➢ Granular Soil

✓ Standard penetration number N_{60} obtained from field needs to be corrected for following cases

- Correction for overburden pressure
- Correction for submergence

Correction for overburden pressure

✓ In granular soils, value of N is affected by effective overburden pressure σ_0'

✓ For that reason, value of N_{60} obtained from field exploration under different effective overburden pressures should be changed to correspond to a standard value of σ_0'

$$(N_1)_{60} = C_N N_{60}$$

$(N_1)_{60}$ = value of N_{60} corrected to a standard value of σ_0' [100 kN/m² (2000 lb/ft²)]

C_N = correction factor

Granular Soil

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- For that reason, value of N_{60} obtained from field exploration under different effective overburden pressures should be changed to correspond to a standard value of σ_0'

$$(N_1)_{60} = C_N N_{60}$$

- Several correlations have been developed over the years for the correction factor, C_N
- Most commonly cited relationships are those of **Liao and Whitman (1986)** and **Skempton (1986)**

Liao and Whitman's relationship (1986)

$$C_N = \left[\frac{1}{\left(\frac{\sigma_o'}{p_a}\right)}\right]^{0.5} \quad \text{(SI units)}$$

where σ_0' = effective overburden pressure in kN/m²

p_a = atmospheric oressure (≈ 100 kN/m²)

$$C_N = \sqrt{\frac{1}{\sigma_o'}} \quad \text{(English units)}$$

Skempton's relationship (1986)

For SI units

$$C_N = \frac{2}{1 + \left(\frac{\sigma'_o}{p_a}\right)}$$ (normally consolidated fine sand)

$$C_N = \frac{3}{2 + \left(\frac{\sigma'_o}{p_a}\right)}$$ (normally consolidated coarse sand)

$$C_N = \frac{1.7}{0.7 + \left(\frac{\sigma'_o}{p_a}\right)}$$ (overconsolidated sand)

where σ_0' = effective overburden pressure in kN/m^2

Skempton's relationship (1986)

For English units

$$C_N = \frac{2}{1+\sigma_0'} \quad \text{(normally consolidated fine sand)}$$

$$C_N = \frac{3}{2+\sigma_0'} \quad \text{(normally consolidated coarse sand)}$$

$$C_N = \frac{1.7}{0.7+\sigma_0'} \quad \text{(overconsolidated sand)}$$

Correction for submergence

- ✓ In very fine, silty, saturated sand an apparent increase in resistance occurs
- ✓ Terzaghi and Peck have recommended use of an equivalent penetration resistance $(N_1)_{60}$ in place of actually observed value of N_{60}, when N_{60} is greater than 15

$$(N_1)_{60} = 15 + \frac{1}{2}(N_{60} - 15)$$

Correlations between N_{60} and Relative Density

- Kulhawy and Mayne (1990) modified an empirica relationship for relative density that was given by Marcuso and Bieganousky (1977), which can be expressed as

$$D_r(\%) = 12.2 + 0.75\left[222N_{60} + 2311 - 711OCR - 779\left(\frac{\sigma'_o}{p_a}\right) - 50C_u^2\right]^{0.5}$$

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where

D_r = relative density

σ'_o = effective overburden pressure

C_u = uniformity coefficient of sand

$$OCR = \frac{\text{preconsolidation pressure}, \sigma'_c}{\text{effective overburden pressure}, \sigma'_o}$$

p_a = atmospheric pressure

- Meyerhof (1957) developed a correlation between D_r and N_{60} as

$$N_{60} = \left[17 + 24\left(\frac{\sigma'_o}{p_a}\right)\right] D_r^2$$

or

$$D_r = \left\{ \frac{N_{60}}{\left[17 + 24\left(\frac{\sigma'_o}{p_a}\right)\right]} \right\}^{0.5}$$

➢ Cubrinovski and Ishihara (1999) also proposed a correlation between N_{60} and relative density of sand, D_r that can be expressed as

$$D_r(\%) = \left[\frac{N_{60}\left(0.23 + \frac{0.06}{D_{50}}\right)^{1.7}}{9} \left(\frac{1}{\frac{\sigma'_o}{p_a}} \right) \right]^{0.5} (100)$$

where

p_a = atmospheric pressure ($\approx 100\ kN/m^2$)
D_{50} = sieve size through which 50% of the soil will pas

Correlations between $(N_1)_{60}$ and Relative Density

Corrected standard penetration number, $(N_1)_{60}$	Relative density, D_r (%)
0–5	0–5
5–10	5–30
10–30	30–60
30–50	60–95

- Kulhawy and Mayne (1990) correlated corrected standard penetration number and relative density of sand in form

$$D_r(\%) = \left[\frac{(N_1)_{60}}{C_p C_A C_{OCR}}\right]^{0.5} (100)$$

where

C_p = grain-size correlations factor $= 60 + 25 \log D_{50}$

C_A = correlation factor for aging $= 1.2 + 0.05 \log\left(\frac{t}{100}\right)$

C_{OCR} = correlation factor for overconsolidation $= OCR^{0.18}$

D_{50} = diameter through which 50% soil will pass through (mm)

t = age of soil since deposition (years)

OCR = overconsolidation ratio

- Peck, Hanson, and Thornburn (1974) gave a correlat between $(N_1)_{60}$ and ϕ' in a graphic form, which can approximated as (Wolff, 1989)

$$\phi' \,(\text{deg}) = 27.1 + 0.3(N_1)_{60} - 0.00054(N_1)^2_{60}$$

- Schmertmann (1975) provided the correlation between N_{60}, σ_0' and ϕ'. Mathematically, correlation can be approximated as (Kulhawy and Mayne, 1990)

$$\phi' = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma_o'}{p_a}\right)}\right]^{0.34}$$

where

N_{60} = field standard penetration number
σ_o' = effective overburden pressure
p_a = atmospheric pressure in the same unit as σ_o'
ϕ' = soil friction angle

- Hatanaka and Uchida (1996) provided a simple correlation between $(N_1)_{60}$ and ϕ' that can be expressed as

$$\phi' = \sqrt{20(N_1)_{60}} + 20$$

- Following qualifications should be noted when standard penetration resistance values are used in preceding correlations to estimate soil parameters:

- Hatanaka and Uchida (1996) provided a simple correlation between $(N_1)_{60}$ and ϕ' that can be expressed as

$$\phi' = \sqrt{20(N_1)_{60}} + 20$$

- Following qualifications should be noted when standard penetration resistance values are used in preceding correlations to estimate soil parameters:
  - Equations are approximate
  - Because soil is not homogeneous, values of obtained from a given borehole vary widely
  - In soil deposits that contain large boulders and gravel, standard penetration numbers may be erratic and unreliable

Correlation between Modulus of Elasticity and N_{60}

$$\frac{E_s}{p_a} = \alpha N_{60}$$

where

p_a = atmospheric pressure (same unit as E_s)

$$\alpha = \begin{cases} 5 \text{ for sands with fines} \\ 10 \text{ for clean normally consolidated sand} \\ 15 \text{ for clean overconsolidated sand} \end{cases}$$

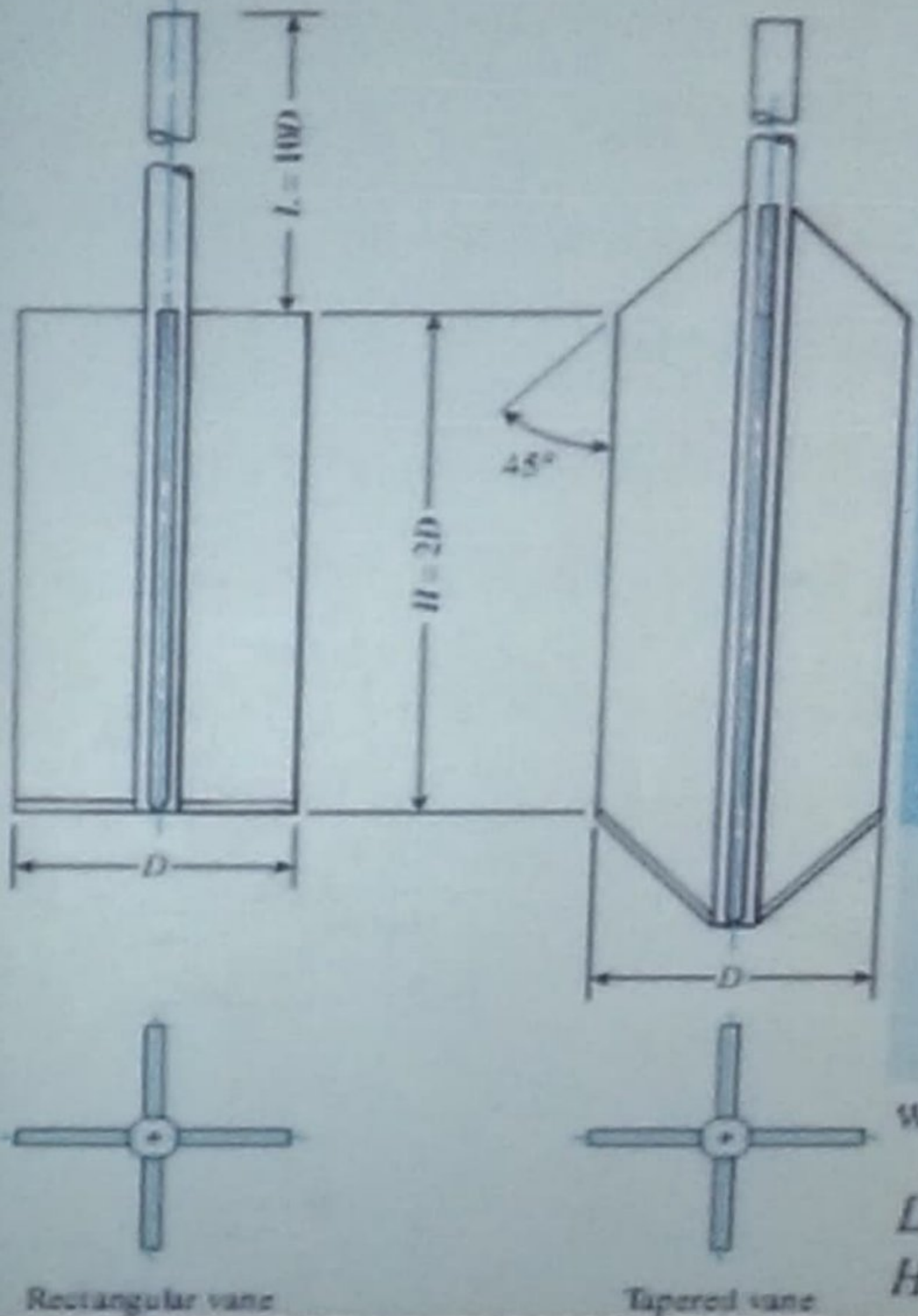
Sources of error in SPT

- Although approximate, with correct interpretation standard penetration test provides a good evaluation of soil properties
- Primary sources of error in standard penetration tests are
  - Inadequate cleaning of borehole
  - Careless measurement of blow count
  - Eccentric hammer strikes on drill rod
  - Inadequate maintenance of water head in borehole

Other In Situ Tests

Vane Shear Test (VST)

- ✓ Vane shear test (ASTM D-2573) may be used during drilling operation to determine in situ undrained shear strength (c_u) of clay soils - particularly soft clays
- ✓ Vane shear apparatus consists of four blades on end of a rod
- ✓ Height, H, of vane is twice of diameter, D
- ✓ Vane can be either rectangular or tapered
- ✓ Vanes of apparatus are pushed into soil at bottom of a borehole without disturbing soil appreciably
- ✓ Torque is applied at top of rod to rotate vanes at a standard rate of 0.1^{0} /sec
- ✓ Rotation will induce failure in a soil of cylindrical shape surrounding vanes
- ✓ Maximum torque, T, applied to cause failure is measured



$$T = f(c_u, H, \text{and } D)$$

or $$c_u = \frac{T}{K}$$

Where

T is in N.m, c_u is in kN/m^2

and

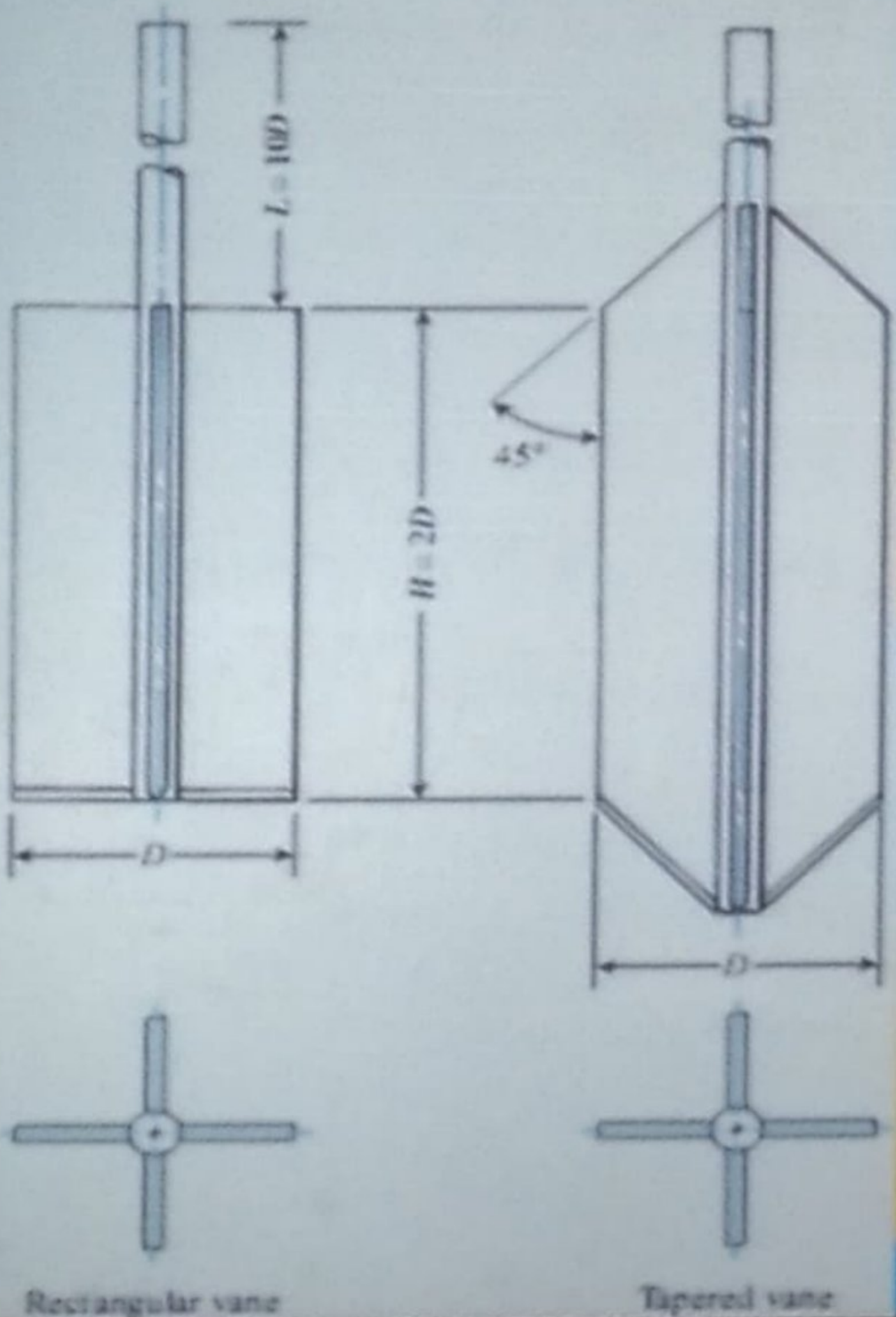
K = a constant with a magnitude depending on dimension and shape of vane

$$K = \left(\frac{\pi}{10^6}\right)\left(\frac{D^2H}{2}\right)\left(1 + \frac{D}{3H}\right)$$

where

D = diameter of vane in cm

H = measured height of vane in cm



$$T = f(c_u, H, \text{and } D)$$

or $$c_u = \frac{T}{K}$$

In English units, if c_u and are expressed in and lb/ft and lb-ft, respectively, then

$$K = \left(\frac{\pi}{1728}\right)\left(\frac{D^2H}{2}\right)\left(1 + \frac{D}{3H}\right)$$

where

D = diameter of vane in inch

H = measured height of vane in inch

- ✓ Field vane shear tests are moderately rapid and economical and are used extensively in field soil-exploration programs
- ✓ Test gives good results in soft and medium-stiff clays and gives excellent results in determining properties of sensitive clays
- ✓ Sources of significant error in field vane shear test are poor calibration of torque measurement and damaged vanes
- ✓ Other errors may be introduced if rate of rotation of vane is not properly controlled
- ✓ For actual design purposes, undrained shear strength values obtained from field vane shear tests [$c_{u(VST)}$] are too high, and it is recommended that they be corrected according to following equation

$$c_{u(corrected)} = \lambda c_{u(VST)}$$

- Several correlations have been given previously for correction factor
- Most commonly used correlation for λ is that given by Bjerrum (1972), which can be expressed

$$\lambda = 1.7 - 0.54\log[PI(\%)]$$

- Morris and Williams (1994) provided following correlation

$$\lambda = 1.18e^{-0.08(PI)} + 0.57 \text{ (for PI} > 5)$$

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$$\lambda = 1.18e^{-0.08(PI)} + 0.57 \text{ (for PI} > 5)$$

$$\lambda = 7.01e^{-0.08(LL)} + 0.57 \text{ (where LL is in \%)}$$

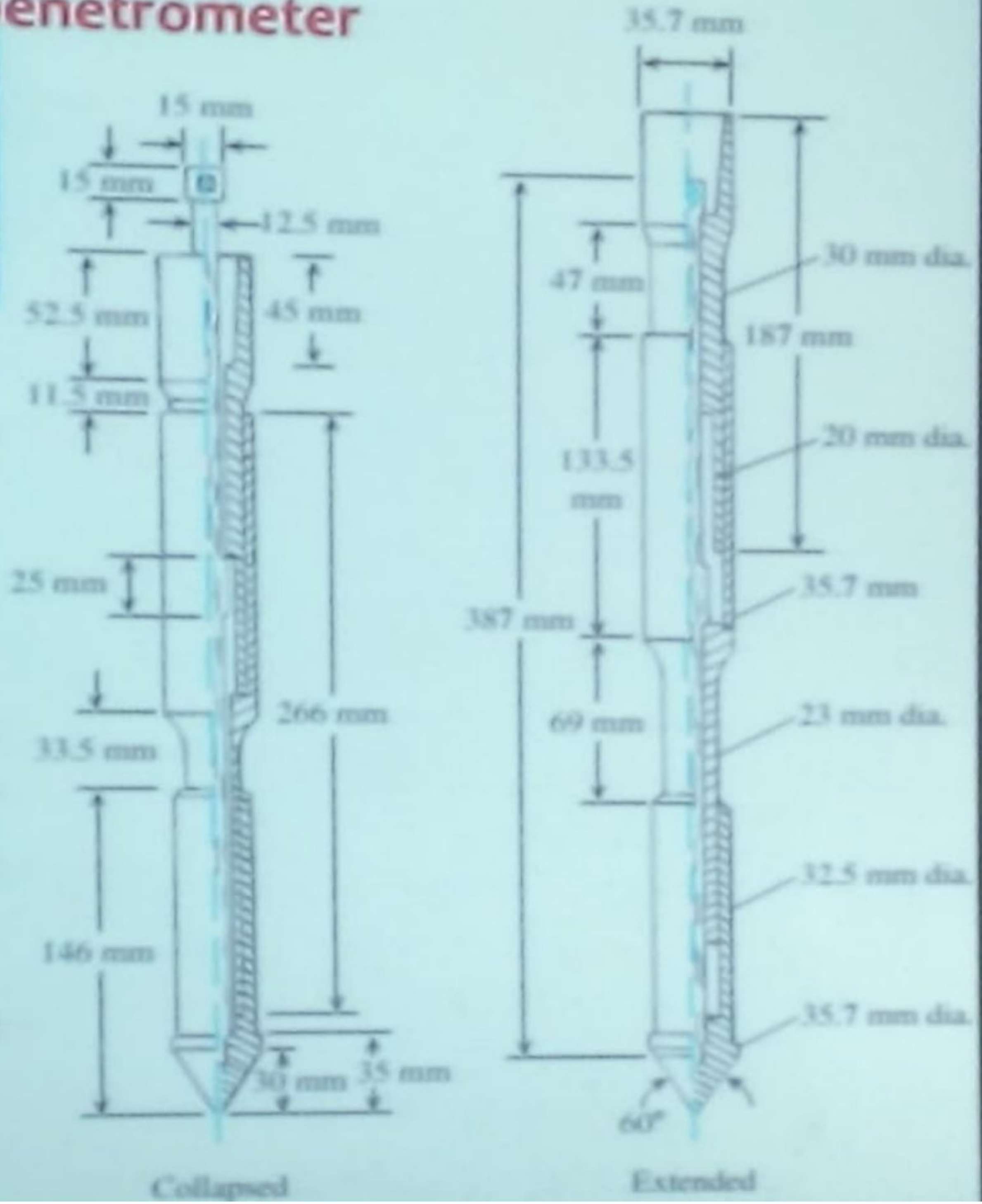
Cone Penetration Test

- ✓ Cone penetration test (CPT), originally known as Dutch cone penetration test, is a versatile sounding method
- ✓ Can be used to determine materials in a soil profile and estimate their engineering properties
- ✓ Test is also called static penetration test, and no boreholes are necessary to perform it
- ✓ In original version, a cone with a base area of 10 cm^2 was pushed into ground at a steady rate of about 20 mm/sec, and resistance to penetration (called point resistance) was measured

- ✓ Cone penetrometers in use at present to measure
  - **Cone resistance (q_c)** to penetration developed by cone, which is equal to vertical force applied to cone, divided by its horizontally projected area
  - **Frictional resistance (f_c)** which is resistance measured by a sleeve located above cone with local soil surrounding it
- ✓ Frictional resistance is equal to vertical force applied to sleeve, divided by its surface area - actually, sum of friction and adhesion
- ✓ Generally, two types of penetrometers are used to measure q_c and f_c
  - Mechanical friction-cone penetrometer
  - Electric friction-cone penetrometer

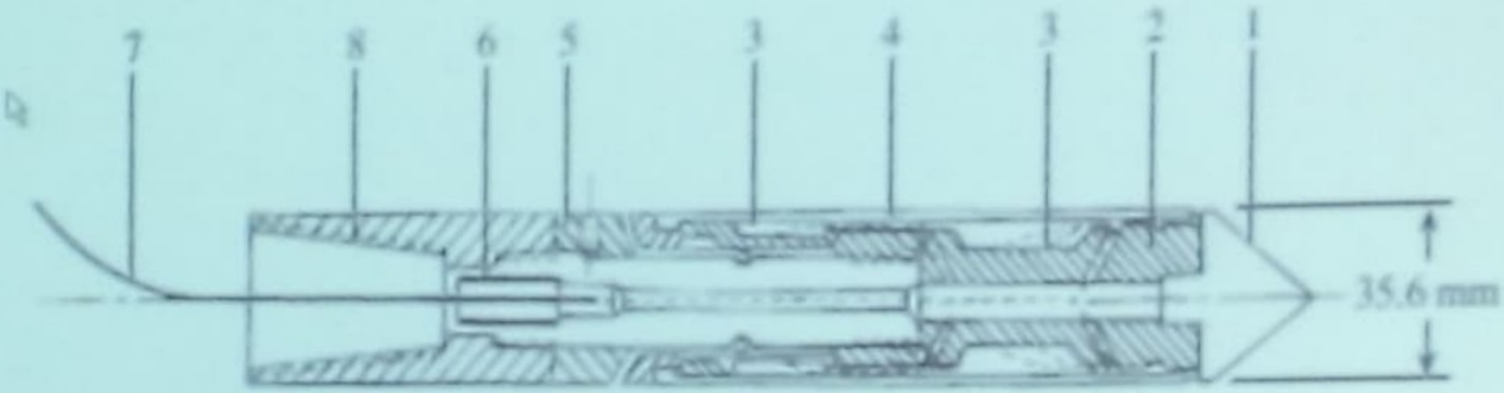
Mechanical friction-cone penetrometer

- ✓ Tip of this penetrometer is connected to an inner set of rods
- ✓ Tip is first advanced about 40 mm, giving cone resistance
- ✓ With further thrusting, tip engages friction sleeve
- ✓ As inner rod advances, rod force is equal to sum of vertical force on cone and sleeve
- ✓ Subtracting force on cone gives side resistance



Electric friction-cone penetrometer

- ✓ Tip of this penetrometer is attached to a string of steel rods
- ✓ Tip is pushed into ground at rate of 20 mm/sec
- ✓ Wires from transducers are threaded through center of rods and continuously measure cone and side resistances



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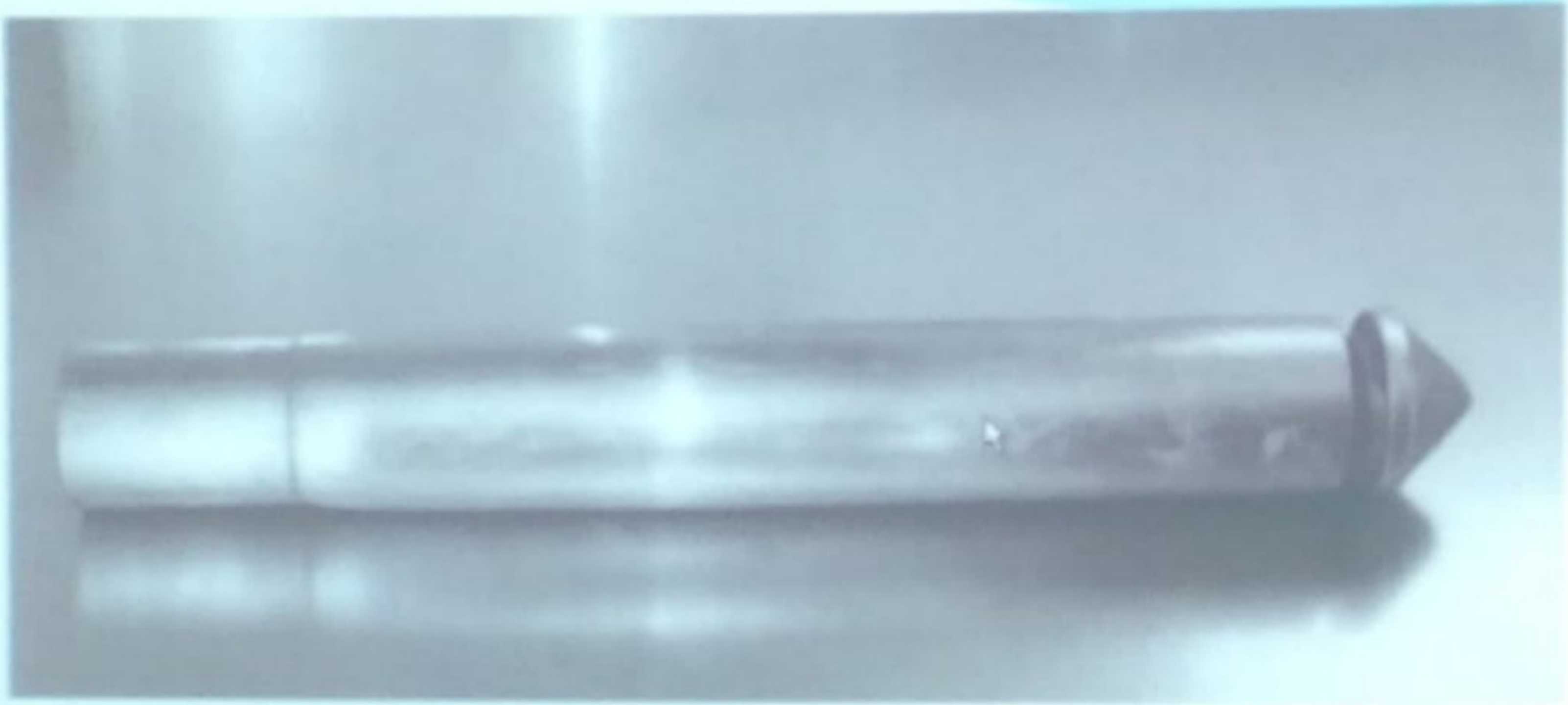
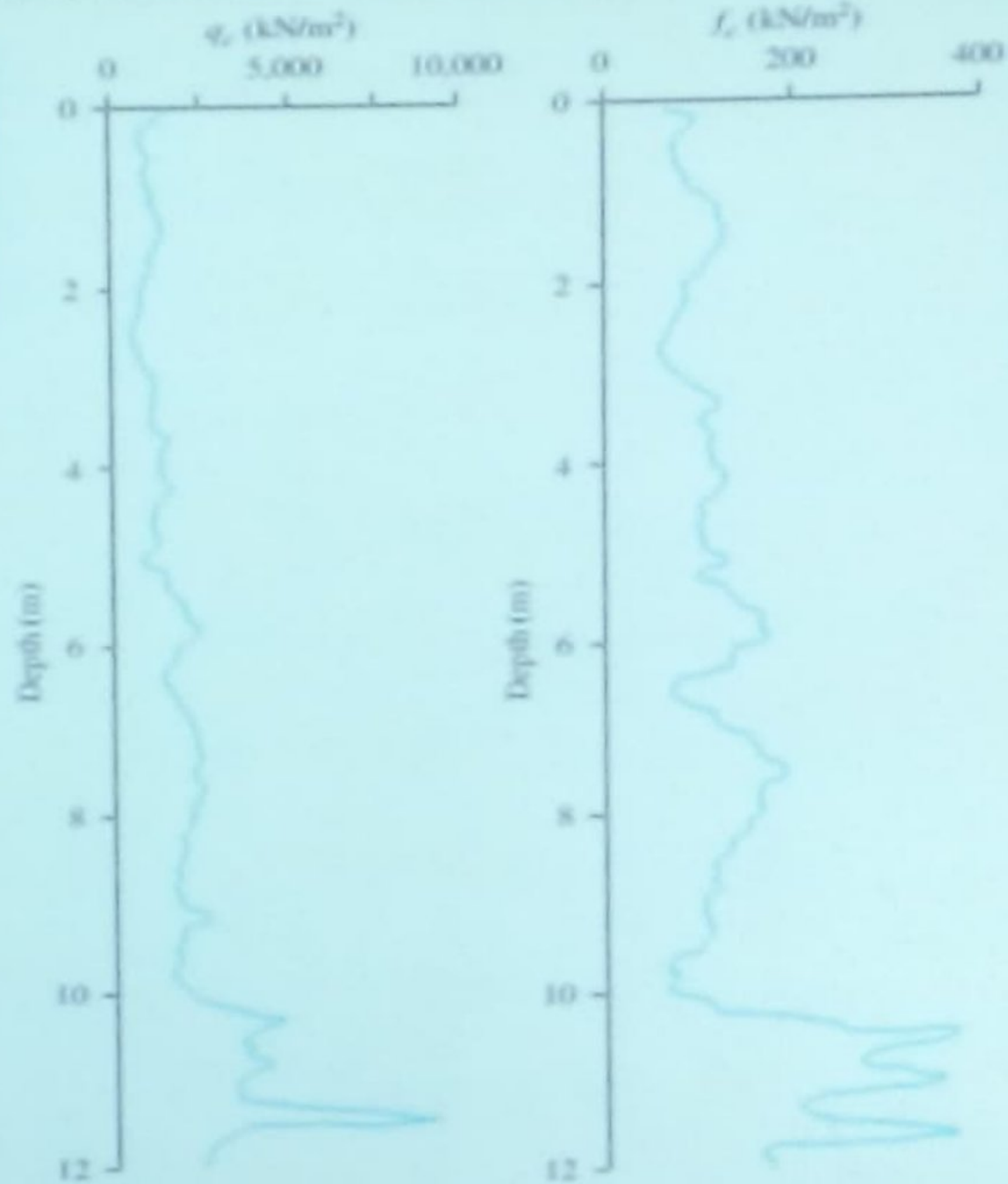




Cone penetrometer test with friction measurement
(electric friction-cone penetrometer)

- Several correlations that are useful in estimating properties of soils encountered during an exploration program have been developed for point resistance (q_c)
- Friction ratio (F_r) obtained from cone penetration tests
- Friction ratio is defined as

$$F_r = \frac{\text{frictional resistance}}{\text{cone resistance}} = \frac{f_c}{q_c}$$

- In a more recent study on several soils in Greece, Anagnostopoulos *et al.* (2003) expressed as

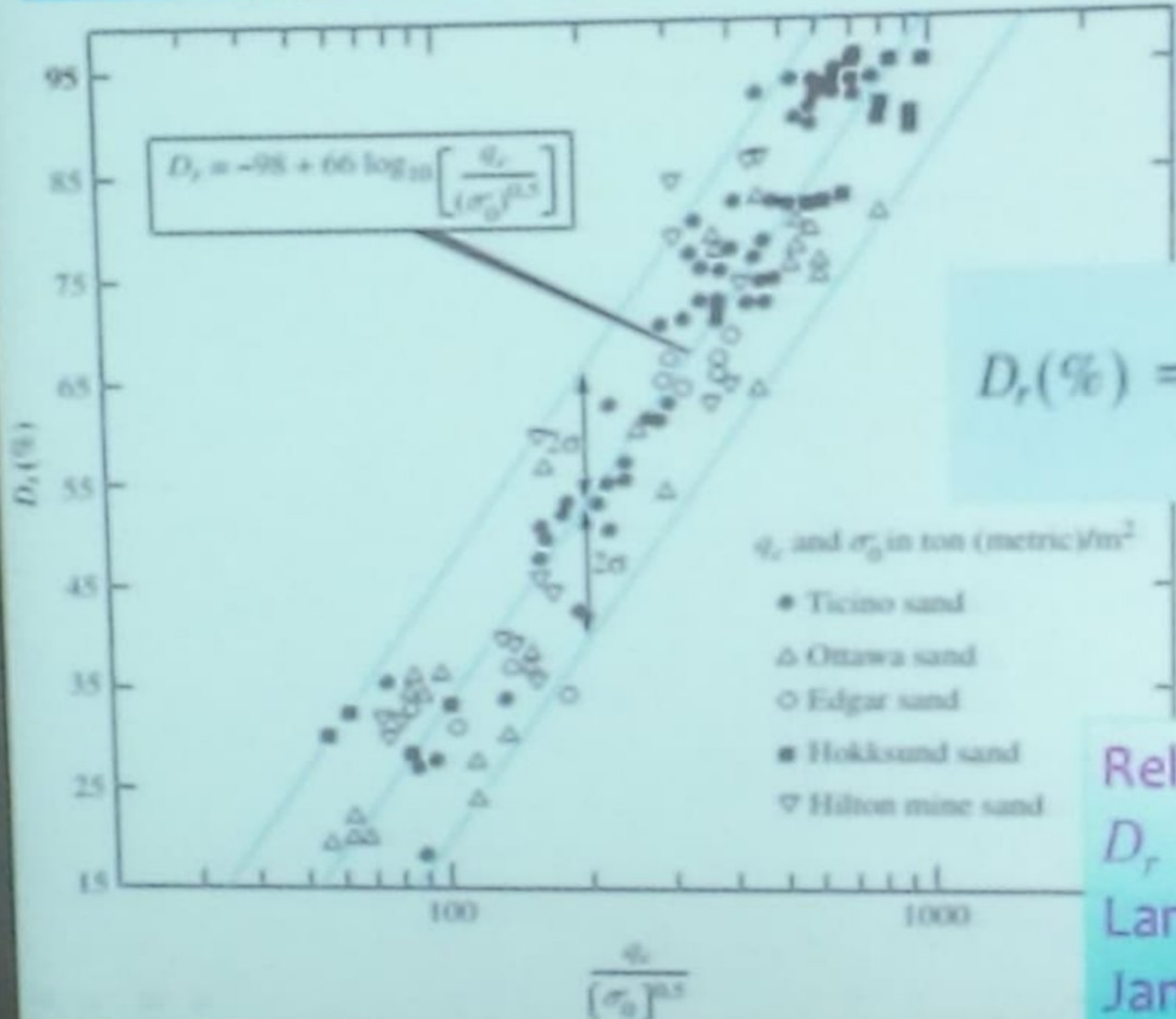
$$F_r(\%) = 1.45 - 1.36 \log D_{50} \text{ (electric cone)}$$

$$F_r(\%) = 0.7811 - 1.611 \log D_{50} \text{ (mechanical cone)}$$

where D_{50} = size through which 50% of soil will pass through (ranged from 0.001 mm to about 10 mm)

Correlation between Relative Density (D_r) and q_c for Sand

- ✓ Lancellotta (1983) and Jamiolkowski *et al.* (1985) showed that relative density D_r of normally consolidated sand, and q_c can be correlated according to formula



$$D_r(\%) = A + B \log_{10}\left(\frac{q_c}{\sqrt{\sigma'_o}}\right)$$

Relationship between D_r and q_c (Based on Lancellotta, 1983, and Jamiolski *et al.*, 1985)

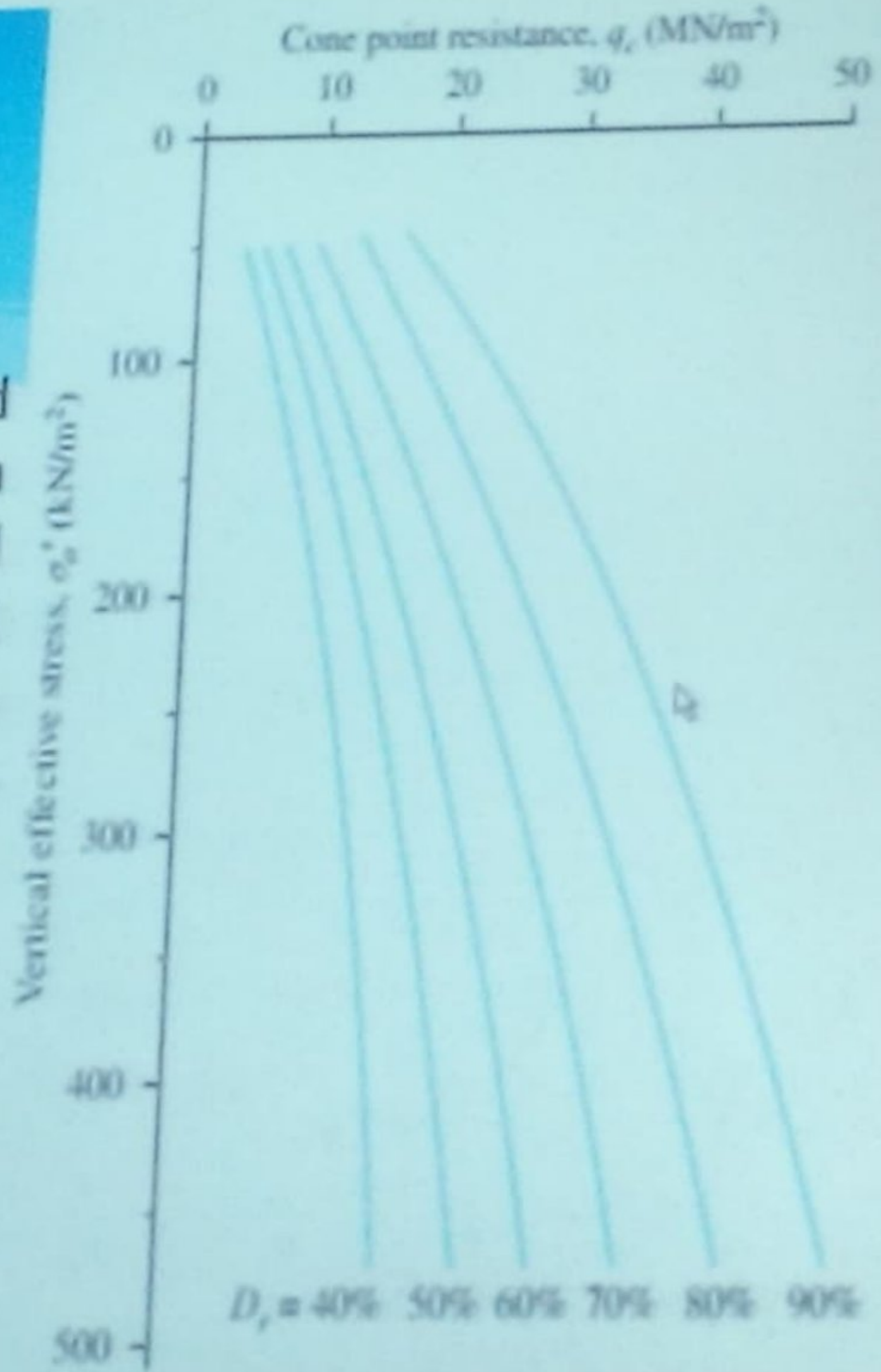
- Preceding relationship can be rewritten as (Kulhawy and Mayne, 1990)

$$D_r(\%) = 68\left[\log\left(\frac{q_c}{\sqrt{p_a \cdot \sigma'_0}}\right) - 1\right]$$

where

p_a = atmospheric pressure ($\approx$ 100 kN/m^2)
σ'_o = vertical effective stress

- Baldi *et al.* (1982), and Robertson and Campanella (1983) recommended empirical relationship between vertical effective stress (σ_0'), relative density (D_r), and q_c for normally consolidated sand



- Kulhawy and Mayne (1990) proposed following relationship to correlate D_r, q_c, and vertical effective stress σ_0'

$$D_r = \sqrt{\left[\frac{1}{305 Q_c \mathrm{OCR}^{1.8}}\right]\left[\frac{\frac{q_c}{p_a}}{\left(\frac{\sigma_o'}{p_a}\right)^{0.5}}\right]}$$

In this equation,

OCR = overconsolidation ratio
p_a = atmospheric pressure
Q_c = compressibility factor

The recommended values of Q_c are as follows:

Highly compressible sand = 0.91
Moderately compressible sand = 1.0
Low compressible sand = 1.09

Correlation between q_c and drained friction angle (ϕ') for sand

- On basis of experimental results, Robertson and Campanella (1983) suggested variation of D_r, σ_0' and ϕ' for normally consolidated quartz sand
- This relationship can be expressed as (Kulhawy and Mayne, 1990)

$$\phi' = \tan^{-1}\left[0.1 + 0.38 \log\left(\frac{q_c}{\sigma_o'}\right)\right]$$

- Based on cone penetration tests on soils in Venice Lagoon (Italy), Ricceri *et al.* (2002) proposed a similar relationship for soil with classifications of ML and SP-SM as

$$\phi' = \tan^{-1}\left[0.38 + 0.27 \log\left(\frac{q_c}{\sigma_o'}\right)\right]$$

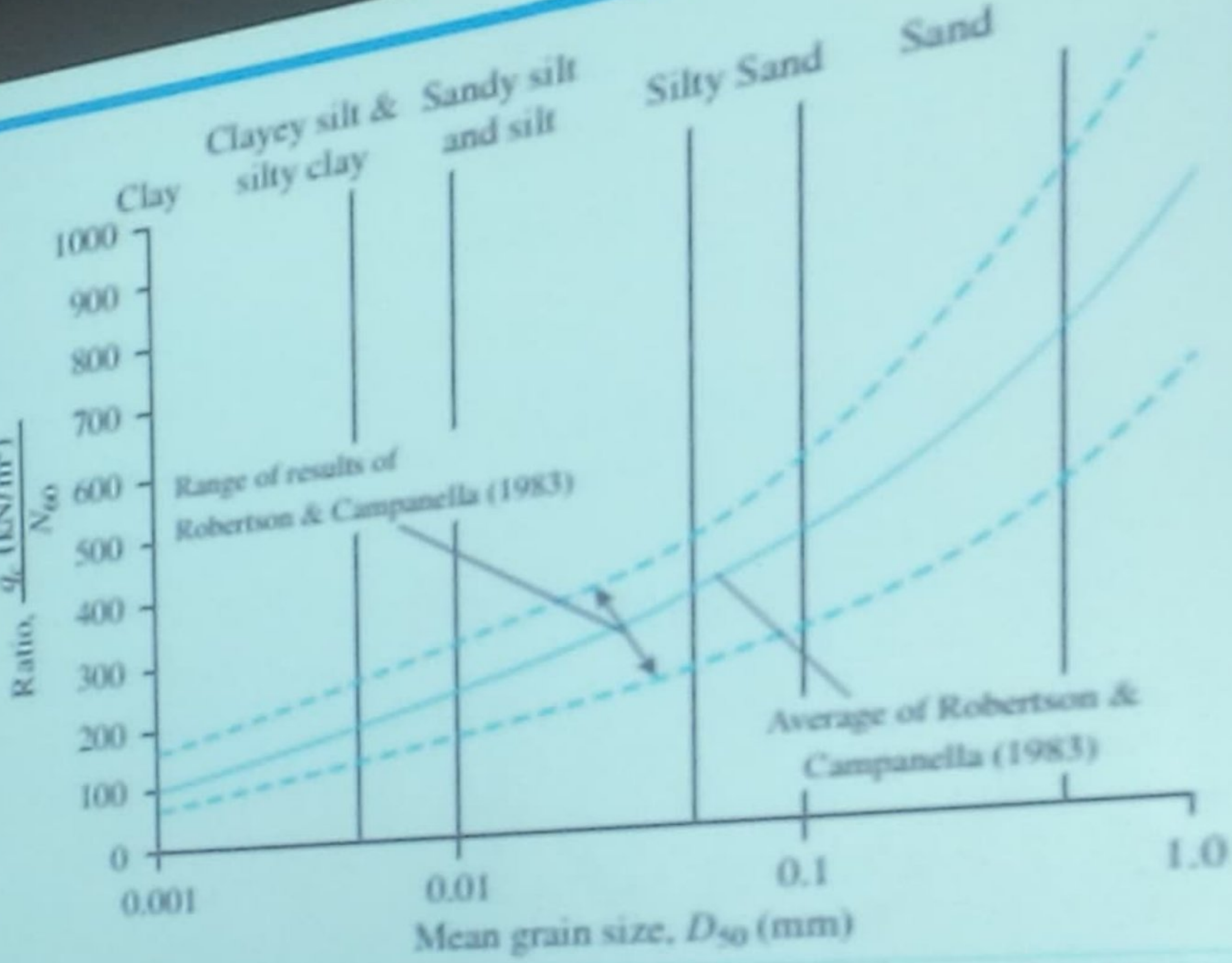
Criteria for assigning group symbols				Group symbol
Coarse-grained soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of course fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $1 > C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	GM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	GC
	Sands 50% or more of course fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $1 > C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	SM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	SC
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 5.3)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 5.3)[e]	ML
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OL zone	OL
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 5.3)	CH
			PI plots below "A" line (Figure 5.3)	MH
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OH zone	OH
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			Pt

[a]Gravels with 5 to 12% fine require dual symbols: GW-GM, GW-GC, GP-GM, GP-GC.

