FOUNDATION ENGINEERING

IV B.Tech II semester
Regulation: IARE R-16

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CO’s	Course outcomes
CO 1	Understand the need and various methods of soil exploration, planning and preparation of soil investigation report
CO 2	Analyze the stability of slopes by various methods
CO 3	Understand various earth pressure theories and stability of retaining walls at various conditions
CO 4	Understand shallow and deep foundations according to various bearing capacity theories and analyze Pile foundations in various different soils
CO 5	Understand various shapes and components of wells and analyze, design according to IRC guidelines

CLOs	Course Learning Outcome
CLO1	Understand the need and methods of Soil Exploration
CLO2	Understand various methods of sampling and boring
CLO3	Learn how to perform field tests such as SPT, DCPT, CPT ,PT
CLO4	Learn how to perform Plate Load test for finding load bearing capacity, settlements of soils

CLOs	Course Learning Outcome
CLO5	Learn how to perform in-situ test using pressure meter
CLO6	Understand the importance of geophysical methods
CLO7	Learn how to prepare Soil investigation Report

UNIT-I
SOIL EXPLORATION

SUB SOIL EXPLORATION

- The process of collection soil data for the assessment soil properties at a site through series of laboratory and field investigation is collectively called Sub-soil Exploration

- Enables the engineers to draw soil profile indicating the sequence of soil strata and the properties of soil involved.

COMMON STAGES IN SITE INVESTIGATION

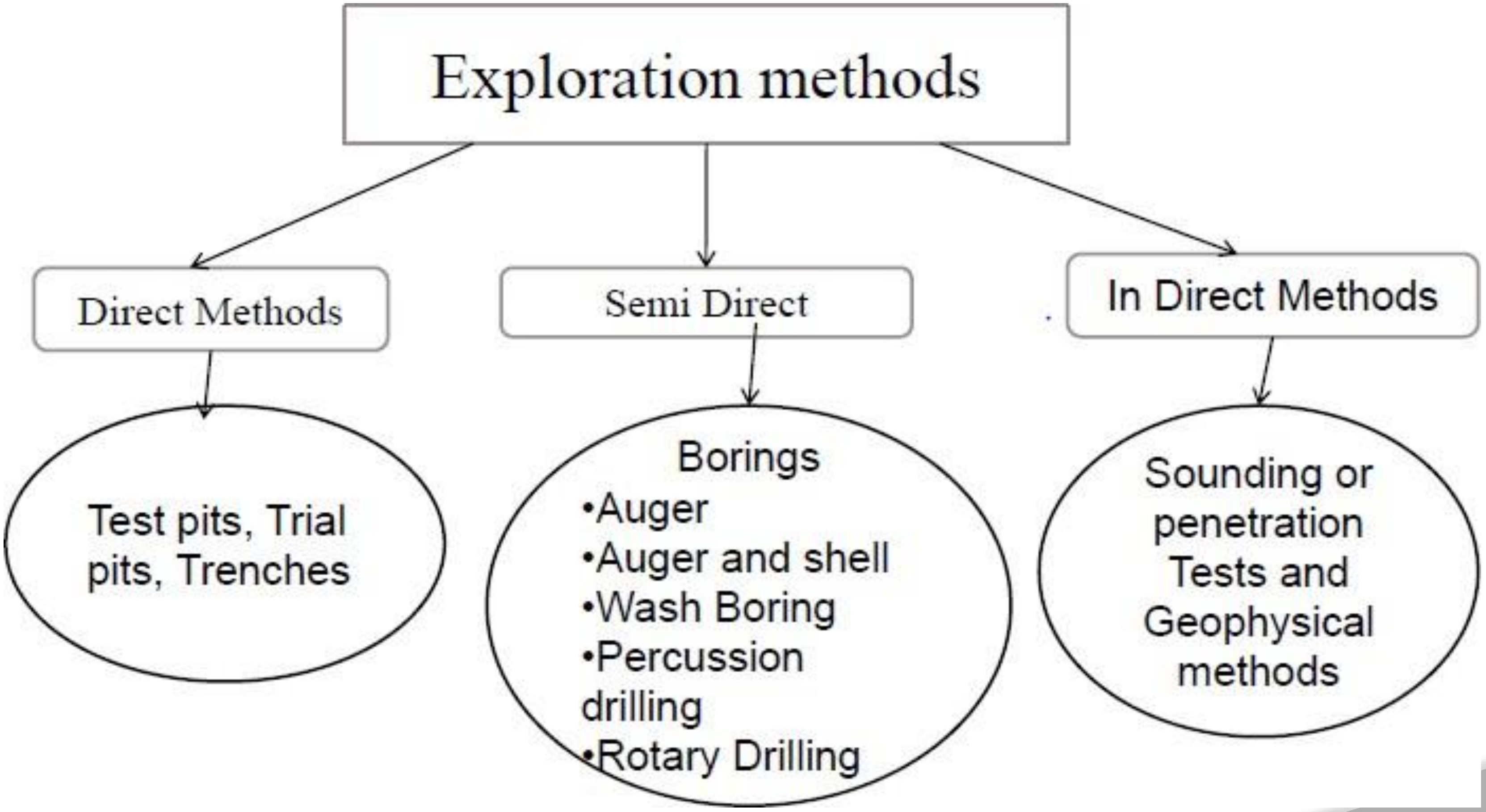
- Desk Study
- Site Reconnaissance
- Field Investigations
-   a) Preliminary Ground Investigation
-   b) Detailed Ground Investigation
- Laboratory Testing
- Report Writing
- Follow up Investigations during design & construction
- Appraisal of performance

