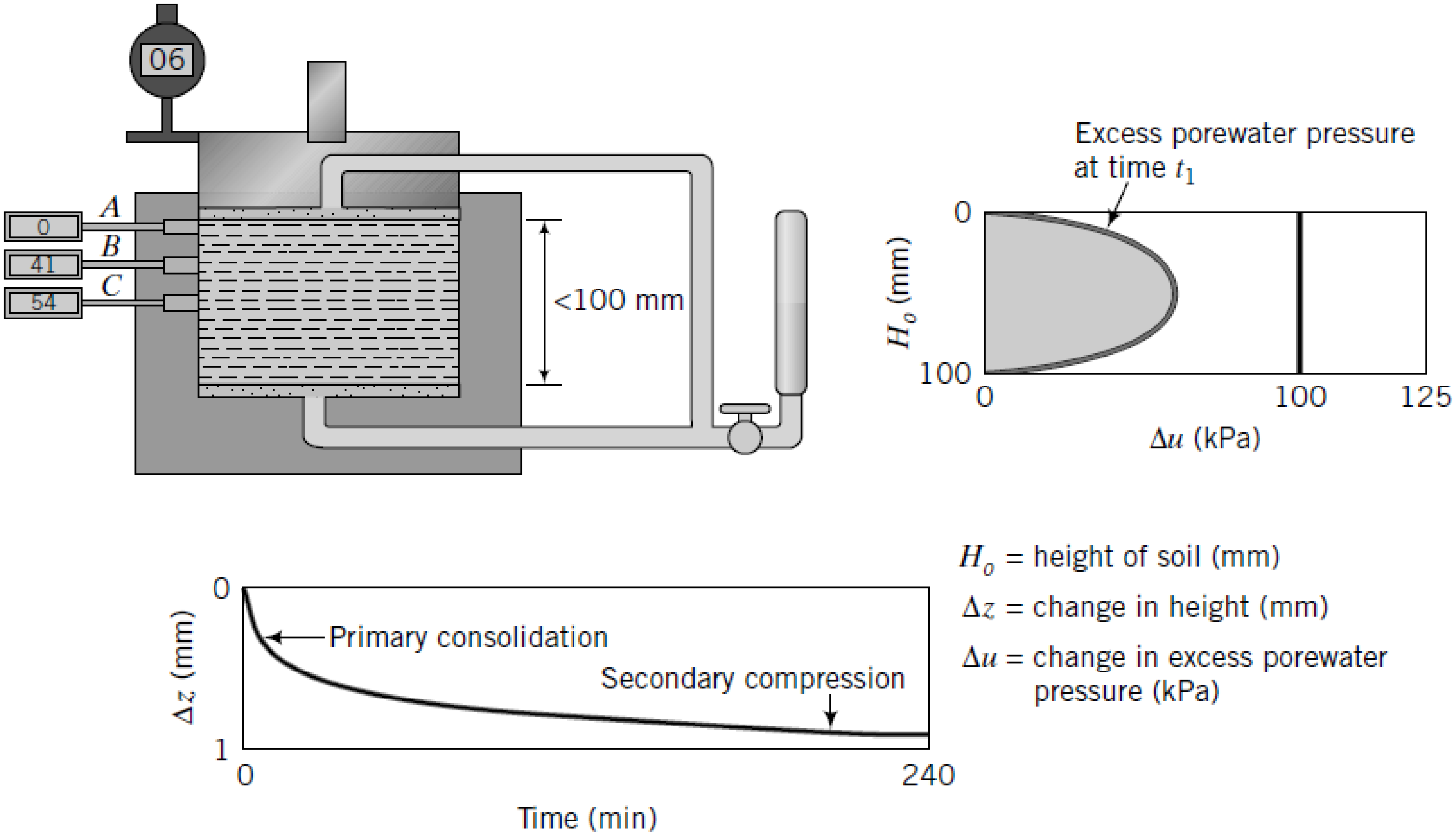


Fig. Excess porewater pressure distribution and settlement during consolidation

1D Consolidation Test cntd

Primary Consolidation in 1D Consolidation Test

- When we open the valve and allow the initial excess porewater to drain, the total volume of soil at time $t1$ decreases by the amount of excess porewater that drains from it.
- At the top and bottom of the soil sample, the excess porewater pressure is zero because these are the drainage boundaries.
- The decrease of initial excess porewater pressure at the middle of soil is the slowest because a water particle must travel from the middle of soil to either the top or bottom boundary to exit the system.