[b]Sands with 5 to 12% fines require dual symbols: SW-SM, SW-SC, SP-SM, SP-SC.

➢ In a more recent study, Lee *et al.* (2004) developed a correlation between ϕ', q_c, and the horizontal effective stress (σ_h') in form

$$\phi' = 15.575\left(\frac{q_c}{\sigma'_h}\right)^{0.1714}$$

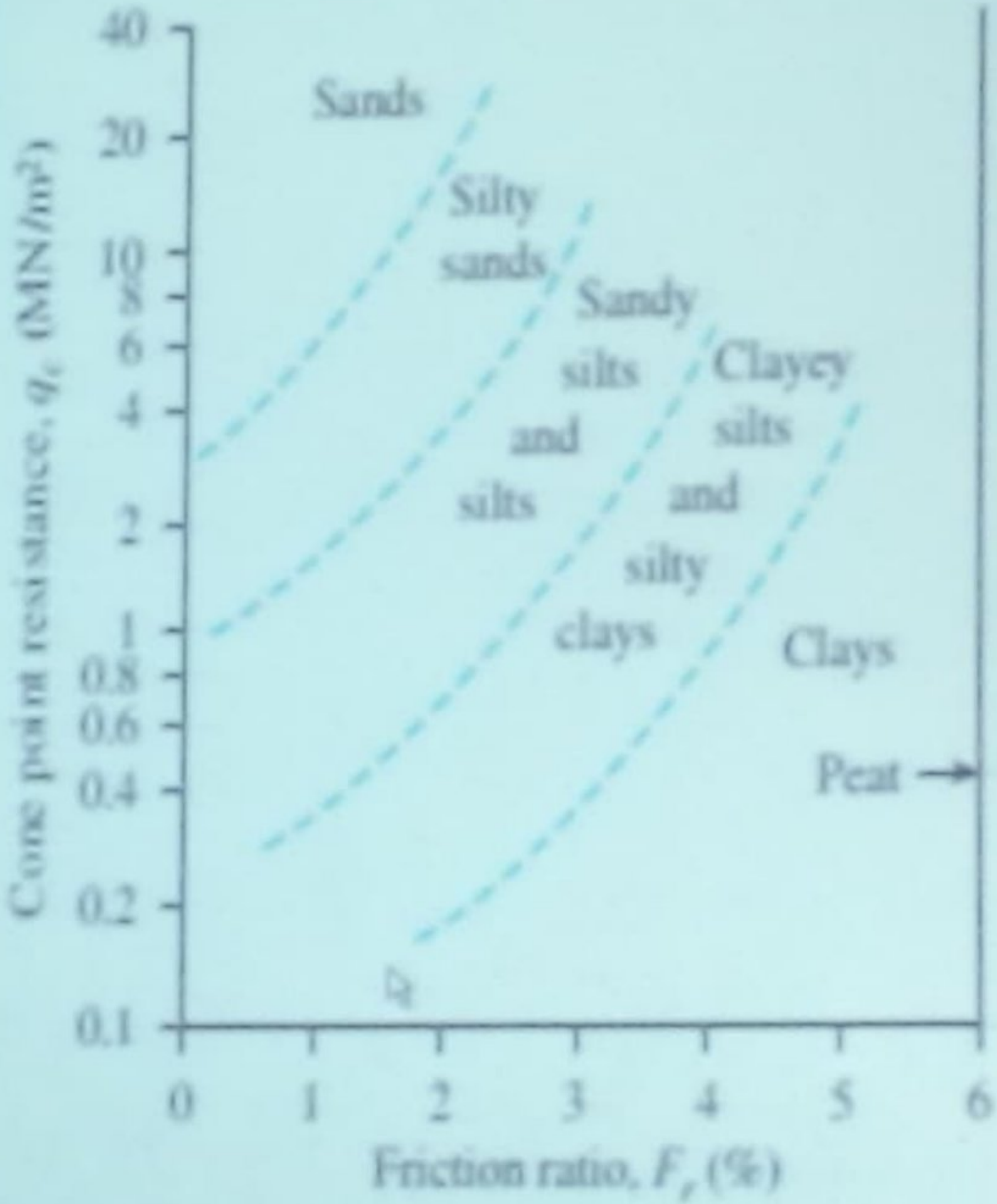


General range of variation of q_c/N_{60} for various types of soil
After Robertson and Campanella (1983)

- Anagnostopoulos *et al.* (2003) provided a similar relationship correlating q_c, N_{60}, and D_{50}

$$\frac{\left(\frac{q_c}{p_a}\right)}{N_{60}} = 7.6429 D_{50}^{0.26}$$

- Robertson and Campanella (1983) provided correlations between q_c and friction ratio F_r to identify various types of soil encountered in field



Correlations for c_u, σ_c', and OCR for Clays

- Undrained shear strength, c_u, can be expressed as

$$c_u = \frac{q_c - \sigma_o}{N_K}$$

where

σ_o = total vertical stress
N_K = bearing capacity factor

N_K = 11 to 19 for normally consolidated clays
= 25 for overconsolidated clay

- According to Mayne and Kemper (1988)

N_K = 15 for electric cone
= 20 for mechanical cone

- Based on tests in Greece, Anagnostopoulos *et al.* (2003) determined

$$N_K = 17.2 \text{ for electric cone}$$
$$= 18.9 \text{ for mechanical cone}$$

- These field tests also showed that

$$c_u = \frac{f_c}{1.26} \text{ (for mechanical cones)}$$

$$c_u = f_c \text{ (for electrical cones)}$$

- Mayne and Kemper (1988) provided correlations for preconsolidation pressure (σ_c') and overconsolidation ratio (OCR) as

$$\sigma_c' = 0.243(q_c)^{0.96}$$

↑ σ_c' in MN/m², ↑ q_c in MN/m²

$$\text{OCR} = 0.37\left(\frac{q_c - \sigma_o}{\sigma_o'}\right)^{1.01}$$

where σ_0 and σ_0' = total and effective stress, respectively

Pressuremeter Test (PMT)

- ✓ An in situ test conducted in a borehole
- ✓ It was originally developed by Menard (1956) to measure strength and deformability of soil
- ✓ It has also been adopted by ASTM as Test Designation 4719
- ✓ Menard-type PMT consists essentially of a probe with three cells
- ✓ Top and bottom ones are guard cells and middle one is measuring cell
- ✓ Test is conducted in a prebored hole with a diameter that is between 1.03 and 1.2 times nominal diameter of probe
- ✓ Probe that is most commonly used has a diameter of 58 mm and a length of 420 mm
- ✓ Probe cells can be expanded by either liquid or gas
- ✓ Guard cells are expanded to reduce end-condition effect on measuring cell, which has a volume (V_0) of 535 cm^3
- ✓ Following are the dimensions for probe diameter and diameter of borehole, as recommended by ASTM

- In order to conduct a test, measuring cell volume V_0, is measured and probe is inserted into borehole
- Pressure is applied in increments and new volume of cell is measured
- Process is continued until soil fails or until pressure limit of device is reached
- Soil is considered to have failed when total volume of expanded cavity (V) is about twice volume of original cavity

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- ✓ Pressure is applied in increments and new volume of cell is measured
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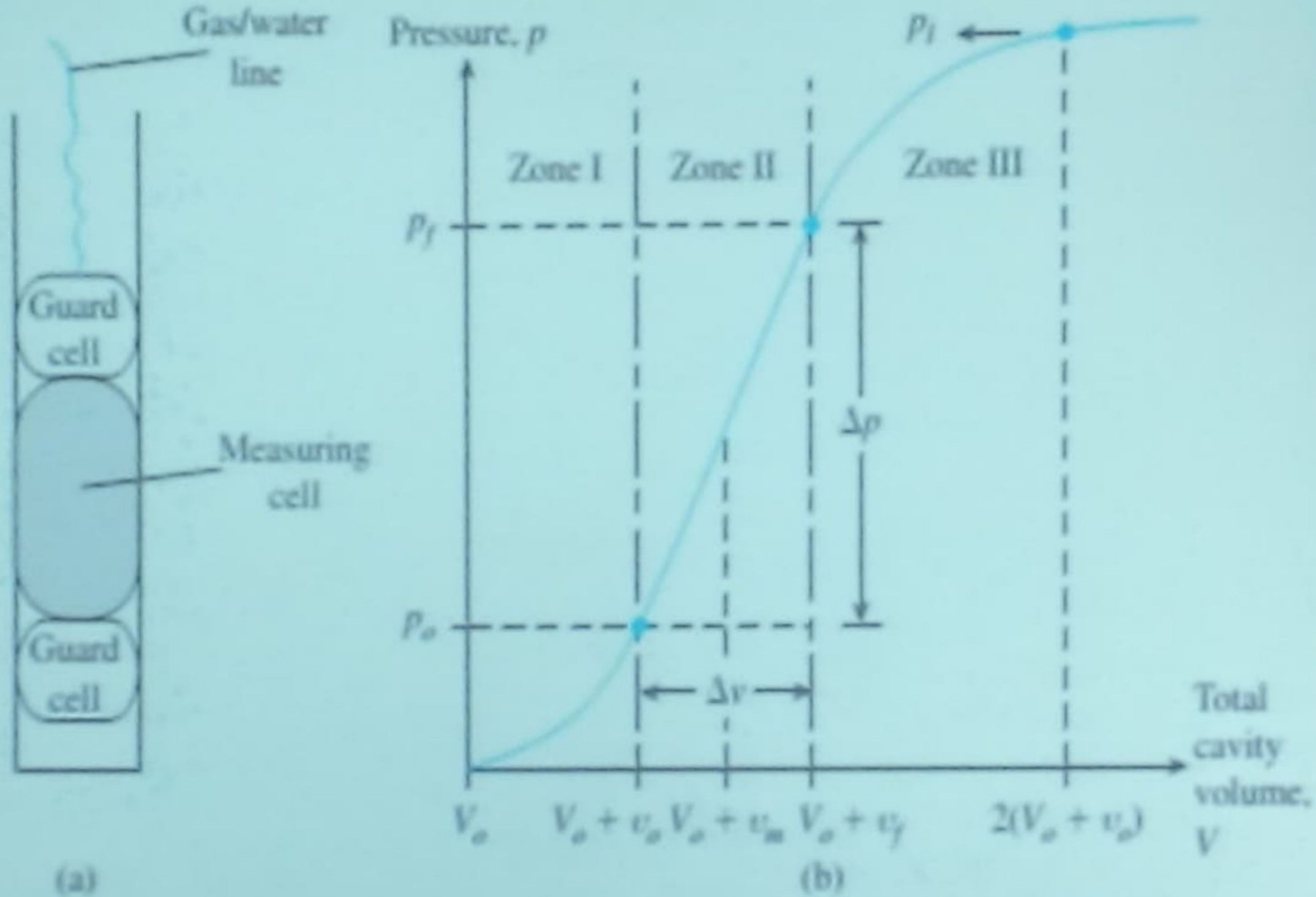
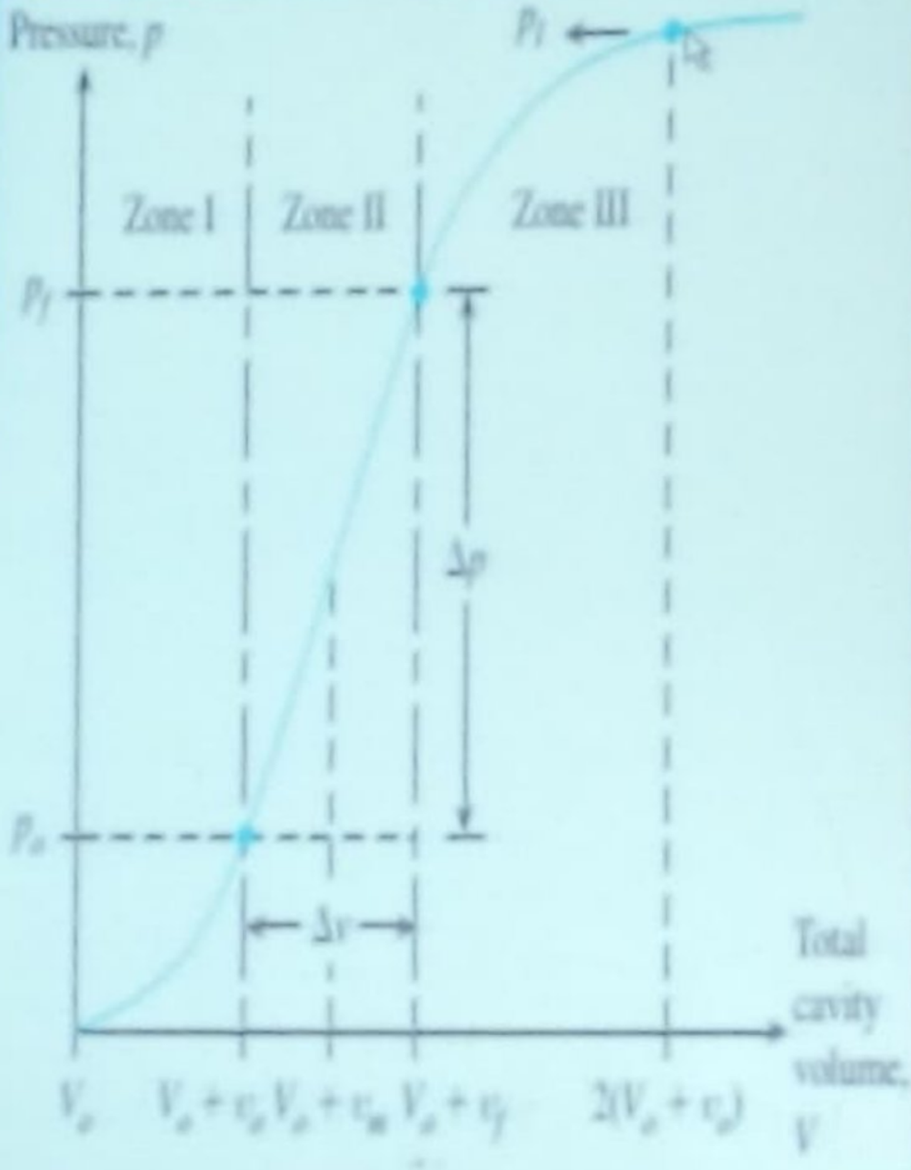
- ✓ Soil is considered to have failed when total volume of expanded cavity (V) is about twice volume of original cavity
- ✓ After completion of test, probe is deflated and advanced for testing at another depth

Probe diameter (mm)	Borehole diameter	
	Nominal (mm)	Maximum (mm)
44	45	53
58	60	70
74	76	89



(a) Pressuremeter; (b) Plot of pressure versus total cavity volume

- Results of pressuremeter test are expressed in graphical form of pressure versus volume
- In figure, Zone I represents reloading portion during which soil around borehole is pushed back into initial state (i.e., state it was in before drilling)
- Pressure p_0 represents in situ total horizontal stress
- Zone II represents a pseudoelastic zone in which cell volume versus cell pressure is practically linear
- p_f pressure represents creep, or yield, pressure
- Zone marked III is plastic zone
- Pressure p_l represents limit pressure



- Pressuremeter modulus, of soil is determined with use of theory of expansion of an infinitely thick cylinder

$$E_p = 2(1 + \mu_s)(V_o + v_m)\left(\frac{\Delta p}{\Delta v}\right)$$

where

$\left(\frac{\Delta p}{\Delta v}\right)$ = Slope of straight-line plot of Zone II

$$v_m = \frac{v_o + v_f}{2}$$

V_o = cell volume corresponding to pressure p_o (that is, cell pressure corresponding to beginning of Zone II)

$$\Delta p = p_f - p_o$$

$$\Delta v = v_f - v_o$$

μ_s = Poisson's ratio (which may be assumed to be 0.33)

- ✓ Limit pressure is usually obtained by extrapolation and not by direct measurement

- Correlations between various soil parameters and results obtained from pressuremeter tests have been developed by various investigators
- Kulhawy and Mayne (1990) proposed that, for clays,

$$\sigma'_c = 0.45 p_l$$

where σ'_c = preconsolidation pressure

- On basis of cavity expansion theory, Baguelin *et al.* (1978) proposed that

$$c_u = \frac{(p_l - p_o)}{N_p}$$

where

c_u = undrained shear strength of a clay

$$N_p = 1 + \ln\left(\frac{E_p}{3c_u}\right)$$

= 5 to 12 (average 8.5)

- Ohya *et al.* (1982) (also Kulhawy and Mayne, 1990) correlated E_p with field standard penetration numbers (N_{60}) for sand and clay as follows:

$$\text{Clay: } E_p(\text{kN/m}^2) = 1930N_{60}^{0.63}$$

$$\text{Sand: } E_p(\text{kN/m}^2) = 908N_{60}^{0.66}$$

Preparation of Boring Logs

- ✓ Detailed information gathered from each borehole is presented in a graphical form called boring log
- ✓ A standard log should provide following information
  - Name and address of drilling company
  - Driller's name
  - Job description and number
  - Number, type, and location of boring
  - Date of boring
  - Subsurface stratification, which can be obtained by visual observation of soil brought out by auger, split-spoon sampler, and thin-walled Shelby tube sampler
  - Elevation of water table and date observed, use of casing and mud losses, and so on
  - Standard penetration resistance and depth of SPT
  - Number, type, and depth of soil sample collected

Boring Log

Name of the Project: Two-story apartment building

Location: Johnson & Olive St. Date of Boring: March 2, 2005

Boring No. 3 Type of Boring: Hollow-stem auger Ground Elevation: 60.8 m

Soil description	Depth (m)	Soil sample type and number	N_{60}	w_n (%)	Comments
Light brown clay (fill)					
Silty sand (SM)	1				
	2	SS-1	9	8.2	
[a]G.W.T. 3.5 m	3	SS-2	12	17.6	LL = 38 PI = 11
Light gray clayey silt (ML)	4, 5	ST-1		20.4	LL = 36 q_u = 112 kN/m^2
	6	SS-3	11	20.6	
Sand with some gravel (SP) End of boring @ 8 m	7, 8	SS-4	27	9	

N_{60} = standard penetration number
w_n = natural moisture content
LL = liquid limit; PI = plasticity index
q_u = unconfined compression strength
SS = split-spoon sample; ST = Shelby tube sample

[a] Groundwater table observed after one week of drilling

Subsoil Exploration Report

- At end of all soil exploration programs, soil and rock specimens collected in field are subject to visual observation and appropriate laboratory testing
- After all required information has been compiled, a soil exploration report is prepared for use by design office and for reference during future construction work
- Although details and sequence of information in such reports may vary to some degree, depending on structure under consideration and person compiling report
- Exploration reports should be well planned and documented, as they will help in answering questions and solving foundation problems that may arise later during design and construction

- Each subsoil exploration report should include following items:
  - A description of scope of investigation
  - A description of proposed structure for which subsoil exploration has been conducted
  - A description of location of site, including any structures nearby, drainage conditions, nature of vegetation on site and surrounding it, and any other features unique to site
  - A description of the geological setting of site
  - Details of field exploration - that is, number of borings, depths of borings, types of borings involved, and so on

➢ Each subsoil exploration report should include following items:

- A general description of subsoil conditions, as determined from soil specimens and from related laboratory tests, standard penetration resistance and cone penetration resistance, and so on
- A description of water table conditions
- Recommendations regarding foundation, including type of foundation recommended, allowable bearing pressure, and any special construction procedure that may be needed; alternative foundation design procedures should also be discussed in this portion of report
- Conclusions and limitations of investigations

AC-V

Ini. wt = 3200 gm.

load = 400 KNm.

time = 10 ± 1 min

$P_a = 1.0706 P_m + 28.638$

P_a = Aactual Load

P_m = machine Load

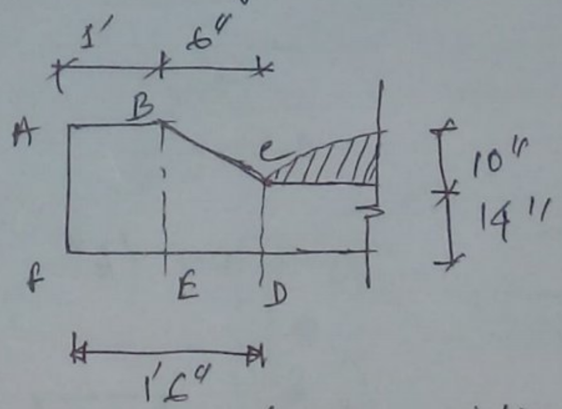
Retain = 2836 gm.

R.C.C LAB

4th day

19-01-17

Kerb/curb/Edge beam Design.



(1) Load coming from railing and rail post

~~D.L from curb~~ = $\left(L \times \frac{5 \times 5}{144} \times 2 + \frac{8 \times 5}{144} \times H_{RP} \times N_{RP}\right) \times 150$

= plf

FOUNDATION

- ✓ Lowest part of a structure generally is referred to as foundation
- ✓ Transfers load from superstructure to footing

FOOTING

- ✓ A footing is a foundation unit constructed in brick work, masonry or concrete under base of a wall or a column for purpose of distributing load over a large area of soil on which it rests on
- ➢ In simple words, foundation means legs and footing means foot of leg

TYPES OF FOUNDATION

- Depending on structure and soil encountered, two types of foundations are used
  - Shallow foundation (**Depth/width ≤ 4**)
    - Spread footing/Isolated footing
      - Square footing
      - Rectangular footing
      - Circular footing
      - Strap footing
      - Combined footing
    - Mat foundation
  - Deep foundation (**Depth/width > 4**)
    - Pile foundation
    - Drilled shaft
    - Well foundation or Caisson

SPREAD FOOTING

- A spread footing is simply an enlargement of a load-bearing wall or column that makes it possible to spread load of structure over a larger area of soil

Square/Rectangular/Circular footing

SPREAD FOOTING

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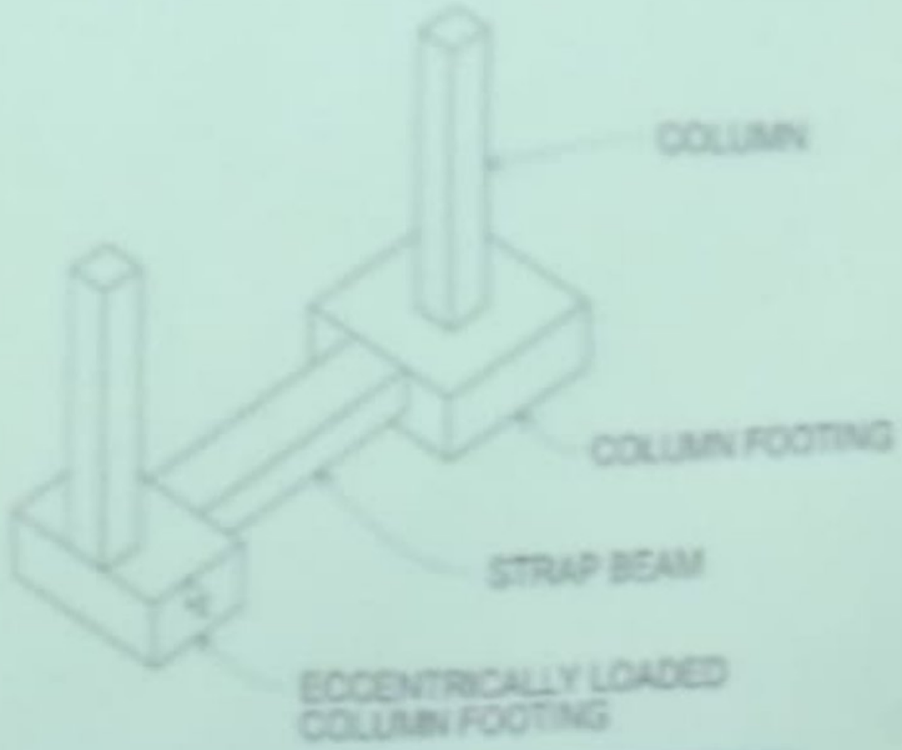

Strap footing

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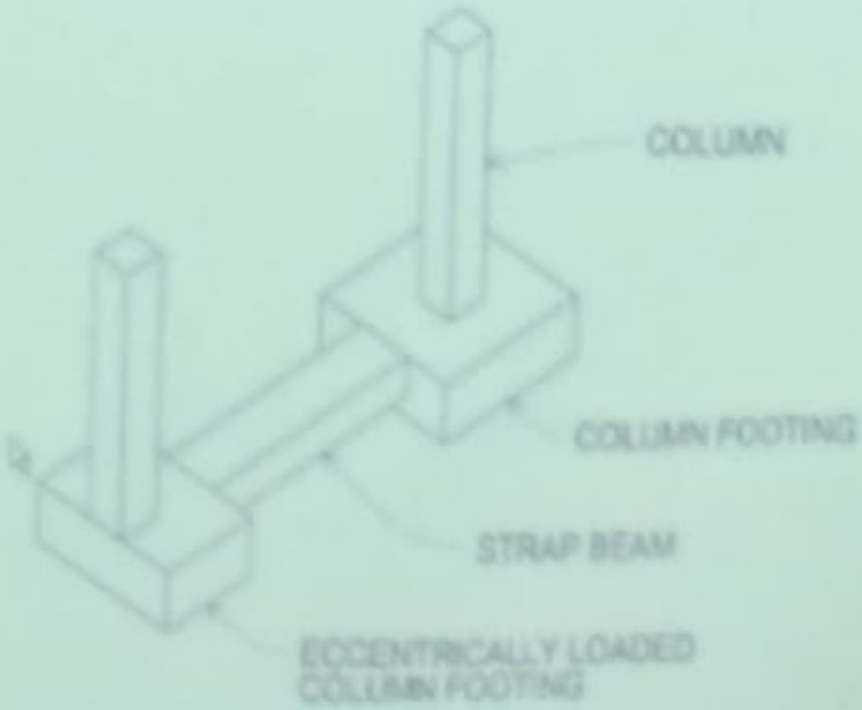

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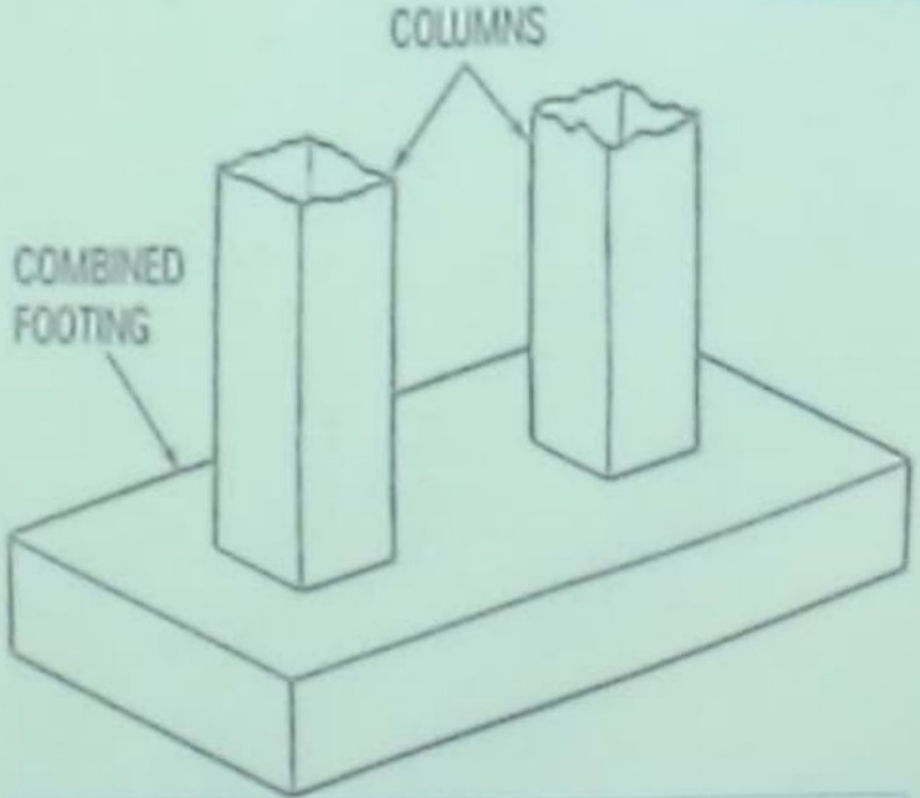
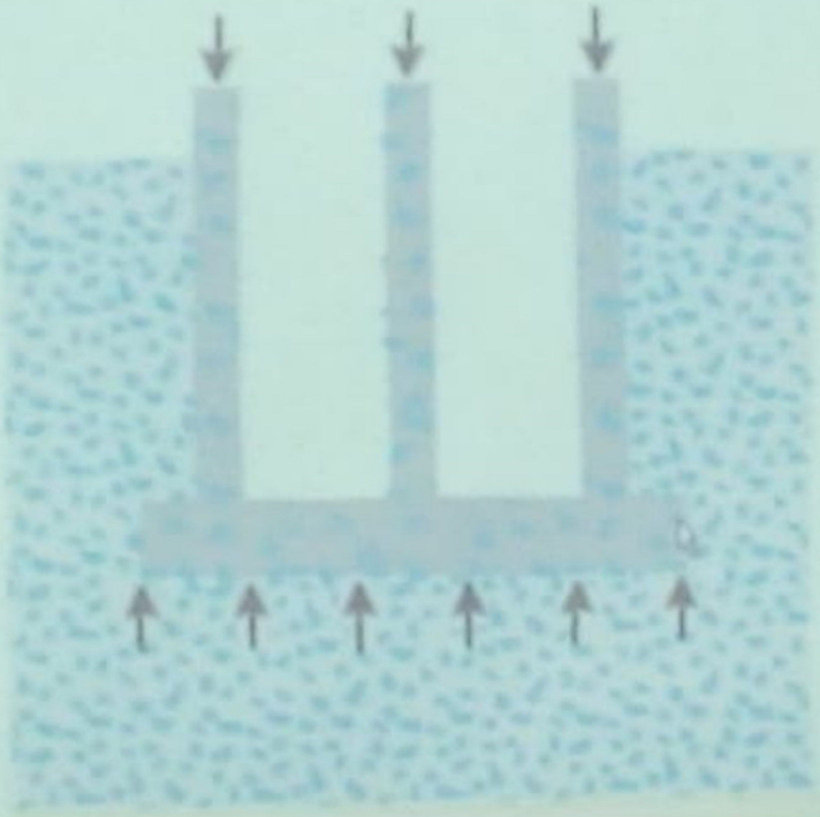


Combined footing

MAT FOUNDATION

- ✓ In soil with low load-bearing capacity, size of spread footings required is impracticably large
- ✓ In that case, it is more economical to construct entire structure over a concrete pad called a mat foundation



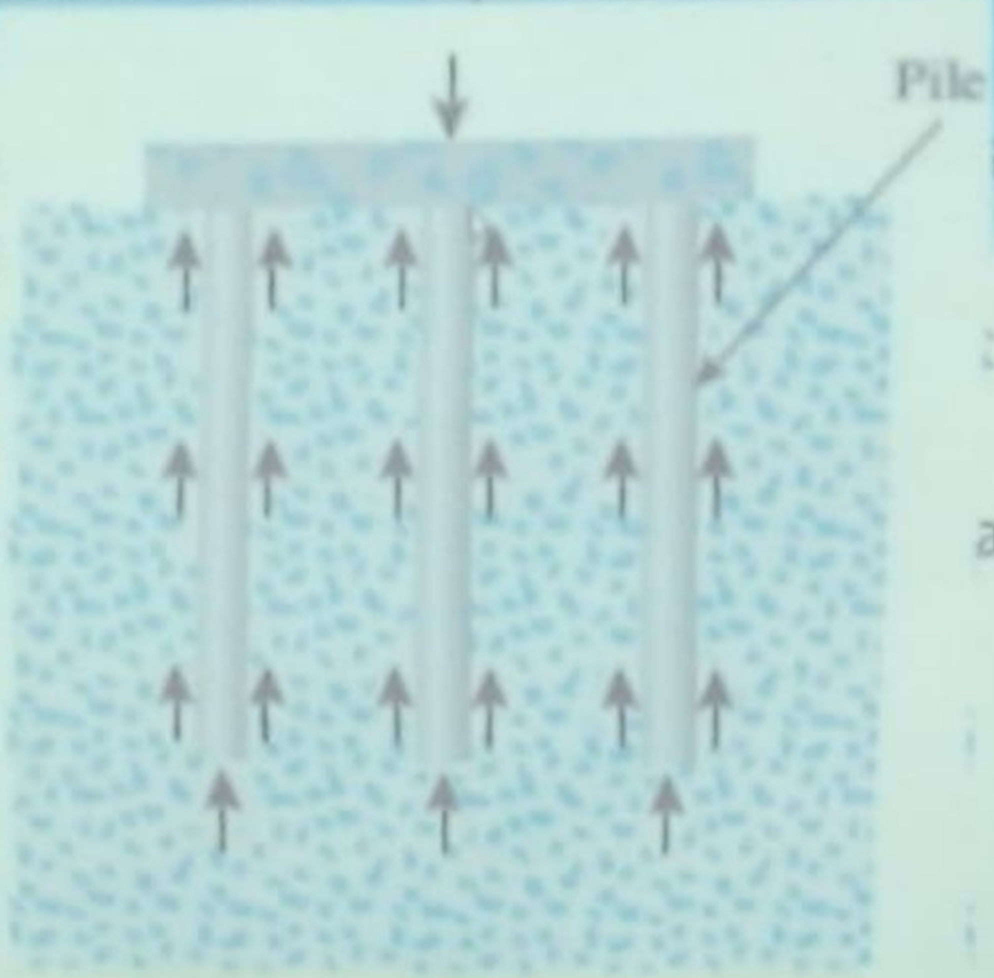
Mat foundation

PILE FOUNDATION

- ✓ Piles are structural members made of timber, concrete, or steel that transmit load of superstructure to lower layers of soil
- ✓ According to how they transmit their load into subsoil, piles can be divided into two categories:
  - **Fiction piles** - superstructure load is resisted by shear stresses generated along surface of pile
  - **End-bearing piles** - load carried by pile is transmitted at its tip to a firm stratum

PILE FOUNDATION

- Piles are structural members made of timber, concrete, or steel that transmit load of superstructure to lower layers of soil
- According t[…]ubsoil, piles can be divid[…]
  - **Fictio**[…]ed by shear stress[…]
  - **End-b**[…]ansmitted at its tip



Pile foundation

DRILLED SHAFT

- ✓ In case of drilled shafts, a shaft is drilled into subsoil and then is filled with concrete
- ✓ A metal casing may be used while shaft is being drilled
- ✓ Casing may be left in place or may be withdrawn during placing of concrete
- ✓ Generally, diameter of a drilled shaft is much larger than that of a pile
- ✓ Distinction between piles and drilled shafts becomes hazy at an approximate diameter of 1 m (3 ft)

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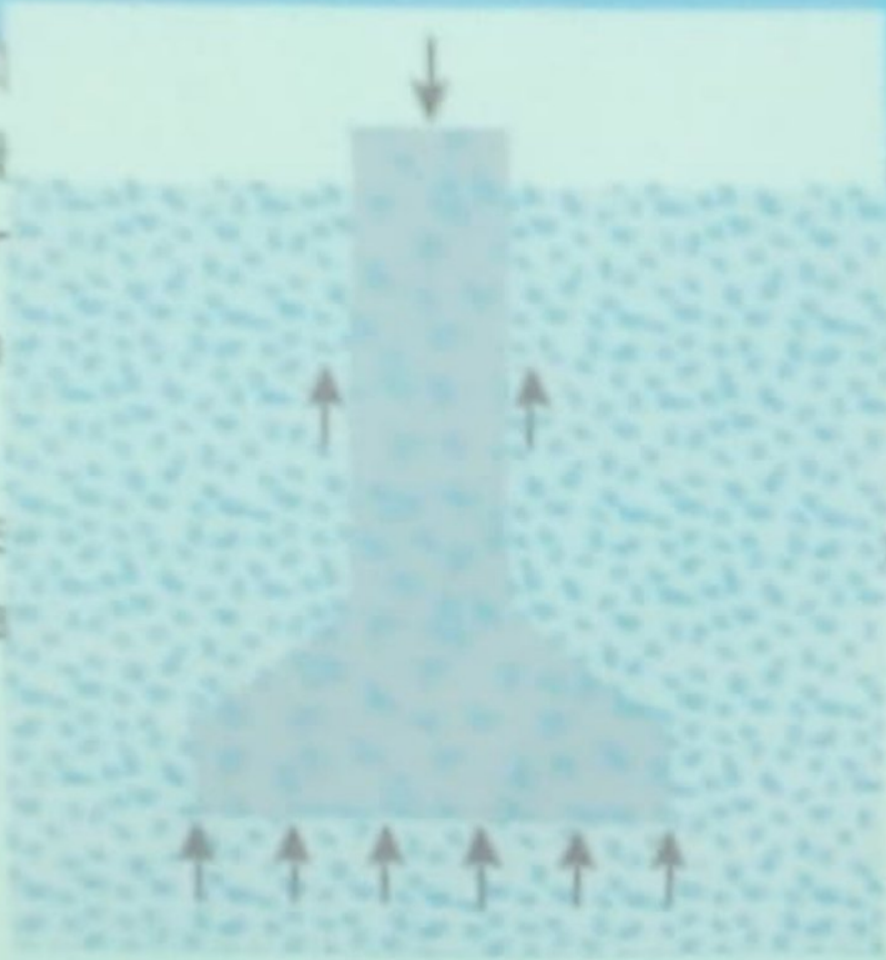
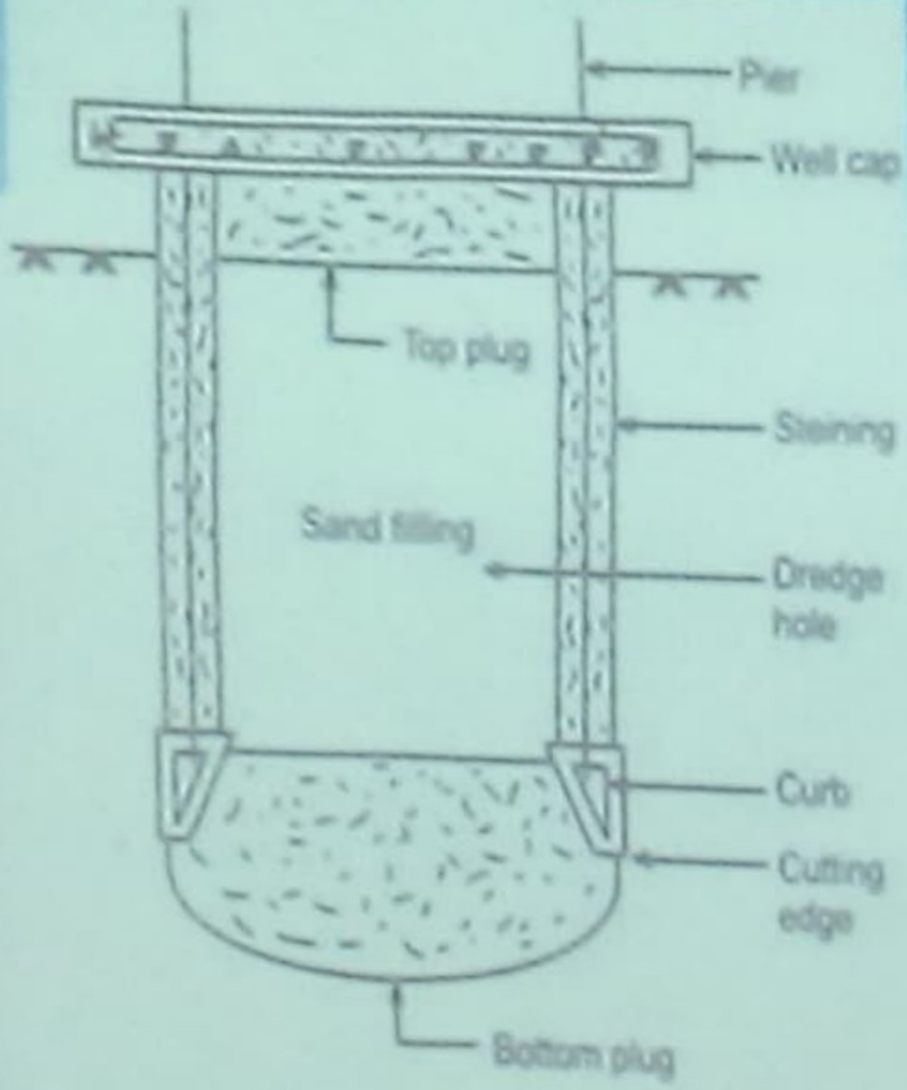


Drilled shaft

WELL FOUNDATION/CAISSON

- ✓ A large watertight retaining structure used to work on foundations of a bridge pier or for construction of a concrete dam
- ✓ These are constructed such that water can be pumped out, keeping the working environment dry and in which construction work may be carried out under water

WELL FOUNDATION/CAISSON



Well foundation

structure used to work on
or construction of a concrete

t water can be pumped out,
nment dry and in which
d out under water

WELL FOUNDATION/CAISSON

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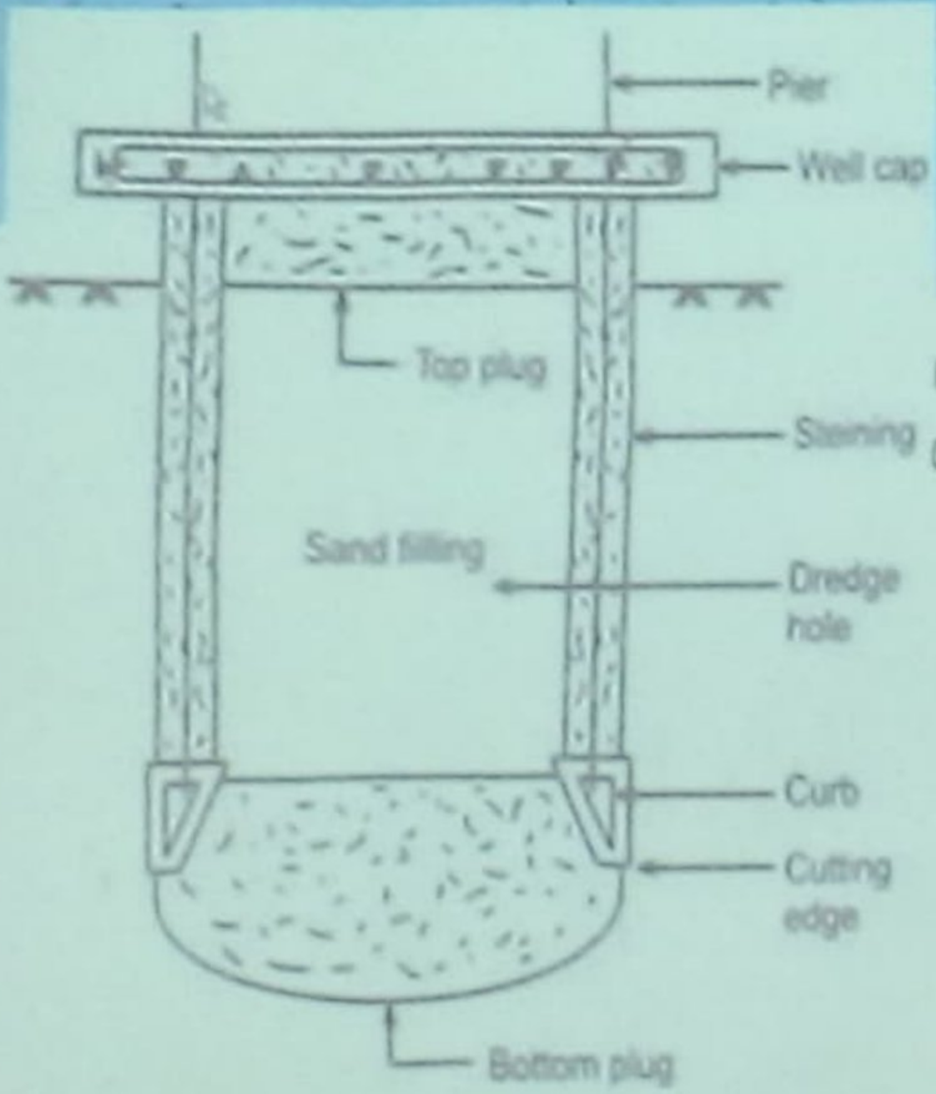
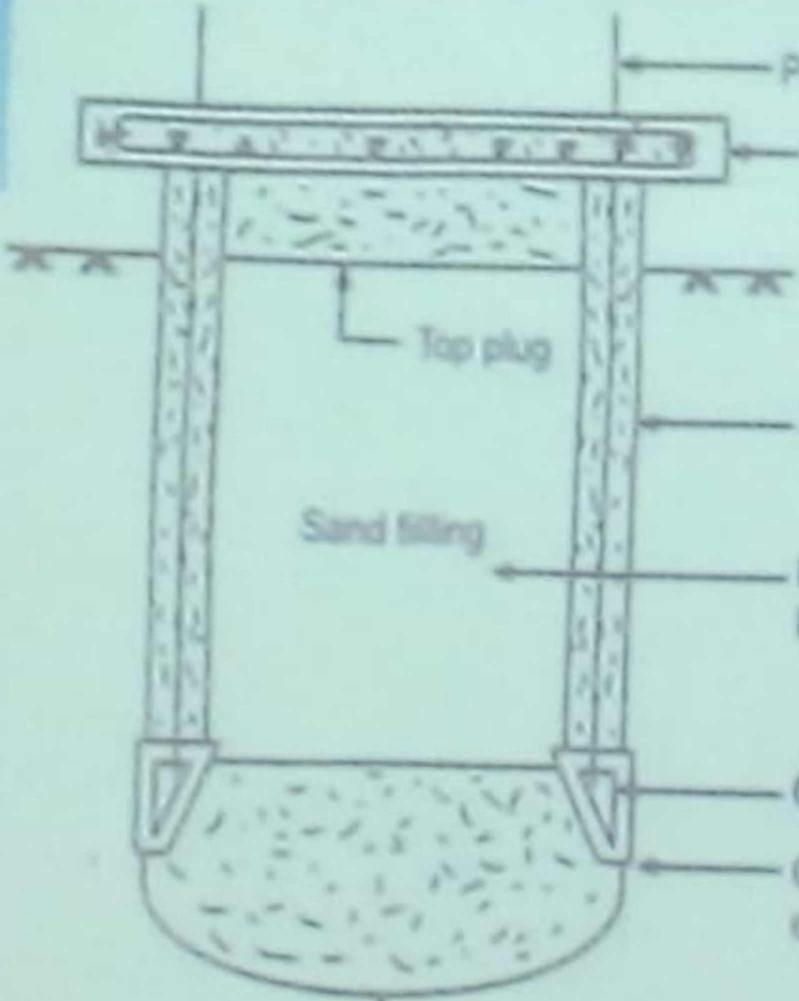
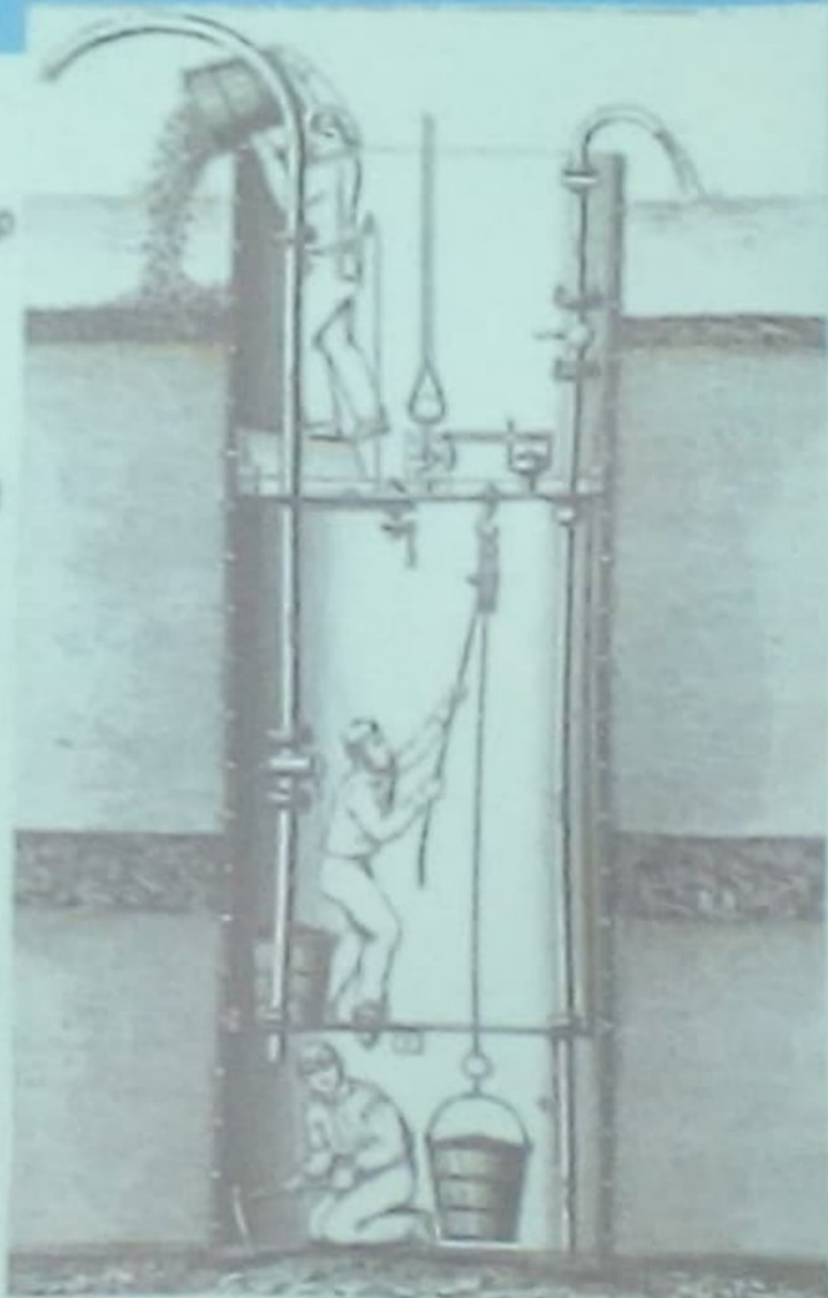