PURPOSE OF SOIL INVESTIGATION

Site investigation provides first hand information for

- Selection of foundation type.
- Design of foundations.
- Contractors to quote realistic and competitive tenders.
- Planning construction techniques.
- Selection of appropriate construction equipment (especially for excavation and foundations).
- Feasibility studies of the site.
- Estimating development cost for the site.
- Study of environmental impacts of the proposed construction.

METHODS OF SOIL EXPLORATION



METHODS OF INVESTIGATION

The methods to determine the sequence, thickness and lateral extent of the soil strata and where appropriate the level of bedrock.

The common methods include

- Test pits
- Shafts and audits
- Boring or drilling

TEST PITS

The excavation of test pits is a simple and reliable method.

- The depth is limited to 4-5m only.
- The in-situ conditions are examined visually
- It is easy to obtain disturbed and undisturbed samples
- Block samples can be cut by hand tools and tube samples can be taken from the bottom of the pit.

BORING OR DRILLING

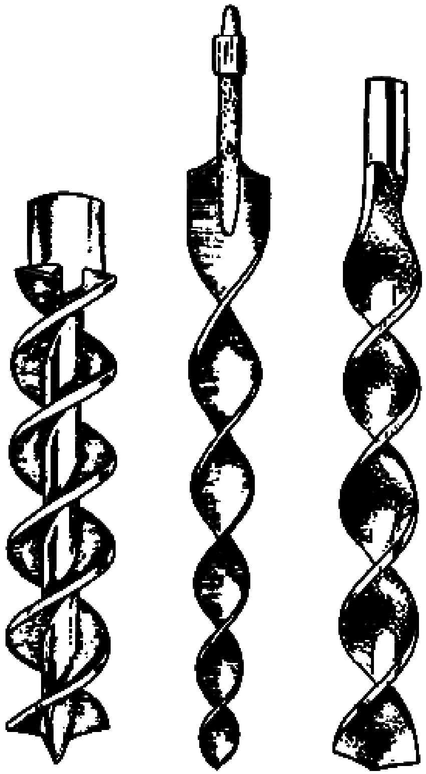
- Boring refers to advancing a hole in the ground.
- Boring is required for the following
- To obtain representative soil and rock samples for laboratory tests.
- Performance of in-situ tests to assess appropriate soil characteristics. Some of the common types of boring are as follows
- Auger boring
- Wash boring
- Rotary drilling
- Core drilling
- Percussion drilling

AUGER BORING

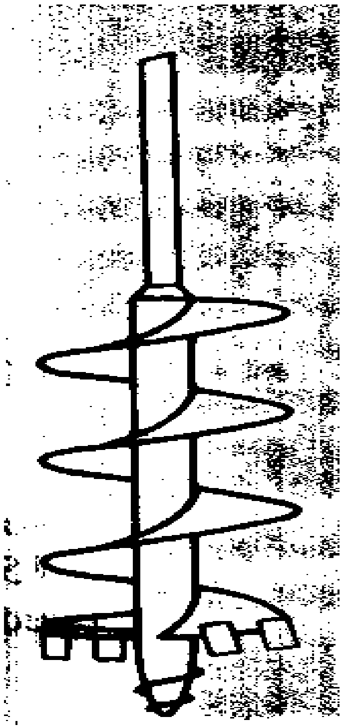
- Hand Auger
- Mechanical Auger
- It is the simplest method of boring used for small projects in soft cohesive soils.
- For hard soil and soil containing gravels boring with hand auger becomes difficult.
- Hand-auger holes can be made up to about 20m depth, although depth greater than about 8-10m is usually not practical.