1D Consolidation Test cntd

- Most of the settlement occurs shortly after the valve is opened. The rate of settlement, $\Delta z/t$, is also much faster soon after the valve is opened compared with later times.
- Before the valve is opened, an initial hydraulic head, $\Delta u_o/\gamma_w$, is created by the applied vertical stress. When the valve is opened, the initial excess porewater is forced out of the soil by this initial hydraulic head.
- With time, the initial hydraulic head decreases and, consequently, smaller amounts of excess porewater are forced out.

1D Consolidation Test cntd

- An analogy can be drawn with a pipe containing pressurized water that is ruptured. A large volume of water gushes out as soon as the pipe is ruptured, but soon after, the flow becomes substantially reduced.
- We will call the initial settlement response soon after the valve is opened the early time response, or primary consolidation.
- Primary consolidation is the change in volume of the soil caused by the expulsion of water from the voids and the transfer of load from the excess porewater pressure to the soil particles.

1D Consolidation Test cntd

Secondary Consolidation in 1D Consolidation Test

- Theoretically, primary consolidation ends when $\Delta u_o = 0$. The later time settlement response is called secondary compression, or creep.
- Secondary compression is the change in volume of a fine-grained soil caused by the adjustment of the soil fabric (internal structure) after primary consolidation has been completed.
- The term "consolidation" is reserved for the process in which settlement of a soil occurs from changes in effective stresses resulting from decreases in excess porewater pressure.

1D Consolidation Test cntd

Secondary Consolidation in 1D Consolidation Test

- The rate of settlement from secondary compression is very slow compared with that from primary consolidation.
- We have separated primary consolidation and secondary compression. In reality, the distinction is not clear because secondary compression occurs as part of the primary consolidation phase, especially in soft clays. The mechanics of consolidation is still not fully understood, and to make estimates of settlement it is convenient to separate primary consolidation and secondary compression.

1D Consolidation Test cntd

1D Consolidation Tests - Summary

- Measures the **rate** and **amount of volume change** with the application of load on a laboratory specimen.
- The test is performed in a **consolidometer** also called and **oedometer**.
- The soil sample is fully submerged in water.
- Loads are applied in steps in such a way that the successive load intensity is twice the preceding one.
- Each load is allowed to stand until compression has practically ceased.
- The time elapsed is also measured.

1D Consolidation Test cntd

Some of the simple parameters determined (needed)

- Moisture content and weight of the sample before test.
- Moisture content and weight of the sample after test.
- Specific gravity of the solids.

Load intensities commonly used: ¼, ½, 1, 2, 4, 8, 16kg/cm^2

Elapsed time:

0, ¼, 1, 2 ¼, 4, 6 ¼, 9, 12 ¼, 16, 20 ¼, 25min……24hrs

- **Outputs**
  - ✓ Time-deformation curve
  - ✓ Void ratio-pressure plot

1D Consolidation Test cntd

Pressure Void Ratio Curves (e-p curve)

- To present the results of the consolidation test graphically, void ratio, e Vs pressure, p is plotted. If the pressure is plotted to a logarithmic scale the result is called an e-logp curve.
- In 1D compression the change in height Δh per unit of original height ho equals the change in volume ΔV per unit of original volume Vo.

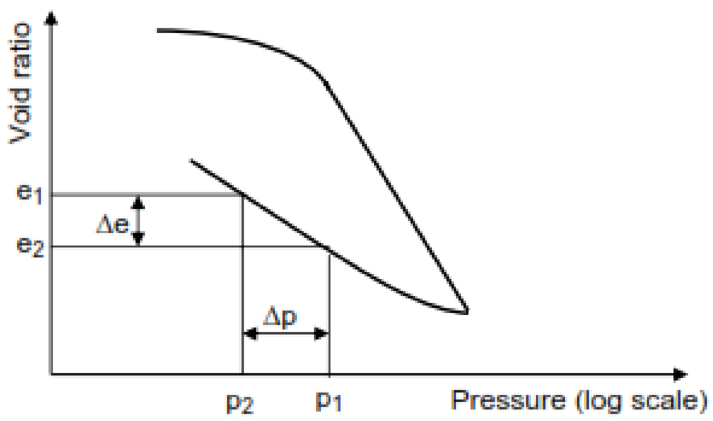
1D Consolidation Test cntd

- **Swelling Index, Cs**
  - Denotes the slope of an expansion or rebound curve of e-logp plot.