Well foundation

WELL FOUNDATION/CAISSON



Well foundation



- To perform satisfactorily, shallow foundations must have two main characteristics:
  - ✓ They have to be safe against overall shear failure in soil that supports them
  - ✓ They cannot undergo excessive displacement, or settlement (term excessive is relative, because degree of settlement allowed for a structure depends on several considerations)

ULTIMATE BEARING CAPACITY

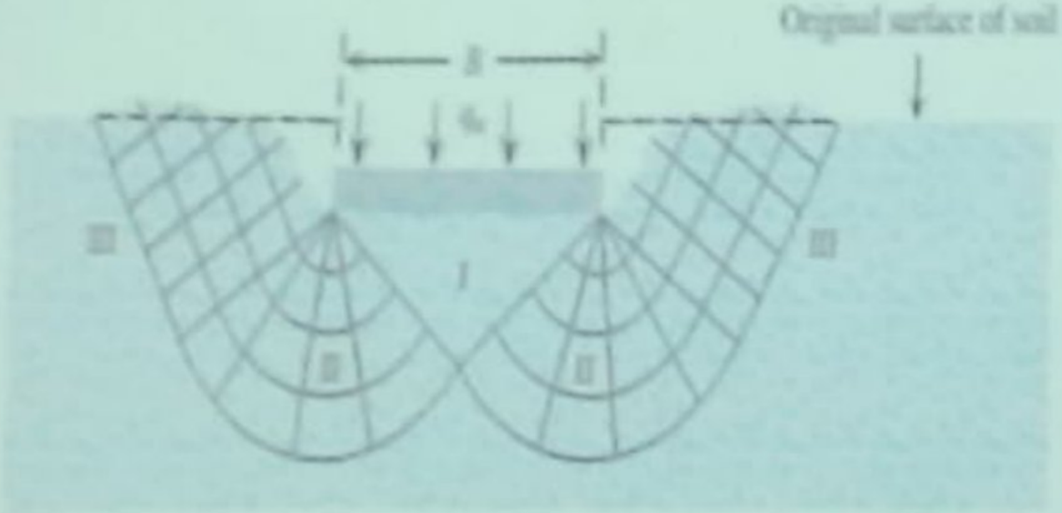
- ✓ Load per unit area of foundation at which shear failure in soil occurs is called ultimate bearing capacity

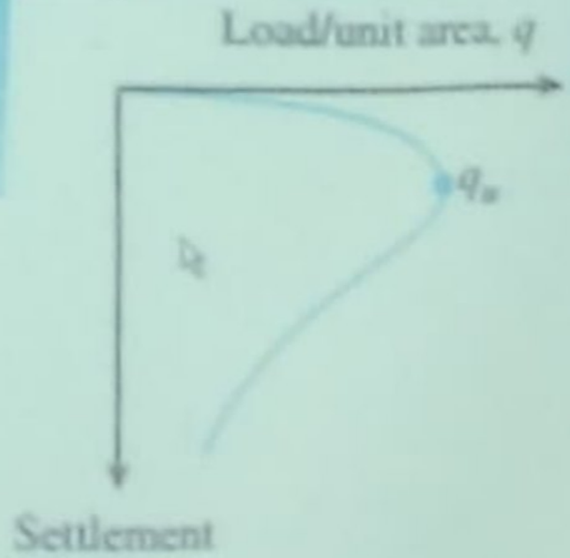
MODE OF SHEAR FAILURE OF SOIL

- General shear failure
- Local shear failure
- Punching shear failure

GENERAL SHEAR FAILURE

- ✓ Failure surface in soil will extend to ground surface
- ✓ Occurs in a dense sand or stiff cohesive soil

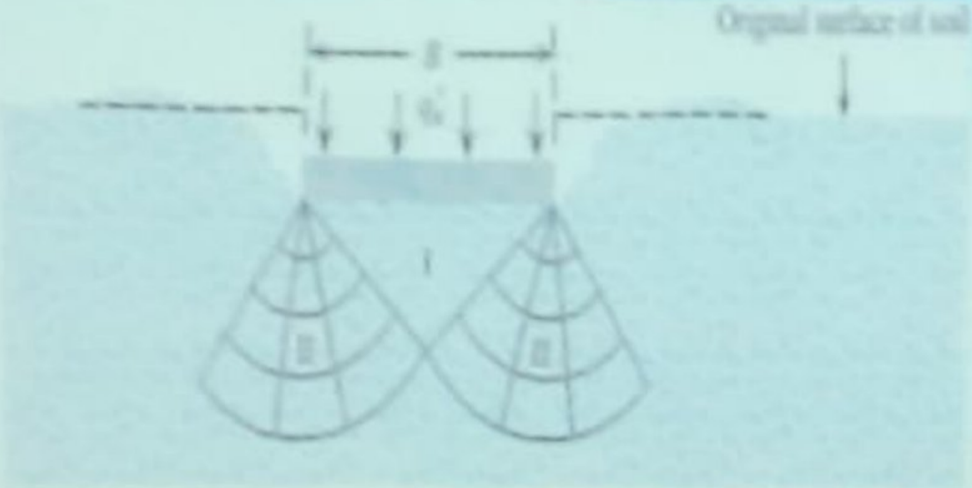


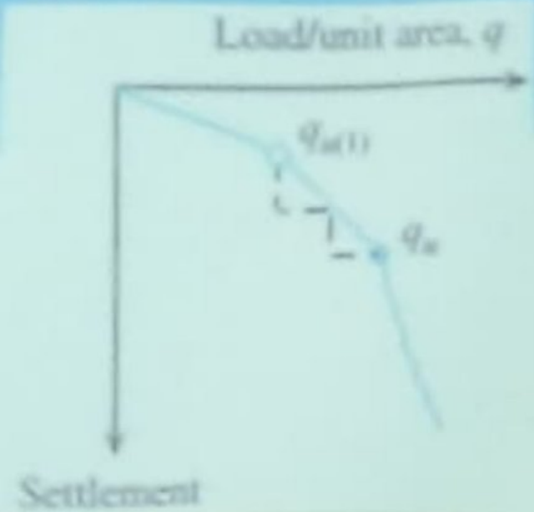


- ✓ If a load is gradually applied to foundation, settlement will increase
- ✓ At a certain point - when load per unit area equals q_u - a sudden failure in soil supporting foundation will take place
- ✓ This load per unit area, is usually referred to as ultimate bearing capacity of the foundation
- ✓ When such sudden failure in soil takes place, it is called general shear failure

LOCAL SHEAR FAILURE

- Failure surface in soil will gradually extend outward from foundation and do not or little extend to ground surface
- Occurs in sand or clayey soil of medium compaction

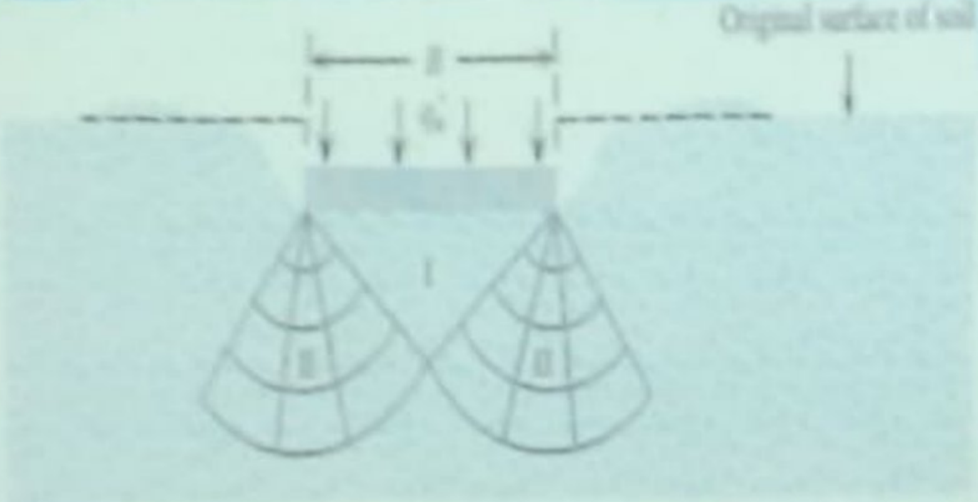


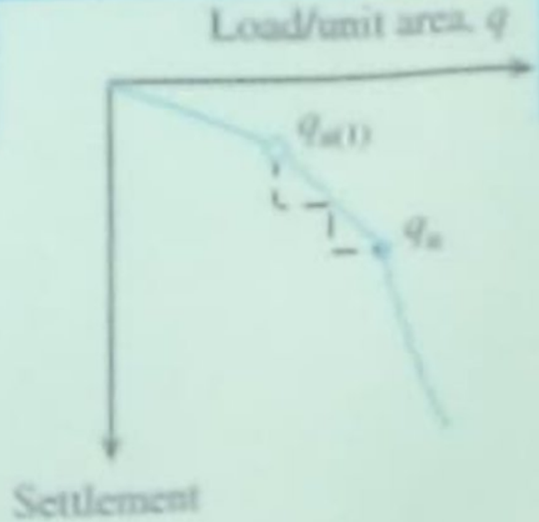


- When load per unit area on foundation equals $q_{u(1)}$ movement of foundation will be accompanied by sudden jerks
- A considerable movement of foundation is then required for failure surface in soil to extend to ground surface (as shown by broken lines in figure)
- load per unit area at which this happens is ultimate bearing capacity, q_u

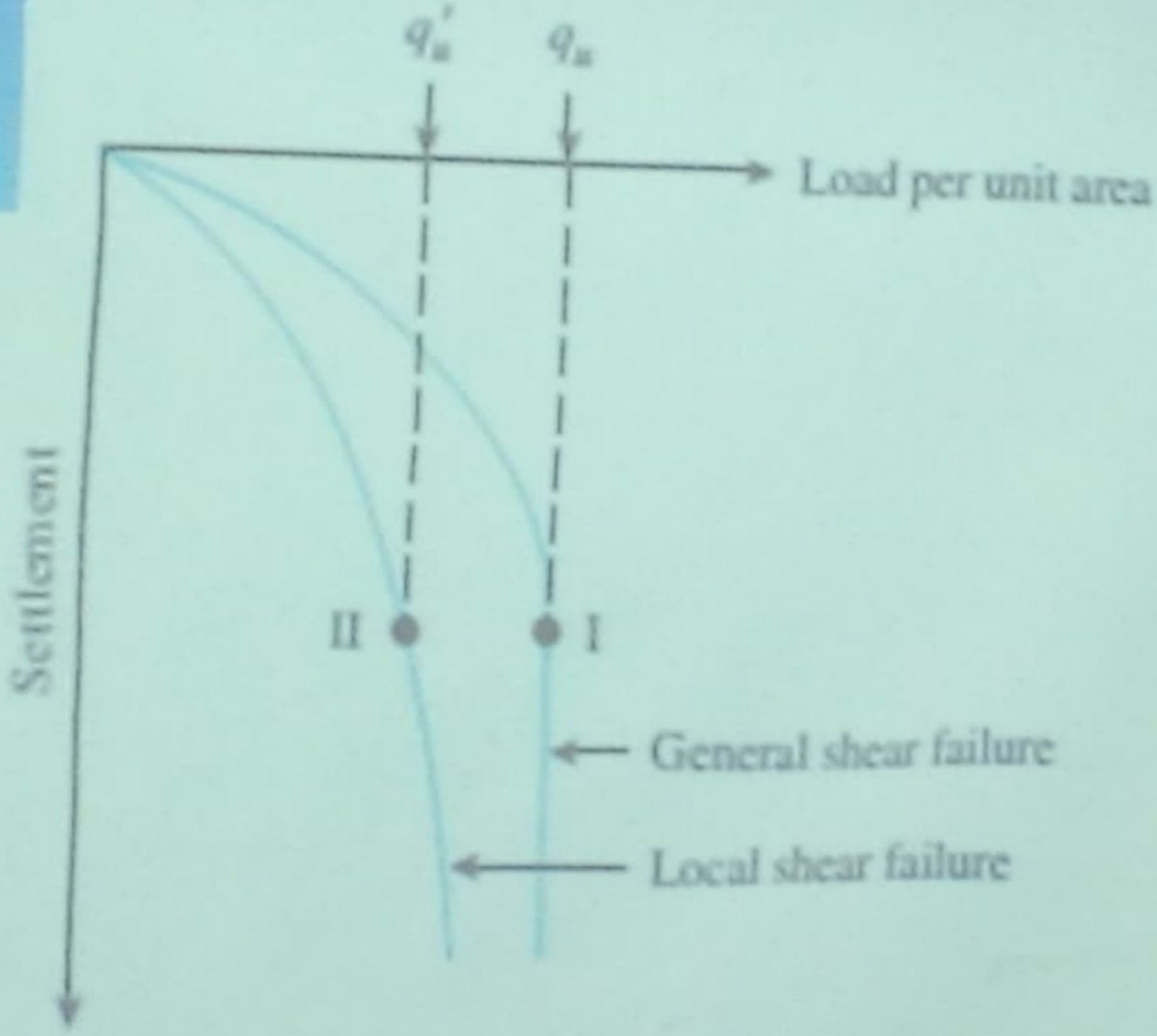
LOCAL SHEAR FAILURE

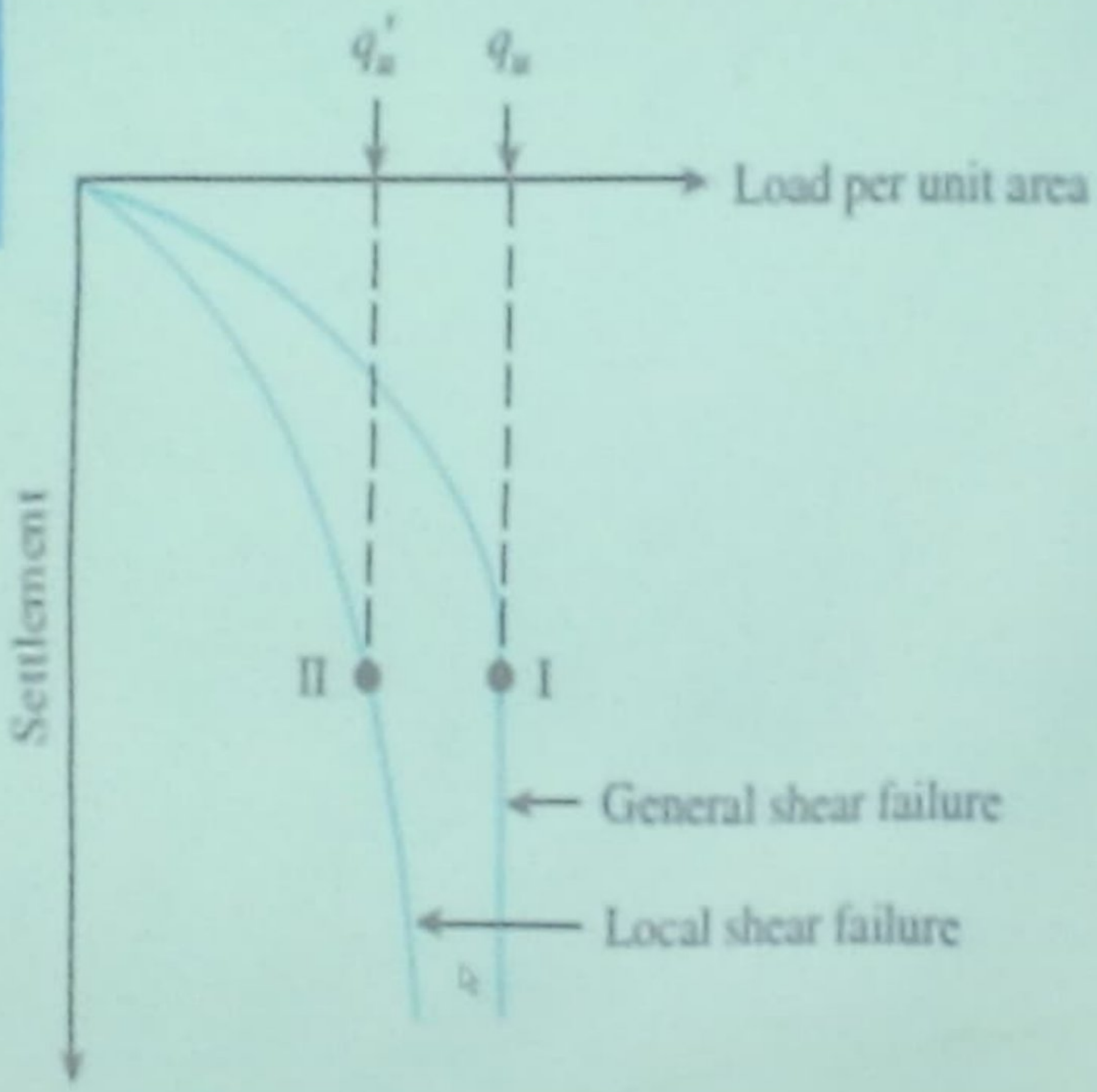
- Failure surface in soil will gradually extend outward from foundation and do not or little extend to ground surface
- Occurs in sand or clayey soil of medium compaction





- Beyond that point, an increase in load will be accompanied by a large increase in foundation settlement
- Load per unit area of foundation, $q_{u(1)}$ is referred to as first failure load (Vesic, 1963)
- A peak value of q is not realized in this type of failure

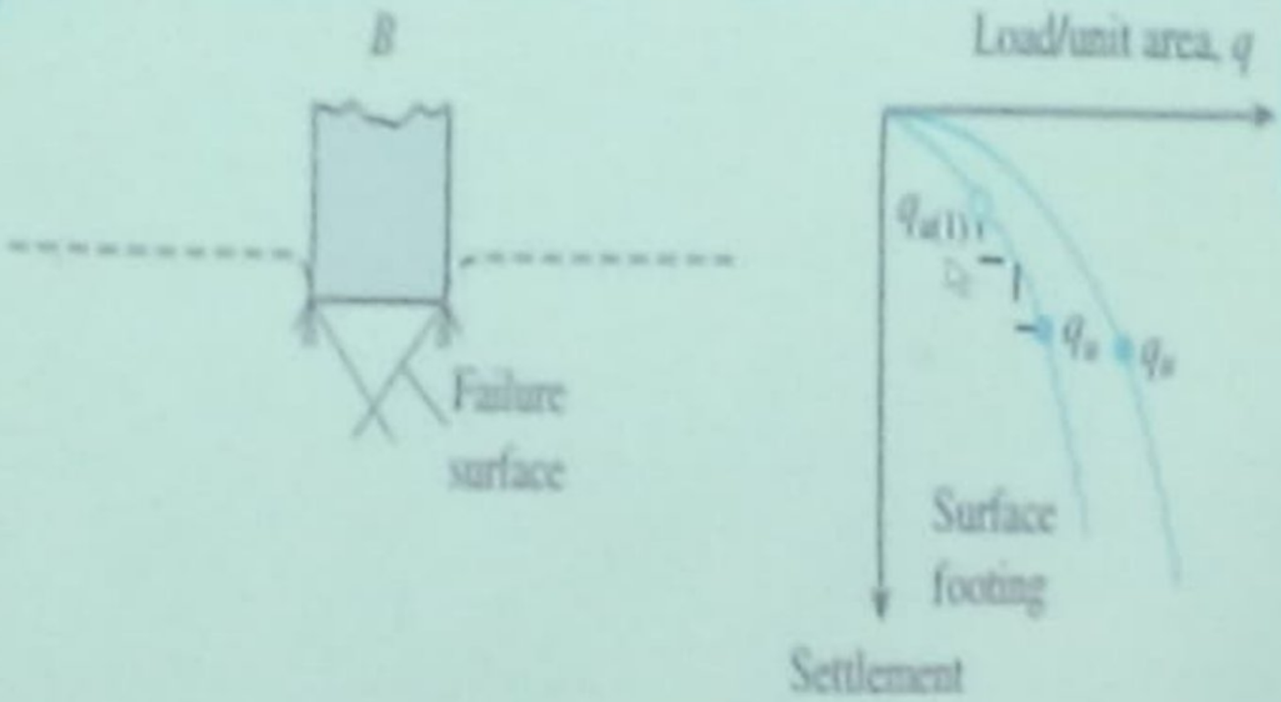

q_u'
q_u
Load per unit area
Settlement
II
I
General shear failure
Local shear failure



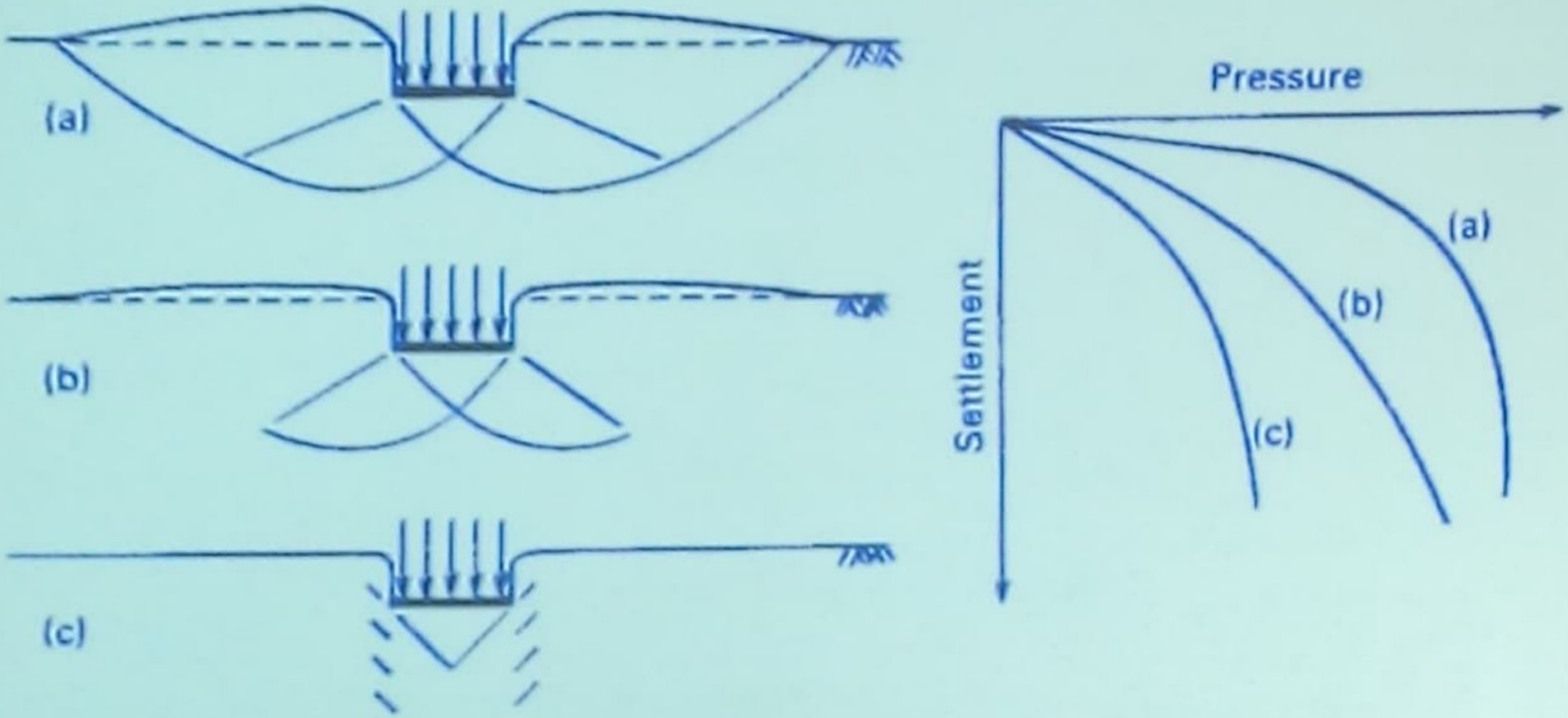
q_u'
q_u
Load per unit area
Settlement
II
I
General shear failure
Local shear failure

PUNCHING SHEAR FAILURE

- ✓ Failure surface in soil will not extend to ground surface
- ✓ Occurs in a fairly loose soil



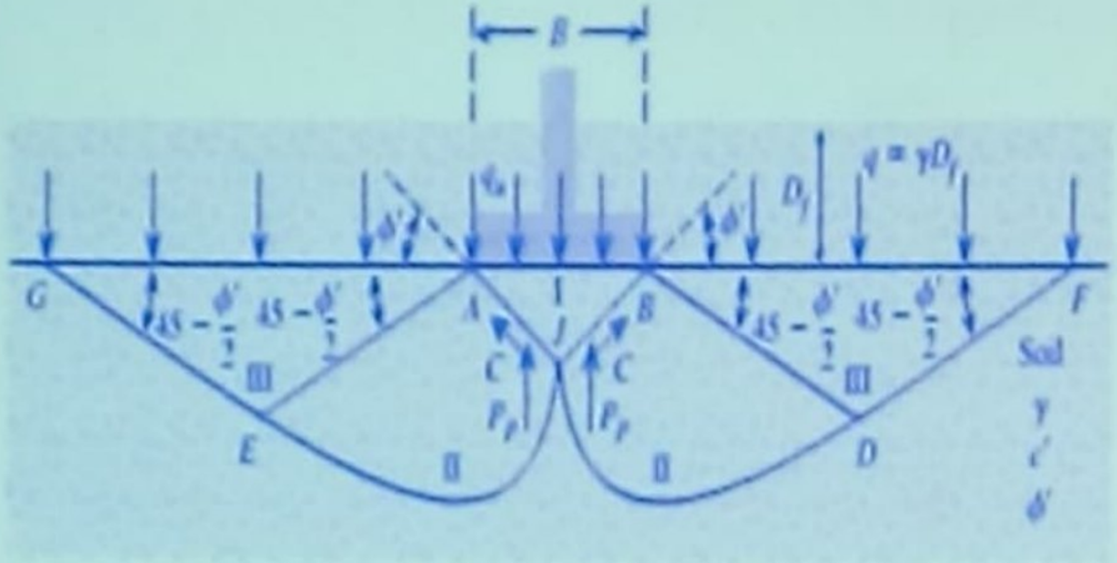
- ✓ Beyond ultimate failure load, q_u, load - settlement plot will be steep and practically linear

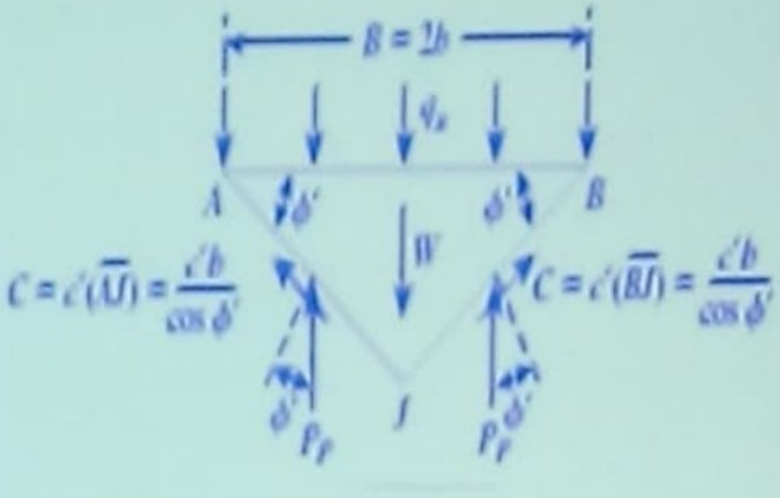


Modes of failure: (a) general shear, (b) local shear, (c) punching shear.

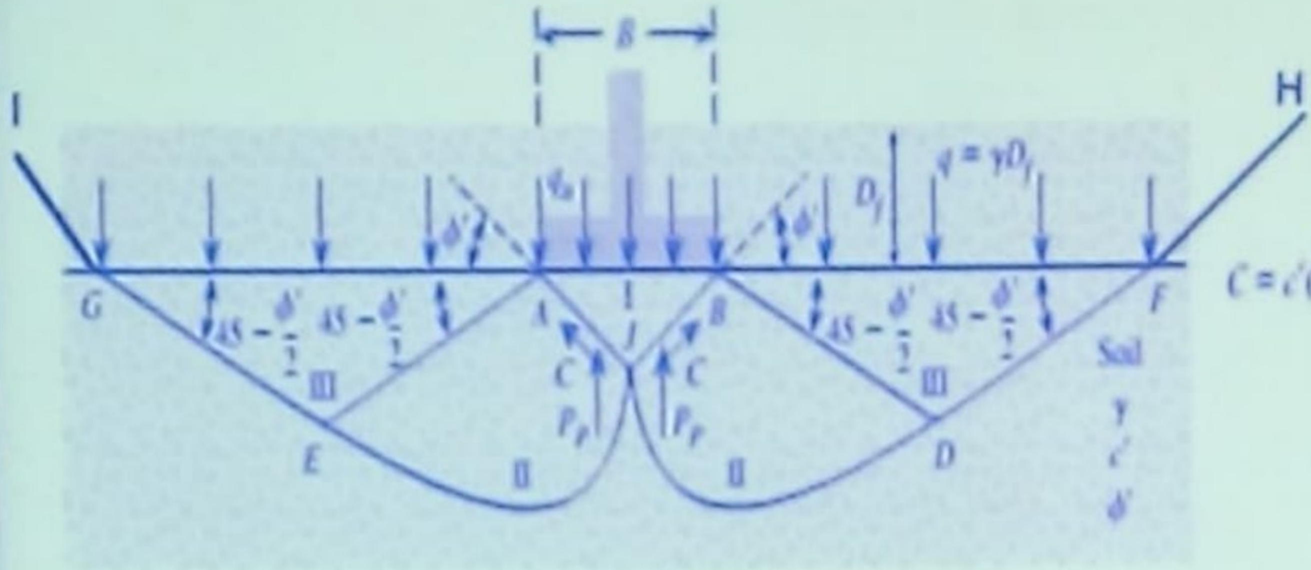
TERZAGHI'S BEARING CAPACITY THEORY

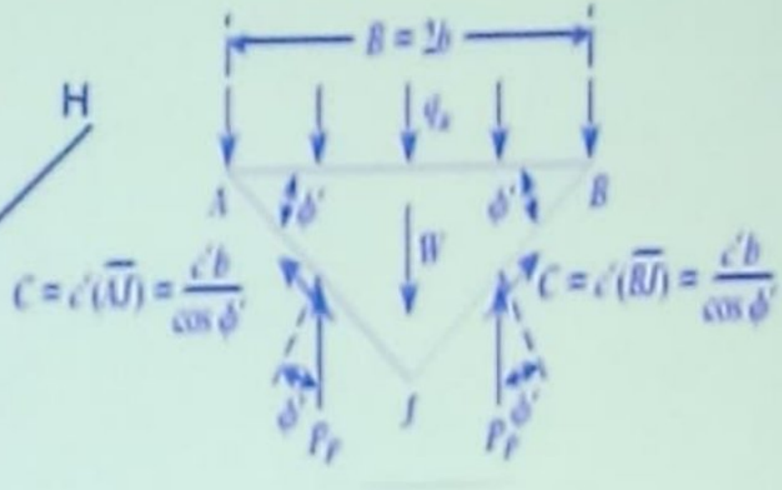
- ✓ Terzaghi (1943) was first to present a comprehensive theory for evaluation of ultimate bearing capacity of rough shallow foundations
- ✓ According to this theory, a foundation is shallow if its depth, is less than or equal to its width
- ✓ Later investigators, however, have suggested that foundations with equal to 3 to 4 times their width may be defined as shallow foundations



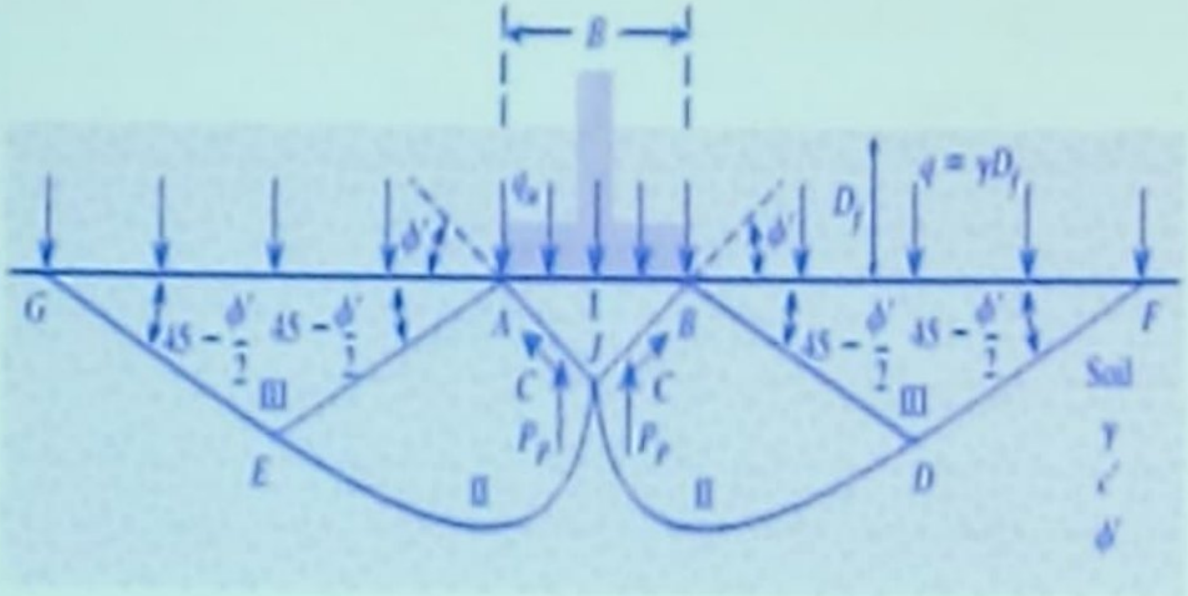


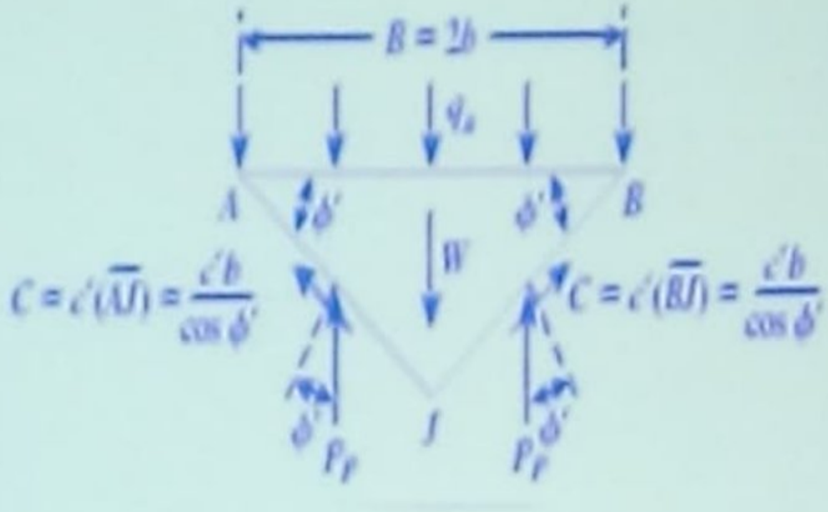
- ✓ Terzaghi suggested that for a continuous, or strip foundation (width to length ratio approaches zero), failure surface in soil at ultimate load may be assumed to be similar to that shown in Figure (case of general shear failure)
- ✓ Effect of soil above bottom of foundation may also be assumed to be replaced by an equivalent surcharge, $q = \gamma D_f$ (where γ is unit weight of soil)





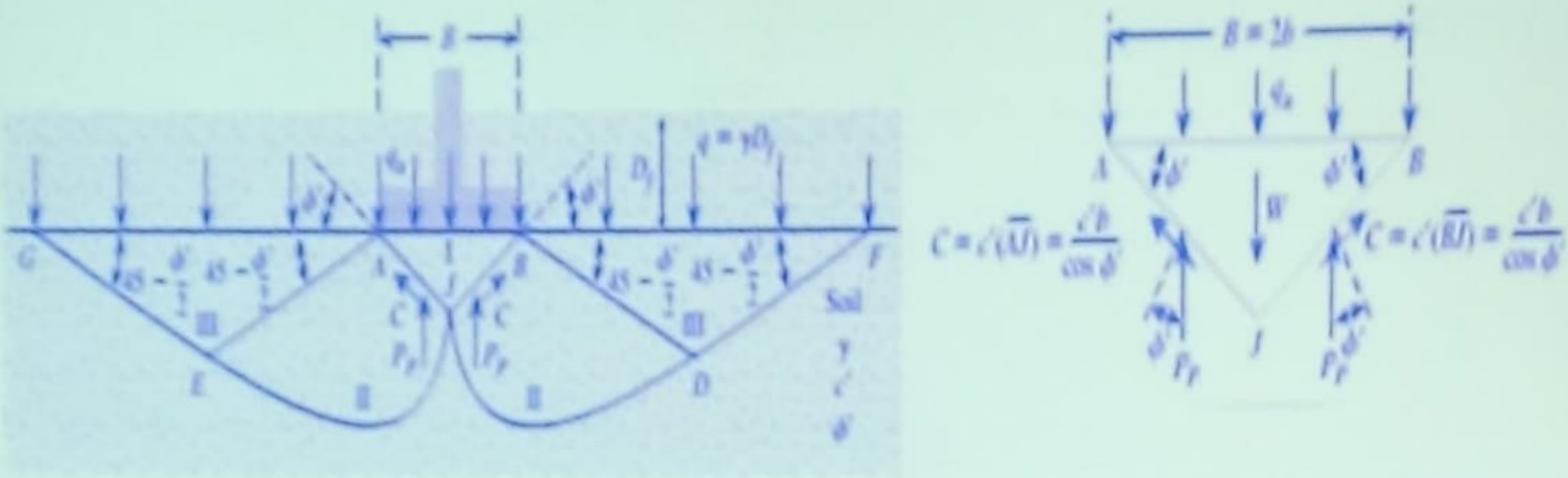
- ✓ Failure zone under foundation can be separated into three parts
  - ➡ Triangular zone ABJ (Zone I) immediately under foundation (elastic zone)
  - ➡ Radial shear zones AJE and BJD (Zone II), with curves JE and JD being arcs of a logarithmic spiral
  - ➡ Two triangular Rankine passive zones AEG and BDF (Zone III)
- ✓ Angles BAJ and ABJ are assumed to be equal to soil friction angle ϕ'



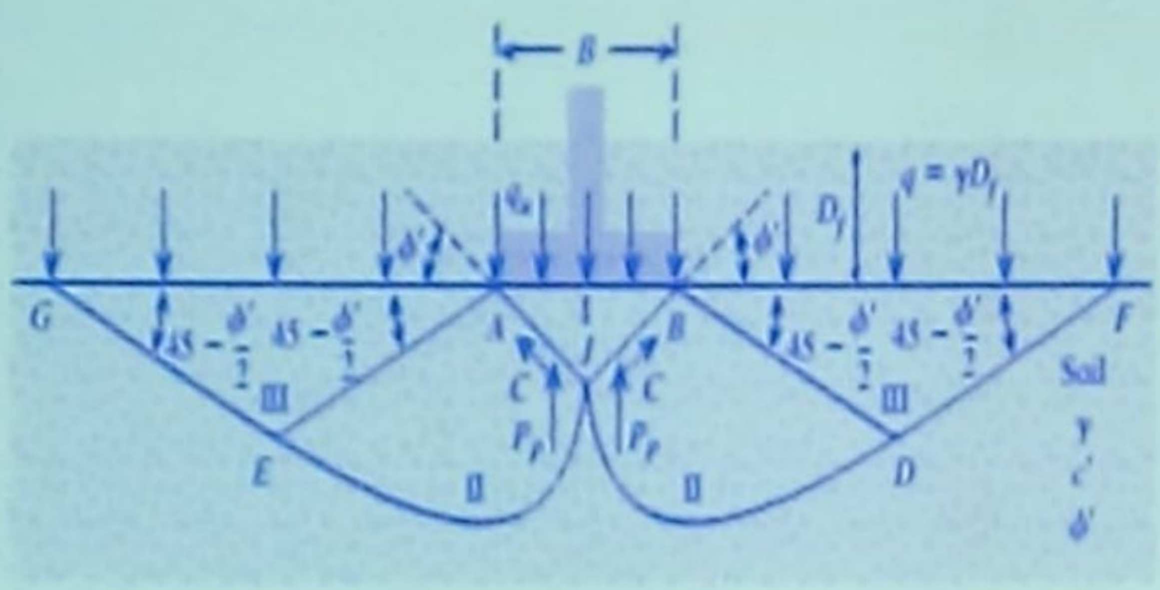


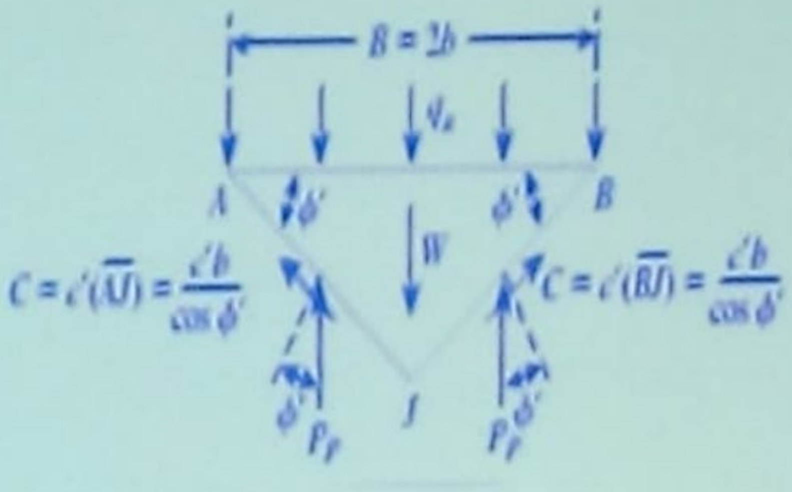
- ✓ With replacement of soil above bottom of foundation by an equivalent surcharge q, shear resistance of soil along failure surfaces GI and FH was neglected
- ✓ Equation of arcs of the logarithmic spirals JD and JE may be given as

$$r = r_0 e^{\theta \tan\emptyset'}$$



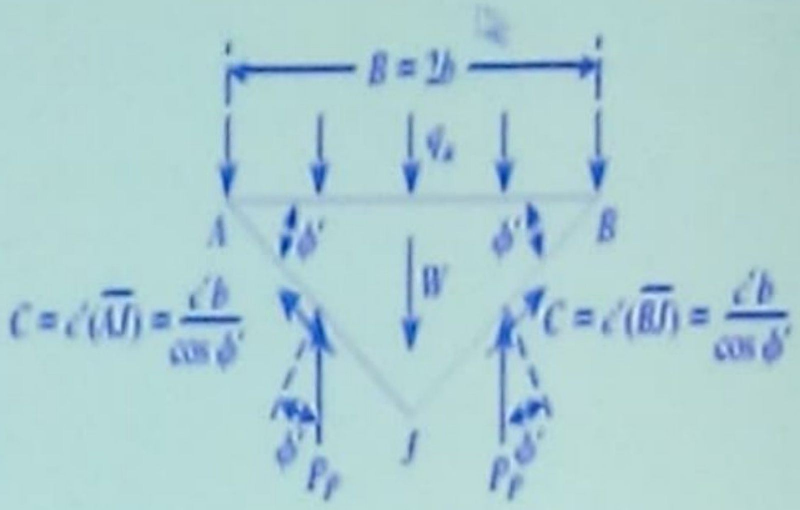
- If the load per unit area, q_u, is applied to footing and general shear failure occurs, passive force P_p is acting on each of faces of soil wedge ABJ
- This concept is easy to conceive of if we imagine that AJ and BJ are two walls that are pushing soil wedges AJEG and BJDF, respectively, to cause passive failure
- P_p should be inclined at an angle δ' (which is the angle of wall friction) to perpendicular drawn to wedge faces (that is, AJ and BJ)





- ✓ In this case, δ' should be equal to angle of friction of soil, φ' Because AJ and BJ are inclined at an angle φ' to horizontal, direction of P_p should be vertical

- ✓ Now let us consider free-body diagram of wedge ABJ as shown in Figure
- ✓ Considering unit length of the footing, we have, for equilibrium



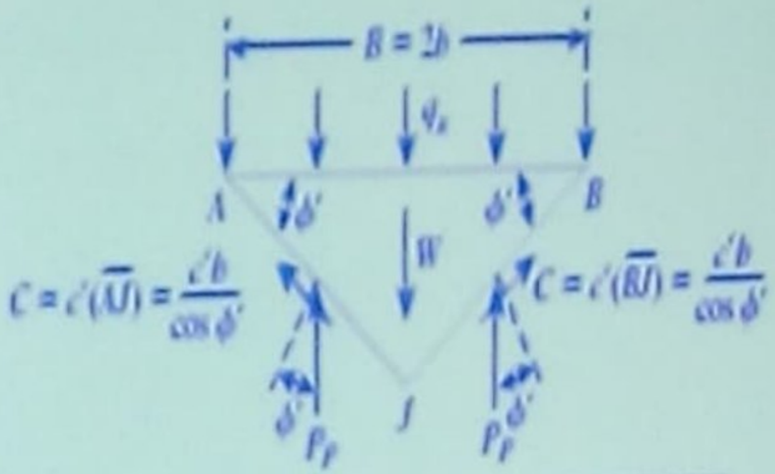
$$(q_u)(2b)(1) = -W + 2C\sin\phi' + 2P_p$$

where $b = B/2$

W = weight of soil wedge $ABJ = \gamma b^2 \tan\phi'$

C = cohesive force acting along each face, AJ and BJ, that is equal to the unit cohesion times the length of each face $= c'b/(\cos\phi')$

- Now let us consider free-body diagram of wedge ABJ as shown in Figure
- Considering unit length of the footing, we have, for equilibrium



- The passive pressure is sum of contribution of weight of soil γ, cohesion c', and surcharge q, and can be expressed as

$$q_u = \frac{P_p}{b} + c' \tan \phi' - \frac{\gamma b}{2} \tan \phi'$$

The passive pressure in Eq. (16.2) is the sum of the contribution of soil γ, cohesion c', and surcharge q and can be expressed as

$$P_p = \frac{1}{2}\gamma(b \tan \phi')^2 K_\gamma + c'(b \tan \phi')K_c + q(b \tan \phi')K_q$$

where K_γ, K_c, and K_q are earth-pressure coefficients that are functions of th angle, ϕ'.