HAND AUGER

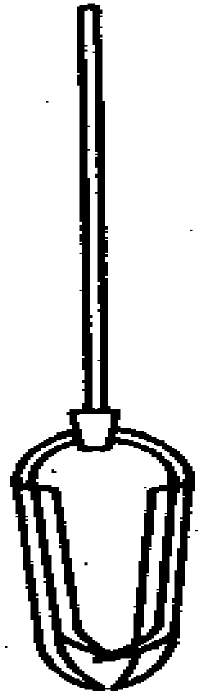
- The length of the auger blade varies from 0.3-0.5m.
- The auger is rotated until it is full of soil, then it is with drawn to remove the soil and the soil type present at various depths is noted.
- Repeated with drawl of auger for soil removal makes boring difficult below 8-10m depth.
- The soil samples collected in this manner are disturbed samples and can be used for classification test.
- Auger boring may not be possible in very soft clay or coarse and because the hole tends to collapse when auger is removed



(a) Helical (worm types) Augers



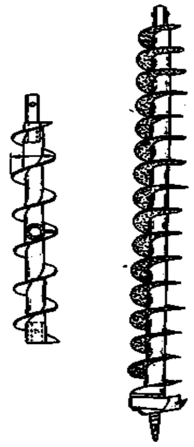
b) Short flight Auger



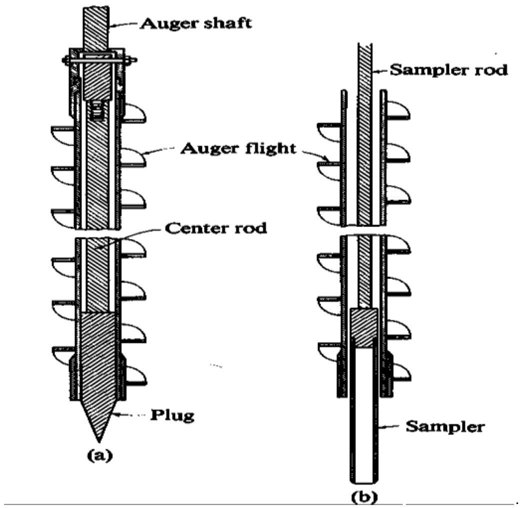
c) post hole Auger

MECHANICAL AUGER

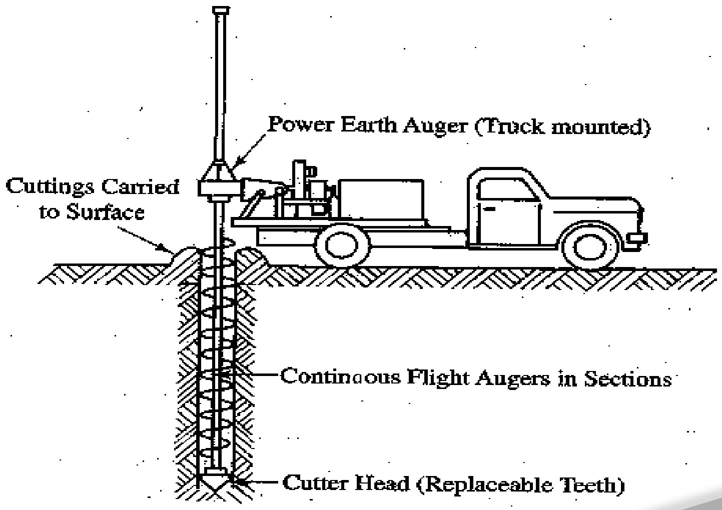
- Mechanical Auger means power operated augers. The power required to rotate the auger depends on the type and size of auger and the type of soil.
- Downwards pressure can be applied hydraulically, mechanically or by dead weight



a. Continuous Flight Auger



b. Hallow-stem auger plugged



c.Truck mounted auger boring machine

- The diameter of the flight auger usually is between 75 to 300mm,although diameters up to 1m and bucket augers up to2m are available.
- Borehole depths up to 50m are possible with continuous-flight augers.
- The most common method is to use continuous flight augers. Continuous flight augers can be solid stem or hollow stem with internal diameter of 75-150mm.

WASH BORING

- Water with high pressure pumped through hallow boring rods is released from narrow holes in a chisel attach to the lower end of the rods.
- The soil is loosened and broken by the water jet and the up-down moment of the chisel.
- The soil particles are carried in suspension to the surface between the rock and the borehole sites.
- The rods are raised and drop for chopping action of the chisel by means of winch.
- Wash boring can be used in most type of soil but the progress is slow in coarse gravel strata.