$$\boxed{Cs = \frac{e_3 - e_4}{\log P_4 - \log P_3}}$$

$$e_3 - e_4 = Cs \log {}^{P_4}/_{P_3}$$

$$e_4 = e_3 - Cs \log {}^{P_4}/_{P_3}$$



Coeefficient of Consolidation, Cv

- coefficient containing the physical constants of a soil affecting its rate of volume change.
- Indicates the combined effects of permeability and compressibility for a given void ratio change.

$$\boxed{C_v = \frac{k}{\gamma_\omega \, m_v} = \frac{k(1+e)}{\gamma_\omega a_v}}$$

C_v = Coefficient of consolidation

k = Coefficient of permeability

m_v= Coefficient of volume compressibility = $\frac{a_v}{(1+e)}$

a_v= Coefficient of compressibility

γ_ω = Unit weight of water

1D Consolidation Test cntd

Determination of Void Ratio at the End of a Loading Step

To determine $\boldsymbol{\sigma'_{zc}}$, $\boldsymbol{C_c}$, $\boldsymbol{C_r}$, and $\boldsymbol{C_\alpha}$, we need to know the void ratio for each loading step.

At the end of the consolidation test, we determined the water content (*w*) of the soil sample.

Using these data, initial height (*Ho*), and the specific gravity (*Gs*) of the soil sample, you can calculate the void ratio for each loading step as follows:

1D Consolidation Test cntd

1. Calculate the final void ratio, $e_{fin} = \omega G_s$, where ω is the water content determined at the end of the test.

2. Calculate the total consolidation settlement of the soil sample during the test, $(\Delta z)_{fin} = d_{fin} - d_i$, where d_{fin} is the final displacement gage reading and d_i is the displacement gage reading at the start of the test.

3. Back-calculate the initial void ratio as

$$e_o = \frac{e_{fin} + \frac{(\Delta z)_{fin}}{H_o}}{1 - \frac{(\Delta z)_{fin}}{H_o}}$$

1D Consolidation Test cntd

4. Calculate e for each loading step using

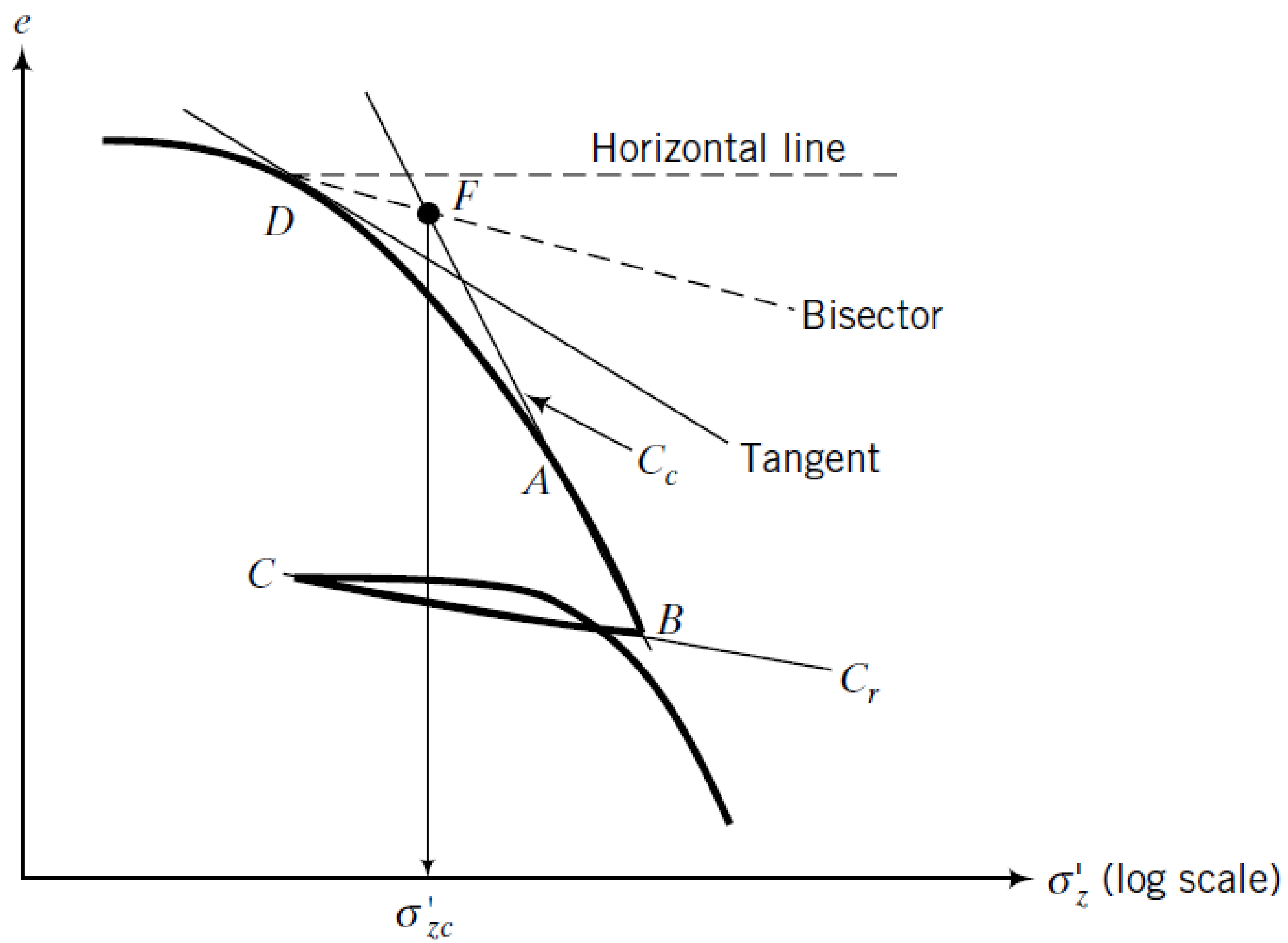
$$e = e_o - \Delta e = e_o - \frac{\Delta z}{H_o}(1 + e_o) = e_o\left(1 - \frac{\Delta z}{H_o}\right)$$