Combining Eqs. (16.3) and (16.4), we obtain

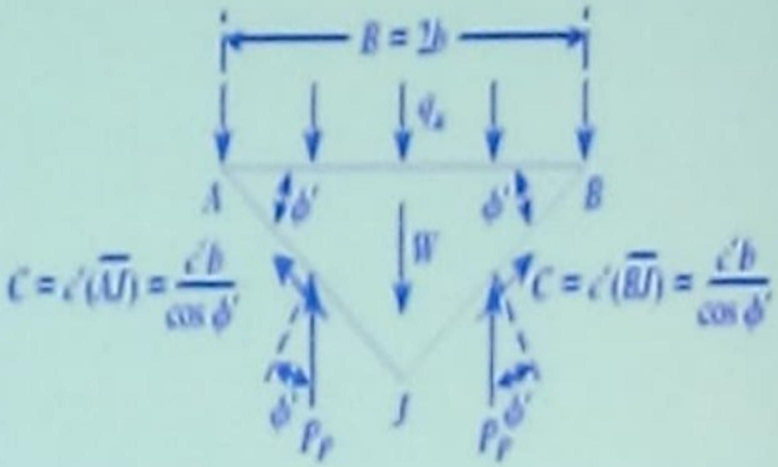
$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

where

$$N_c = \tan \phi'(K_c + 1)$$

$$N_q = K_q \tan \phi'$$

- Now let us consider free-body diagram of wedge ABJ as shown in Figure
- Considering unit length of the footing, we have, for equilibrium



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$$P_p = \tfrac{1}{2}\gamma(b \tan \phi')^2 K_\gamma + c'(b \tan \phi')K_c + q(b \tan \phi')K_q$$

Where, K_γ, K_c, and K_q are earth-pressure coefficients that are functions of soil friction angle, φ'

$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

$$N_c = \tan\phi'(K_c + 1)$$

$$N_q = K_q \tan\phi'$$

$$N_\gamma = \tfrac{1}{2}\tan\phi'(K_\gamma \tan\phi' - 1)$$

$$N_c = \tan\phi'(K_c + 1)$$

$$N_q = K_q \tan\phi'$$

$$N_\gamma = \tfrac{1}{2}\tan\phi'(K_\gamma \tan\phi' - 1)$$

- ✓ N_c, N_q, and N_γ are, respectively, contributions of cohesion, surcharge, and unit weight of soil to ultimate load-bearing capacity
- ✓ It is extremely tedious to evaluate K_c, K_q, and K_γ
- ✓ For this reason, Terzaghi used an approximate method to determine ultimate bearing capacity, q_u

$$N_c = \tan \phi'(K_c + 1)$$

$$N_q = K_q \tan \phi'$$

$$N_\gamma = \tfrac{1}{2} \tan \phi'(K_\gamma \tan \phi' - 1)$$

- ✓ N_c, N_q, and N_γ are, respectively, contributions of cohesion, surcharge, and unit weight of soil to ultimate load-bearing capacity
- ✓ It is extremely tedious to evaluate K_c, K_q, and K_γ
- ✓ For this reason, Terzaghi used an approximate method to determine ultimate bearing capacity, q_u
- ✓ Principles of this approximation are followings

1. If $c' = 0$ and surcharge $(q) = 0$ (that is, $D_f = 0$), then

$$q_u = q_\gamma = \tfrac{1}{2}\gamma B N_\gamma$$

$$N_c = \tan \phi'(K_c + 1)$$

$$N_q = K_q \tan \phi'$$

$$N_\gamma = \tfrac{1}{2} \tan \phi'(K_\gamma \tan \phi' - 1)$$

- ✓ N_c, N_q, and N_γ are, respectively, contributions of cohesion, surcharge, and unit weight of soil to ultimate load-bearing capacity
- ✓ It is extremely tedious to evaluate K_c, K_q, and K_γ
- ✓ For this reason, Terzaghi used an approximate method to determine ultimate bearing capacity, q_u
- ✓ Principles of this approximation are followings

1. If $\gamma = 0$ (that is, weightless soil) and $q = 0$, then

$$q_u = q_c = c'N_c$$

- ✓ By method of superimposition, when effects of unit weight of soil, cohesion, and surcharge are considered, we have

$$q_u = q_c + q_q + q_\gamma = c'N_c + qN_q + \tfrac{1}{2}\gamma B N_\gamma$$

- ✓ This equation is referred to as Terzaghi's bearing capacity equation
- ✓ Terms N_c, N_q, and N_γ are called bearing capacity factors

$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

Contribution of: Shear Strength ($c'N_c$), Surcharge (qN_q), Soil Self Weight ($\frac{1}{2}\gamma B N_\gamma$)

$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma$$ Strip footing

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$$ Square footing

$$q_u = 1.3c'N_c + qN_q + 0.3\gamma BN_\gamma$$ Circular footing

Equations were derived on assumption that bearing capacity failure of soil takes place by general shear failure

$$N_c = \cot\phi'\left[\frac{e^{2(3\pi/4-\phi'/2)\tan\phi'}}{2\cos^2\left(\frac{\pi}{4}+\frac{\phi'}{2}\right)}-1\right] = \cot\phi'(N_q-1)$$

$$N_q = \frac{e^{2(3\pi/4-\phi'/2)\tan\phi'}}{2\cos^2\left(45+\frac{\phi'}{2}\right)}$$

$$N_\gamma = \frac{1}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi'}-1\right)\tan\phi'$$

where $K_{p\gamma}$ = passive pressure coefficient

- In case of local shear failure, we may assume that

$$\overline{c}' = \tfrac{2}{3}c'$$

and $$\tan\overline{\phi}' = \tfrac{2}{3}\tan\phi'$$

$$q'_u = \overline{c}'N'_c + qN'_q + \tfrac{1}{2}\gamma BN'_\gamma$$ Strip footing

$$q'_u = 1.3\overline{c}'N'_c + qN'_q + 0.4\gamma BN'_\gamma$$ Square footing

$$q'_u = 1.3\overline{c}'N'_c + qN'_q + 0.3\gamma BN'_\gamma$$ Circular footing

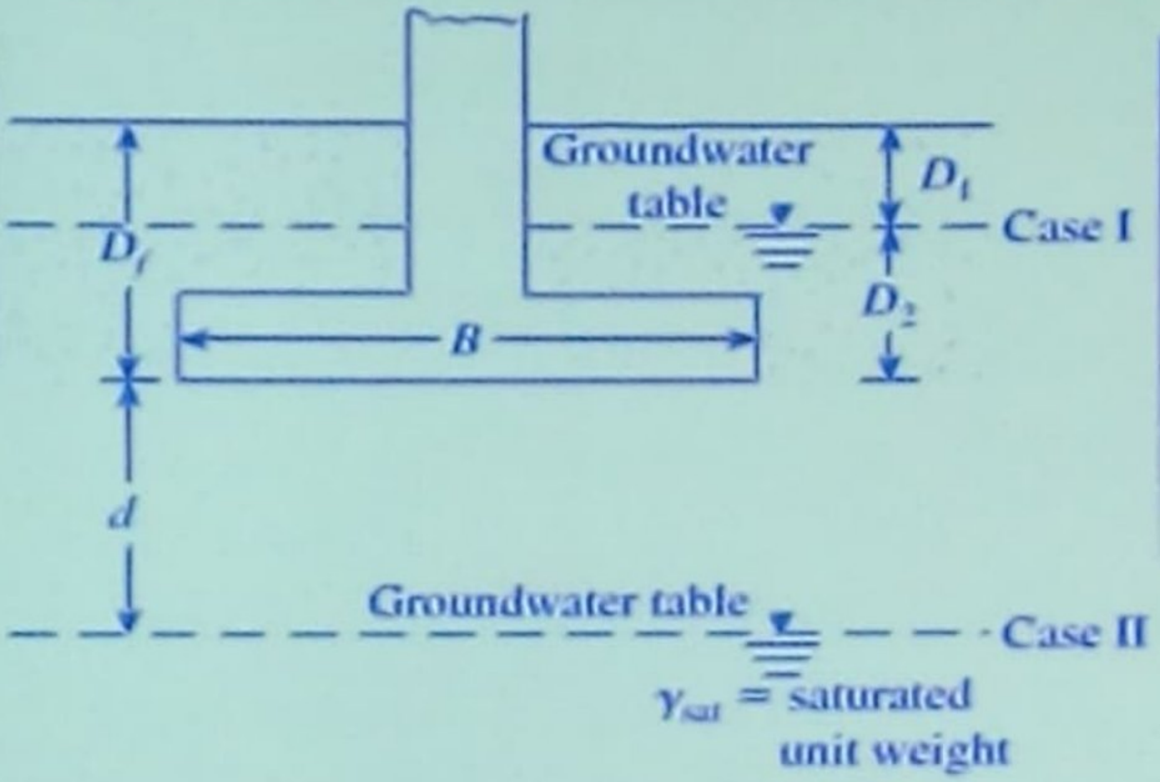
substituting $\overline{\phi}' = \tan^{-1}(\tfrac{2}{3}\tan\phi')$ for ϕ'

Limitations of Terzaghi's Bearing Capacity Formula

- ✓ $D_f \leq B$
- ✓ No sliding between footing and soil
- ✓ Soil is a homogeneous semi-infinite mass
- ✓ Failure plane angle is equal to ϕ'
- ✓ Not applicable for inclined load & rectangular foundation
- ✓ No resistance of soil above the level of base of foundation

Effect of Groundwater Table

- ✓ In developing bearing capacity equations it is assumed that groundwater table is located at a depth much greater than width, *B* of footing
- ✓ However, if groundwater table is close to footing, some changes are required in second and third terms of bearing capacity equations
- ✓ Three different conditions can arise regarding location of groundwater table with respect to bottom of foundation



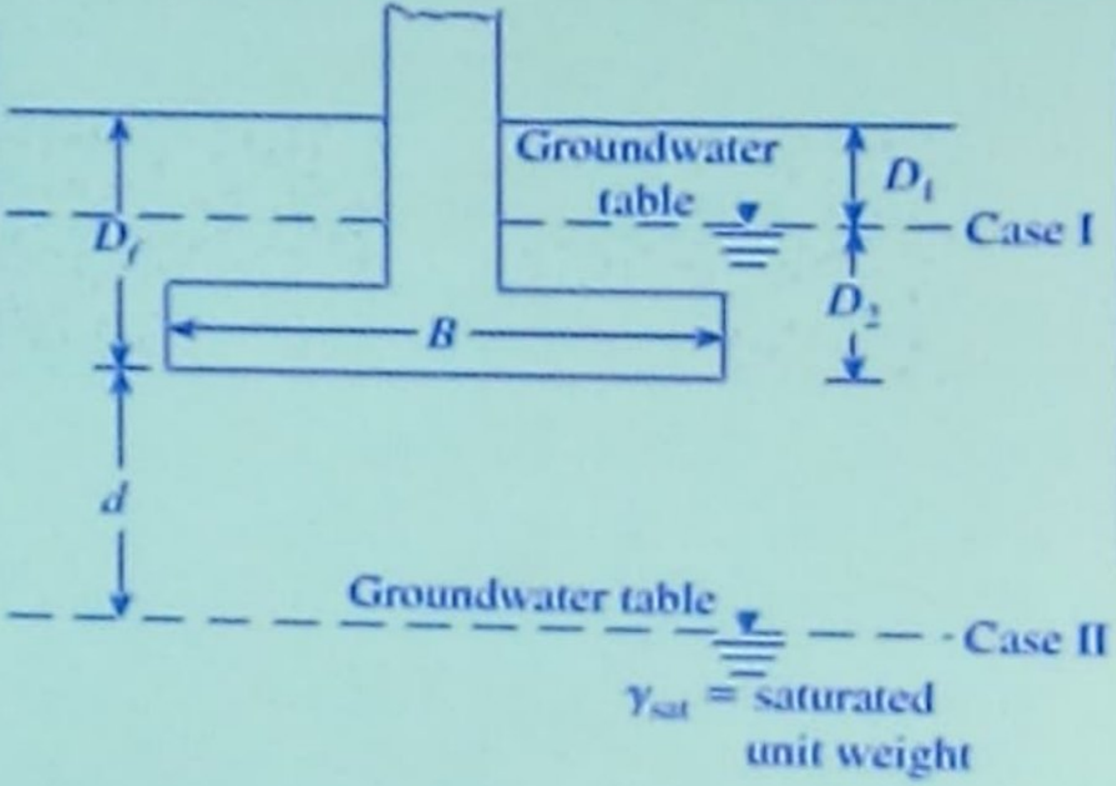
Case I

- ✓ If the water table is located so that $0 \leq D_1 \leq D_f$ factor q in bearing capacity equations takes form

$$q = \text{effective surcharge} = D_1\gamma + D_2(\gamma_{sat} - \gamma_w)$$

where $\gamma' = \gamma_{sat} - \gamma_w$ = effective unit weight of soil

- ✓ Also, unit weight of soil, γ, that appears in third term of bearing capacity equations should be replaced by γ'



Case II

For a water table located so that $0 \leq d \leq B$

$$q = \gamma D_f$$

In this case, factor γ in last term of bearing capacity equations must be replaced by factor

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$$

Case III

- ✓ When water table is located so that $d > B$, water will have no effect on ultimate bearing capacity

Factor of Safety

- Calculating gross allowable load-bearing capacity of shallow foundations requires application of a factor of safety (FS) to gross ultimate bearing capacity, or

$$q_{all} = \frac{q_u}{FS}$$

However, some practicing engineers prefer to use a factor of safety such that

$$\text{Net stress increase on soil} = \frac{\text{net ultimate bearing capacity}}{FS}$$

- Net ultimate bearing capacity is defined as ultimate pressure per unit area of foundation that can be supported by soil in excess of pressure caused by surrounding soil at foundation level
- If difference between unit weight of concrete used in foundation and unit weight of soil surrounding is assumed to be negligible, then

$$q_{net(u)} = q_u - q$$

where

$q_{net(u)}$ = net ultimate bearing capacity

$q = \gamma D_f$

$$\text{So} \quad q_{all(net)} = \frac{q_u - q}{FS}$$

Factor of safety as defined should be at least 3 in all cases

General Bearing Capacity Equation

- Meyerhof (1963) suggested following form of general bearing capacity equation

$$q_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

In this equation:

c' = cohesion
q = effective stress at the level of the bottom of the foundation
γ = unit weight of soil
B = width of foundation (= diameter for a circular foundation)
$F_{cs}, F_{qs}, F_{\gamma s}$ = shape factors
$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors
$F_{ci}, F_{qi}, F_{\gamma i}$ = load inclination factors
N_c, N_q, N_γ = bearing capacity factors

Bearing Capacity Factors

$$N_q = \tan^2\left(45 + \frac{\phi'}{2}\right)e^{\pi \tan \phi'}$$ (Reissner, 1924)

$$N_c = (N_q - 1)\cot \phi'$$ (Prandtl,1921)

$$N_\gamma = 2(N_q + 1)\tan \phi'$$ (Vesic, 1973)

Shape factor

$$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi'$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right)$$

Depth factor $\frac{D_f}{B} \leq 1$

For $\phi = 0$:

$$F_{cd} = 1 + 0.4\left(\frac{D_f}{B}\right)$$

$$F_{qd} = 1$$

$$F_{\gamma d} = 1$$

For $\phi' > 0$:

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$$

$$F_{qd} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \left(\frac{D_f}{B}\right)$$

$$F_{\gamma d} = 1$$

Depth factor

$$\frac{D_f}{B} > 1$$

For $\phi = 0$:

$$F_{cd} = 1 + 0.4 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radians}}$$

$$F_{qd} = 1$$

$$F_{\gamma d} = 1$$

For $\phi' > 0$:

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$$

$$F_{qd} = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radians}}$$

$$F_{\gamma d} = 1$$

Inclination factor

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta^\circ}{90^\circ}\right)^2$$

Meyerhof (1963); Hanna and Meyerhof (1981)

$$F_{\gamma i} = \left(1 - \frac{\beta}{\phi'}\right)$$

β = inclination of the load on the foundation with respect to the vertical

A square foundation is 2 m × 2 m in plan. The soil supporting the foundation has a friction angle of $\phi' = 25°$ and $c' = 20\ \text{kN/m}^2$. The unit weight of soil, γ, is $16.5\ \text{kN/m}^3$. Determine the allowable gross load on the foundation with a factor of safety (FS) of 3. Assume that the depth of the foundation (D_f) is 1.5 m and that general shear failure occurs in the soil.

$$q_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

Since the load is vertical, $F_{ci} = F_{qi} = F_{\gamma i} = 1$ [redacted] for $\phi' = 25°$, $N_c = 20.72$, $N_q = 10.66$, and $N_\gamma = 10.88$.

$$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = 1 + \left(\frac{2}{2}\right)\left(\frac{10.66}{20.72}\right) = 1.514$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi' = 1 + \left(\frac{2}{2}\right)\tan 25 = 1.466$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{2}{2}\right) = 0.6$$

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right)$$

$$= 1 + (2)(\tan 25)(1 - \sin 25)^2\left(\frac{1.5}{2}\right) = 1.233$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} = 1.233 - \left[\frac{1 - 1.233}{(20.72)(\tan 25)}\right] = 1.257$$

$$F_{\gamma d} = 1$$

$$q_u = (20)(20.72)(1.514)(1.257)(1)$$

$$+(1.5 \times 16.5)(10.66)(1.466)(1.233)(1)$$

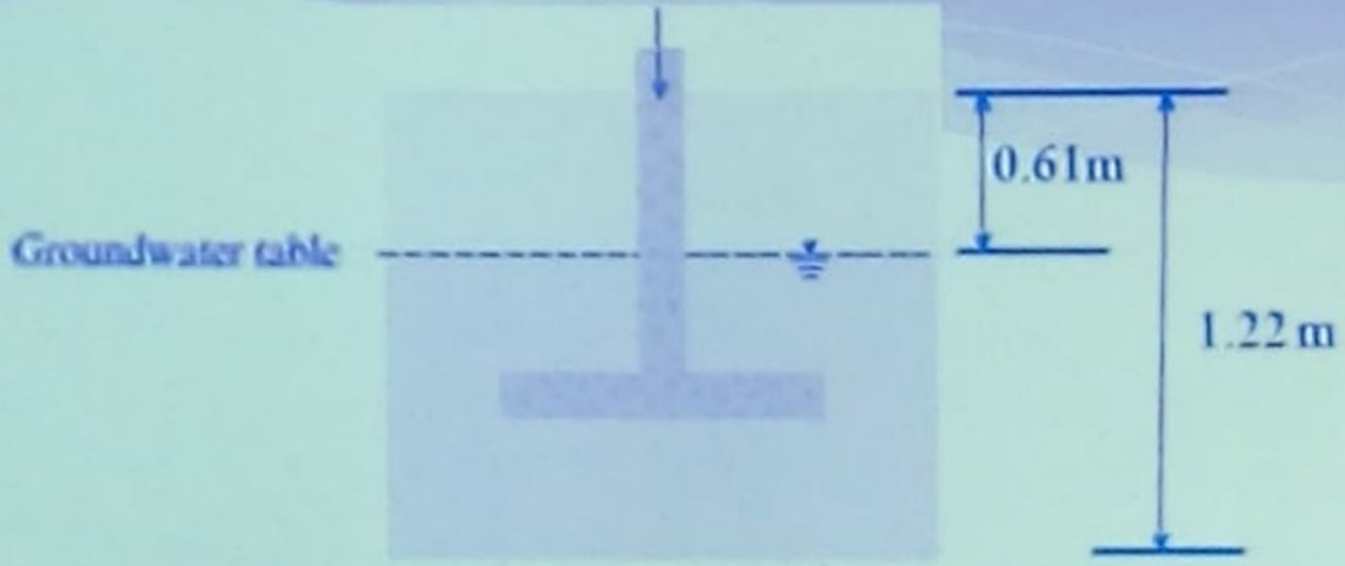
$$+ \frac{1}{2}(16.5)(2)(10.88)(0.6)(1)(1)$$

$$= 788.6 + 476.9 + 107.7 = 1373.2 \text{ kN/m}^2$$

$$q_{all} = \frac{q_u}{FS} = \frac{1373.2}{3} = 457.7\ \text{kN/m}^2$$

$$Q = (457.7)(2 \times 2) = 1830.8\ \text{kN}$$

A square foundation ($B \times B$) has to be constructed as shown in Figure . Assume that $\gamma = 16.5\ \text{kN/m}^3$, $\gamma_{sat} = 18.55\ \text{kN/m}^3$, $\phi' = 34°$, $D_f = 1.22$ m, and $D_1 = 0.61$ m. The gross allowable load, Q_{all}, with FS = 3 is 667.2 kN. Determine the size of the footing.



$$q_{all} = \frac{Q_{all}}{B^2} = \frac{667.2}{B^2}\ \text{kN/m}^2 \quad \text{.....................(a)}$$

$$q_{all} = \frac{q_u}{FS} = \frac{1}{3}\left(qN_qF_{qs}F_{qd} + \frac{1}{2}\gamma' BN_\gamma F_{\gamma s}F_{\gamma d}\right) \quad \text{(with } c' = 0\text{).}$$

For $\phi' = 34°$

$N_q = 29.44$ and $N_\gamma = 41.06$.

$$F_{qs} = 1 + \frac{B}{L}\tan\phi' = 1 + \tan 34 = 1.67$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4 = 0.6$$

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\frac{D_f}{B}$$

$$= 1 + 2\tan 34(1 - \sin 34)^2\frac{4}{B} = 1 + \frac{1.05}{B}$$

$$F_{\gamma d} = 1$$

$$q = (0.61)(16.5) + 0.61(18.55 - 9.81) = 15.4 \text{ kN/m}^2$$

$$q_{all} = \frac{1}{3}\left[(15.4)(29.44)(1.67)\left(1 + \frac{1.05}{B}\right) + \left(\frac{1}{2}\right)(18.55 - 9.81)(B)(41.06)(0.6)(1)\right]$$

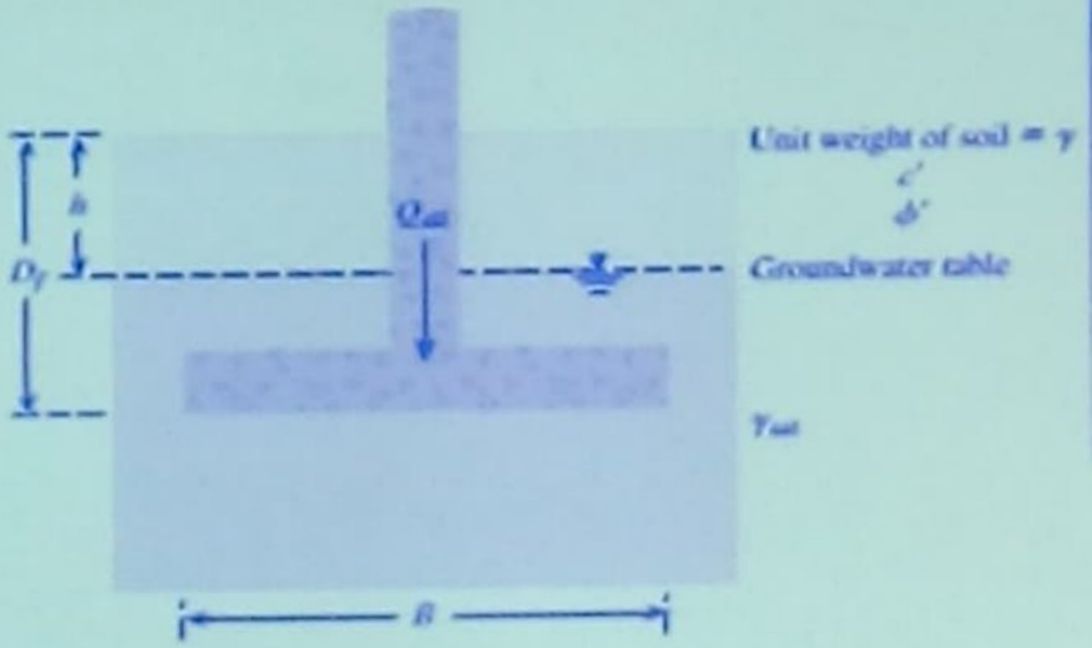
$$= 252.38 + \frac{265}{B} + 35.89B \quad \text{..........................(b)}$$

Combining Eqs. (a) and (b) results in

$$\frac{667.2}{B^2} = 252.38 + \frac{265}{B} + 35.89B$$

By trial and error, we find that $B \approx 1.3$ m.

Assignment



Given:

$\gamma = 100\ lb/ft^3, \gamma_{sat} = 120\ lb/ft^3, c' = 0, \emptyset' = 30^0, D_f = 5$ ft

$Q_{all} = 40{,}000$ lb, and FS = 3

Determine size of footing when:

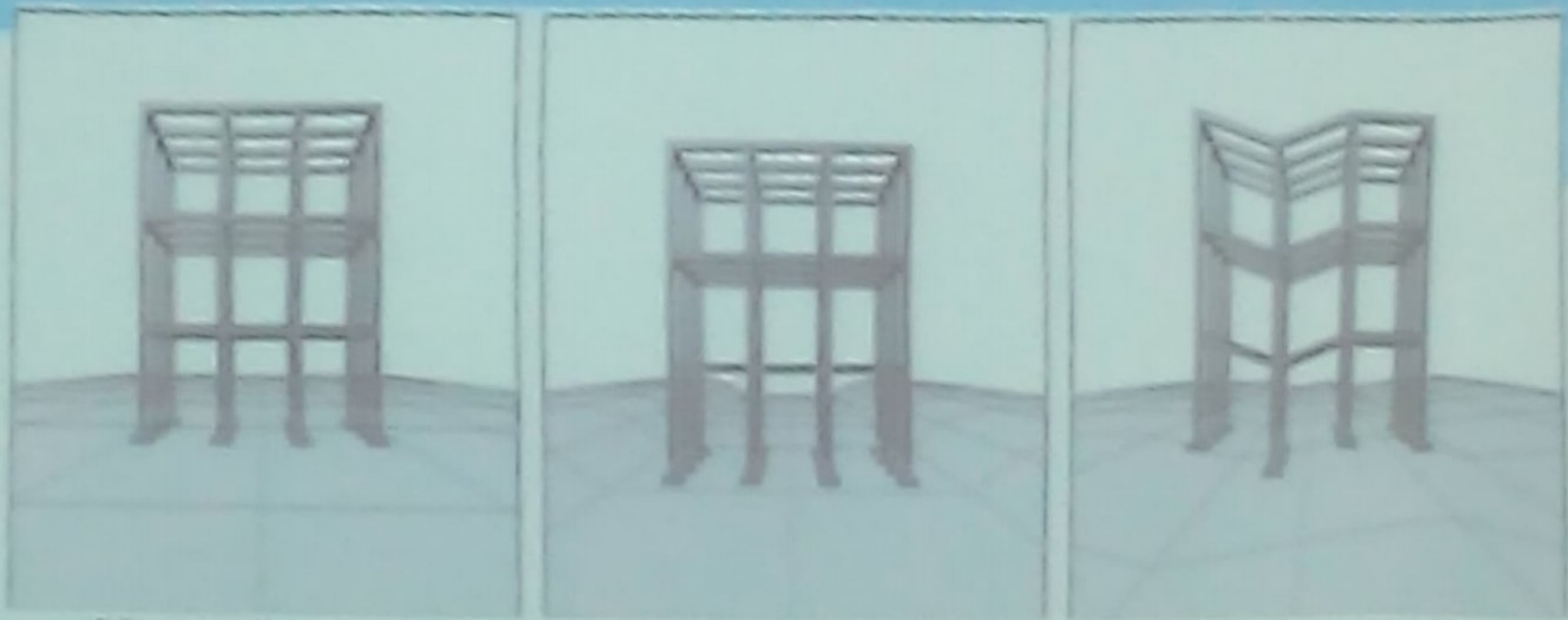
(i) $h = 0$ ft

(ii) $h = 2$ ft

(iii) $h = 5$ ft

SETTLEMENT
OF
SHALLOW FOUNDATI

SETTLEMENTS OF FOUNDATIONS



No settlement | Total settlement | Differential settlement

- Uniform settlement is usually of little consequence in a building, but differential settlement can cause severe structural damage

SETTLEMENT

- Definition

  Total vertical deformation at surface resulting from :

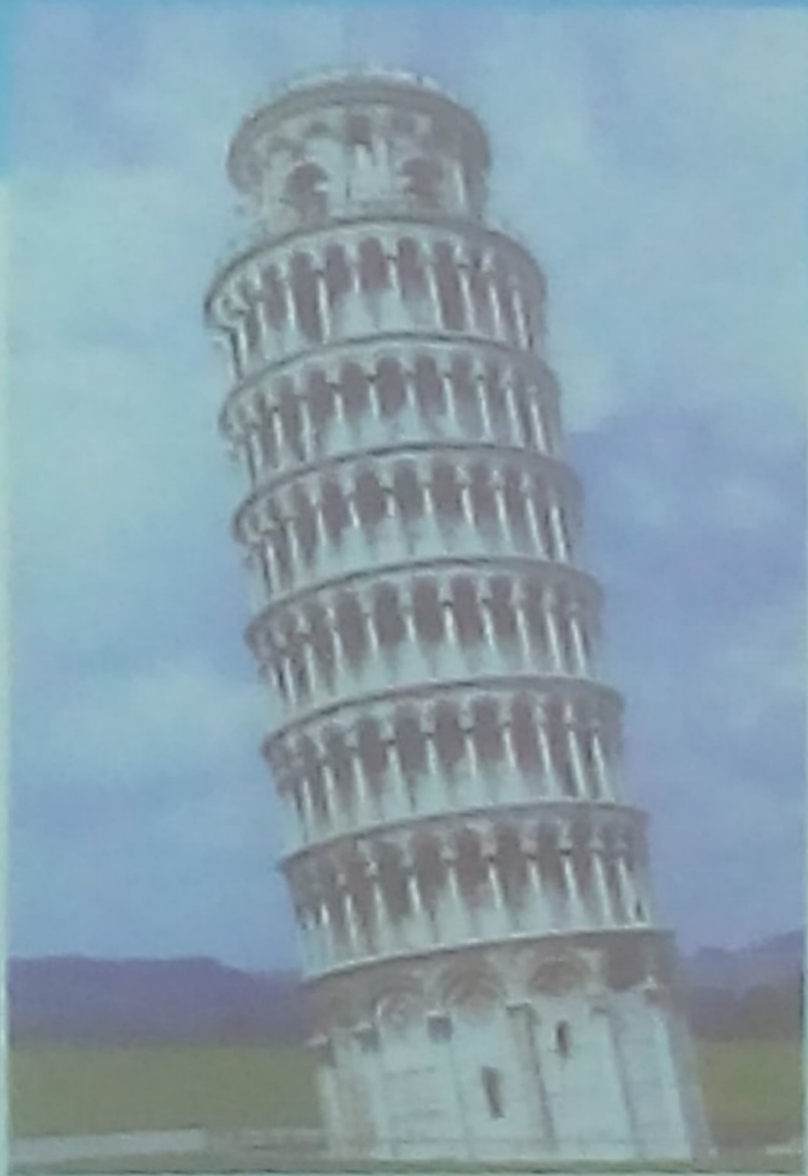
  - External Load
  - Dewatering
- Settlement Components
  - Immediate Settlement or Elastic Settlement- S_e
  - Primary Consolidation Settlement - S_c
  - Secondary Consolidation Settlement (Creep) - S_s

$$S = S_e + S_c + S_s$$

SETTLEMENT

- Purpose
  - Study settlement behavior
  - Determine settlement value and time
  - Study settlement influence to structure stability

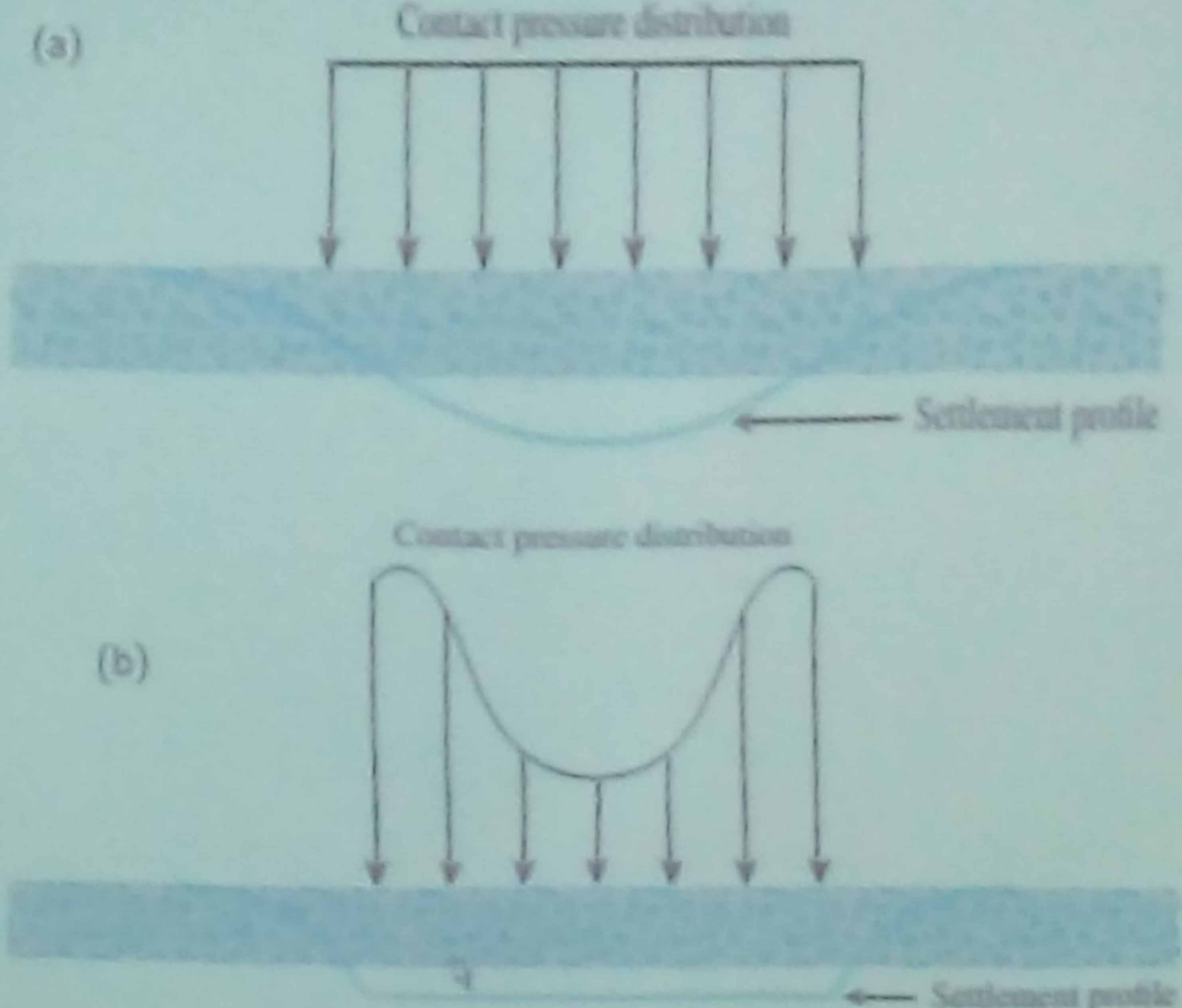
SETTLEMENT INFLUENCE



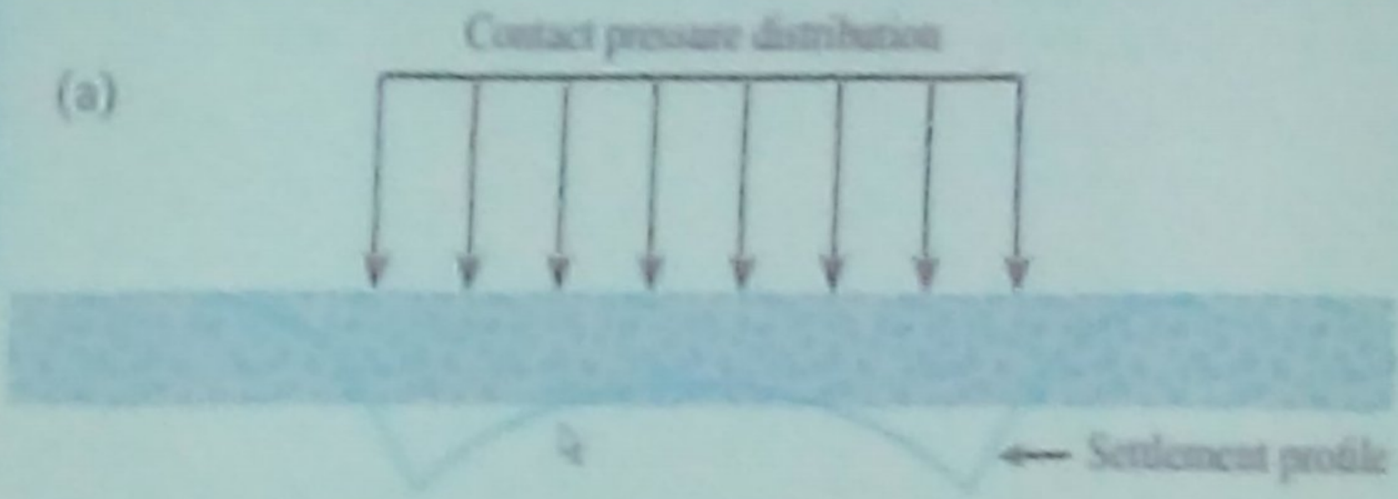
The Leaning Tower of Pisa, Italy

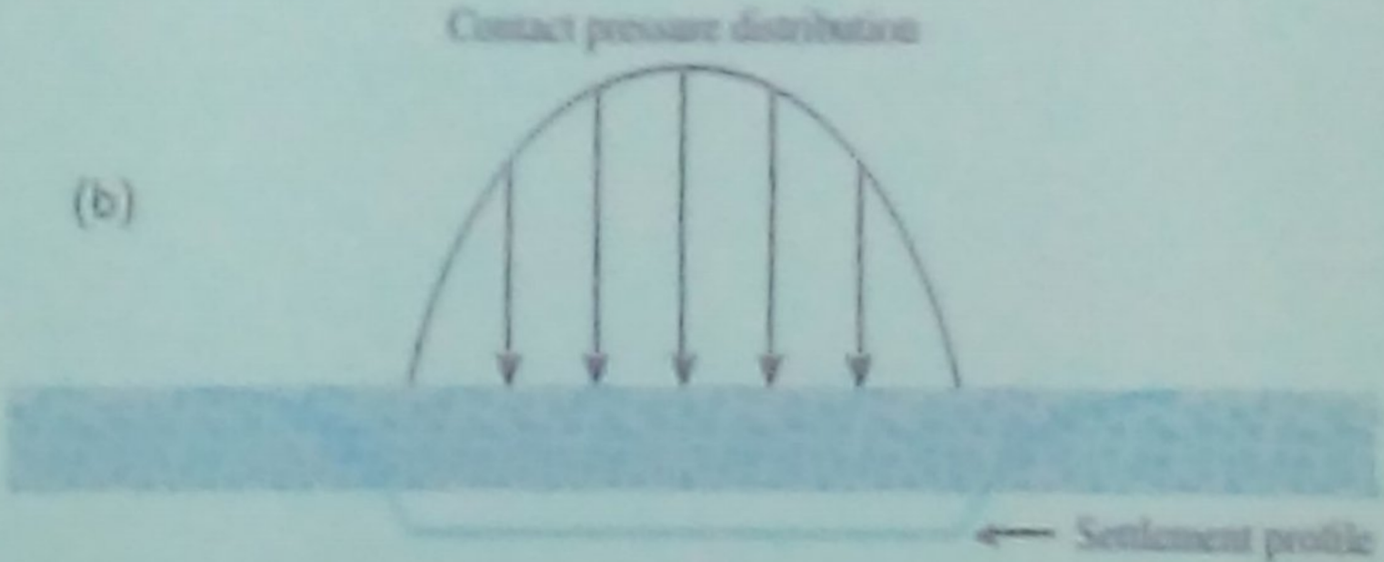
IMMEDIATE SETTLEMENT

- ✓ Defined as settlement which occurred directly after application of a load, without a change in moisture content
- ✓ Caused by soil elasticity behavior
- ✓ Magnitude of contact settlement will depend on flexibility of foundation and type of material on which it is resting
- ✓ For clay, immediate settlement generally very small comparing to consolidation settlement, therefore this immediate settlement mostly ignored
- ✓ Usually considered at sand or sandy soil
- ✓ Elastic settlement calculations generally are based on equations derived from theory of elasticity



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





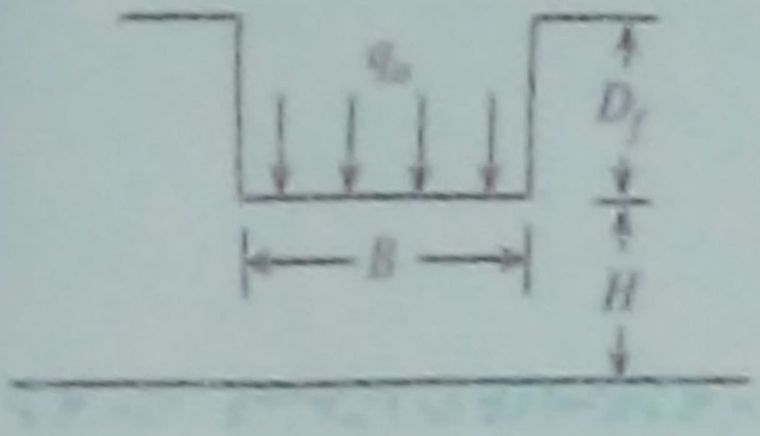
Elastic settlement profile and contact pressure in sand:
(a) flexible foundation; (b) rigid foundation

Elastic Settlement of Foundations on Saturated Clay ($\mu_s = 0.5$)