- The accurate identification of soil strata is difficult due to mixing of the material has they are carried to the surface.
- The method is unacceptable for obtaining soil samples.
- It is only used for advancing the borehole to enable tube sample to be taken or field test to be carried at the hole bottom.
- The advantage is that the soil immediately below the hole remains relatively un-disturbed

GEOPHYSICAL METHODS

Electromagnetic Wave Techniques

- Ground Penetrating Radar(GPR)
- Electromagnetic Conductivity (EM)
- Surface Resistivity (SR)
- Magnetometer Surveys (MT)

Mechanical Wave Measurements

- Crosshole Tests (CHT)
- Downhole Tests (DHT)
- Spectral Analysis of Surface Waves
- Seismic Refraction

How Many Borings & How Deep?

- NO hard-and-fast rule exists for determining the number of borings or the depth to which borings are to be advanced.

HOW MANY BORINGS?

Conventional Wisdom

The number (density) of borings will increase

- As soil variability increases
- As the loads increase
- For more critical/significant structures

Rules of Thumb

- Soft Soils - Space 30 to 60 m
- As soils become harder, spacing may be increased up to 150m

HOW MANY BORINGS?

Structure or Project	Subsurface Variability	Spacing of Borings (m)
Highway Subgrade	Irregular	50
	Average	150
	Uniform	300
Multistory Building	Irregular	15
	Average	30
	Uniform	45

Source: Sowers 1979

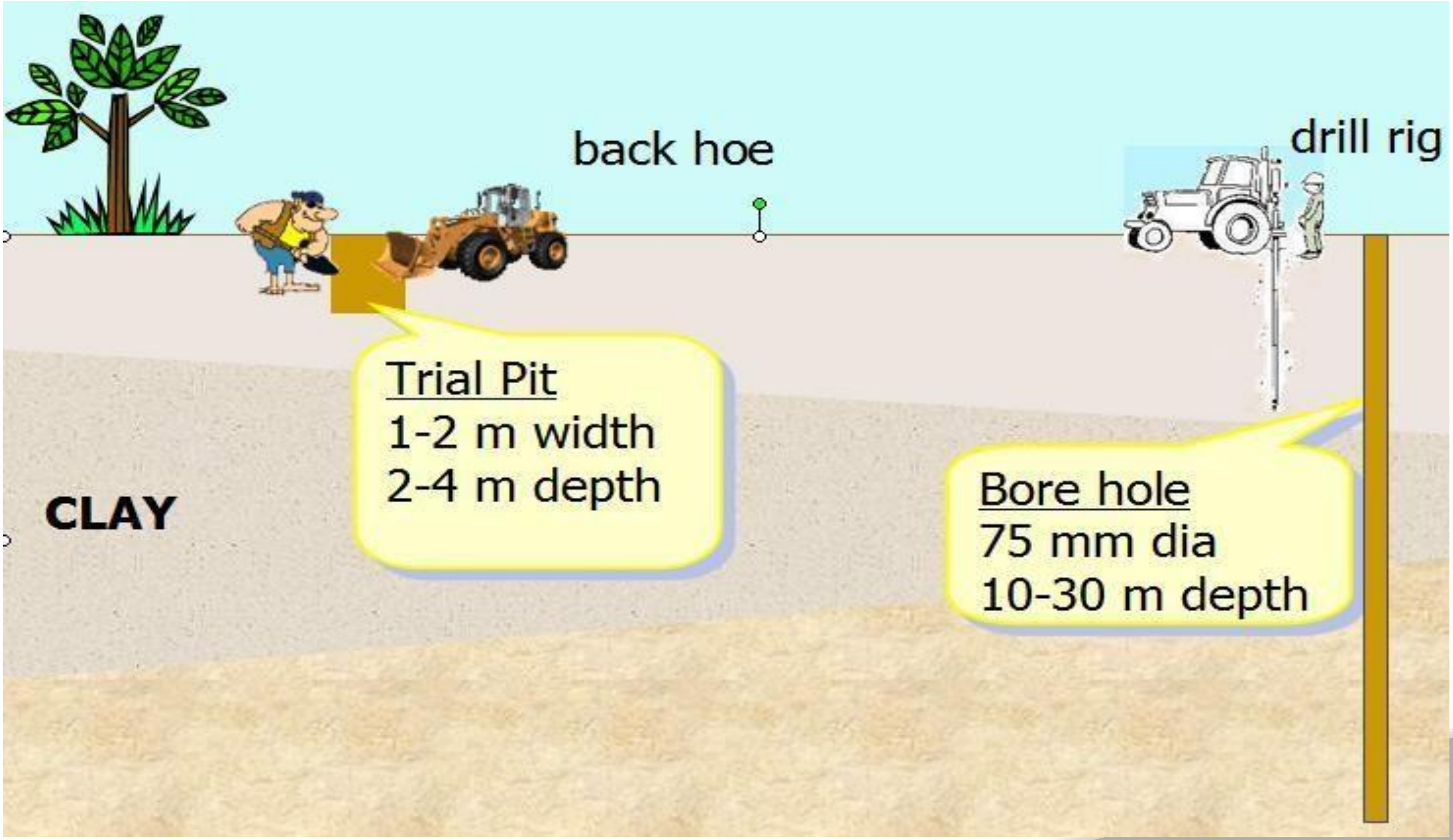
HOW MANY BORINGS?