For saturated soil, $e_o = \omega G_s$

$$e = \omega G_s\left(1 - \frac{\Delta z}{H_o}\right)$$

1D Consolidation Test cntd

Determination of the Past Maximum Vertical Effective Stress - using a method proposed by Casagrande (1936)



1D Consolidation Test cntd

1. Identify the point of maximum curvature, point *D*, on the initial part of the curve.

2. Draw a horizontal line through *D*.

3. Draw a tangent to the curve at *D*.

4. Bisect the angle formed by the tangent and the horizontal line at *D*.

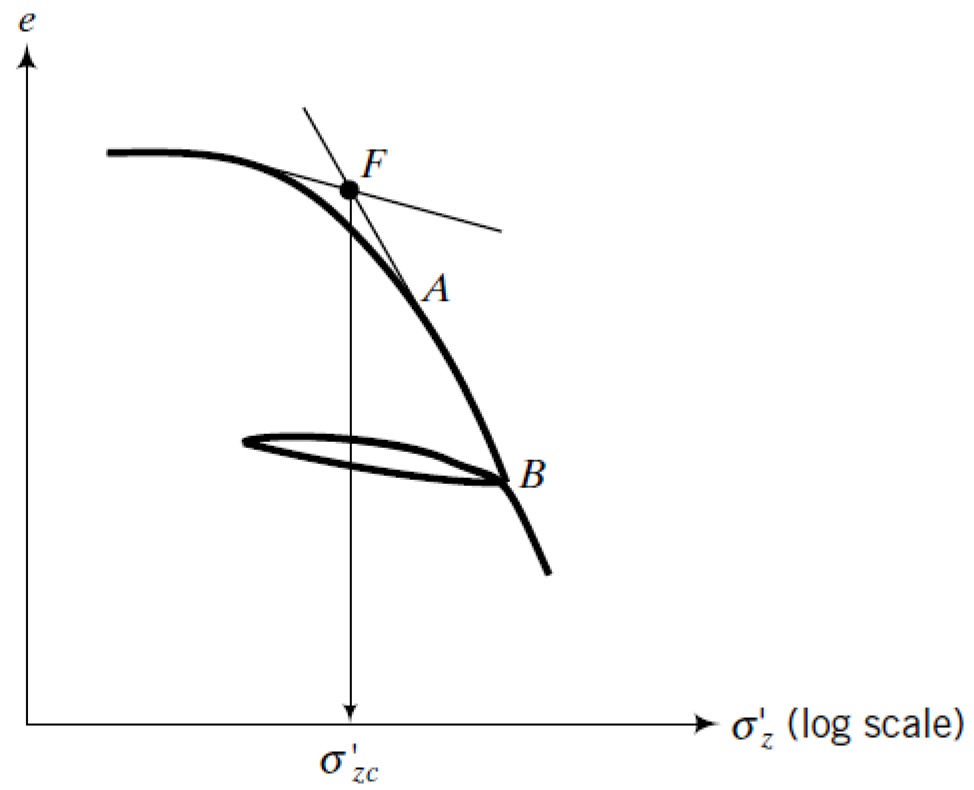
5. Extend backward the straight portion of the curve (the normal consolidation line), *BA*, to intersect the bisector line at *F*.