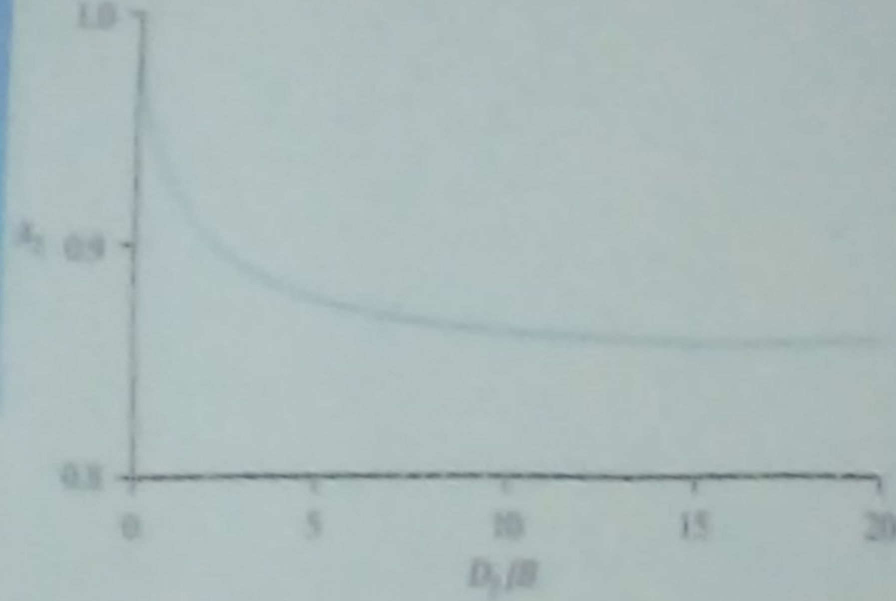
- Janbu et al. (1956) proposed an equation for evaluating average settlement of flexible foundations on saturated clay soils (Poisson's ratio, $\mu_s = 0.5$)

$$S_e = A_1 A_2 \frac{q_o B}{E_s}$$

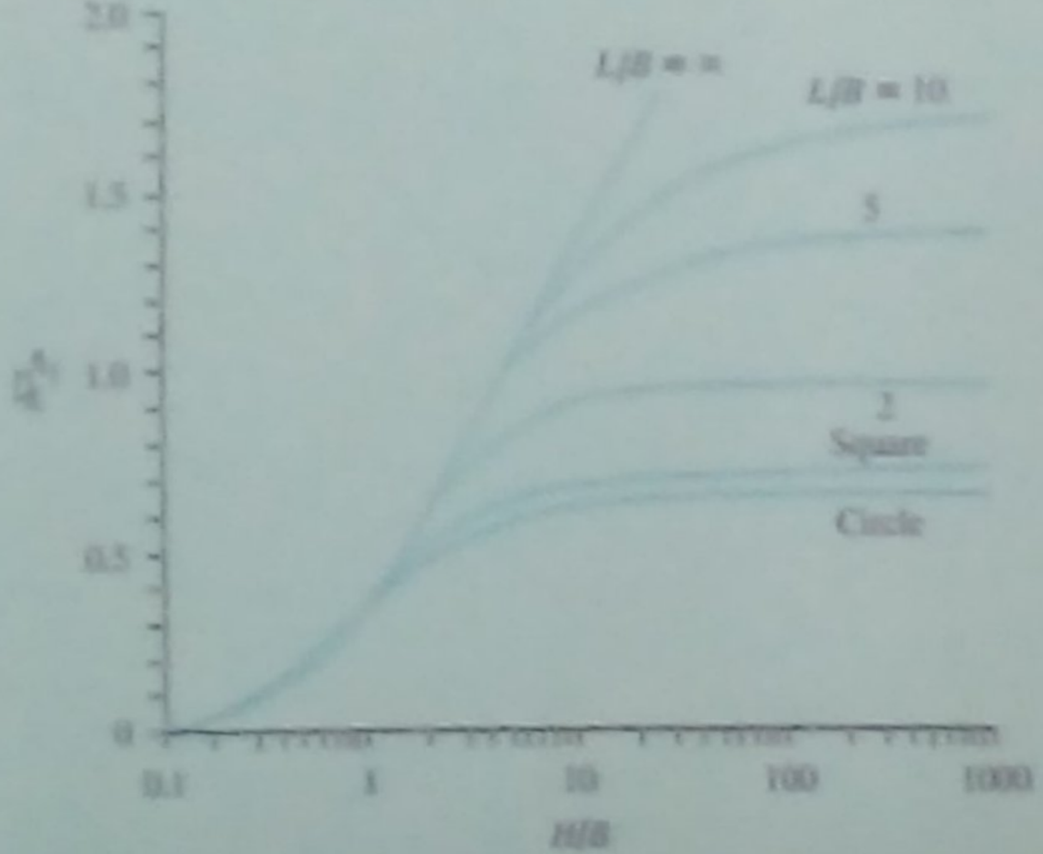
where A_1 is a function of H/B and L/B and A_2 is a function of D_f/B



q_u
D_f
B
H



1.0
0.9
0.8
0
5
10
15
20
D_f/B



2.0
1.5
1.0
0.5
0
$L/B = \infty$
$L/B = 10$
5
2
Square
Circle
0.1
1
10
100
1000
H/B

✓ Modulus of elasticity (E_s) for clays can, in general, be given as

$$E_s = \beta c_u$$

where c_u = undrained shear strength

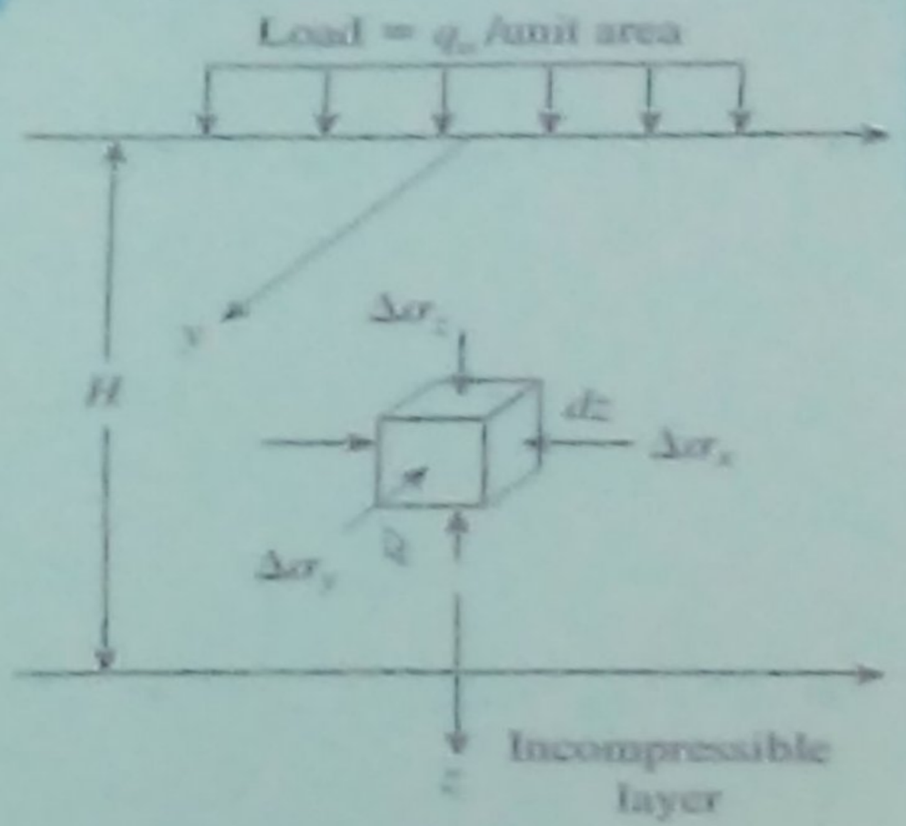
✓ Parameter β is primarily a function of plasticity index and overconsolidation ratio

✓ Following Table provides a general range for β based on that proposed by Duncan and Buchignani (1976)

Plasticity Index	β				
	OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
<30	1500–600	1380–500	1200–580	950–380	730–300
30 to 50	600–300	550–270	580–220	380–180	300–150
>50	300–150	270–120	220–100	180–90	150–75

✓ Settlement Based on the Theory of Elasticity

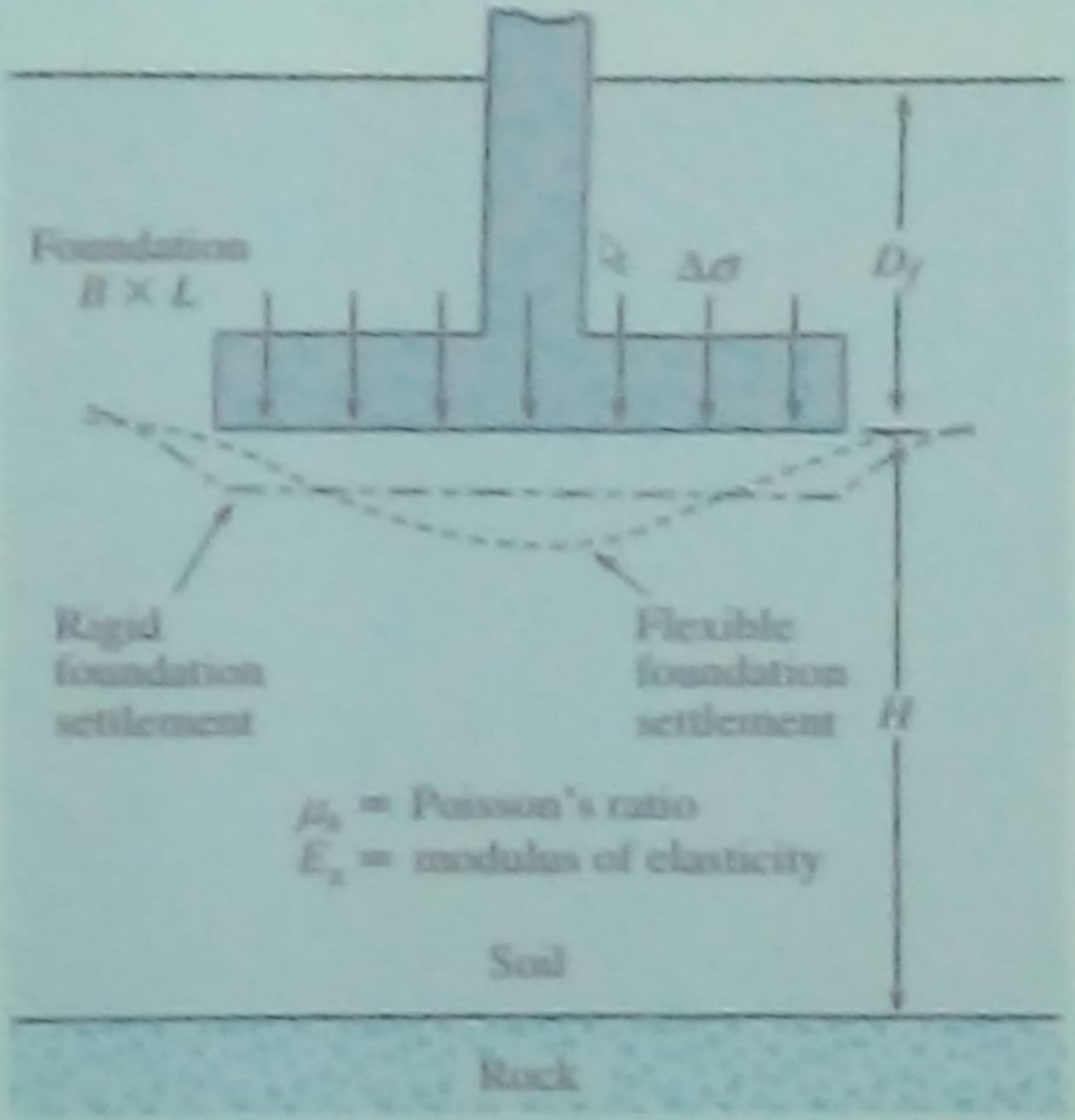
✓ Elastic settlement of a shallow foundation can be estimated by using theory of elasticity



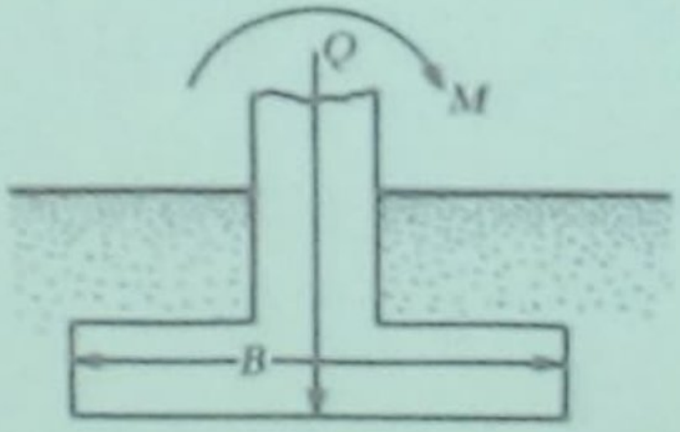
✓ From Hooke's law, we obtain

$$S_e = \int_0^H \varepsilon_z dz = \frac{1}{E_s}\int_0^H (\Delta\sigma_z - \mu_s\Delta\sigma_x - \mu_s\Delta\sigma_y)dz$$

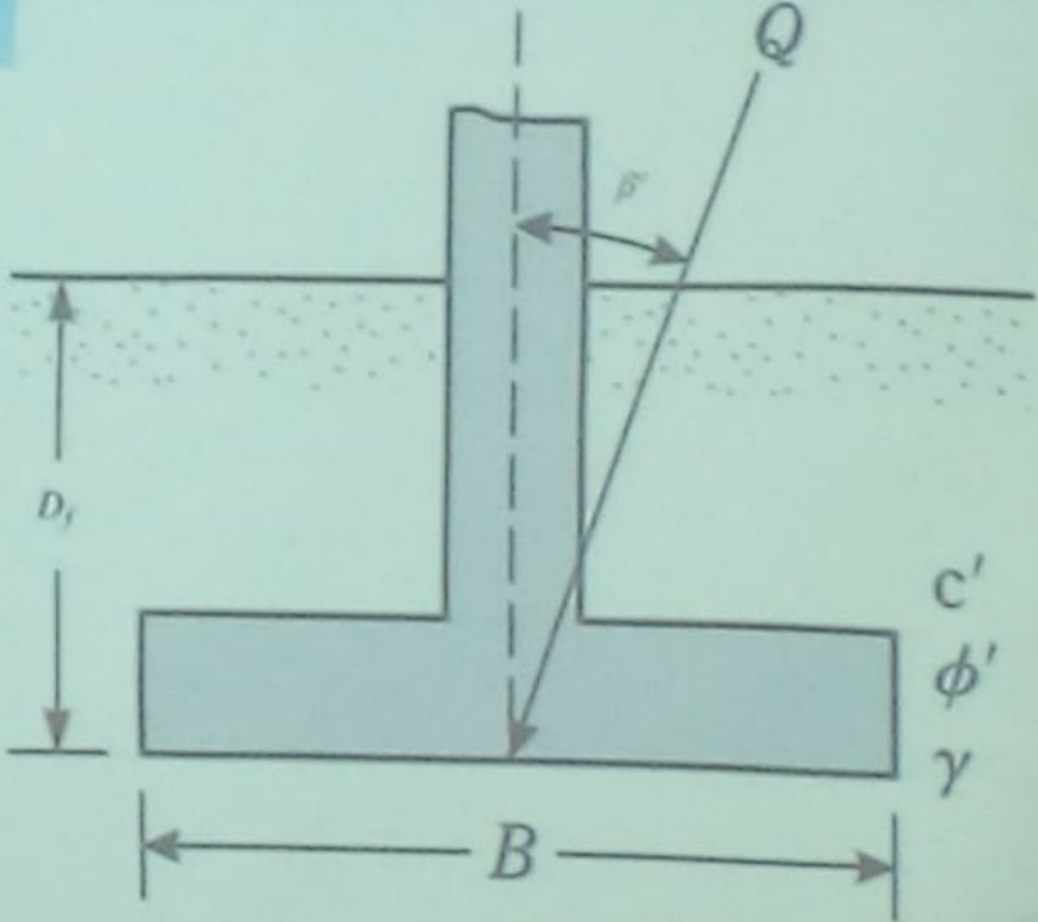
IMMEDIATE SETTLEMENT



FOOTINGS WITH ECCENTRIC OR INCLINED LOADINGS



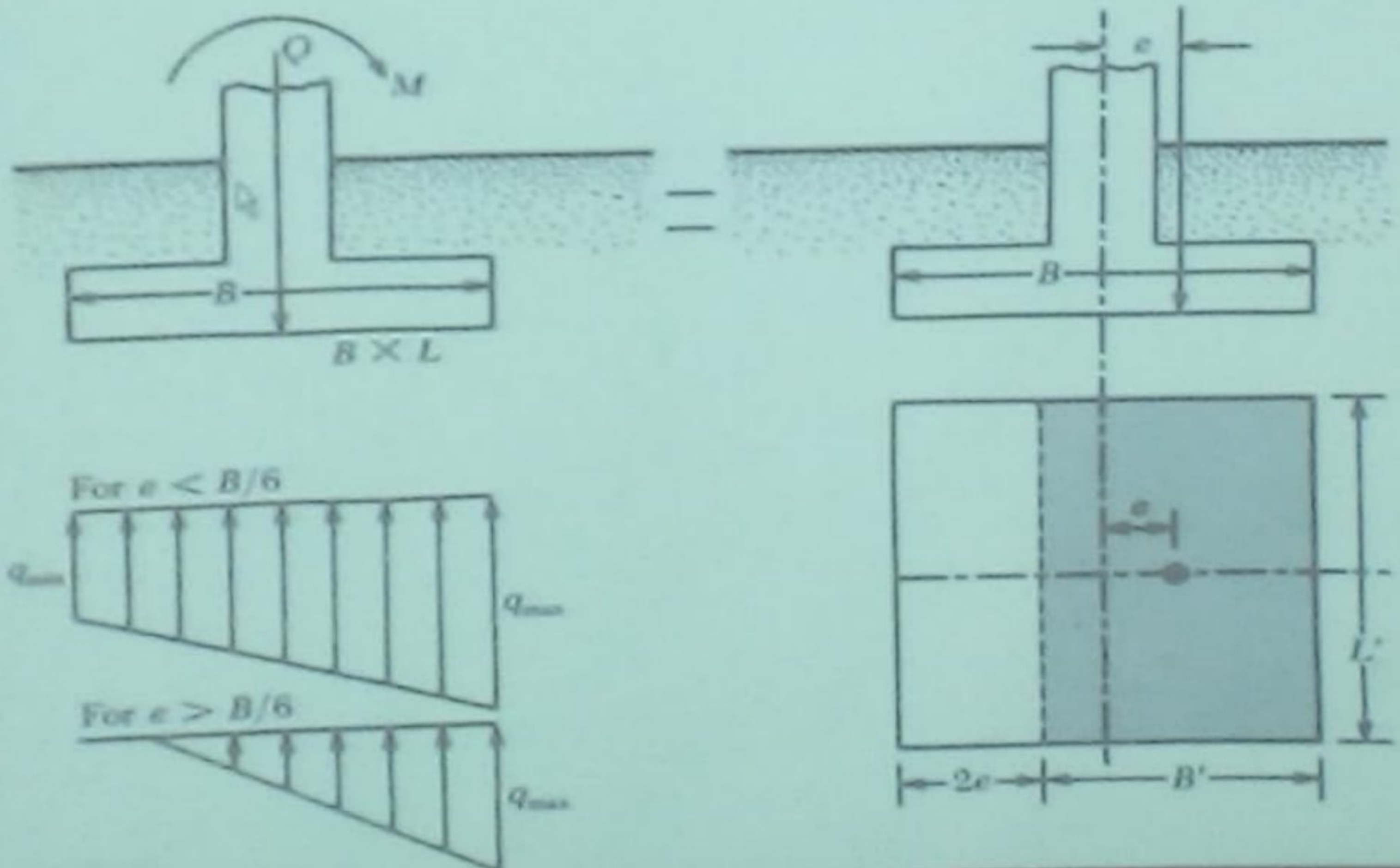
Eccentricity



Inclination

Footing with One Way Eccentricity

- ✓ In most instances, foundations are subjected to moments in addition to the vertical load as shown below
- ✓ In such cases distribution of pressure by foundation upon soil is not uniform



The effective width is now,

$B' = B - 2e$

whereas the effective length is Still,

$L' = L$

Nominal distribution of pressure is

$$q_{max} = \frac{Q}{BL} + \frac{6M}{B^2L} \qquad q_{min} = \frac{Q}{BL} - \frac{6M}{B^2L}$$

where

Q = total vertical load
M = moment on the foundation

$$e = \frac{M}{Q}$$

$$q_{max} = \frac{Q}{BL}\left(1 + \frac{6e}{B}\right) \qquad q_{min} = \frac{Q}{BL}\left(1 - \frac{6e}{B}\right)$$

Note that, when eccentricity $e = B/6$, q_{min} is **zero**

For $e > B/6$, q_{min} is negative, which means that **tension** will develop

- ✓ Because soil cannot take any tension, there will then be a separation between foundation and soil underlying it
- ✓ Nature of pressure distribution on soil will be as shown in Figure

For $e > B/6$

$$q_{max} = \frac{4Q}{3L(B - 2e)}$$

$$FS = \frac{Q_{ult}}{Q}$$

Ultimate Bearing Capacity under Eccentric Loading

– One-Way Eccentricity

- Following is a step-by-step procedure for determining ultimate load that soil can support and factor of safety against bearing capacity failure:

Step 1. Determine effective dimensions of foundation:

$$B' = \text{effective width} = B - 2e$$

$$L' = \text{effective length} = L$$

- ✓ Note that if eccentricity were in direction of length of foundation, value of L' would be equal to $L - 2e$ and value of B' would equal B
- ✓ Smaller of two dimensions (i.e., B' and L') is effective width of foundation

Ultimate Bearing Capacity under Eccentric Loading
– One-Way Eccentricity

- Following is a step-by-step procedure for determining ultimate load that soil can support and factor of safety against bearing capacity failure:

Step 1. Determine effective dimensions of foundation:

Step 2. Use ultimate bearing capacity equation:

$$q'_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma B'N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

- ✓ To evaluate F_{cs}, F_{qs} and $F_{\gamma s}$ use relationships given earlier with effective length and effective width dimensions instead of L and B, respectively
- ✓ To determine **F_{cd}, F_{qd} and $F_{\gamma d}$** use relationships given earlier **without replacing B with B'**

Ultimate Bearing Capacity under Eccentric Loading
– One-Way Eccentricity

- Following is a step-by-step procedure for determining ultimate load that soil can support and factor of safety against bearing capacity failure:

Step 1. Determine effective dimensions of foundation:

Step 2. Use ultimate bearing capacity equation:

Step 3. Calculate total ultimate load foundation can sustain:

$$Q_{ult} = q'_u \overbrace{(B')(L')}^{A'}$$

where A' = effective area

Ultimate Bearing Capacity under Eccentric Loading
– One-Way Eccentricity

- Following is a step-by-step procedure for determining ultimate load that soil can support and factor of safety against bearing capacity failure:

Step 1. Determine effective dimensions of foundation:

Step 2. Use ultimate bearing capacity equation:

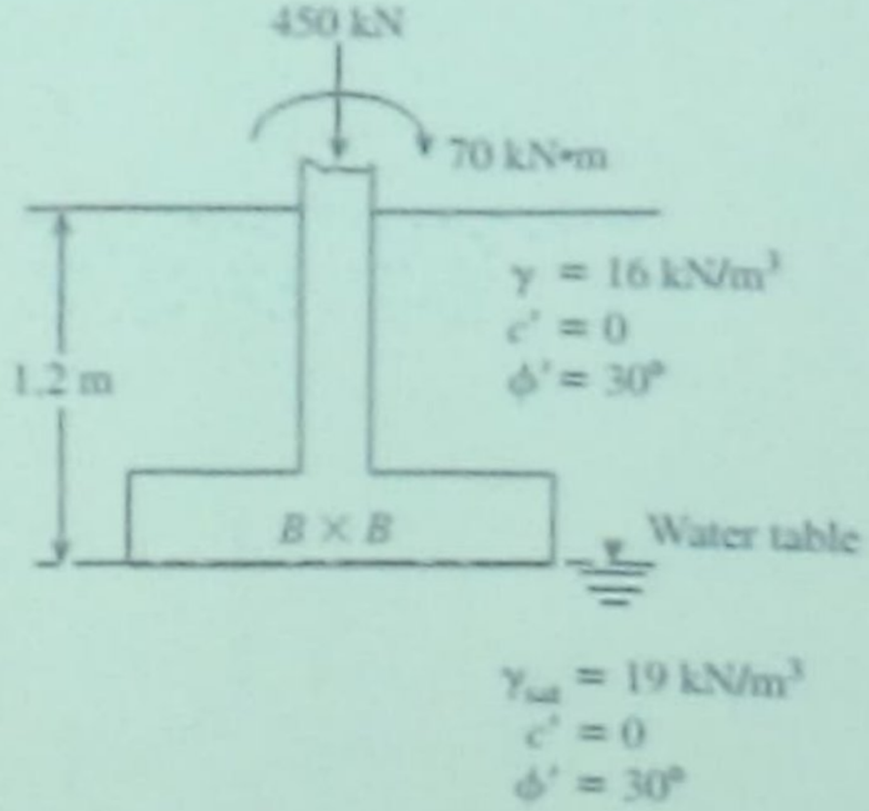
Step 3. Calculate total ultimate load foundation can sustain:

Step 4. Determine FS against bearing capacity failure:

$$FS = \frac{Q_{ult}}{Q}$$

where Q = total vertical load

A square footing is shown in Figure below. Use FS = 6, and determine size of footing.



Consolidation Settlement

- ✓ Consolidation settlement occurs over time in saturated clayey soils subjected to an increased load caused by construction of foundation
- ✓ On basis of one-dimensional consolidation settlement equations we write

$$S_{c(p)} = \int \varepsilon_z dz$$

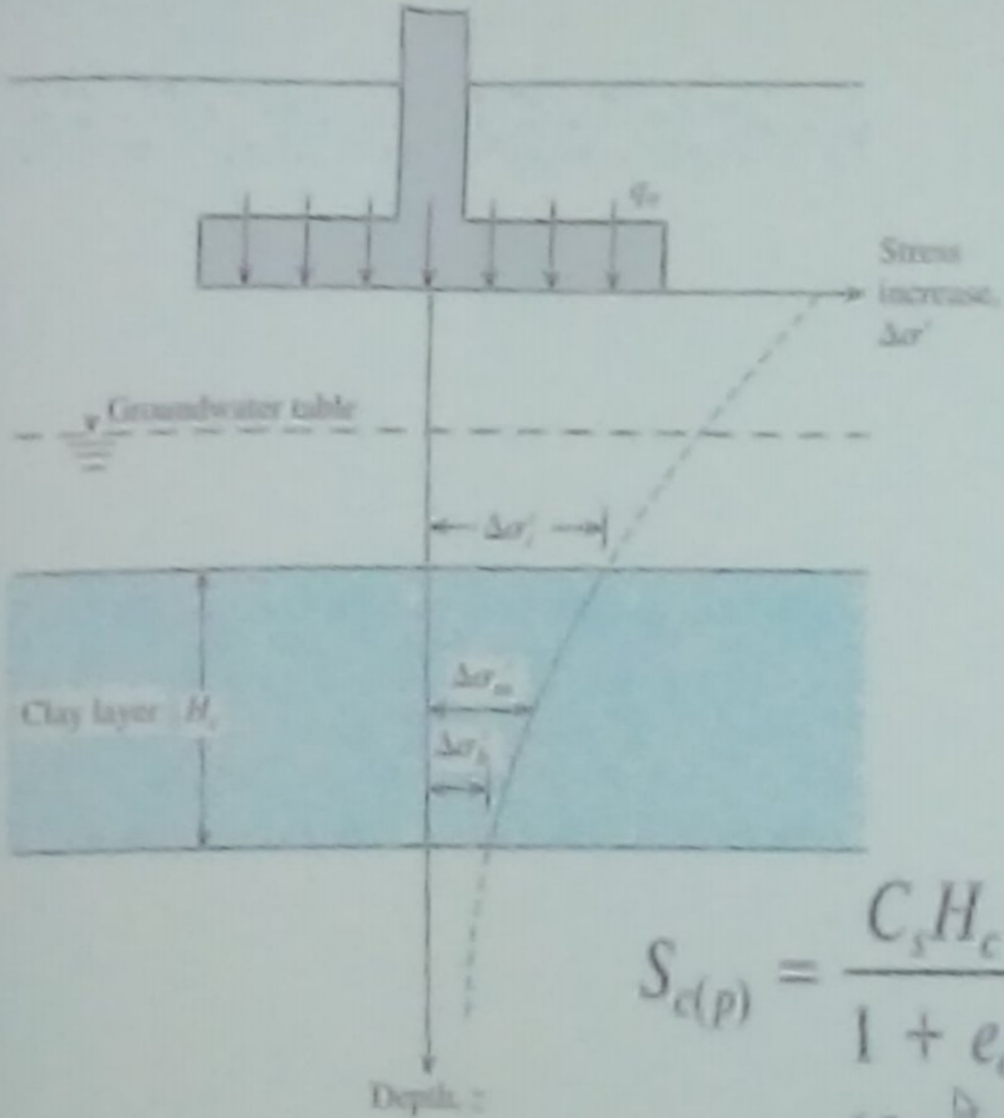
where

$$\varepsilon_z = \text{vertical strain}$$

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

$$\Delta e = \text{change of void ratio}$$

$$= f(\sigma'_o, \sigma'_c, \text{ and } \Delta\sigma')$$



$$S_{c(p)} = \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$

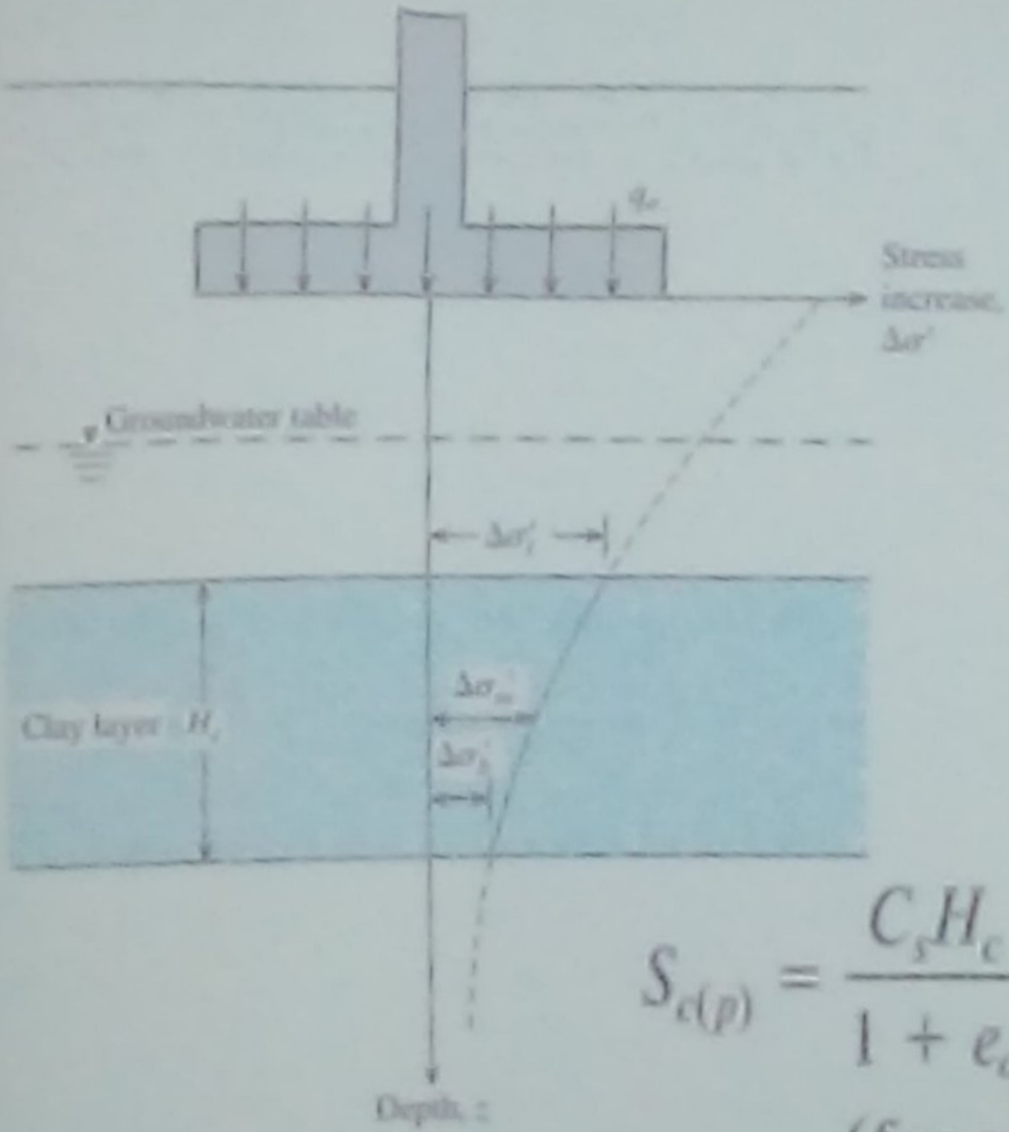
(for normally consolidated clays)

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$

(for overconsolidated clays with $\sigma'_o + \Delta\sigma'_{av} < \sigma'_c$)

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_c}$$

(for overconsolidated clays with $\sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'_{av}$)



$$S_{c(p)} = \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$

(for normally consolidated clays)

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$

(for overconsolidated clays with $\sigma'_o + \Delta\sigma'_{av} < \sigma'_c$)

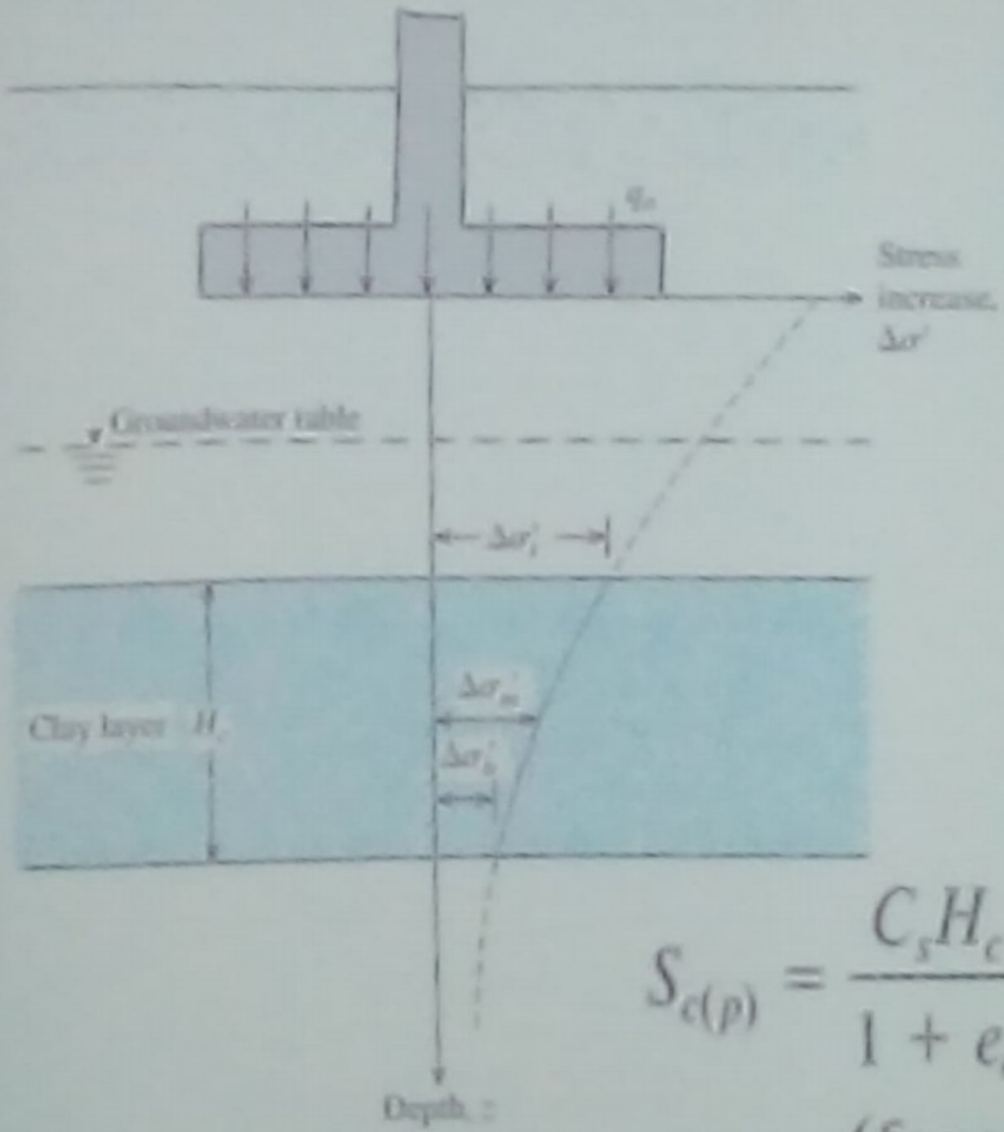
$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_c}$$

(for overconsolidated clays with $\sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'_{av}$)

σ'_c = preconsolidation pressure

e_o = initial void ratio of the clay layer

C_c = compression index



$$S_{c(p)} = \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$

(for normally consolidated clays)

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$

(for overconsolidated clays with $\sigma'_o + \Delta\sigma'_{av} < \sigma'_c$)

$$S_{c(p)} = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_c}$$

(for overconsolidated clays with $\sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'_{av}$)

C_s = swelling index

H_c = thickness of the clay layer

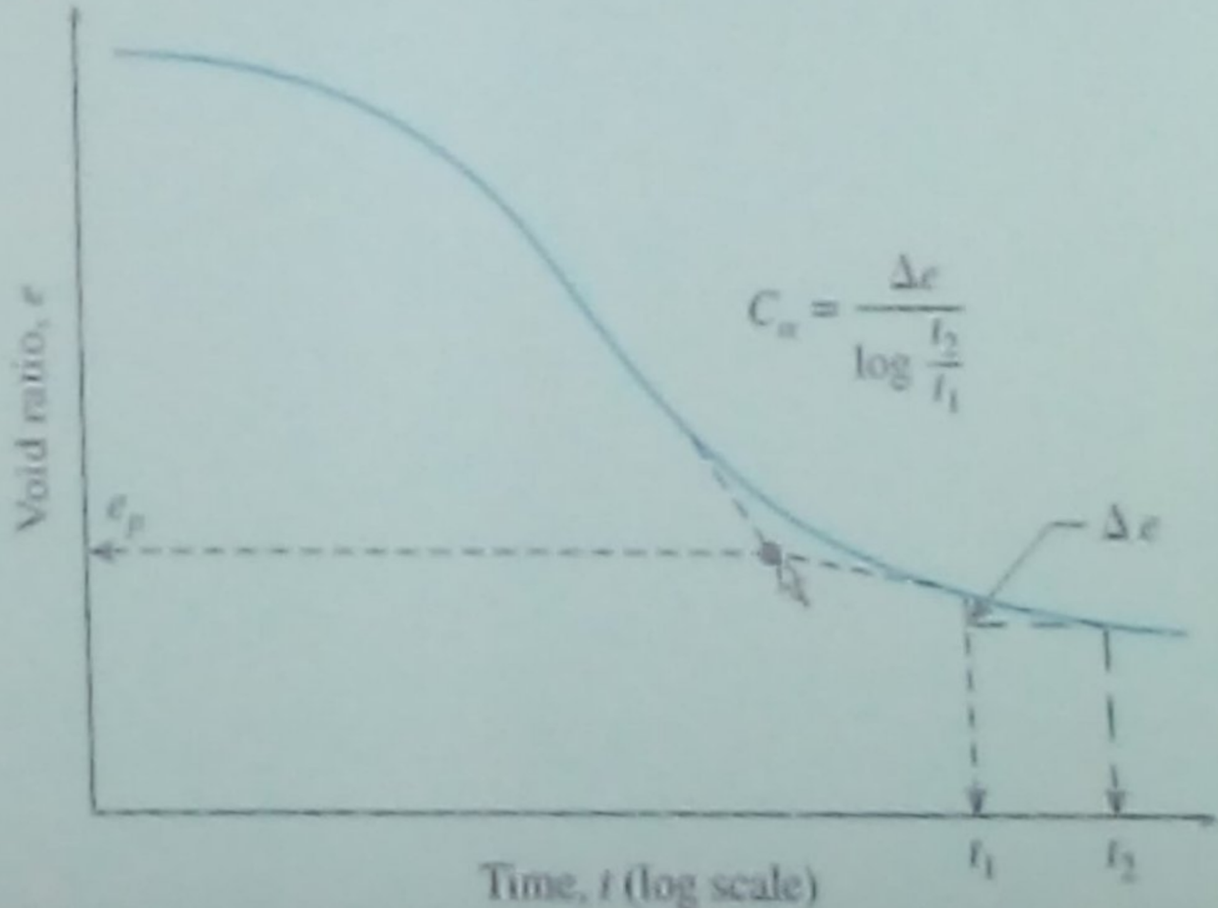
- ✓ Note that increase in effective pressure, $\Delta\sigma'$ on clay layer is not constant with depth
- ✓ Magnitude of $\Delta\sigma'$ will decrease with increase in depth measured from bottom of foundation
- ✓ However, average increase in pressure may be approximated by

$$\Delta\sigma'_{av} = \frac{1}{6}(\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b)$$

where $\Delta\sigma_t'$, $\Delta\sigma_m'$ and $\Delta\sigma_b'$ are, respectively, effective pressure increases at top, middle, and bottom of clay laye[r] that are caused by construction of foundation

Settlement Due to Secondary Consolidation

- At end of primary consolidation (i.e., after complete dissipation of excess pore water pressure) some settlement is observed that is due to plastic adjustment of soil fabrics
- This stage of consolidation is called secondary consolidation
- A plot of deformation against logarithm of time during secondary consolidation is practically linear



$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log (t_2/t_1)}$$

where

C_α = secondary compression index
Δe = change of void ratio
t_1, t_2 = time

- ✓ Magnitude of secondary consolidation can be calculated as

$$S_{c(s)} = C'_\alpha H_c \log(t_2/t_1)$$

where

$C'_\alpha = C_\alpha/(1 + e_p)$
e_p = void ratio at the end of primary con
H_c = thickness of clay layer

- Mesri (1973) correlated C_α' with natural moisture content (w) of several soils, from which it appears that

$$C_\alpha' \approx 0.0001w$$

where w = natural moisture content, in percent

For most overconsolidated soils, C_α' varies between 0.0005 to 01

- Mesri and Godlewski (1977) compiled magnitude of C_α/C_c (C_c = compression index) for a number of soils
- Based on their compilation, it can be summarized that

For inorganic clays and silts:

$$C_\alpha/C_c \approx 0.04 \pm 0.01$$

For organic clays and silts:

$$C_\alpha/C_c \approx 0.05 \pm 0.01$$

For peats:

$$C_\alpha/C_c \approx 0.075 \pm 0.01$$

- Secondary consolidation settlement is more important in case of all organic and highly compressible inorganic soils
- In overconsolidated inorganic clays, secondary compression index is very small and of less practical significance

Calculate the settlement of the 10-ft-thick clay layer (Figure 11.30) that will result from the load carried by a 5-ft-square footing. The clay is normally consolidated. Use the weighted average method to calculate the average increase of effective pressure in the clay layer.