Subsurface Conditions	Structure Footprint Area for Each Exploratory Boring (m^2)	(ft^2)
Poor quality and/or erratic	100–300	1,000–3,000
Average	200–400	2,000–4,000
High quality and uniform	300–1,000	3,000–10,000

HOW DEEP?

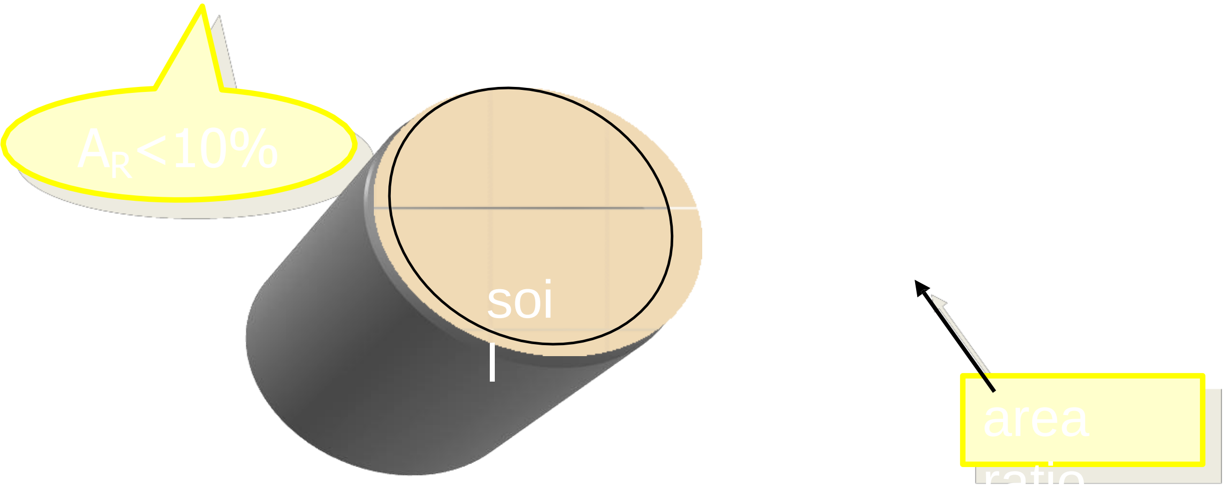
Subsurface Conditions	Minimum Depth of Borings (S = number of stories; D = anticipated depth of foundation) (m)	(ft)
Poor	$6\,S^{0.7} + D$	$20\,S^{0.7} + D$
Average	$5\,S^{0.7} + D$	$15\,S^{0.7} + D$
Good	$3\,S^{0.7} + D$	$10\,S^{0.7} + D$

SOIL BORING


back hoe
drill rig
Trial Pit
1-2 m width
2-4 m depth
CLAY
Bore hole
75 mm dia
10-30 m depth

DISTURBED VS UNDISTURBED

Good quality samples necessary.