6. The abscissa of *F* is the past maximum vertical effective stress, $\boldsymbol{\sigma'}_{zc}$.

1D Consolidation Test cntd

A simpler method that is also used in practice is to project the straight portion of the initial recompression curve to intersect the backward projection of the normal consolidation line at *F*.

The abscissa of *F* is $\boldsymbol{\sigma'_{zc}}$.



1D Consolidation Test cntd

Determination of Compression and Recompression Indices

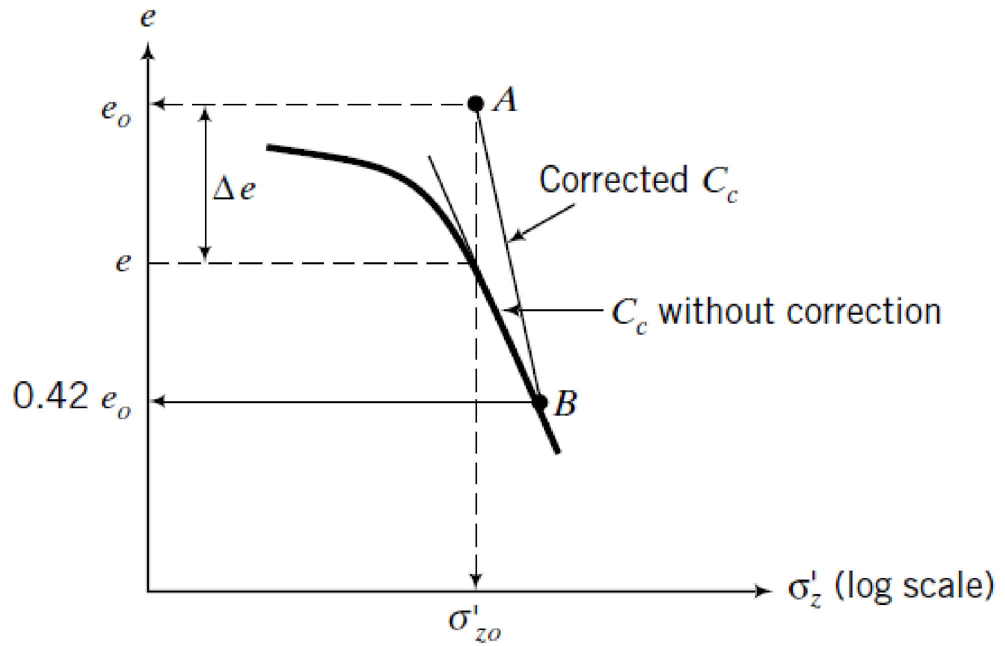
The slope of the normal consolidation line, *BA*, gives the compression index, *Cc*.

To determine the recompression index, *Cr*, draw a line (*BC*) approximately midway between the unloading and reloading curves.