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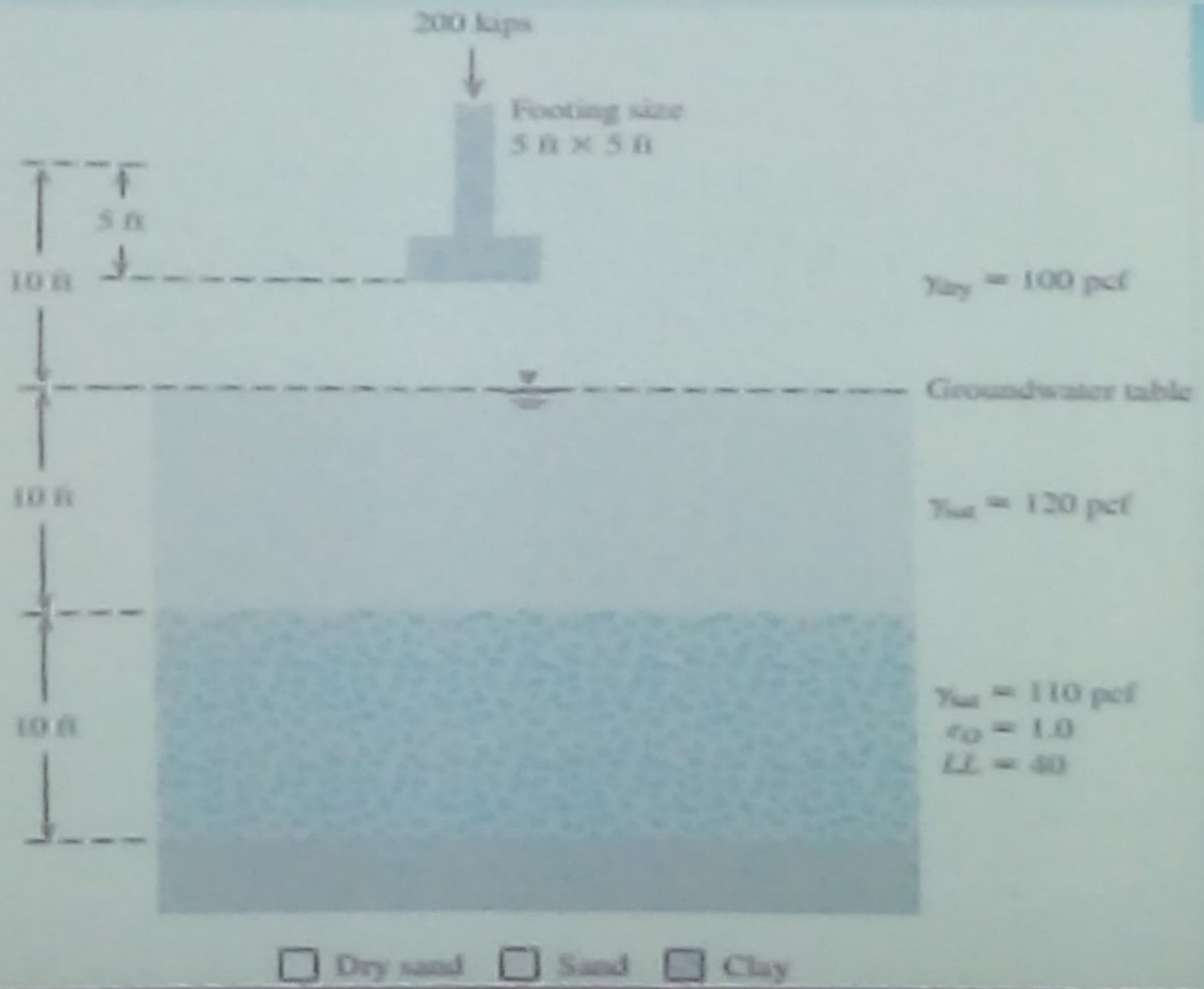


Maximum and minimum super-elevation

- ✓ Depends on (a) slow moving vehicle and (b) heavy loaded trucks with high CG a maximum super-elevation of 7 percent for plain and rolling terrain is recommended
- ✓ On hilly terrain 10 percent and on urban road 4 percent
- ✓ Minimum super elevation is 2-4 percent for drainage purpose, especially for large radius of the horizontal curve

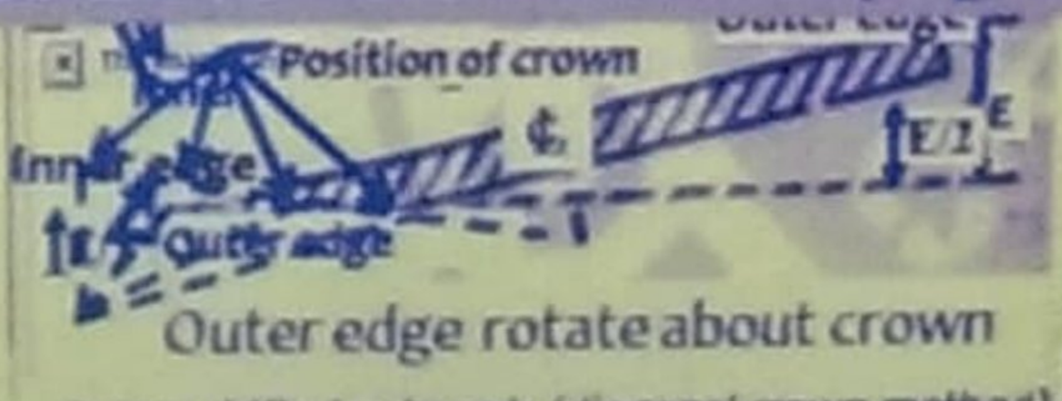
Attainment of super-elevation

- ✓ May be split into two parts
  - Elimination of crown of cambered section
  - Rotation of pavement cross section to attain full superelevation

Rotation about center line: Pavement is rotated such that

Rotation about inner edge: Pavement is rotated raising outer edge as well as centre such that outer edge is raised by full amount of super elevation with respect to inner edge

increasing width of inner half of cross section progressively



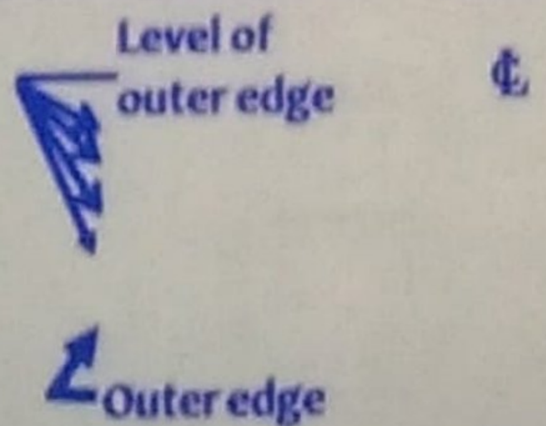
Crown shifted outwards (diagonal crown method)

Attainment of super-elevation

✓ May be split into two parts

- Elimination of crown of cambered section

Rotating outer edge about crown: Outer half of cross slope is rotated about crown at a desired rate such that this surface falls on same plane as inner half



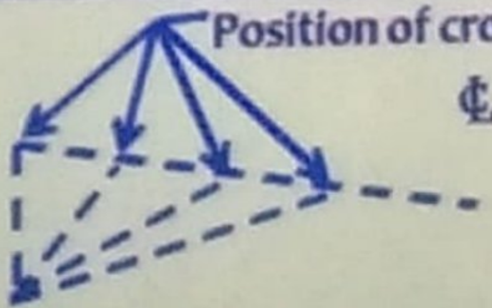
Outer edge rotate about crown

Attainment of super-elevation

- ✓ May be split into two parts
  - ➡ Elimination of crown of cambered section

Shifting position of crown: (also known as diagonal crown method)

Position of crown is progressively shifted outwards, t
increasing width of inner half of cross section progressi

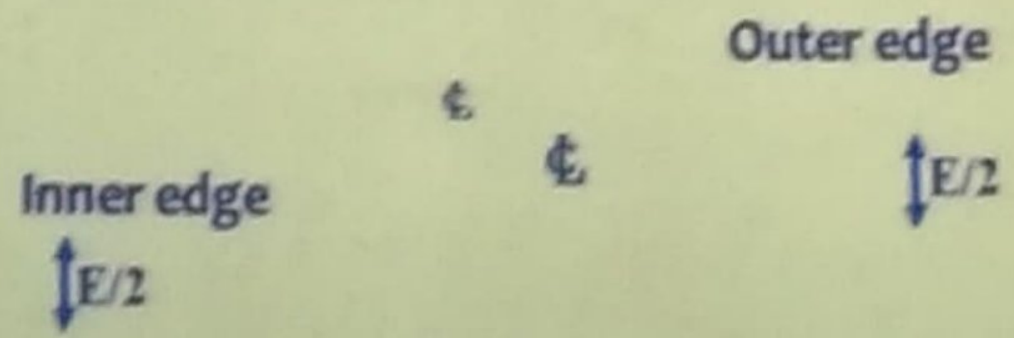


Crown shifted outwards (diagonal crown me

Attainment of super-elevation

- ✓ May be split into two parts
  - Elimination of crown of cambered section
  - Rotation of pavement cross section to attain full superelevation

Rotation about center line: Pavement is rotated such that inner edge is depressed and outer edge is raised both by half total amount of super elevation, i.e., by $\frac{E}{2}$ with respect to centre



Rotating about centre line

Attainment of super-elevation

✓ May be split into two parts

- Elimination of crown of cambered section
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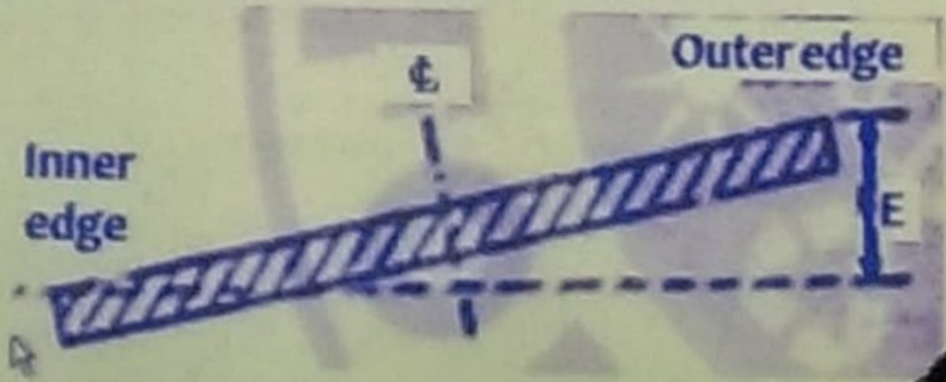
Rotation about inner edge: Pavement is rotated raising outer edge as well as centre such that outer edge is raised by full amount of super elevation with respect to inner edge



Rotating about inner edge

Radius of Horizontal Curve

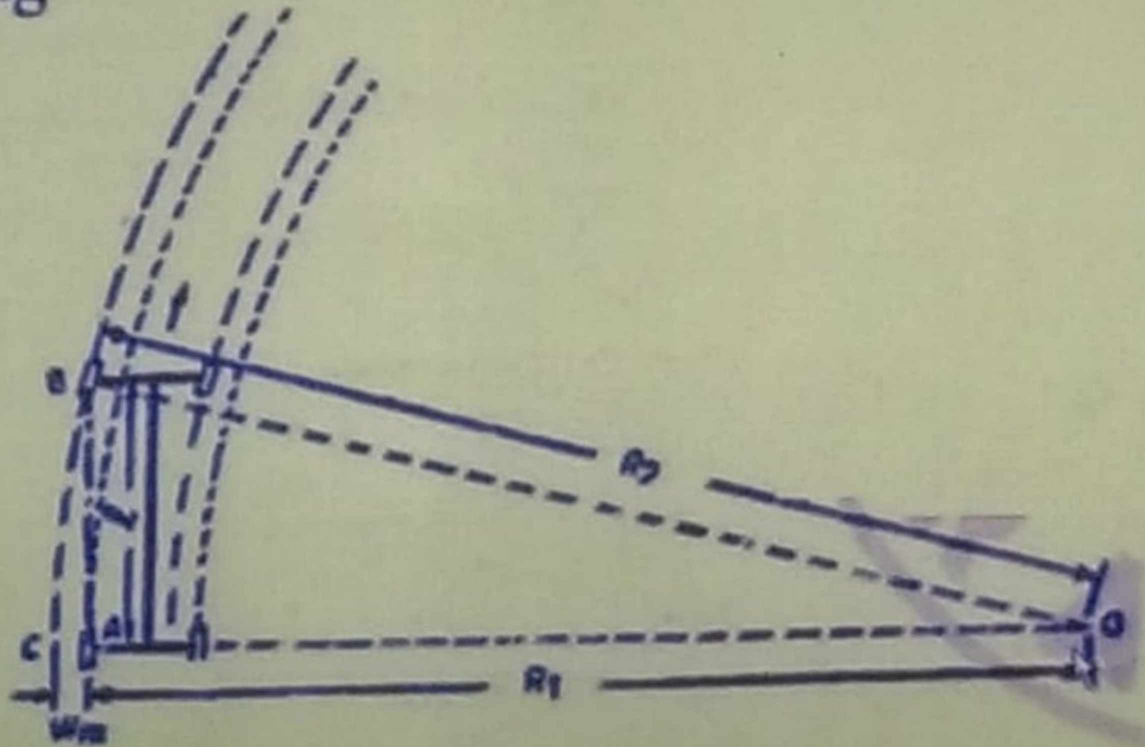
- ✓ Radius of horizontal curve is an important design aspect of geometric design
- ✓ Maximum comfortable speed on a horizontal curve depends on radius of curve
- ✓ Although it is possible to design curve with maximum super elevation and coefficient of friction, it is not desirable because re-alignment would be required if design speed is increased in future
- ✓ Therefore, a ruling minimum radius R_{ruling} can be derived by assuming maximum super elevation and coefficient of friction

$$R_{ruling} = \frac{v^2}{g(e+f)}$$

- ✓ Ideally, radius of curve should be higher than R_{ruling}
- ✓ Very large curves are not desirable because Setting out large curves in field becomes difficult, it also enhances driving strain

Widening of pavement on horizontal curve

- ✓ Extra widening refers to additional width of carriageway that is required on a curved section of a road over and above that required on a straight alignment
- ✓ Object of providing extra widening of pavements on horizontal curves are due to following reasons
  - Off tracking

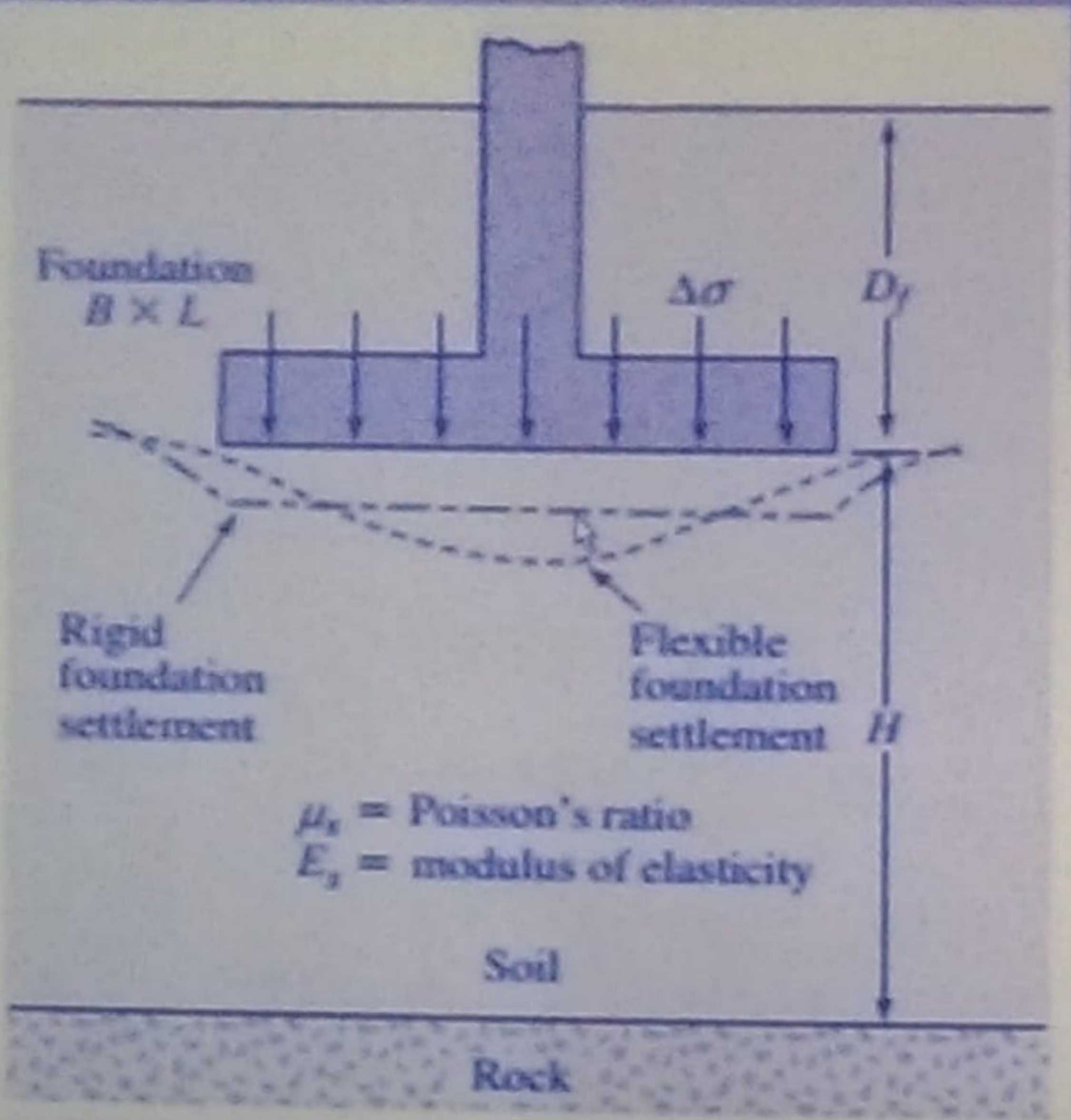


Widening of pavement on horizontal curve

- ✓ Extra widening refers to additional width of carriageway that is required on a curved section of a road over and above that required on a straight alignment
- ✓ Object of providing extra widening of pavements on horizontal curves are due to following reasons
  - Off tracking
  - Transverse skidding due to excessive speed
  - Towing vehicle
  - Tendency of driver to follow outer side of curve
  - Greater clearance between crossing or overtaking vehicle

- Required extra widening of pavement at horizontal curves depends on
  - Length of wheel base of vehicle
  - Radius of curve negotiated
  - Psychological factors (function of speed and radius)
- ✓ It has been a practice therefore to provide extra width of pavement on horizontal curves when radius is less than about 300 m

IMMEDIATE SETTLEMENT



Elastic settlement of flexible and rigid foundations

✓ If foundation is perfectly flexible, settlement may be expressed as

$$S_e = \Delta\sigma(\alpha B')\frac{1-\mu_s^2}{E_s} I_s I_f$$

where $\Delta\sigma$ = net applied pressure on the foundation

μ_s = Poisson's ratio of soil

E_s = average modulus of elasticity of the soil $z = 0$ to about $z = 4B$

B' = $B/2$ for center of foundation

= B for corner of foundation

I_s = shape factor (Steinbrenner, 1934)

$$= F_1 + \frac{1-2\mu_s}{1-\mu_s} F_2$$

I_f = depth factor (Fox, 1948) = $f\left(\frac{D_f}{B}, \mu_s, \text{and } \frac{L}{B}\right)$

α = factor that depends on the location on the foundation where settlement is being calculated

$$F_1 = \frac{1}{\pi}(A_0 + A_1)$$

$$F_2 = \frac{n'}{2\pi}\tan^{-1}A_2$$

$$A_0 = m' \ln \frac{(1 + \sqrt{m'^2 + 1})\sqrt{m'^2 + n'^2}}{m'(1 + \sqrt{m'^2 + n'^2 + 1})}$$

$$A_1 = \ln \frac{(m' + \sqrt{m'^2 + 1})\sqrt{1 + n'^2}}{m' + \sqrt{m'^2 + n'^2 + 1}}$$

$$A_2 = \frac{m'}{n'\sqrt{m'^2 + n'^2 + 1}}$$

For calculation of settlement at the *center* of the foundation:

$$\alpha = 4$$

$$m' = \frac{L}{B}$$

$$n' = \frac{H}{\left(\frac{B}{2}\right)}$$

Variation of I_f with D_f/B, B/L, and μ_s

		B/L		
μ_s	D_f/B	0.2	0.5	1.0
0.3	0.2	0.95	0.93	0.90
	0.4	0.90	0.86	0.81
	0.6	0.85	0.80	0.74
	1.0	0.78	0.71	0.65
0.4	0.2	0.97	0.96	0.93
	0.4	0.93	0.89	0.85
	0.6	0.89	0.84	0.78
	1.0	0.82	0.75	0.69
0.5	0.2	0.99	0.98	0.96
	0.4	0.95	0.93	0.89
	0.6	0.92	0.87	0.82
	1.0	0.85	0.79	0.72

✓ Elastic settlement of a rigid foundation can be estimated as

$$S_{e(\text{rigid})} \approx 0.93 S_{e(\text{flexible, center})}$$

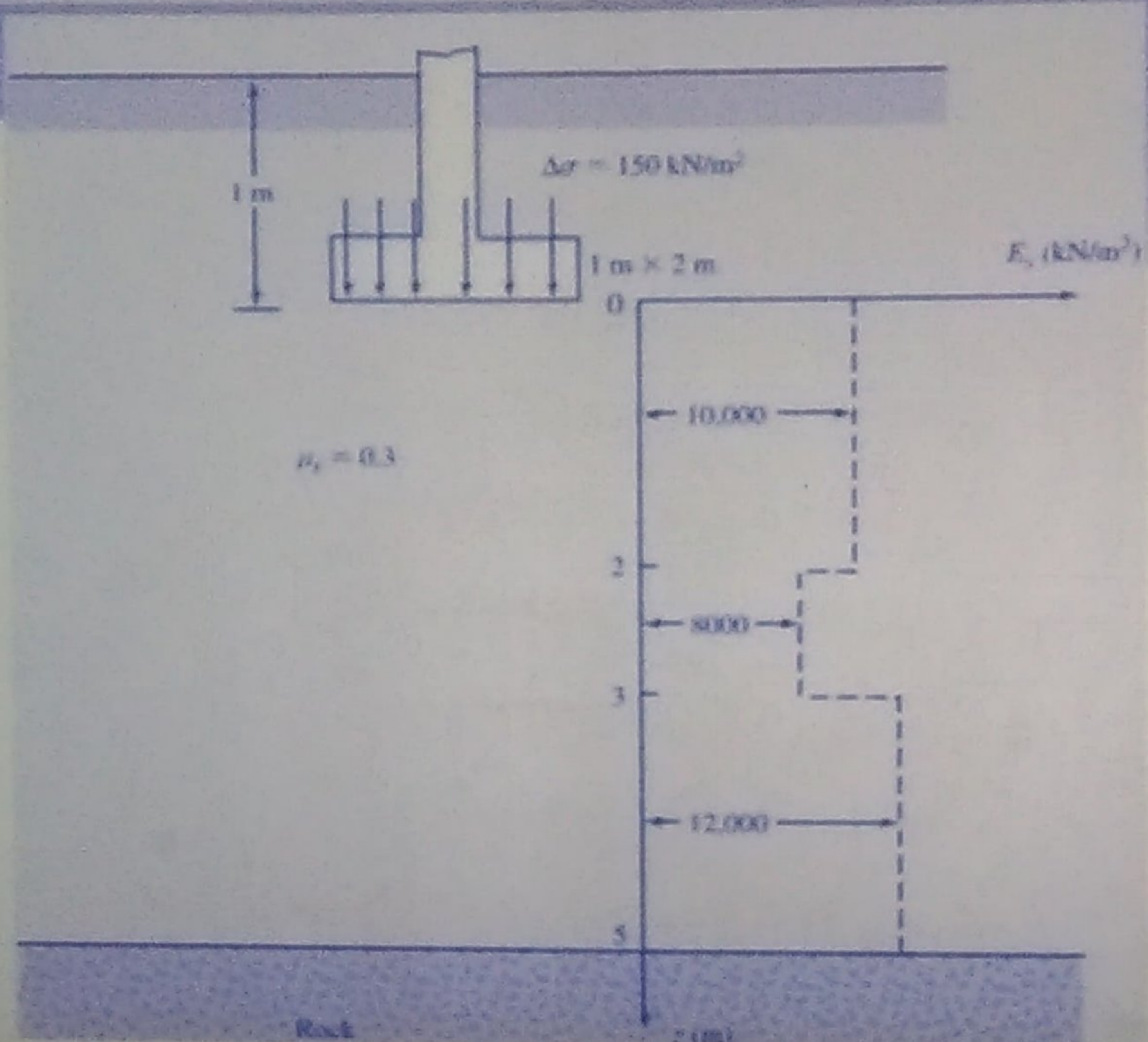
Due to the nonhomogeneous nature of soil deposits, the magnitude of E_s may vary with depth. For that reason, Bowles (1987) recommended using a weighted average value of E_s

$$E_s = \frac{\Sigma E_{s(i)} \Delta z}{\bar{z}}$$

Where $E_{s(i)}$ = soil modulus of elasticity within a depth Δz

$\bar{z}$ = H or $5B$, whichever is smaller

A rigid shallow foundation 1 m x 2 m is shown in Figure. Calculate the elastic settlement at the center of the foundation.



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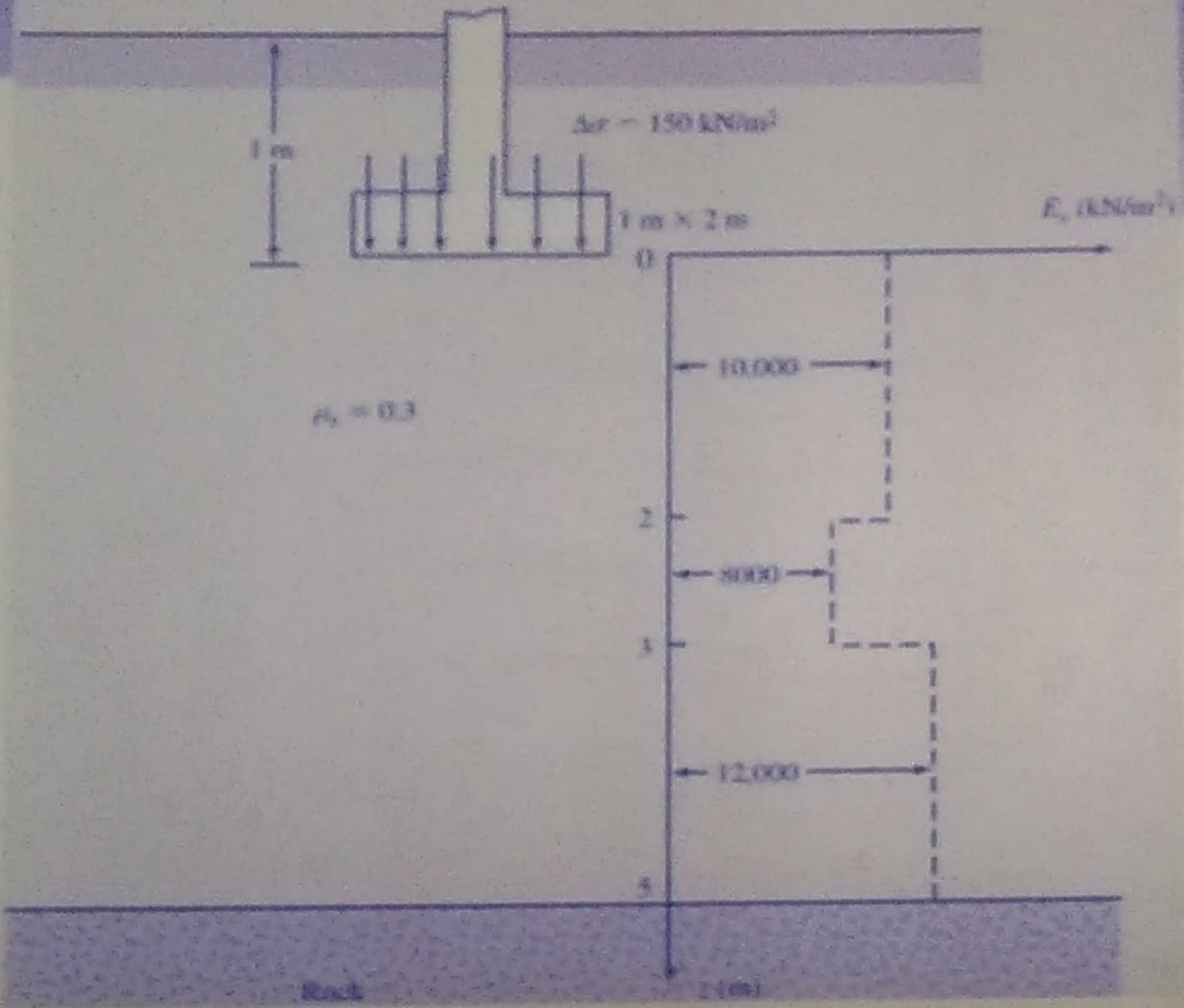


Improved Equation for Elastic Settlement

- ✓ In 1999, Mayne and Poulos presented an improved formula for calculating elastic settlement of foundations
- ✓ Formula takes into account rigidity of foundation, depth of embedment of foundation, increase in modulus of elasticity of soil with depth, and location of rigid layers at a limited depth
- ✓ To use Mayne and Poulos's equation, one needs to determine equivalent diameter of a rectangular foundation, or

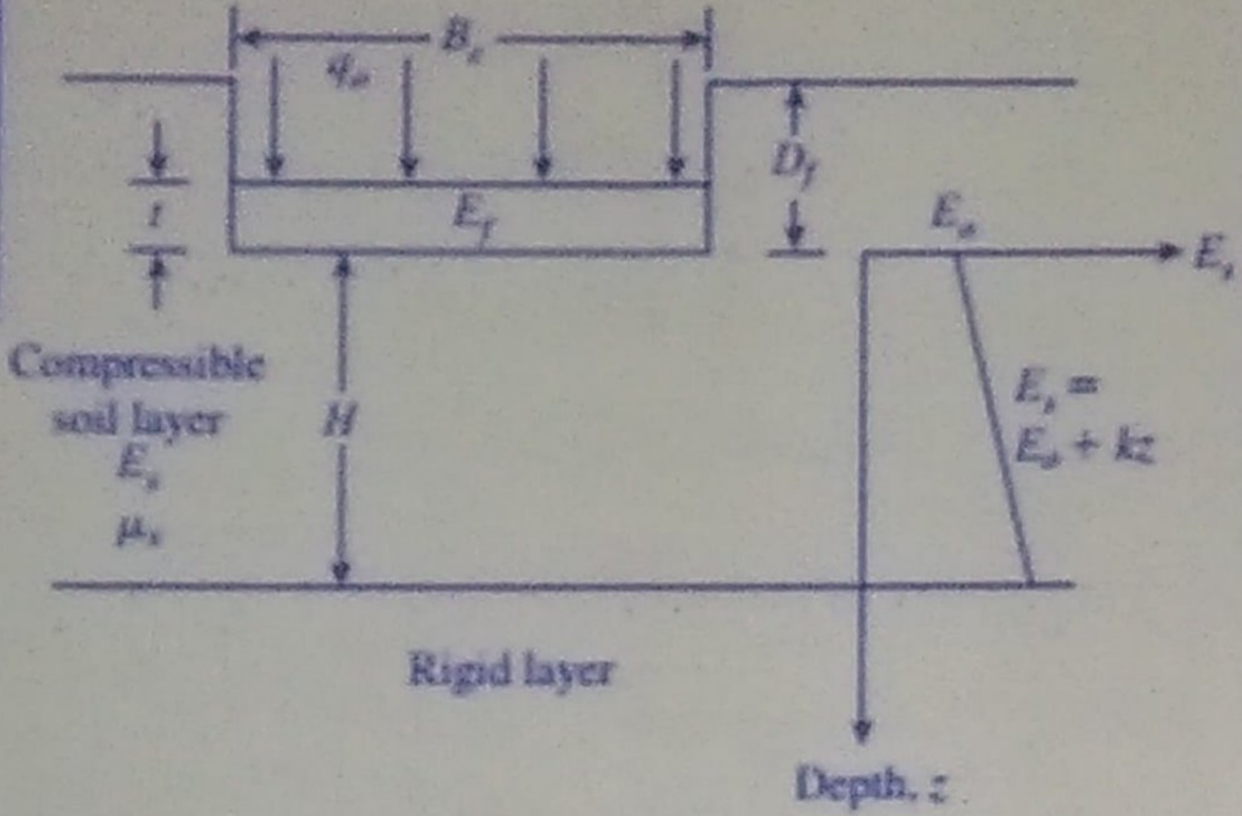
$$B_e = \sqrt{\frac{4BL}{\pi}}$$

where

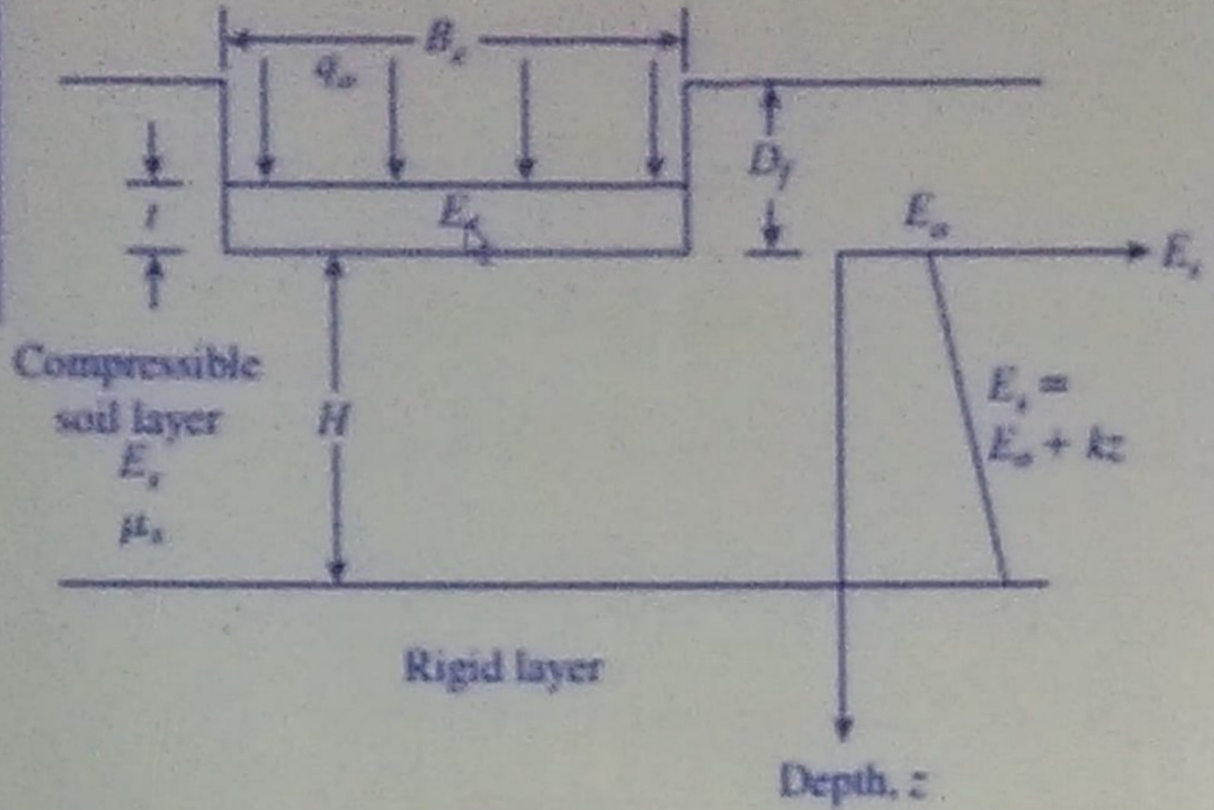
B = width of foundation
L = length of foundation

For circular foundations, $B_e = B$

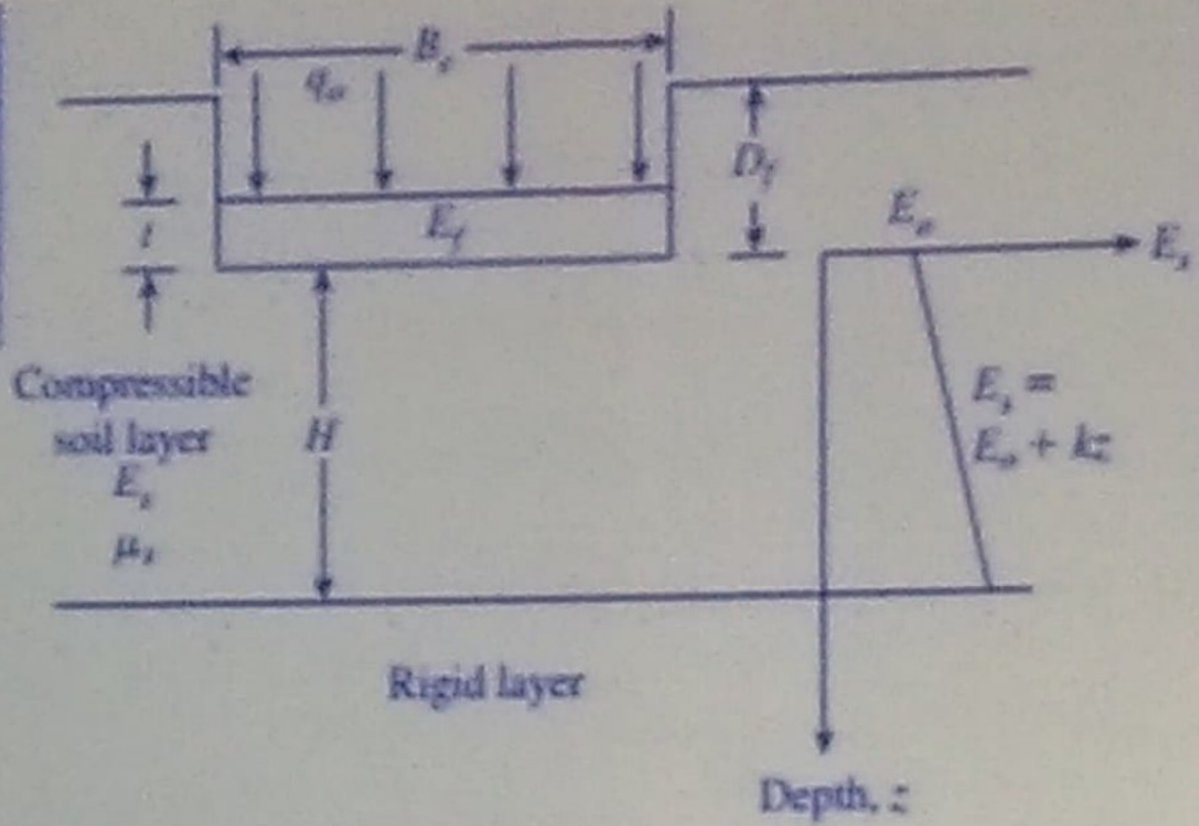
where B = diameter of foundation



B_c
q_o
D_f
t
E_f
E_o
E_s
Compressible soil layer
E_s
μ_s
H
$E_s = E_o + kz$
Rigid layer
Depth, z



- ✓ Figure shows a foundation with an equivalent diameter located at a depth D_f below ground surface
- ✓ Let thickness of foundation be t and modulus of elasticity of foundation material be E_f
- ✓ A rigid layer is located at a depth H below bottom of foundation

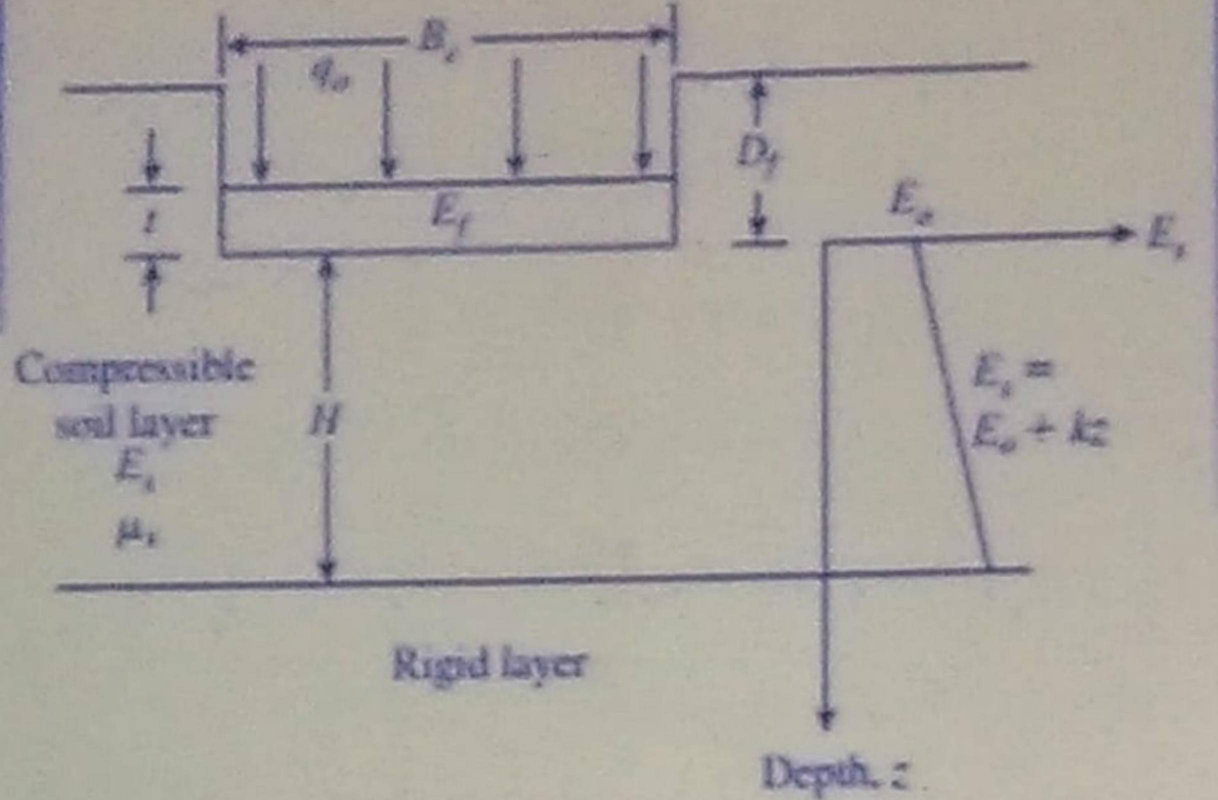


- ✓ Modulus of elasticity of compressible soil layer can be given as

$$E_s = E_o + kz$$

- ✓ With preceding parameters defined, elastic settlement below center of foundation is

$$S_e = \frac{q_o B_e I_G I_F I_E}{E_o}\left(1 - \mu_s^2\right)$$



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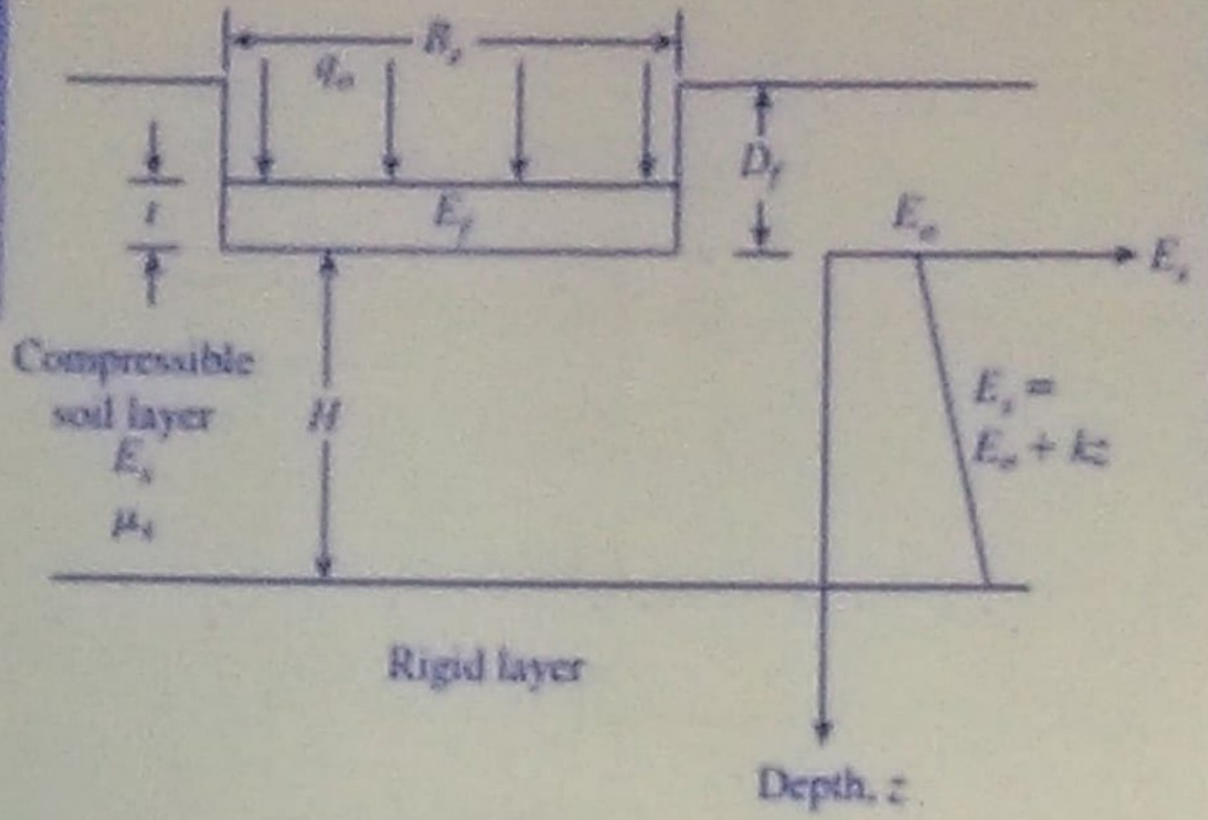
$$S_e = \frac{q_o B_e I_G I_F I_E}{E_o}\left(1 - \mu_s^2\right)$$

where

I_G = influence factor for the variation of E_s with depth

$= f\left(\beta = \frac{E_o}{kB_e}, \frac{H}{B_e}\right)$



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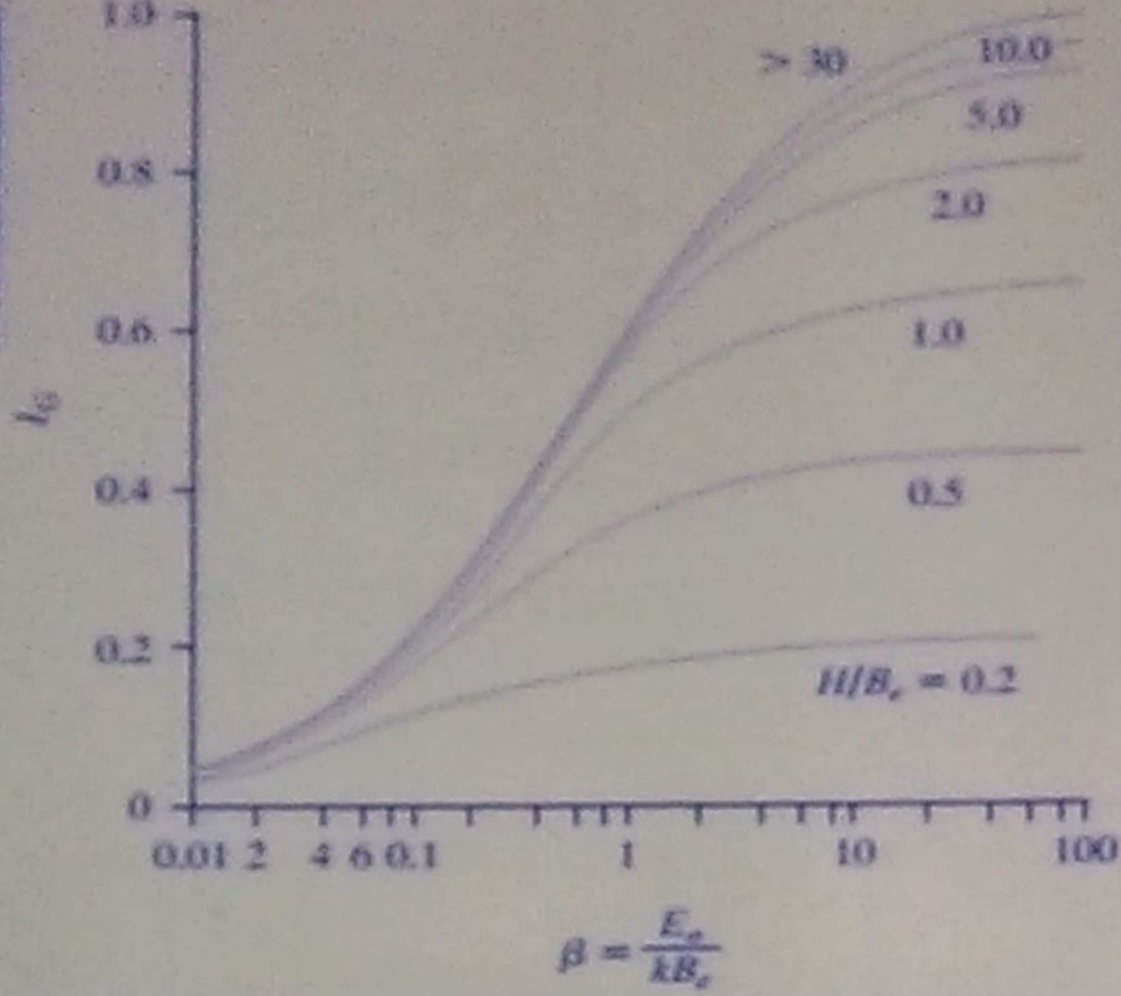
$$S_e = \frac{q_o B_e I_G I_F I_E}{E_o}\left(1 - \mu_s^2\right)$$