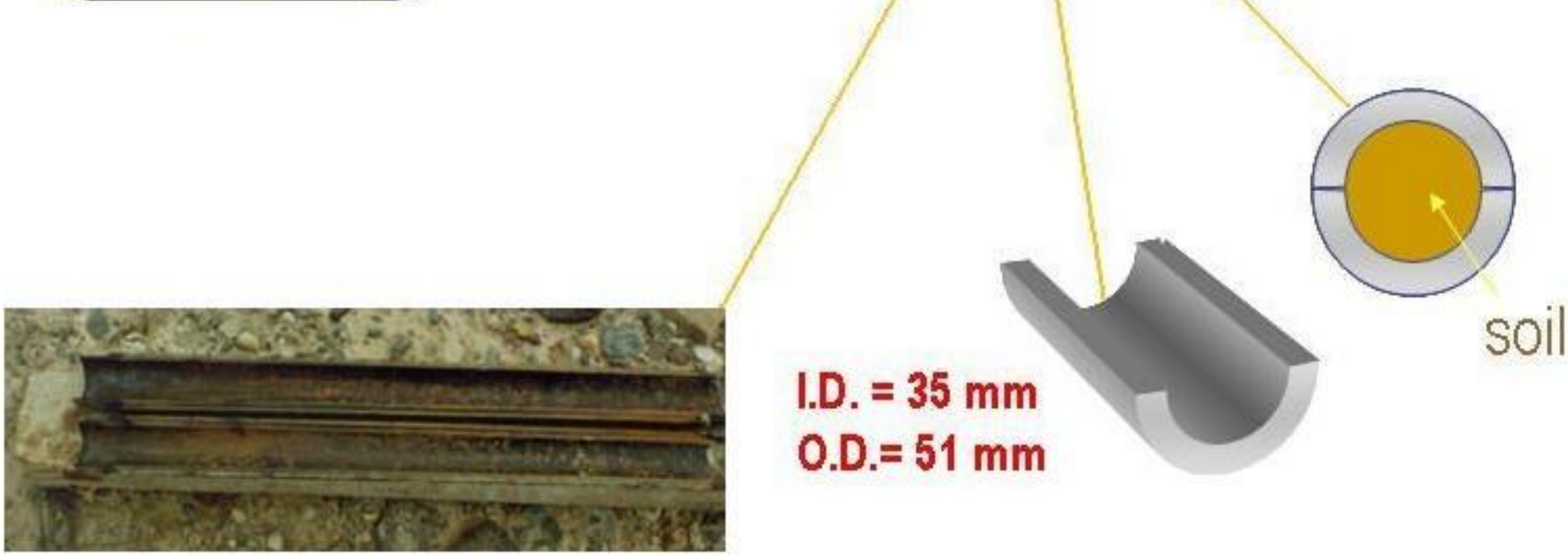


Thicker the wall, greater the disturbance.

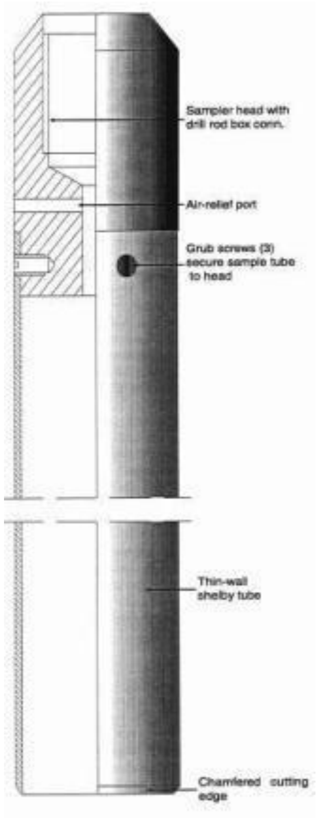
DISTURBED VS UNDISTURBED

- samples (disturbed) collected in <u>split-spoon sampler</u>

A_R = 112%; use for classification



SHELBY TUBE SAMPLER

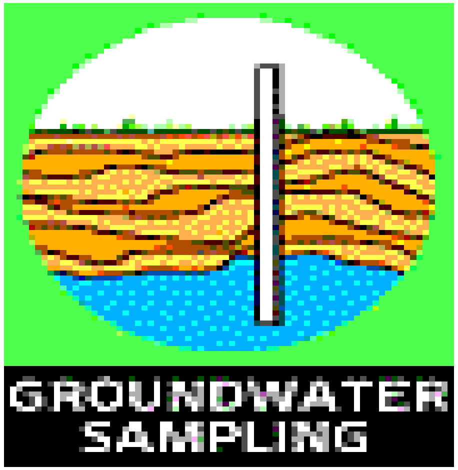


COMMON SAMPLING METHODS

Sampler	*Disturbed / Undisturbed*	*Appropriate Soil Types*	*Method of Penetration*	*% Use in Practice*
Split-Barrel (Split Spoon)	Disturbed	Sands, silts, clays	Hammer driven	85
Thin-Walled Shelby Tube	Undisturbed	Clays, silts, fine-grained soils, clayey sands	Mechanically Pushed	6
Continuous Push	Partially Undisturbed	Sands, silts, & clays	Hydraulic push with plastic lining	4
Piston	Undisturbed	Silts and clays	Hydraulic Push	1
Pitcher	Undisturbed	Stiff to hard clay, silt, sand, partially weather rock, and frozen or resin impregnated granular soil	Rotation and hydraulic pressure	<1
Denison	Undisturbed	Stiff to hard clay, silt, sand and partially weather rock	Rotation and hydraulic pressure	<1
Modified California	Disturbed	Sands, silts, clays, and gravels	Hammer driven (large split spoon)	<1
Continuous Auger	Disturbed	Cohesive soils	Drilling w/ Hollow Stem Augers	<1
Bulk	Disturbed	Gravels, Sands, Silts, Clays	Hand tools, bucket augering	<1
Block	Undisturbed	Cohesive soils and frozen or resin impregnated granular soil	Hand tools	<1