The slope of this line

is the recompression

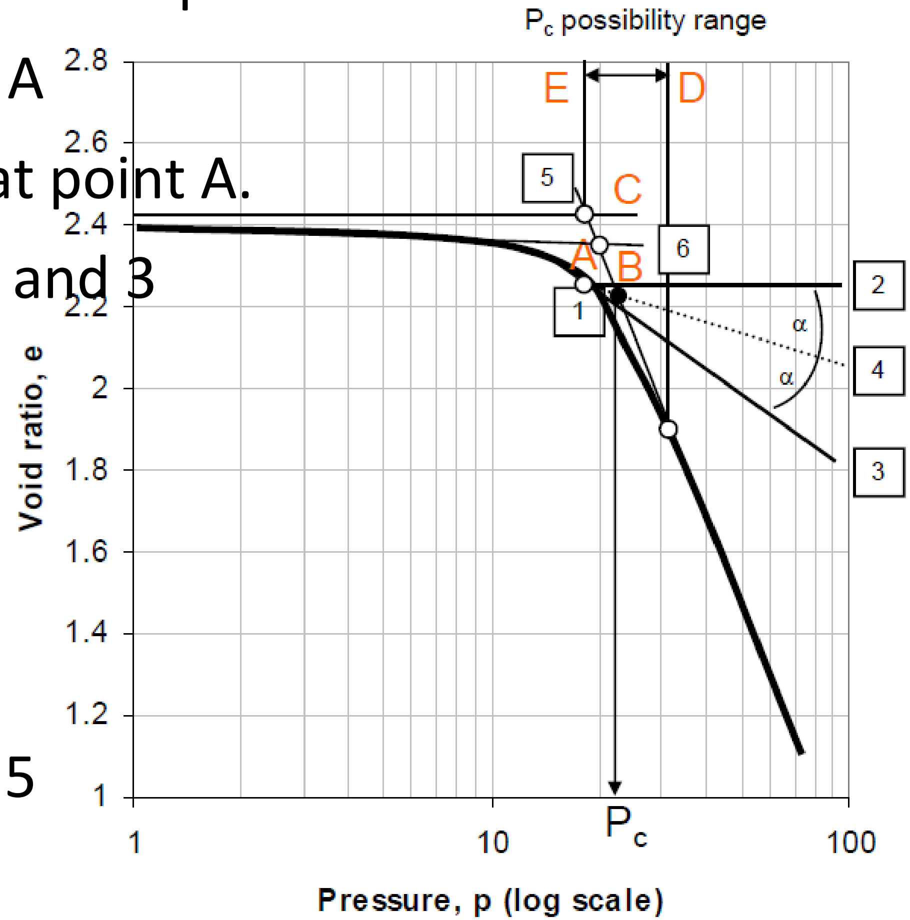
index (*Cr*).