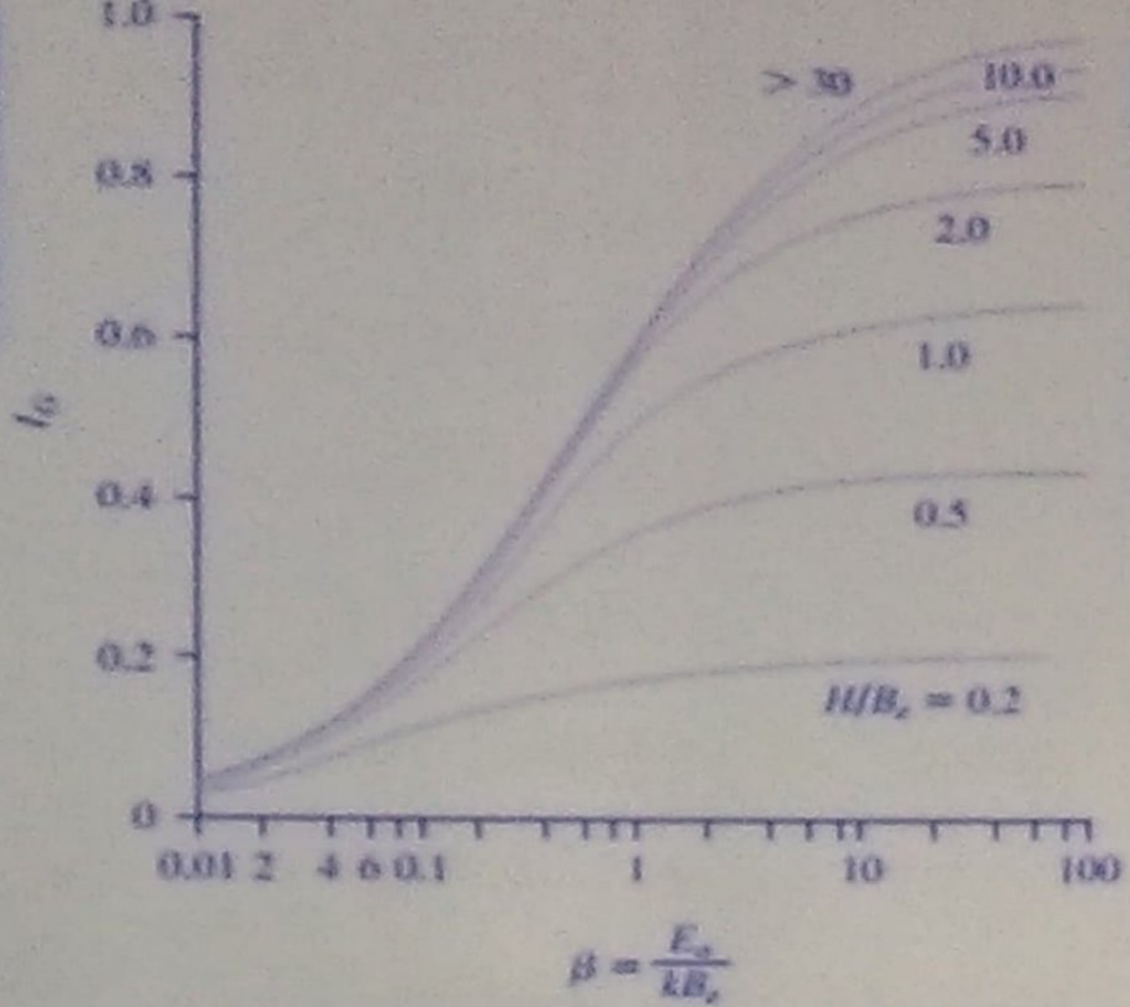
I_F = foundation rigidity correction factor

I_E = foundation embedment correction factor



✓ Foundation rigidity correction factor can be expressed as

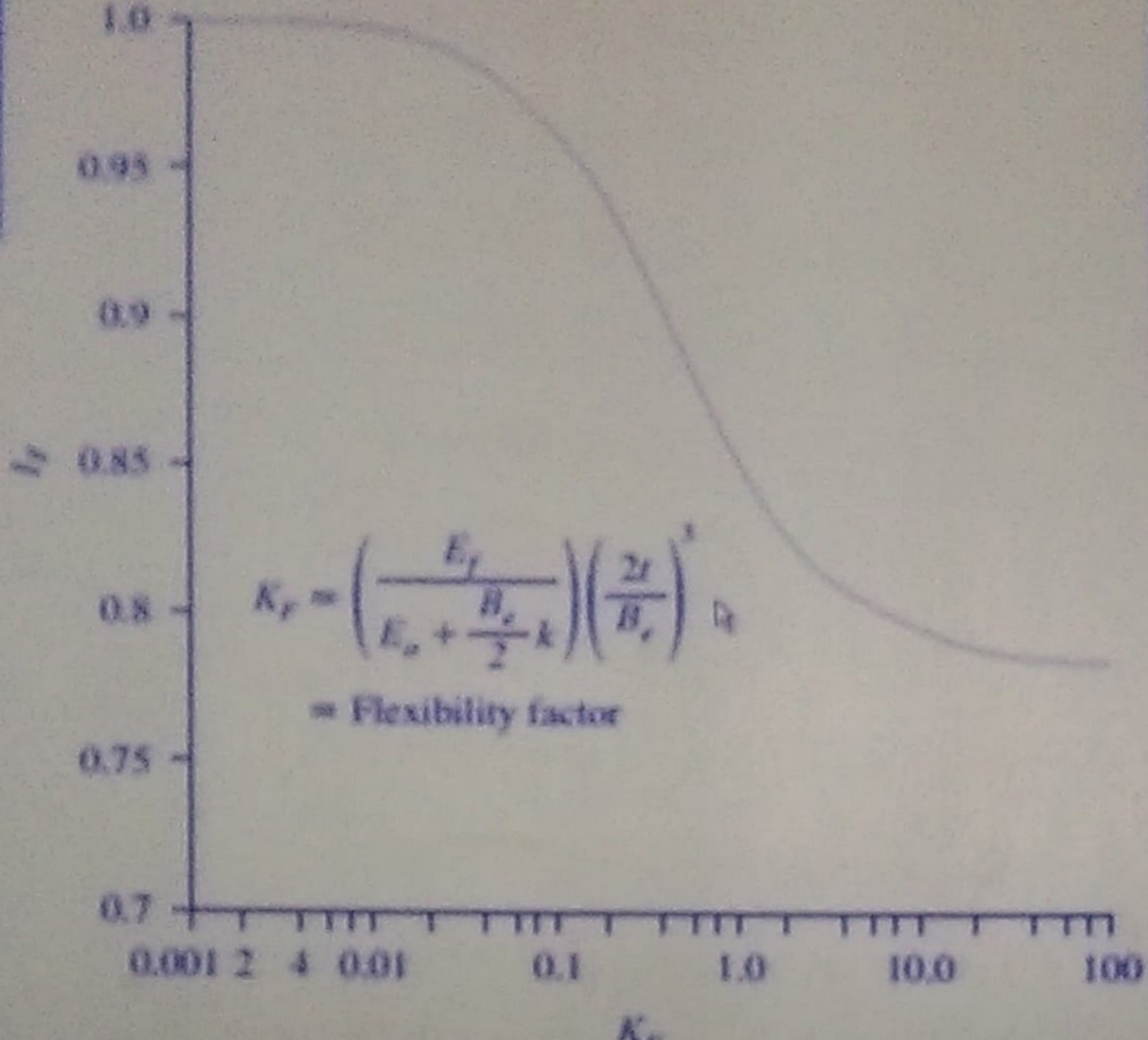
$$I_F = \frac{\pi}{4} + \frac{1}{4.6 + 10\left(\dfrac{E_f}{E_o + \dfrac{B_e}{2}k}\right)\left(\dfrac{2t}{B_e}\right)^3}$$



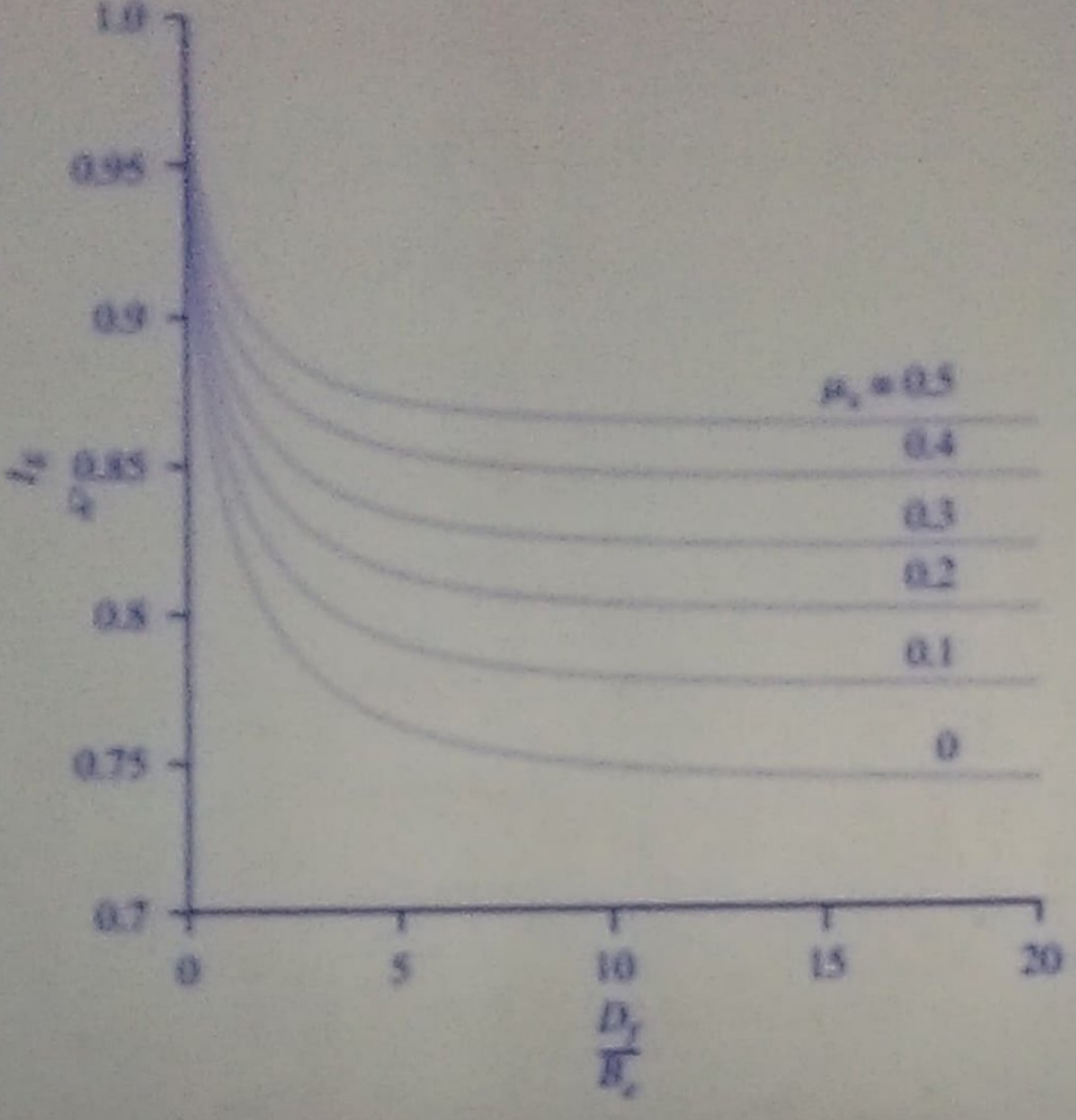
✓ Foundation rigidity correction factor can be expressed as

✓ Similarly embedment correction factor is

$$I_E = 1 - \frac{1}{3.5\exp(1.22\mu_s - 0.4)\left(\dfrac{B_e}{D_f} + 1.6\right)}$$



Variation of rigidity correction factor I_F with flexibility factor K_F



Variation of embedment correction factor I_E with D_f/B_e

Settlement of Foundation on Sand Based on Standard Penetration Resistance

➢ Meyerhof's Method

- ✓ Meyerhof (1956) proposed a correlation for net bearing pressure for foundations with standard penetration resistance, N_{60}
- ✓ Net pressure has been defined as

$$q_{net} = \overline{q} - \gamma D_f$$

where $\overline{q}$ = stress at the level of the foundation

- ✓ According to Meyerhof's theory, for 25 mm (1 in.) of estimated maximum settlement

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.08} \quad (\text{for } B \leq 1.22 \text{ m})$$

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.125}\left(\frac{B + 0.3}{B}\right)^2 \quad (\text{for } B > 1.22 \text{ m})$$

- Since time that Meyerhof proposed his original correlations, researchers have observed that its results are rather conservative
- Later, Meyerhof (1965) suggested that net allowable bearing pressure should be increased by about 50%
- Bowles (1977) proposed that modified form of bearing equations be expressed as

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.05} F_d\left(\frac{S_e}{25}\right) \quad (\text{for } B \leq 1.22 \text{ m})$$

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.08}\left(\frac{B+0.3}{B}\right)^2 F_d\left(\frac{S_e}{25}\right) \quad (\text{for } B > 1.22 \text{ m})$$

where

F_d = depth factor = $1 + 0.33(D_f/B)$
B = foundation width, in meters
S_e = settlement, in mm

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$$S_e(\mathrm{mm}) = \frac{1.25 q_{net}(\mathrm{kN/m^2})}{N_{60} F_d} \quad (\text{for } B \leq 1.22\ \mathrm{m})$$

$$S_e(\mathrm{mm}) = \frac{2 q_{net}(\mathrm{kN/m^2})}{N_{60} F_d} \left(\frac{B}{B + 0.3}\right)^2 \quad (\text{for } B > 1.22\ \mathrm{m})$$

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N_{60} referred to in preceding equations is standard penetration resistance between bottom of foundation and 2B below bottom

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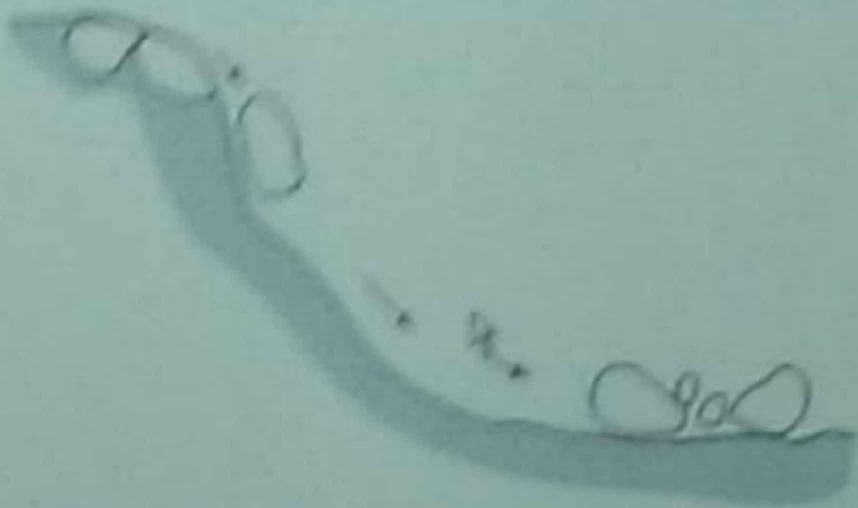
N_{60} referred to in preceding equations is standard penetration resistance between bottom of foundation and $2B$ below bottom

Burland and Burbidge's Method ⟶ (SS)

SLOPE STABILITY

- ✓ An exposed ground surface that stands at an angle with horizontal is called an unrestrained slope
- ✓ Slope can be natural or man-made
- ✓ It can fail in various modes
- ✓ Cruden and Varnes (1996) classified slope failures into following five major categories
    - Fall

This is detachment of soil and/or rock fragments that fall down a slope



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This is detac all down a slo

Soil and rock "fall" in a slope (Courtesy of E.C. Shin, University of Inchon, South Korea)

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- Slope can be natural or man-made
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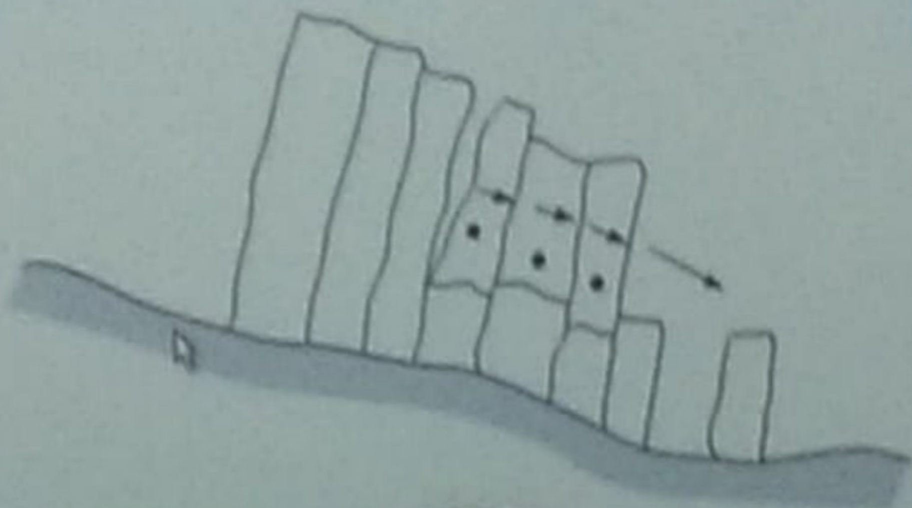
- Cruden and Varnes (1996) classified slope failures into following five major categories
  - Fall
  - Topple

This is a forward rotation of soil and/or rock mass about an ax[is] below center of [g]



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- ✓ Slope can be natural or man-made
- ✓ It can fail in various modes
- ✓ Cruden and Varnes (1996) classified slope failures into following five major categories
  - Fall
  - Topple
  - Slide

This is downward movement of a soil mass occurring on a surface of rupture

Factor of Safety

- ✓ Task of engineer charged with analyzing slope stability is to determine factor of safety
- ✓ Generally, factor of safety is defined as

$$F_s = \frac{\tau_f}{\tau_d} \text{ -----------------------(1)}$$

where F_s = factor of safety with respect to strength

τ_f = average shear strength of soil

τ_d = average shear stress developed along poten failure surface

- ✓ Shear strength of a soil consists of two compone cohesion and friction, and may be written as

$$\tau_f = c' + \sigma' \tan \phi' \text{ ---------------(2}$$

where c' = cohesion

ϕ' = angle of friction

σ' = normal stress on the potential failure surface

✓ In a similar manner, we can write

$$\tau_d = c'_d + \sigma' \tan \phi'_d \quad \text{------------------}(3)$$

where c'_d and φ'_d are, respectively, cohesion and angle of friction that develop along potential failure surface

✓ Substituting Eqs. (2) and (3) into Eq. (1), we get

$$F_s = \frac{c' + \sigma' \tan \phi'}{c'_d + \sigma' \tan \phi'_d} \quad \text{------------------}(4)$$

✓ Factor of safety with respect to cohesion, and factor of safety with respect to friction, are defined as

$$F_{c'} = \frac{c'}{c'_d}$$

- We can see that when $F_{c'}$ becomes equal to $F_{\varphi'}$, it gives factor of safety with respect to strength. Or, if

$$\frac{c'}{c'_d} = \frac{\tan\phi'}{\tan\phi'_d}$$

- Then we can write

$$F_s = F_{c'} = F_{\phi'}$$

- When F_s is equal to 1, slope is in a state of impending failure
- Generally, a value of 1.5 for factor of safety with respect to strength is acceptable for design of a stable slope

- Failure of slopes take place mainly due to
  - Action of gravity force
  - Seepage force within soil
  - Excavation or undercutting of foot
  - Gradual disintegration of structure of soil

Type of slopes

- Infinite slopes
- Finite slopes

Type of slopes

- Infinite slopes

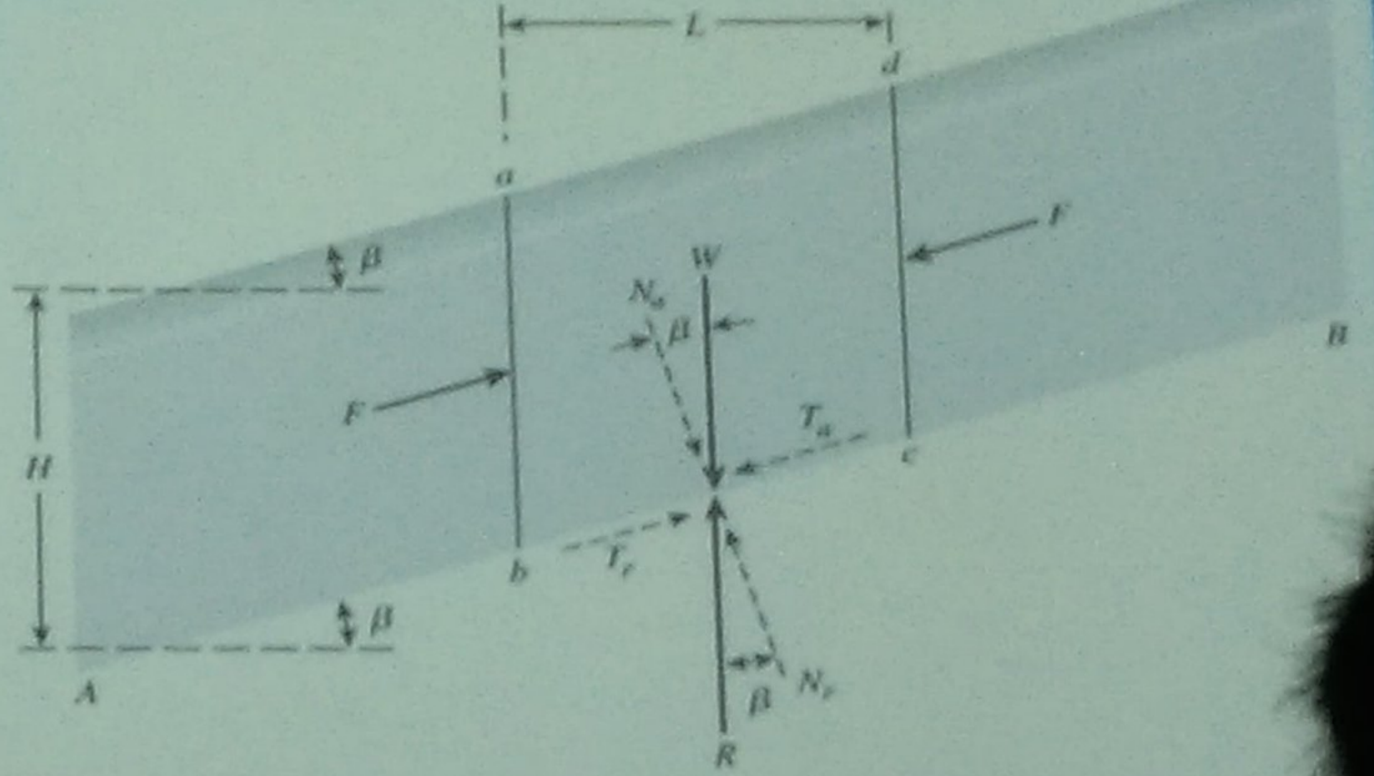
- ✓ Slope represents boundary surface of a semi-infinite soil mass
- ✓ Soil properties for all identical depths below surface are constant
- ✓ Don not exist in nature

Type of slopes

- Infinite slopes
- Finite slopes

- ✓ Slope is of limited extent
- ✓ Inclined faces of earth dam, embankment, cuts etc.

Stability of Infinite Slopes



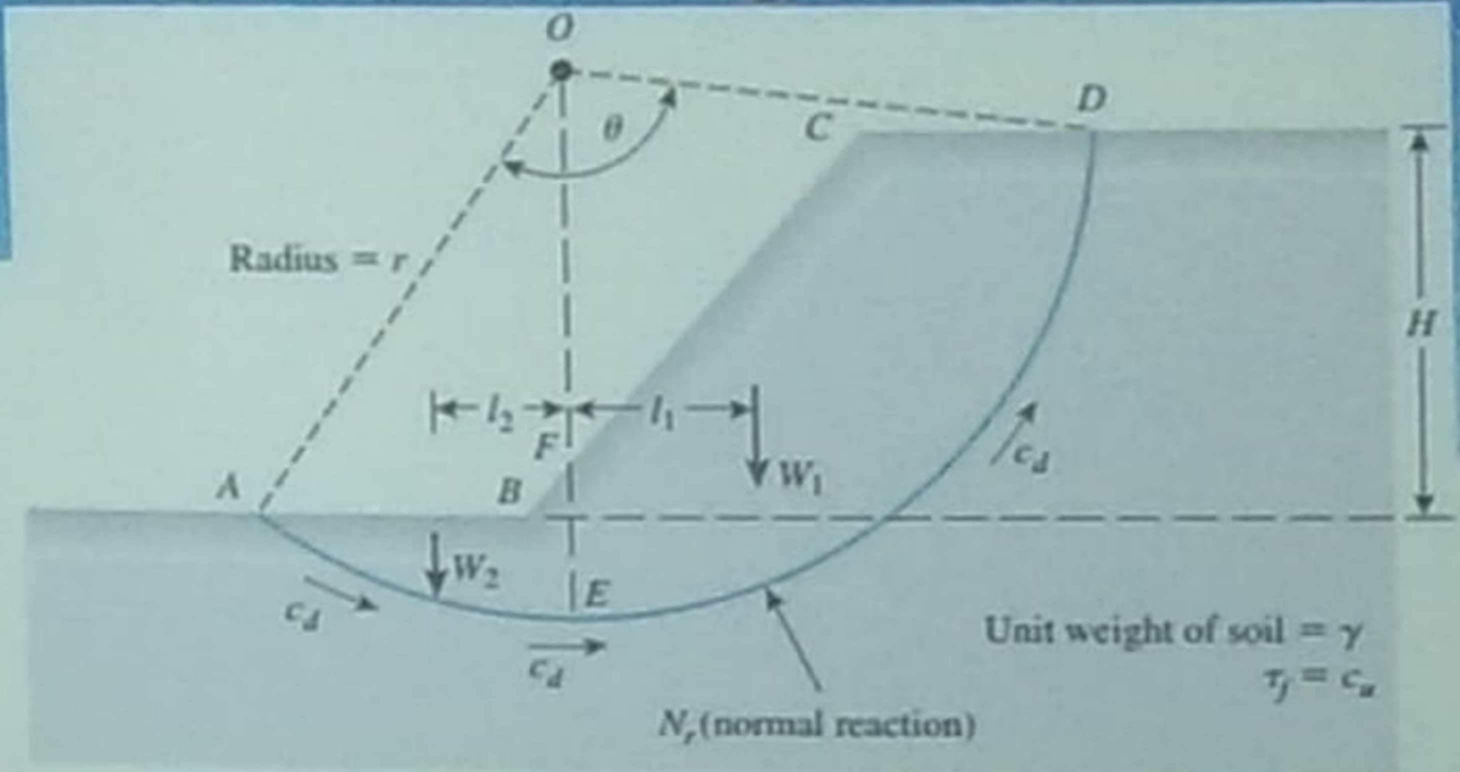
- ✓ Shear strength of soil may be given by

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

Types of Stability Analysis Procedures

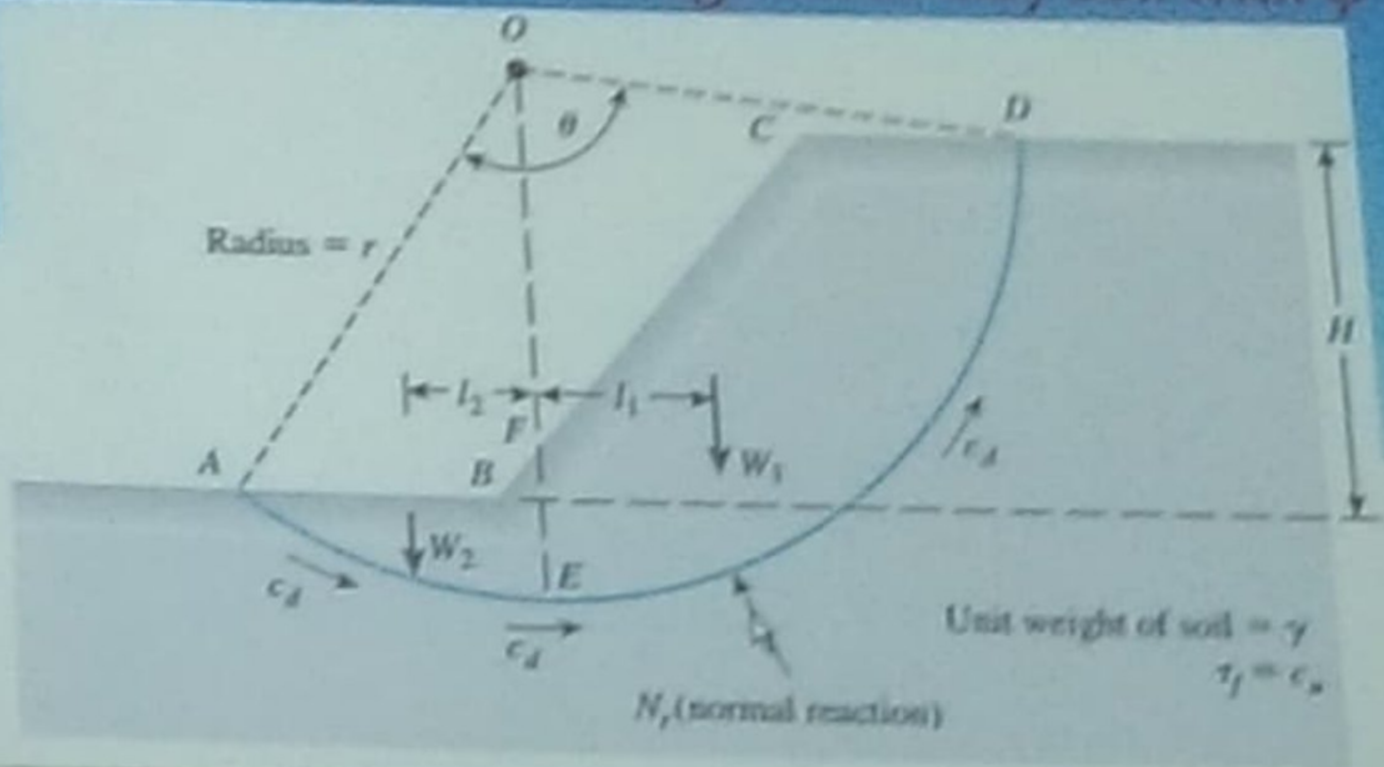
- ✓ Various procedures of stability analysis may, in general, be divided into two major classes:
- ➡ **Mass procedure:** In this case, mass of soil above surface of sliding is taken as a unit
- ✓ This procedure is useful when soil that forms slope is assumed to be homogeneous, although this is not case in most natural slopes
- ➡ **Method of slices:** In this procedure, soil above surface of sliding is divided into a number of vertical parallel slices
- ✓ Stability of each slice is calculated separately
- ✓ This is a versatile technique in which nonhomogeneity of soils and pore water pressure can be taken into consideration
- ✓ It also accounts for variation of normal stress along potential failure surface

Mass Procedure - Slopes in Homogeneous Clay Soil with $\phi = 0$



- ✓ Undrained shear strength of soil is assumed to be constant with depth and may be given by $\tau_f = c_u$
- ✓ To perform stability analysis, we choose a trial potential curve of sliding, AED, which is an arc of a circle that has a radius r and center of circle is located at O
- ✓ Considering a unit length perpendicular to section of slope, we can give weight of soil above curve AED as $W = W_1 + W_2$

Mass Procedure - Slopes in Homogeneous Clay Soil with $\phi = 0$

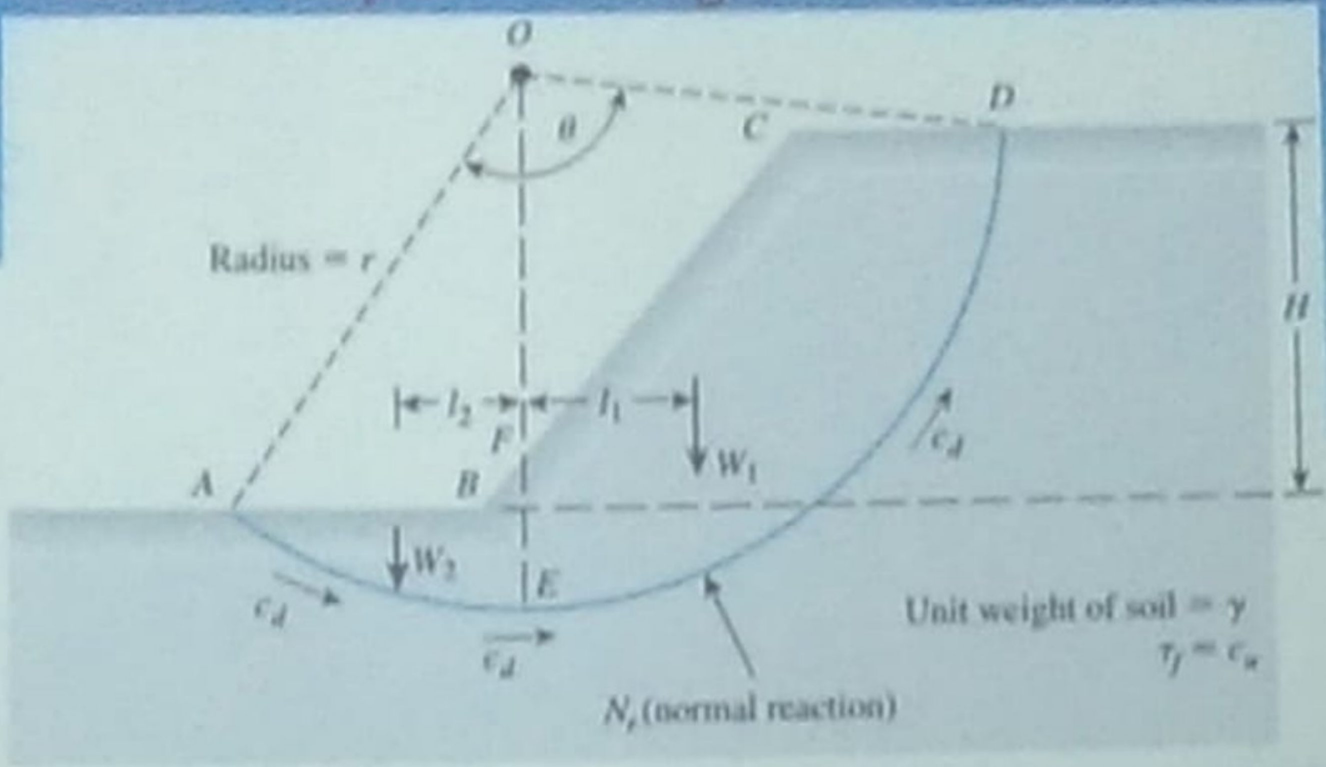


$$W_1 = (\text{Area of } FCDEF)(\gamma)$$
$$W_2 = (\text{Area of } ABFEA)(\gamma)$$

- ✓ Failure of slope may occur by sliding of soil mass
- ✓ Moment of driving force about O to cause slope instability is

  $M_d = W_1 l_1 - W_2 l_2$ where l_1 and l_2 are moment arms
- ✓ Resistance to sliding is derived from cohesion that acts along potential surface of sliding

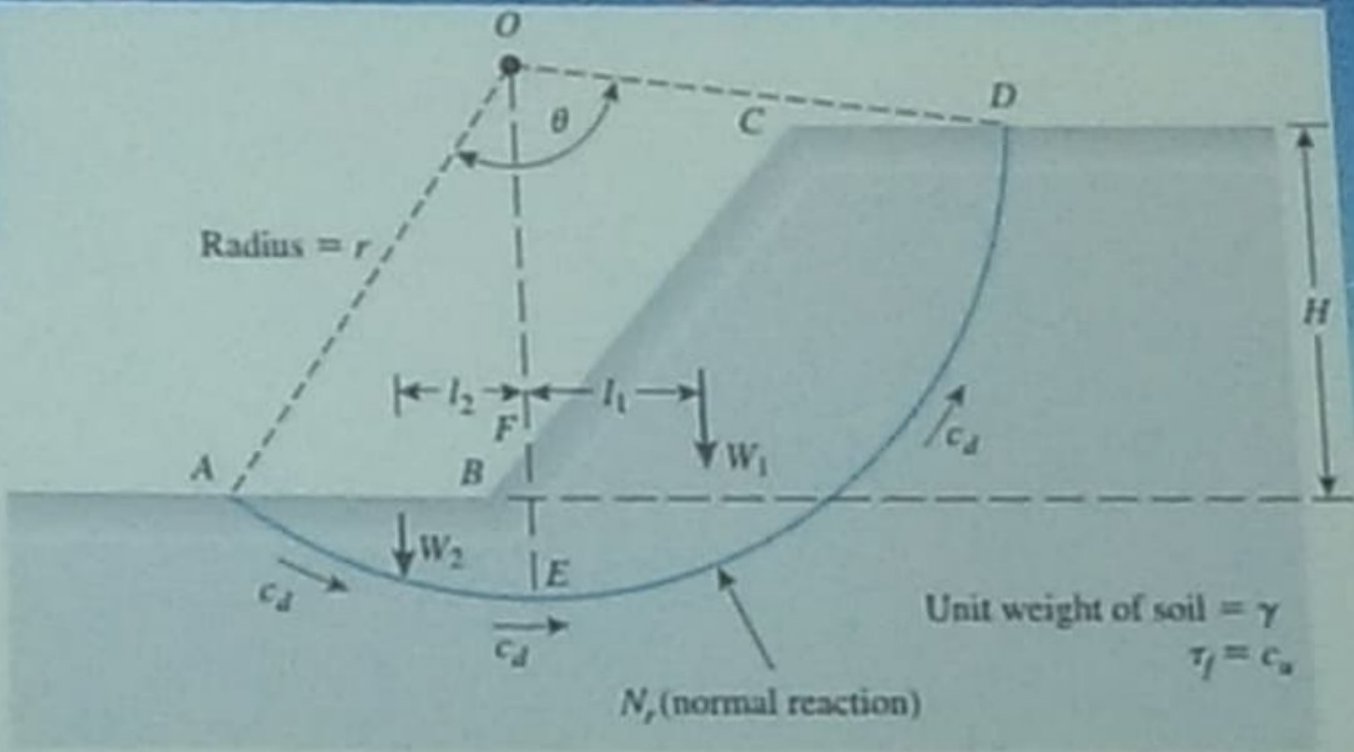
Mass Procedure - Slopes in Homogeneous Clay Soil with $\phi = 0$



- ✓ If c_d is cohesion that needs to be developed, moment of resisting forces about O is $M_R = c_d(\widehat{AED})(1)(r) = c_d r^2\theta$
- ✓ For equilibrium, $M_R = M_d$; thus, $c_d r^2\theta = W_1 l_1 - W_2 l_2$

or $c_d = \dfrac{W_1 l_1 - W_2 l_2}{r^2\theta}$

Mass Procedure - Slopes in Homogeneous Clay Soil with $\phi = 0$



- ✓ Factor of safety against sliding may now be found

$$F_s = \frac{\tau_f}{c_d} = \frac{c_u}{c_d}$$

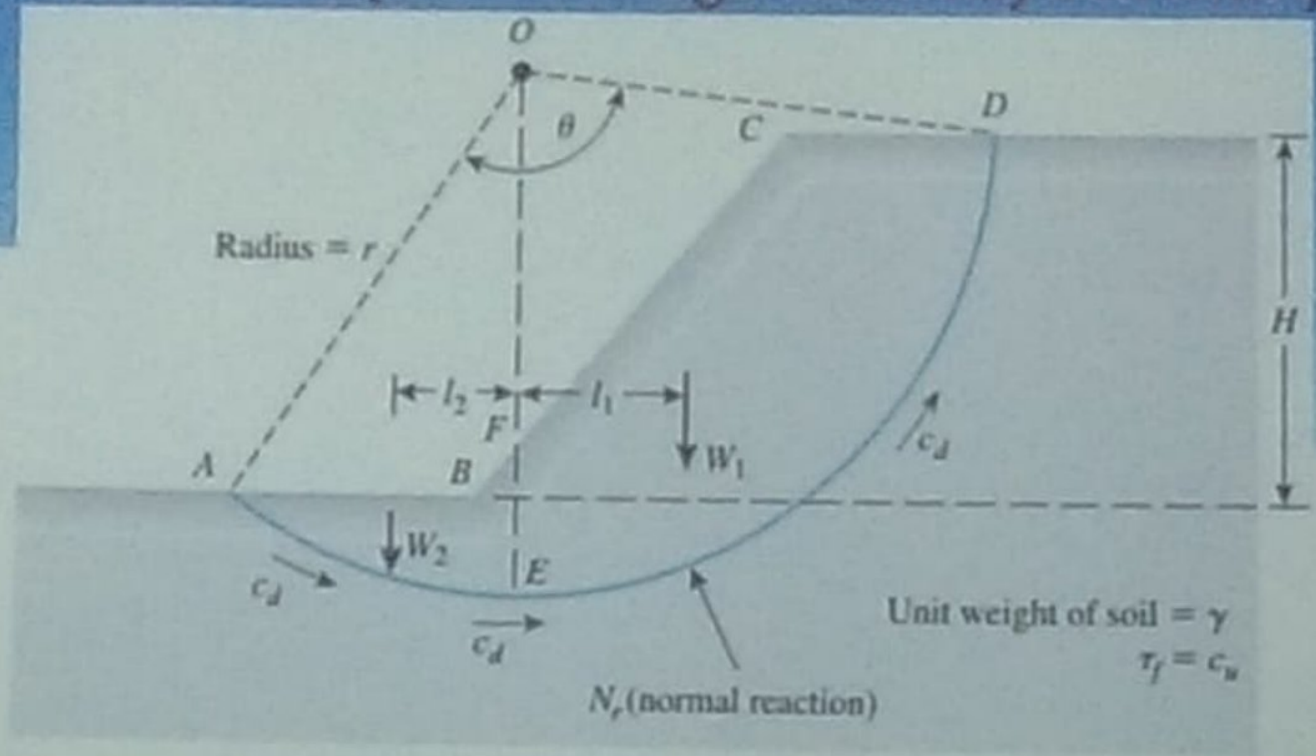
- ✓ For case of critical circles, developed cohesion can be expressed by relationship

$$c_d = \gamma H m$$

or

$$\frac{c_d}{\gamma H} = m$$

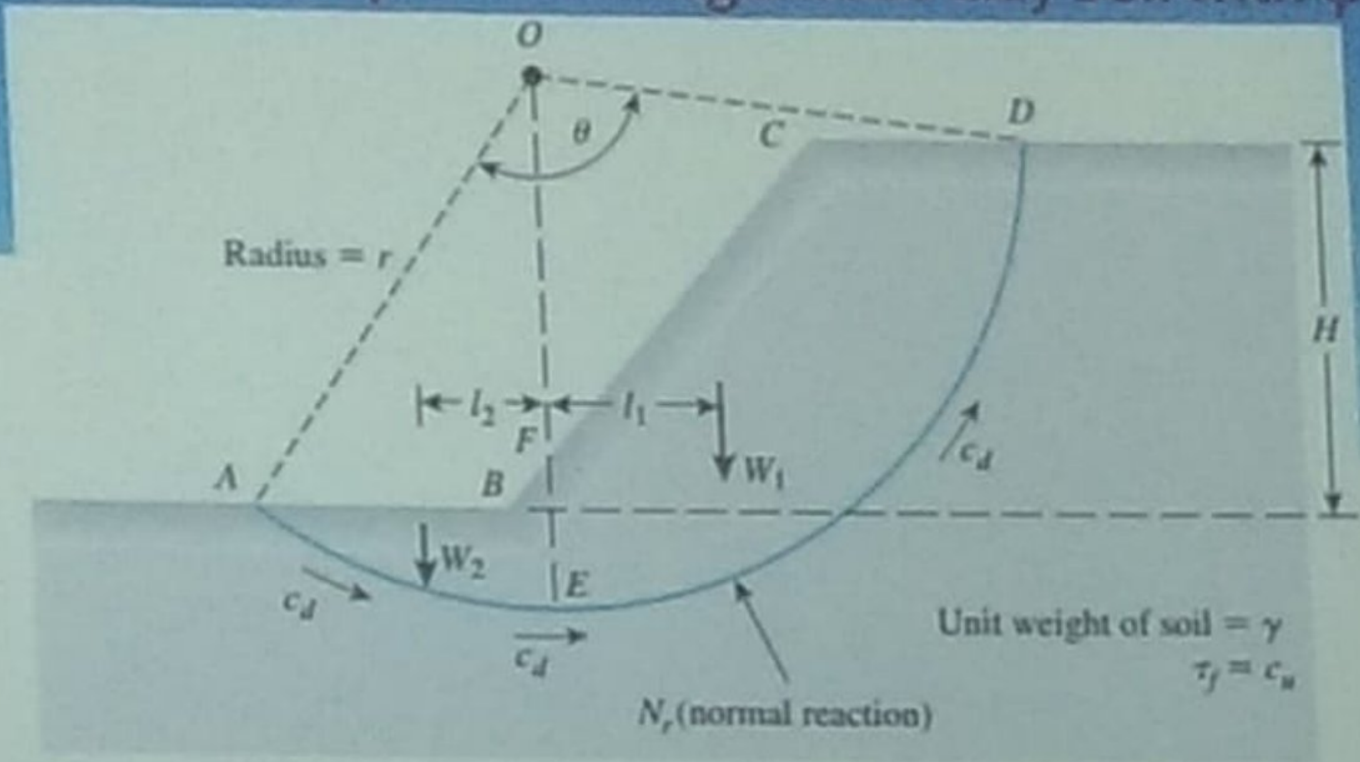
Mass Procedure - Slopes in Homogeneous Clay Soil with $\phi = 0$



✓ Critical height (i.e., $F_s = 1$) of slope can be evaluated by substituting $H = H_{cr}$ and $c_d = cu$ (full mobilization of undrained shear strength) into preceding equation,

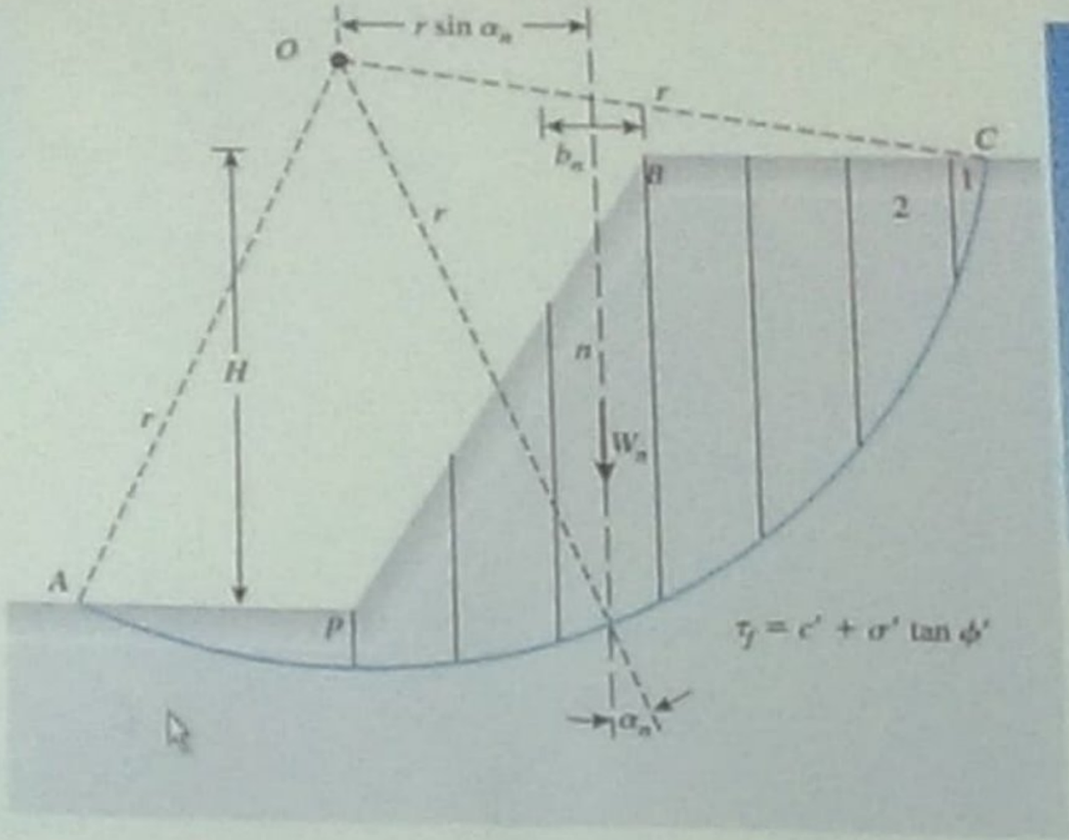
$$\boxed{H_{cr} = \frac{c_u}{\gamma m}}$$