GROUND WATER

- Piezometers
- Monitor Wells & Sampling
- Permeability Tests



IN-SITU TESTING

- When it is difficult to obtain ―undisturbed‖ samples
- Cohesionless soils, Sensitive clays
- In-situ Test Methods

  Standard Penetration Test (SPT)

  Cone Penetration Test (CPT)

  Vane Shear Test (VST)

  Texas Cone Penetration Test (TCP)

STANDARD PENETRATION TEST (SPT)

65 kg Hammer

75 cm free fall

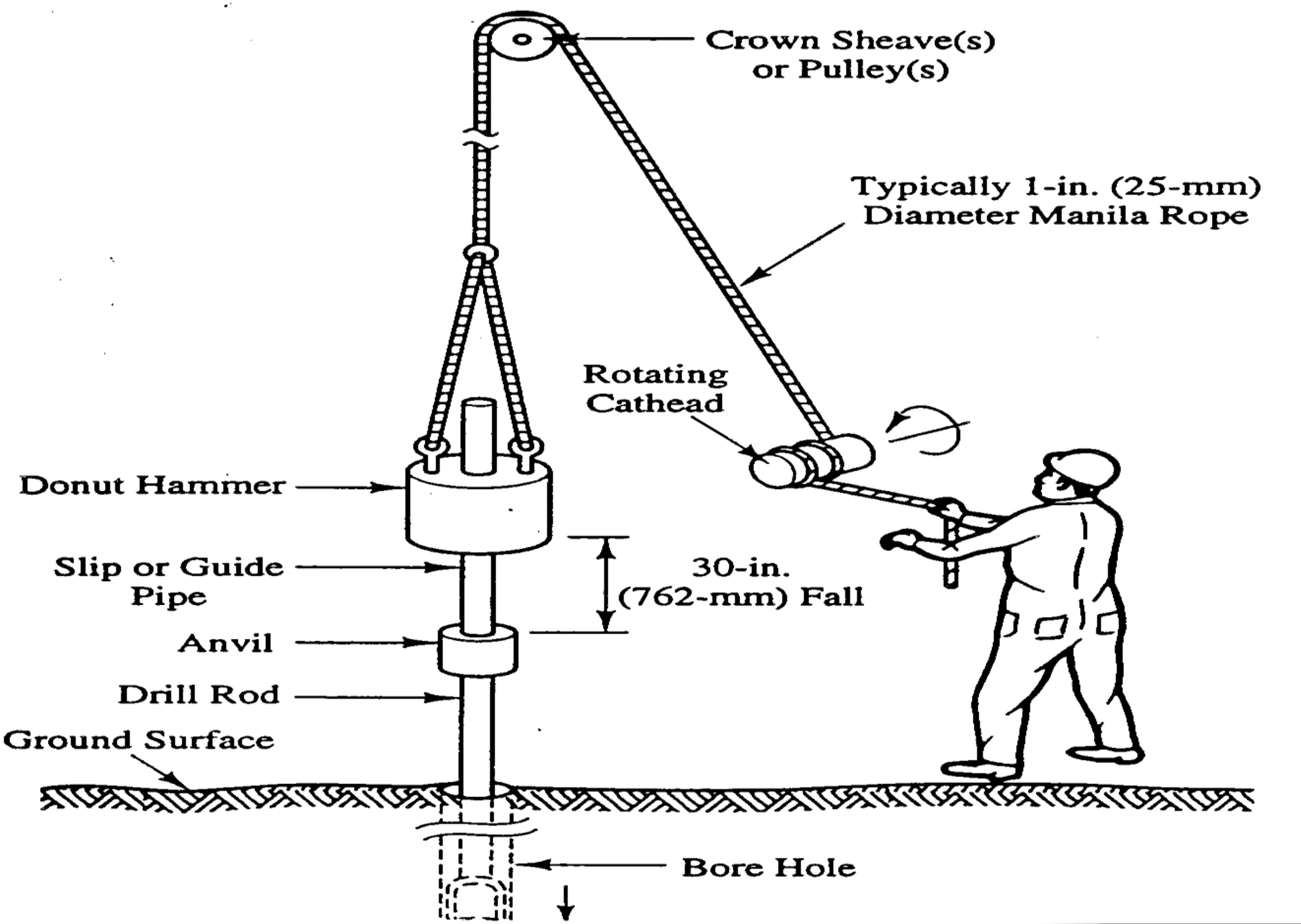
Drive sampler over 150 mm