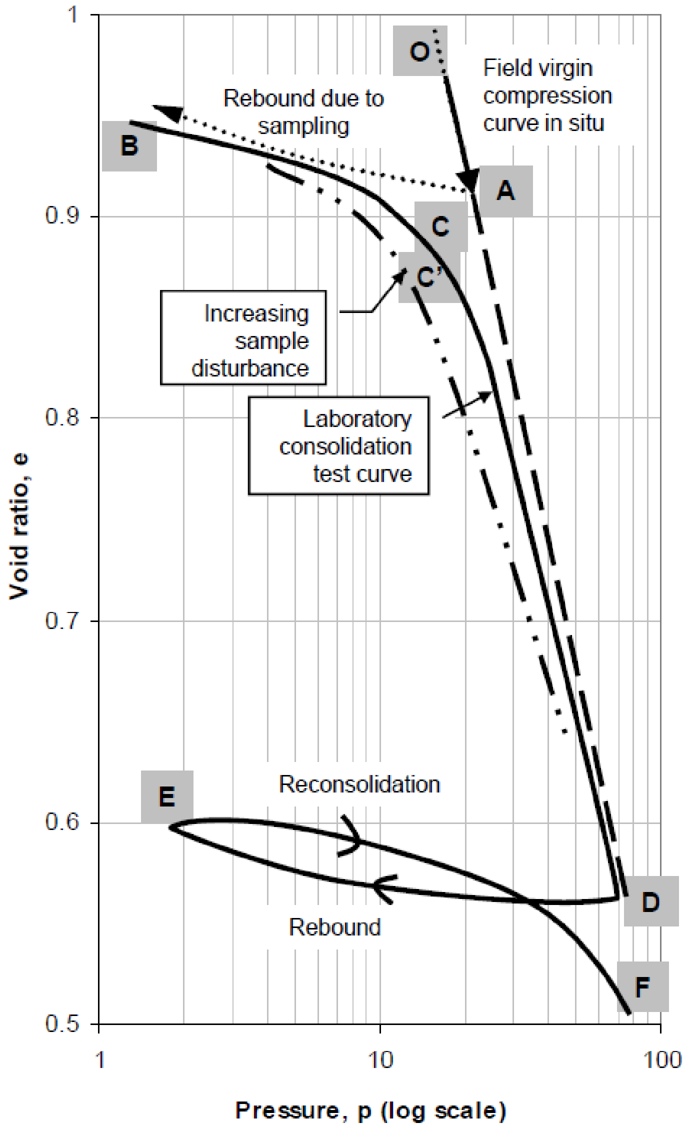
1D Consolidation Test cntd

Determination of σ'_{zc} (Pc) Casagrande Method

1. Choose point with minimum radius → point A
2. Draw horizontal line from point A
3. Draw line tangent to the curve at point A.
4. Bisect the angle made by step 2 and 3
5. Extend the straight line portion of the virgin compression curve up to where it meets the bisector line obtained in step 4.
6. Point of intersection step 4 and 5 is the (most probable) Pc



1D Consolidation Test cntd



Stress-strain history of a sedimentary clay during deposition sampling and reloading in the laboratory by the consolidation test:

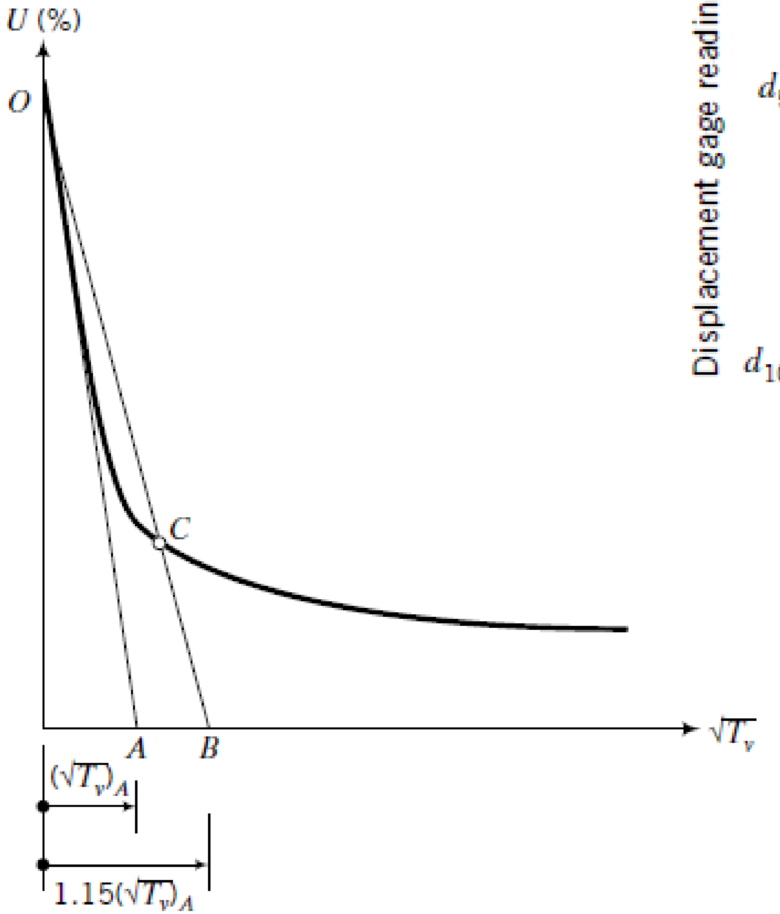
- **OA** represents the relationship between void ratio and the log effective stress of a particular element in the ground during deposition. The process consolidates the element to point **A**. This point represents the in situ *e* vs log *p'o* coordinates of the normally consolidated clay element.
- When the boring is made and soil is sampled, overburden stresses are removed by the sampling operation and the samples rebound or swell along curved **AB**.
- When the sample is transferred from sampling tube into consolidometer ring and then reloaded in the consolidation test, the curve **BC** is obtained.
- About point **C**, the soil structure start to break down and if the loading continuos the laboratory virgin compression curve **CD** is obtained.
- Eventually, the field curve **OAD** and lab curve **BCD** will converge beyond point **D** (approximately 0.4eo according 0.6 to Terzaghi and Peck, 1967)
- If the sampling operation was poor quality and mechanical disturbance to the soil structure occurred, curve **BC'D** would result upon reloading of the sample in the consolidometer.
- The proconsolidation pressure is much more difficult to define when sample disturbance has occurred.