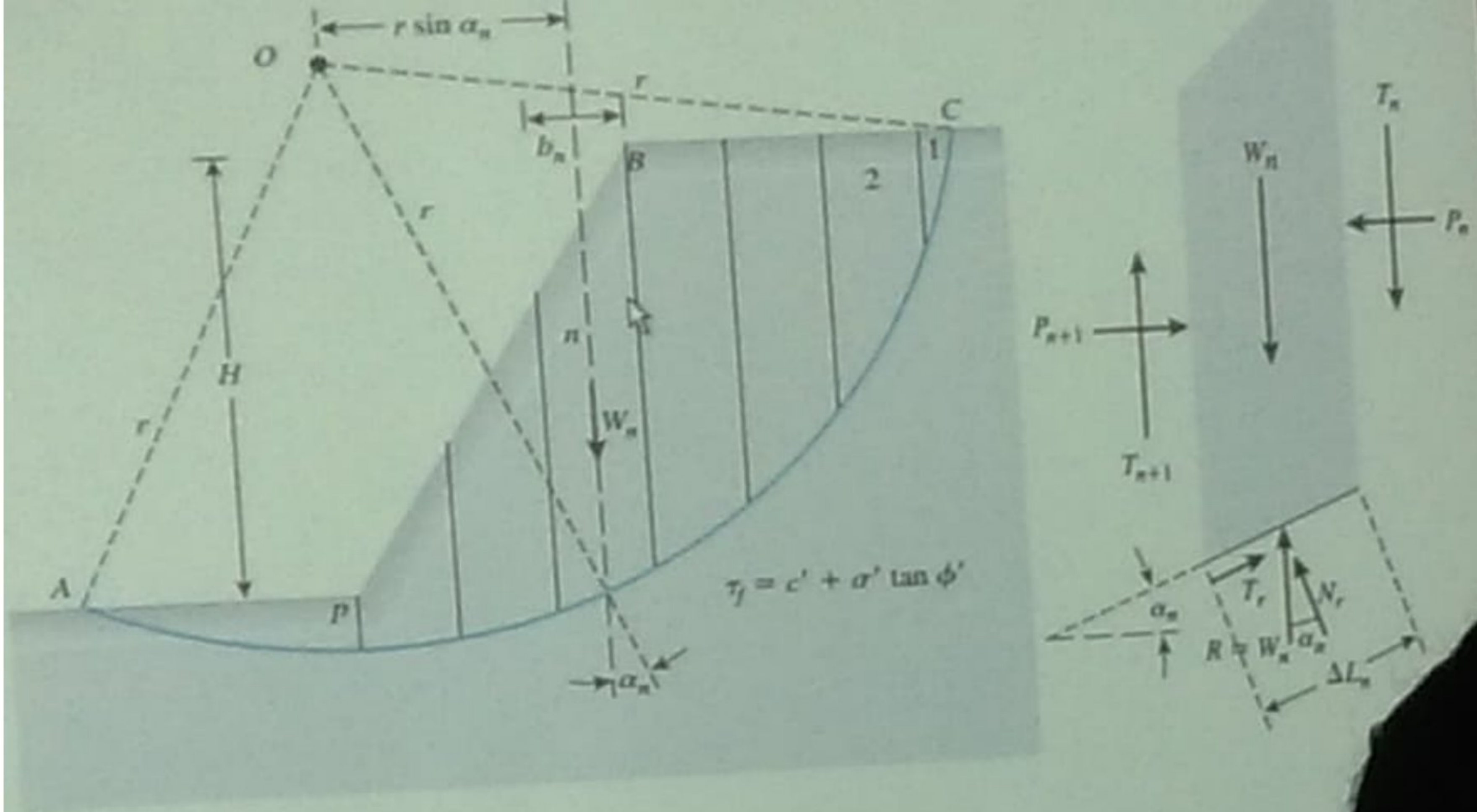
Mass Procedure - Slopes in Homogeneous Clay Soil with $\phi = 0$



Mass Procedure - Slopes in Homogeneous $c' - \phi'$ Soil ⟶ SS

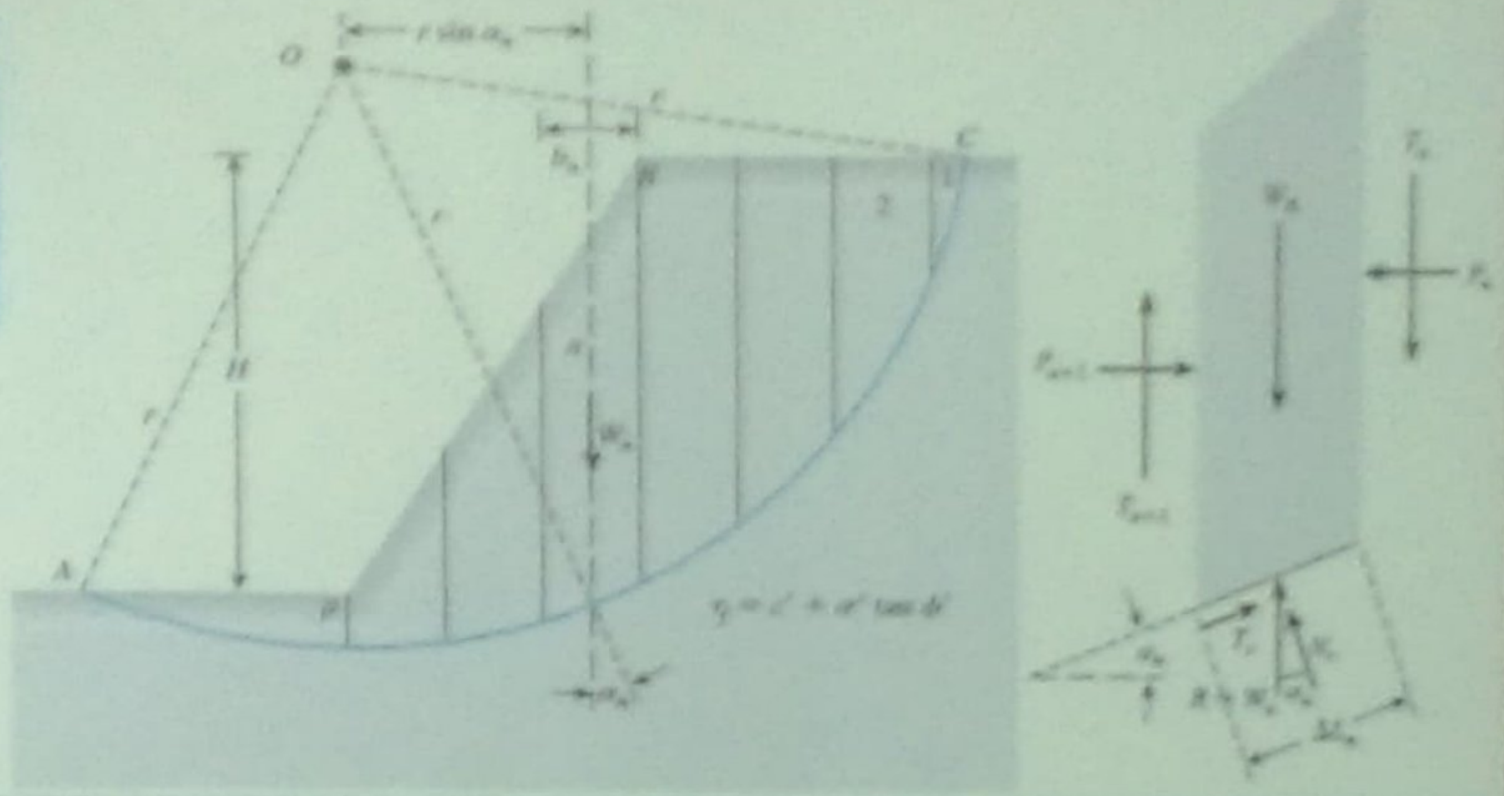
Ordinary Method of Slices





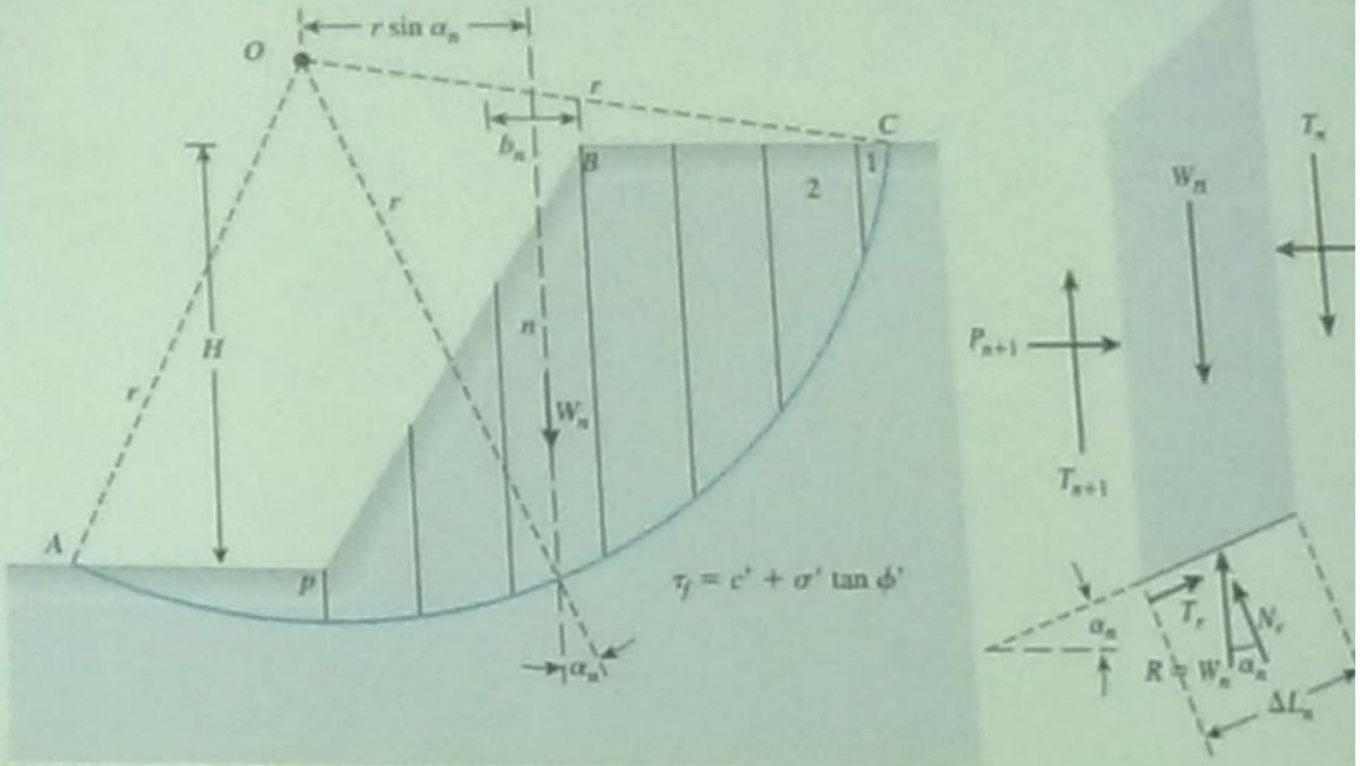
r sin α_n
O
r
C
b_n
B
1
2
r
H
n
W_n
r
A
P
τ_f = c' + σ' tan φ'
α_n
T_n
W_n
P_n
P_{n+1}
T_{n+1}
T_r
N_r
α_n
R = W_n
α_n
ΔL_n

Ordinary Method of Slices



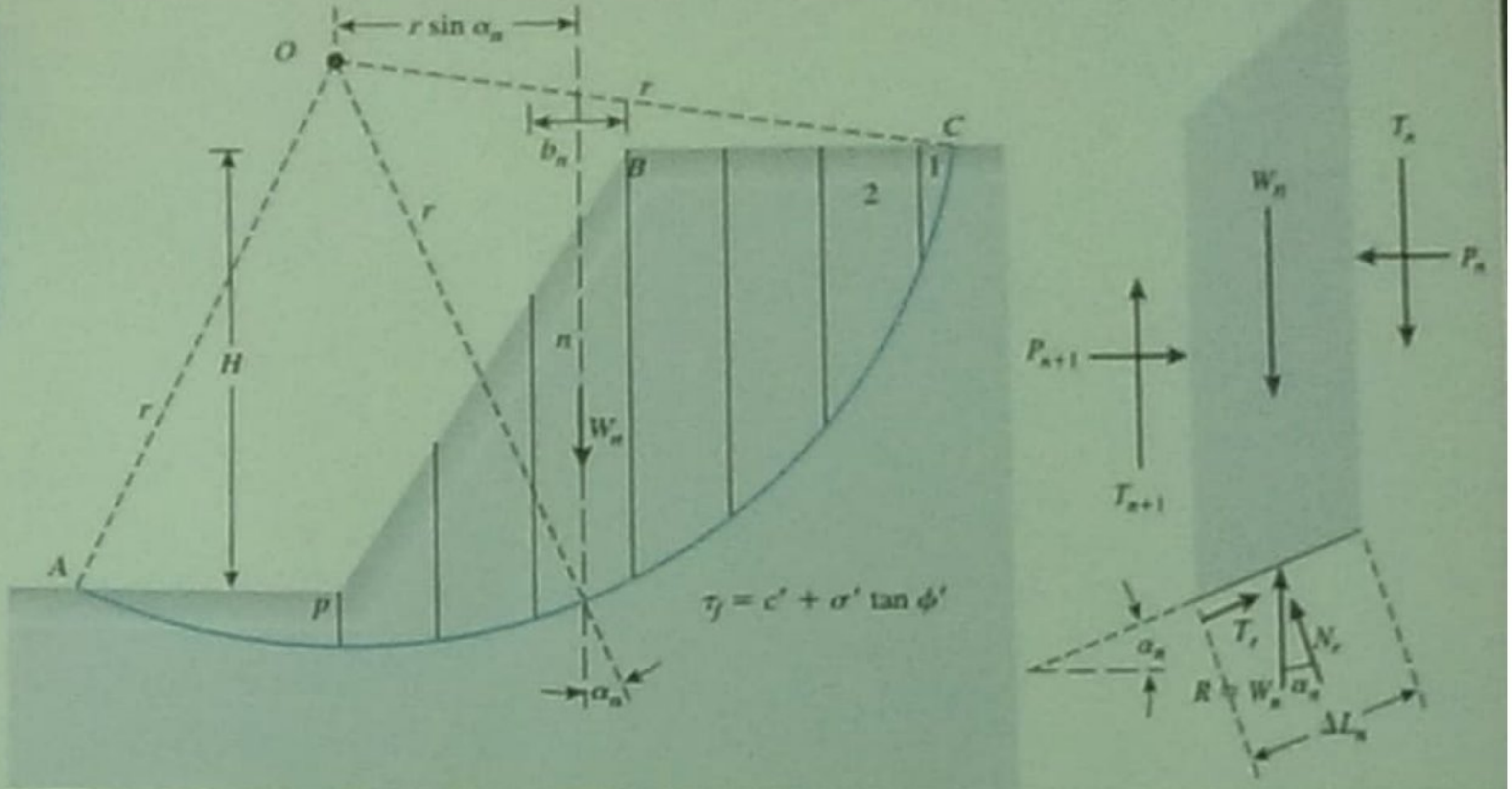
- ✓ AC is an arc of a circle representing trial failure surface
- ✓ Soil above trial failure surface is divided into several vertical slices
- ✓ Width of each slice need not be same
- ✓ Considering a unit length perpendicular to cross section shown, forces that act on a typical slice (nth slice) are shown in Figure
- ✓ W_n is weight of slice
- ✓ Forces N_r and T_r, respectively, are normal and tangential components of reaction R

Ordinary Method of Slices



- ✓ P_n and P_{n+1} are normal forces that act on sides of slice
- ✓ Similarly, shearing forces that act on sides of slice are T_n a T_{n+1}
- ✓ For simplicity, pore water pressure is assumed to be zero
- ✓ Forces P_n, P_{n+1}, T_n, and T_{n+1} are difficult to determine
- ✓ However, we can make an approximate assumptio resultants of P_n and T_n are equal in magnitude to resulta P_{n+1} and T_{n+1} and that their lines of action coincide

Ordinary Method of Slices



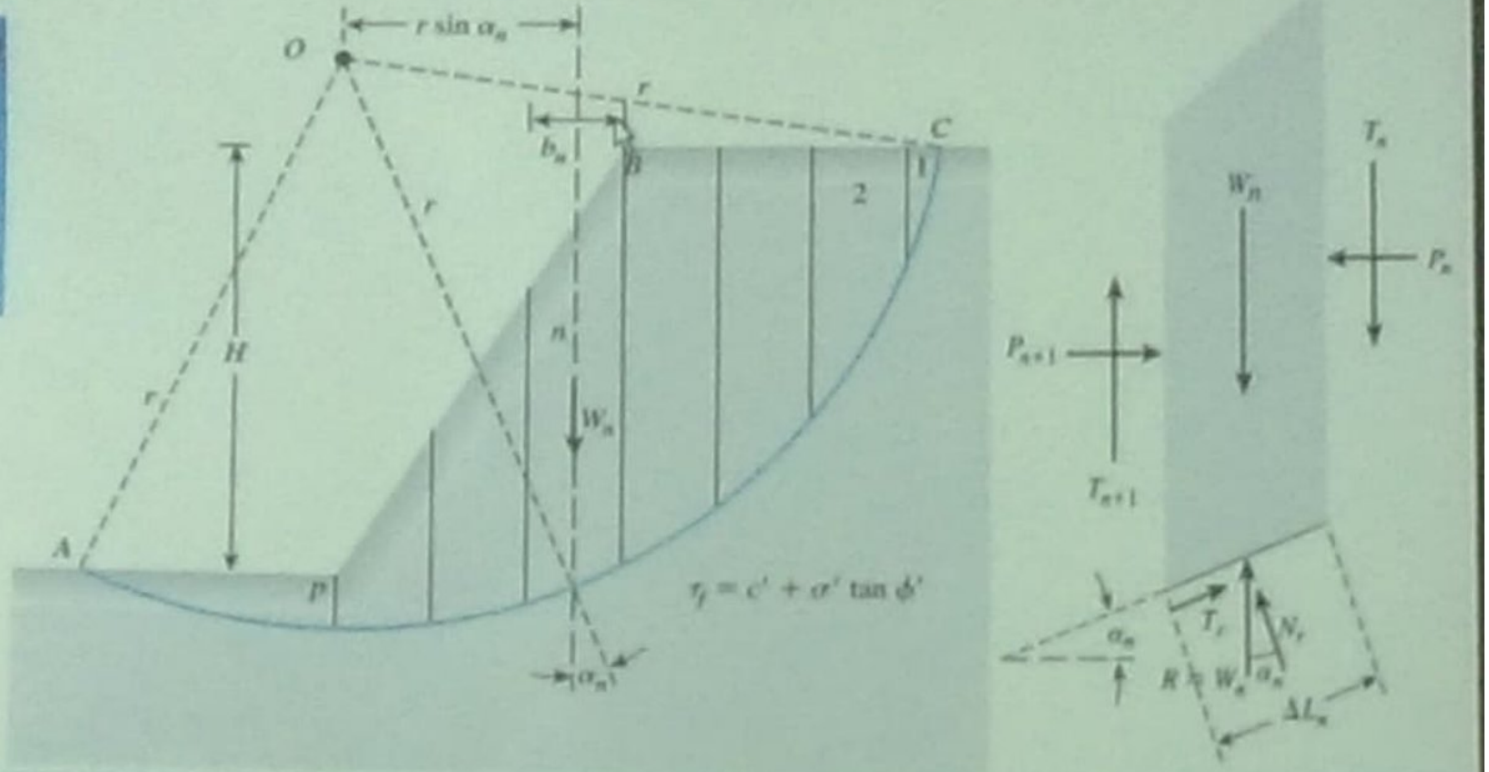
- ✓ For equilibrium consideration, $N_r = W_n \cos \alpha_n$
- ✓ Resisting shear force can be expressed as

$$T_r = \tau_d(\Delta L_n) = \frac{\tau_f(\Delta L_n)}{F_s} = \frac{1}{F_s}[c' + \sigma' \tan \phi']\Delta L_n$$

- ✓ Normal stress, σ', is equal to

$$\frac{N_r}{\Delta L_n} = \frac{W_n \cos \alpha_n}{\Delta L_n}$$

Ordinary Method of Slices



For equilibrium of trial wedge *ABC*, moment of driving force about *O* equals moment of resisting force about *O*, or

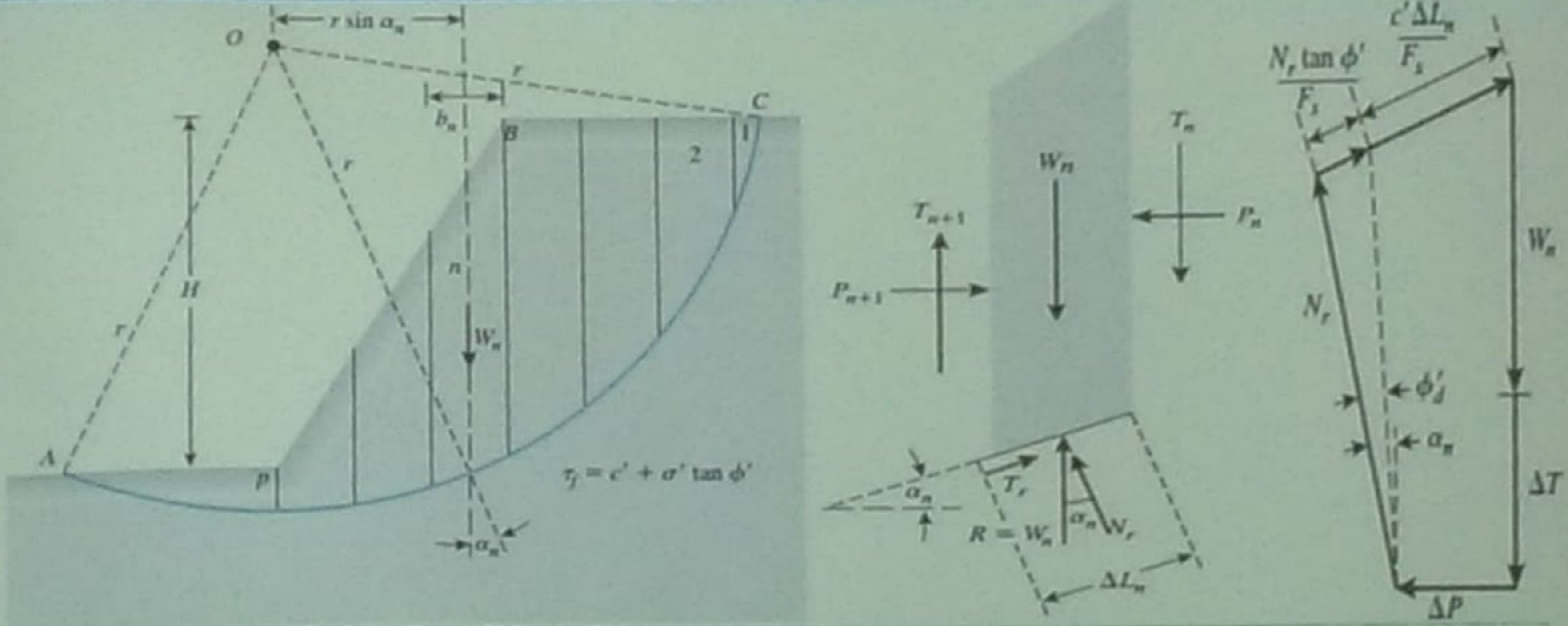
$$\sum_{n=1}^{n=p} W_n r \sin \alpha_n = \sum_{n=1}^{n=p} \frac{1}{F_s}\left(c' + \frac{W_n \cos \alpha_n}{\Delta L_n} \tan \phi'\right)(\Delta L_n)(r)$$

or

$$F_s = \frac{\sum_{n=1}^{n=p} (c' \Delta L_n + W_n \cos \alpha_n \tan \phi')}{\sum_{n=1}^{n=p} W_n \sin \alpha_n}$$

ΔL_n is approx equal to $(b_n)/(c$ where b_n = width slice

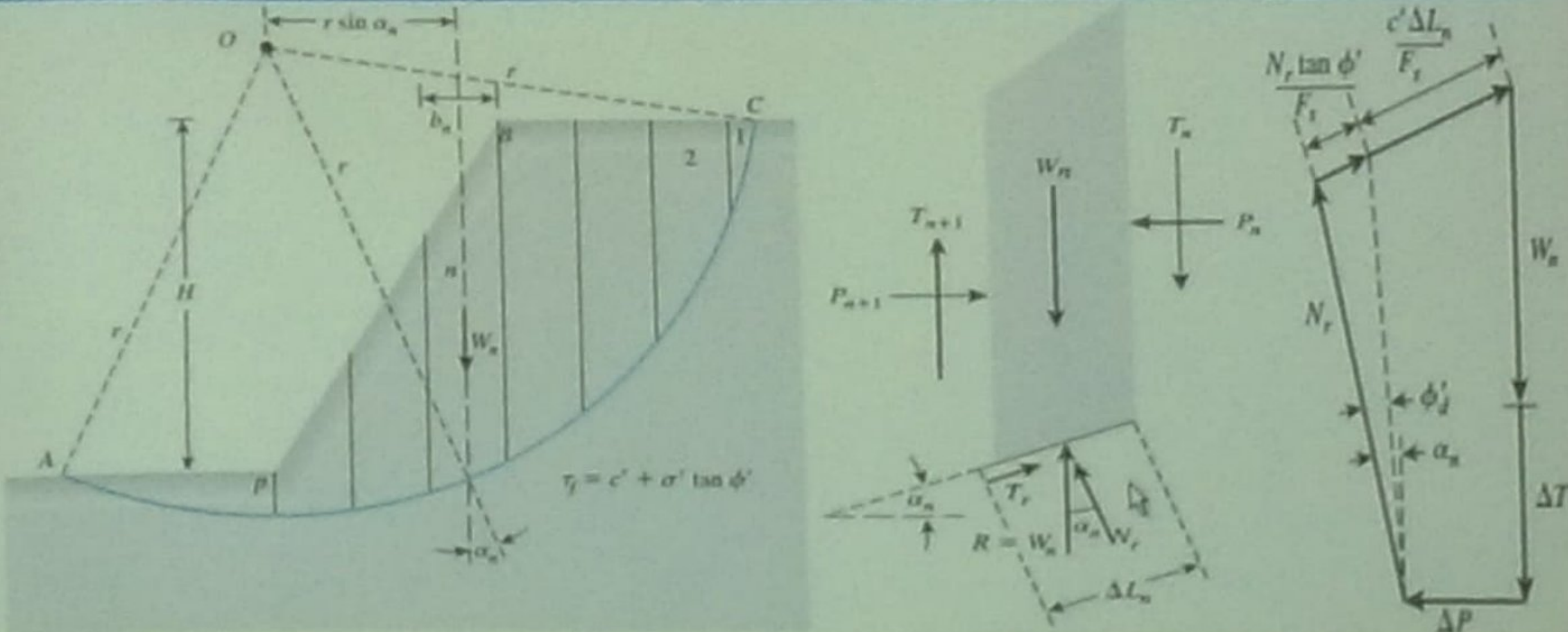
Bishop's Simplified Method of Slices



- ✓ In 1955, Bishop proposed a more refined solution to ordinary method of slices
- ✓ In this method, effect of forces on sides of each slice are accounted for to some degree
- ✓ Forces that act on nth slice have been redrawn in Figure
- ✓ Now, let $P_n - P_{n+1} = \Delta P$ and $T_n - T_{n+1} = \Delta T$
- ✓ Also, we can write

$$T_r = N_r(\tan \phi'_d) + c'_d \Delta L_n = N_r\left(\frac{\tan \phi'}{F_s}\right) + \frac{c' \Delta L_n}{F_s}$$

Bishop's Simplified Method of Slices



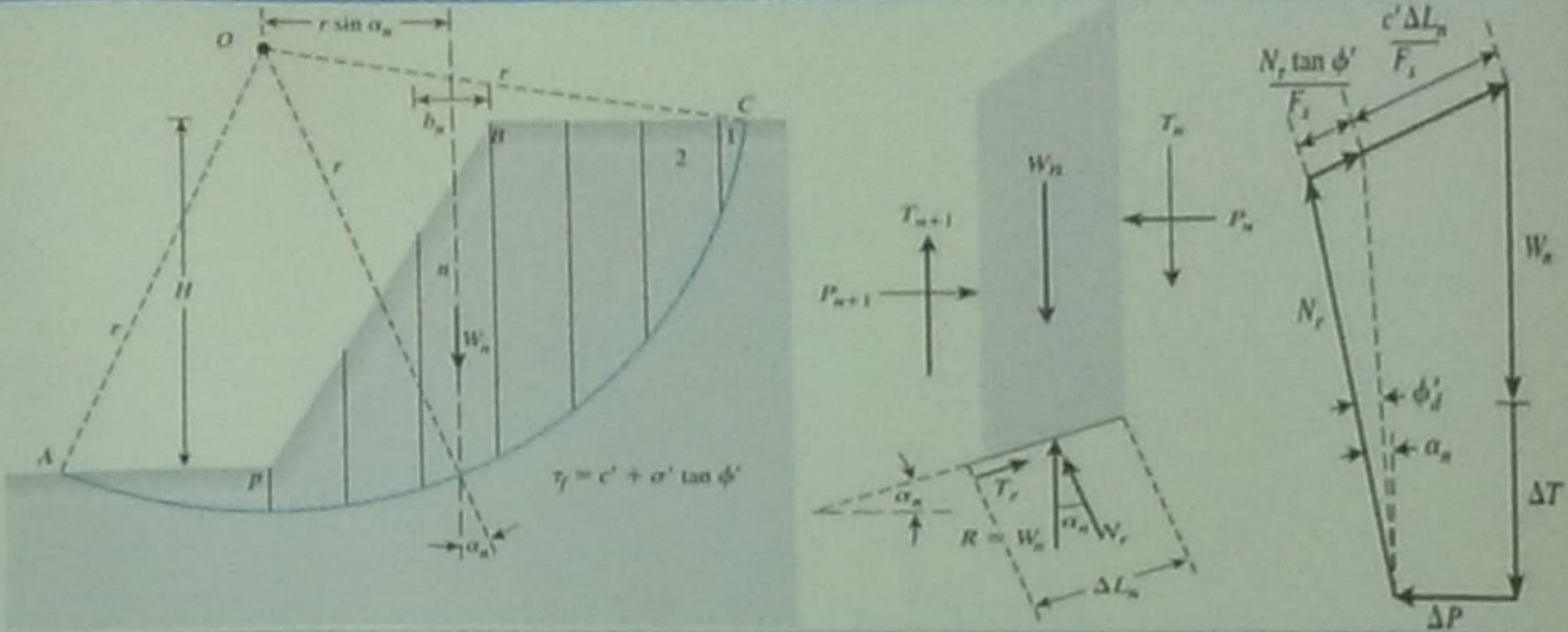
- ✓ Figure (b) shows force polygon for equilibrium of nth slice
- ✓ Summing forces in vertical direction gives

$$W_n + \Delta T = N_r \cos\alpha_n + \left[\frac{N_r \tan\phi'}{F_s} + \frac{c'\Delta L_n}{F_s}\right]\sin\alpha_n$$

or

$$N_r = \frac{W_n + \Delta T - \dfrac{c'\Delta L_n}{F_s}\sin\alpha_n}{\cos\alpha_n + \dfrac{\tan\phi'\sin\alpha_n}{F_s}}$$

Bishop's Simplified Method of Slices

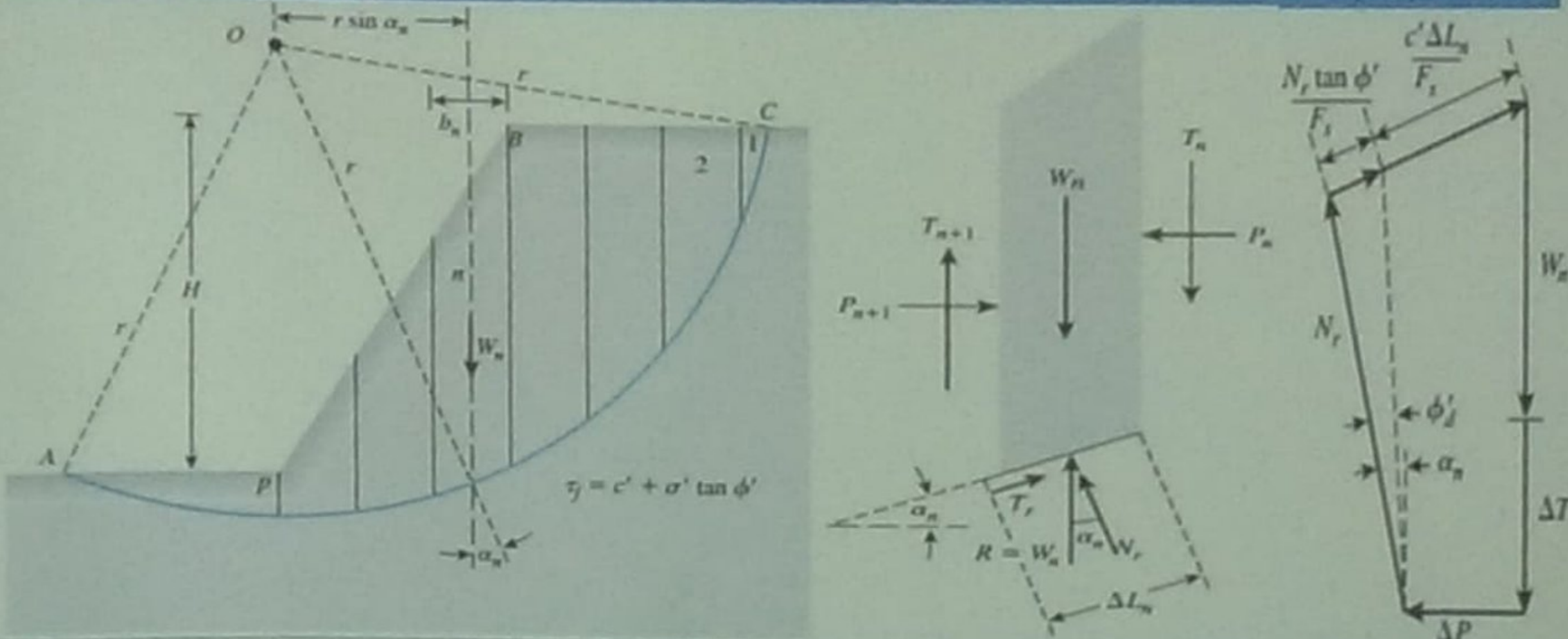


✓ For equilibrium of wedge ABC, taking moment about O gives

$$\sum_{n=1}^{n=p} W_n r \sin \alpha_n = \sum_{n=1}^{n=p} T_r r$$

where $$T_r = \frac{1}{F_s}(c' + \sigma' \tan \phi')\Delta L_n$$

Bishop's Simplified Method of Slices



✓ For equilibrium of wedge ABC, taking moment about O gives

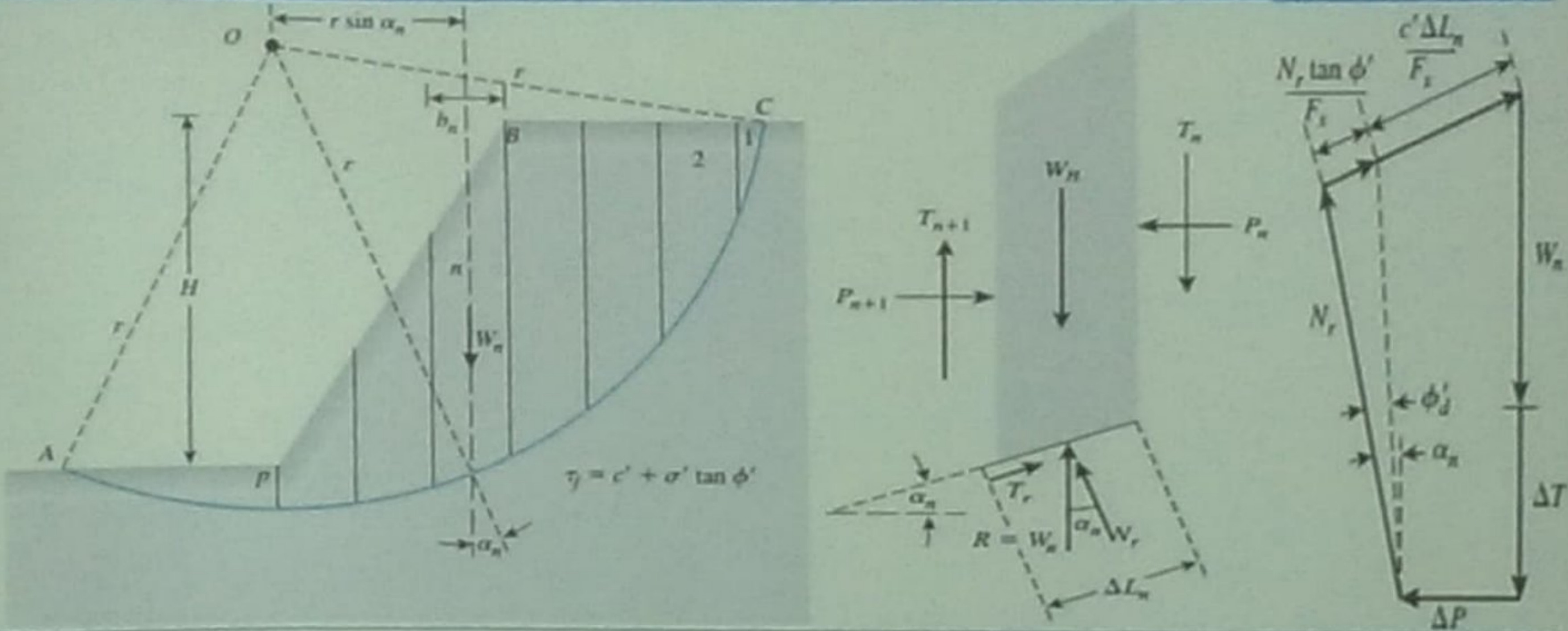
$$\sum_{n=1}^{n=p} W_n r \sin \alpha_n = \sum_{n=1}^{n=p} T_r r$$

where

$$T_r = \frac{1}{F_s}(c' + \sigma' \tan \phi')\Delta L_n$$

$$= \frac{1}{F_s}(c'\Delta L_n + N_r \tan \phi')$$

Bishop's Simplified Method of Slices

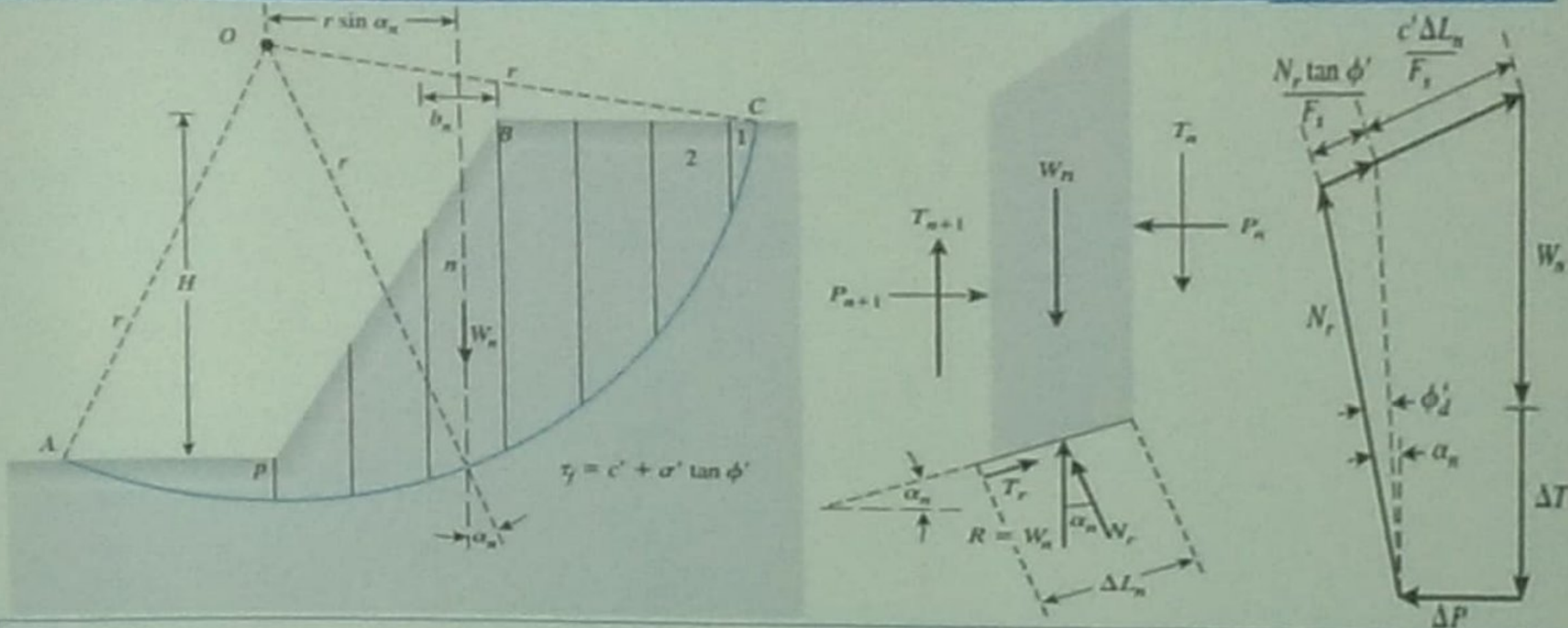


✓ Substitution of T_r and N_r into previous equation gives

$$F_s = \frac{\displaystyle\sum_{n=1}^{n=p} (c'b_n + W_n \tan\phi' + \Delta T \tan\phi') \frac{1}{m_{\alpha(n)}}}{\displaystyle\sum_{n=1}^{n=p} W_n \sin\alpha_n}$$

where $$m_{\alpha(n)} = \cos\alpha_n + \frac{\tan\phi' \sin\alpha_n}{F_s}$$

Bishop's Simplified Method of Slices

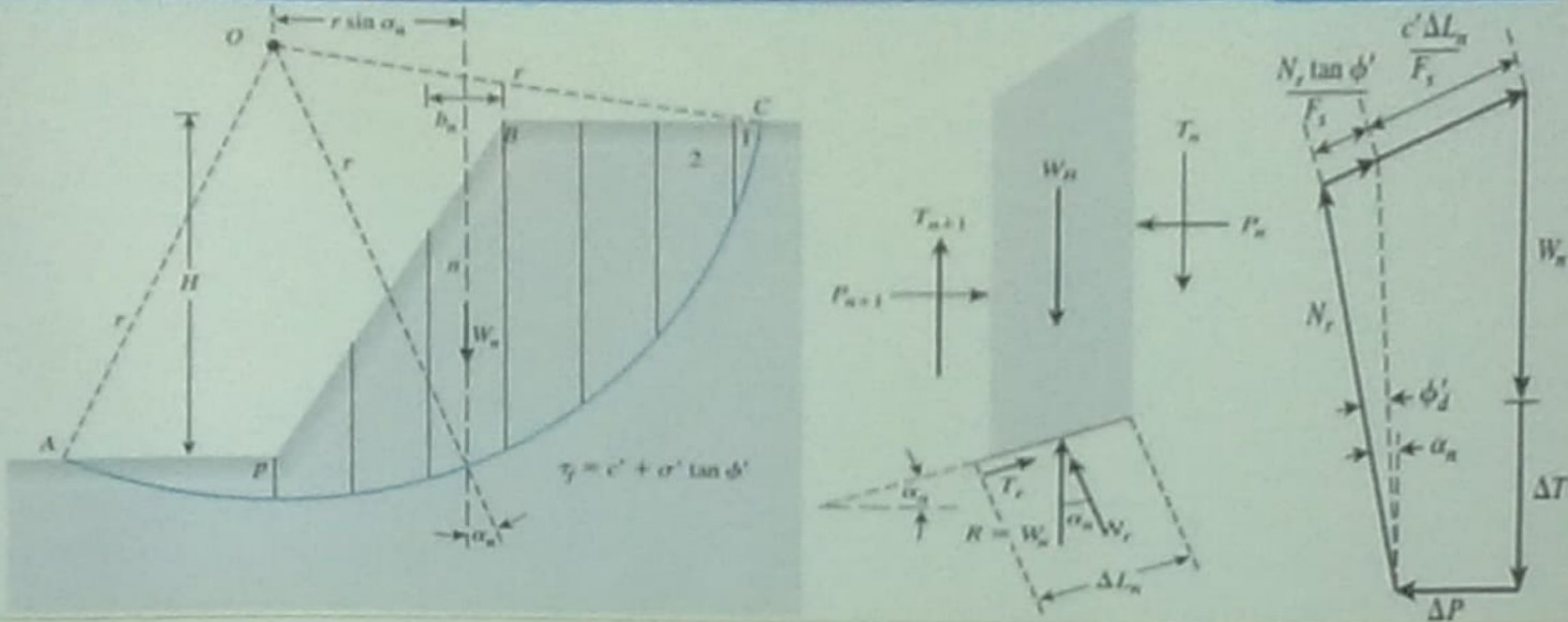


✓ Substitution of T_r and N_r into previous equation gives

$$F_s = \frac{\sum_{n=1}^{n=p} (c'b_n + W_n \tan\phi' + \Delta T \tan\phi')\frac{1}{m_{\alpha(n)}}}{\sum_{n=1}^{n=p} W_n \sin\alpha_n}$$

where $$m_{\alpha(n)} = \cos\alpha_n + \frac{\tan\phi' \sin\alpha_n}{F_s}$$

Bishop's Simplified Method of Slices



- For simplicity, if we let $\Delta T = 0$, preceding Equation becomes

$$F_s = \frac{\displaystyle\sum_{n=1}^{n=p} (c'b_n + W_n \tan\phi')\frac{1}{m_{\alpha(n)}}}{\displaystyle\sum_{n=1}^{n=p} W_n \sin\alpha_n}$$

- Bishop's simplified method is probably most widely used
- When incorporated into computer programs, it yields satisfactory results in most cases