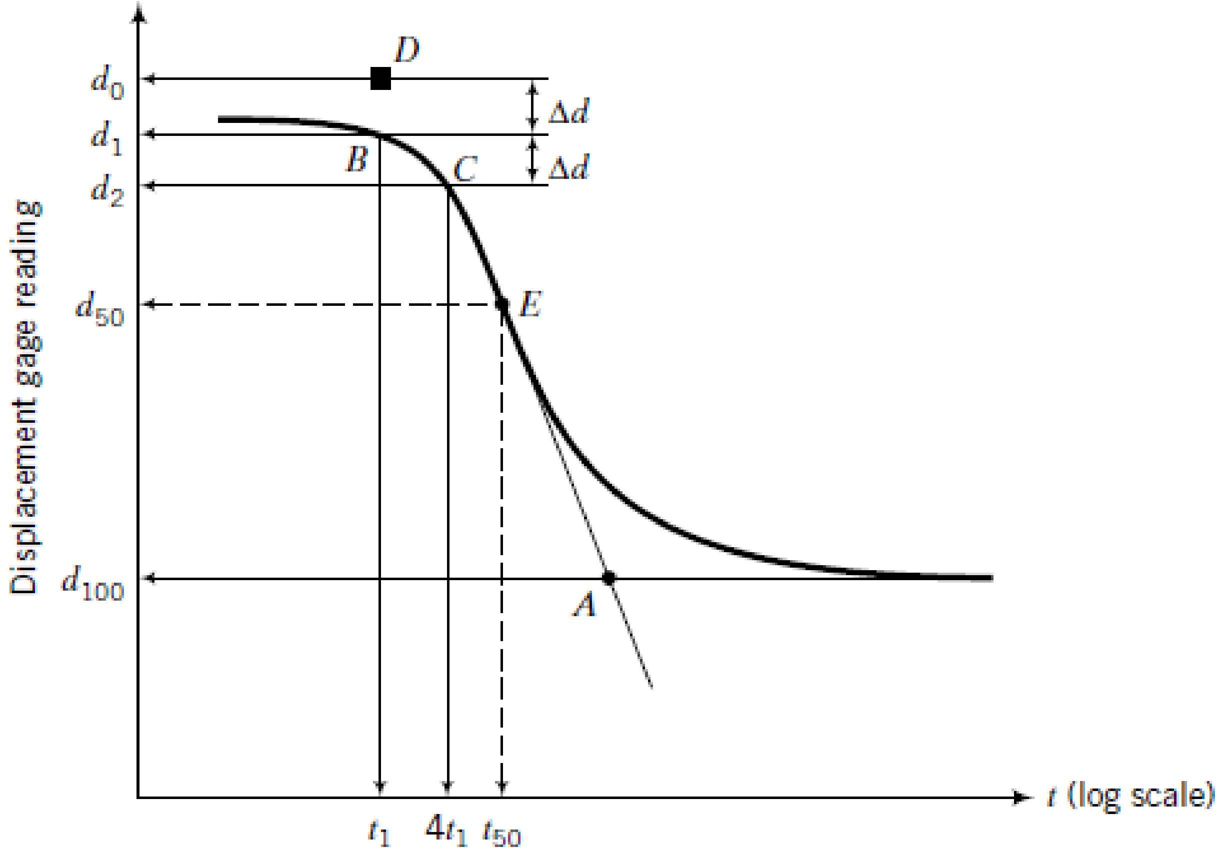
1D Consolidation Test cntd

Determination of the Coefficient of Consolidation

- **Root Time Method** – proposed by Taylor (1942)
  - utilizes the early time response, which theoretically should appear as a straight line in a plot of square root of time versus displacement gage reading.
- **Log Time Method** – proposed by Casagrande and Fadum (1940)

1D Consolidation Test cntd





1D Consolidation Test cntd

Lab vs Field Consolidation

If two layers of the same clay have the same degree of consolidation, then their time factors and coefficients of consolidation are the same.

Hence,

$$T_v = \frac{(C_v t)_{lab}}{(H_{dr}^2)_{lab}} = \frac{(C_v t)_{field}}{(H_{dr}^2)_{field}}$$

$$\frac{t_{field}}{t_{lab}} = \frac{(H_{dr}^2)_{field}}{(H_{dr}^2)_{lab}}$$

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Galatoma!