Recordno.ofblows per

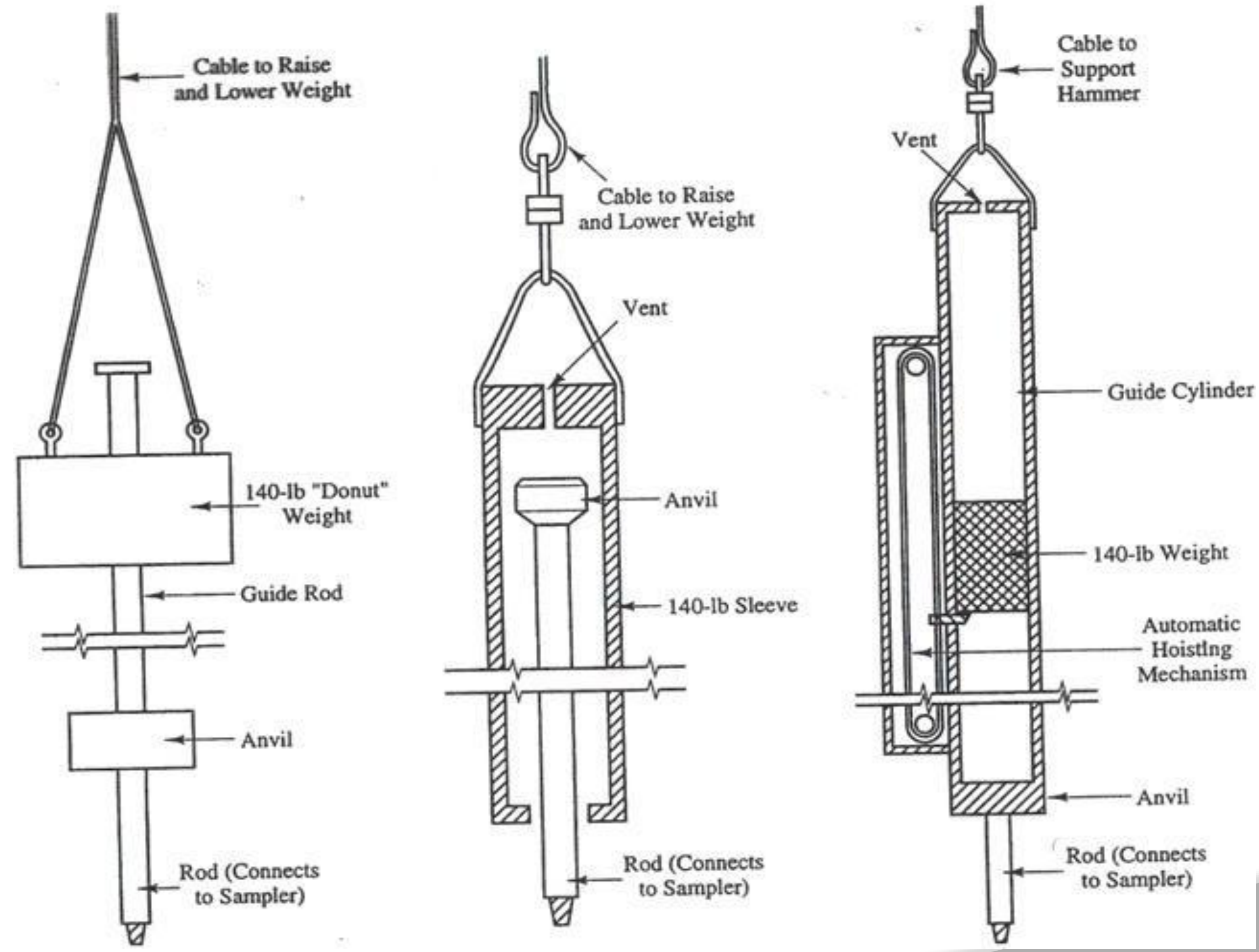
penetration

- SPT blow count=blows

STANDARD PENETRATION TEST (SPT)



TYPES OF SPT HAMMERS



CORRECTIONS TO SPT BLOW COUNTS

Factors affecting SPT blow count are

- Hammer Efficiency
- Borehole diameter
- Type of sampler
- Rod length

CORRECTION DUE TO DILATANCY

Terzaghi and Peck (1967) recommended that the observed N-values be corrected to N' if N > 15 as

$$N' = 15 + \frac{1}{2}(N - 15)$$

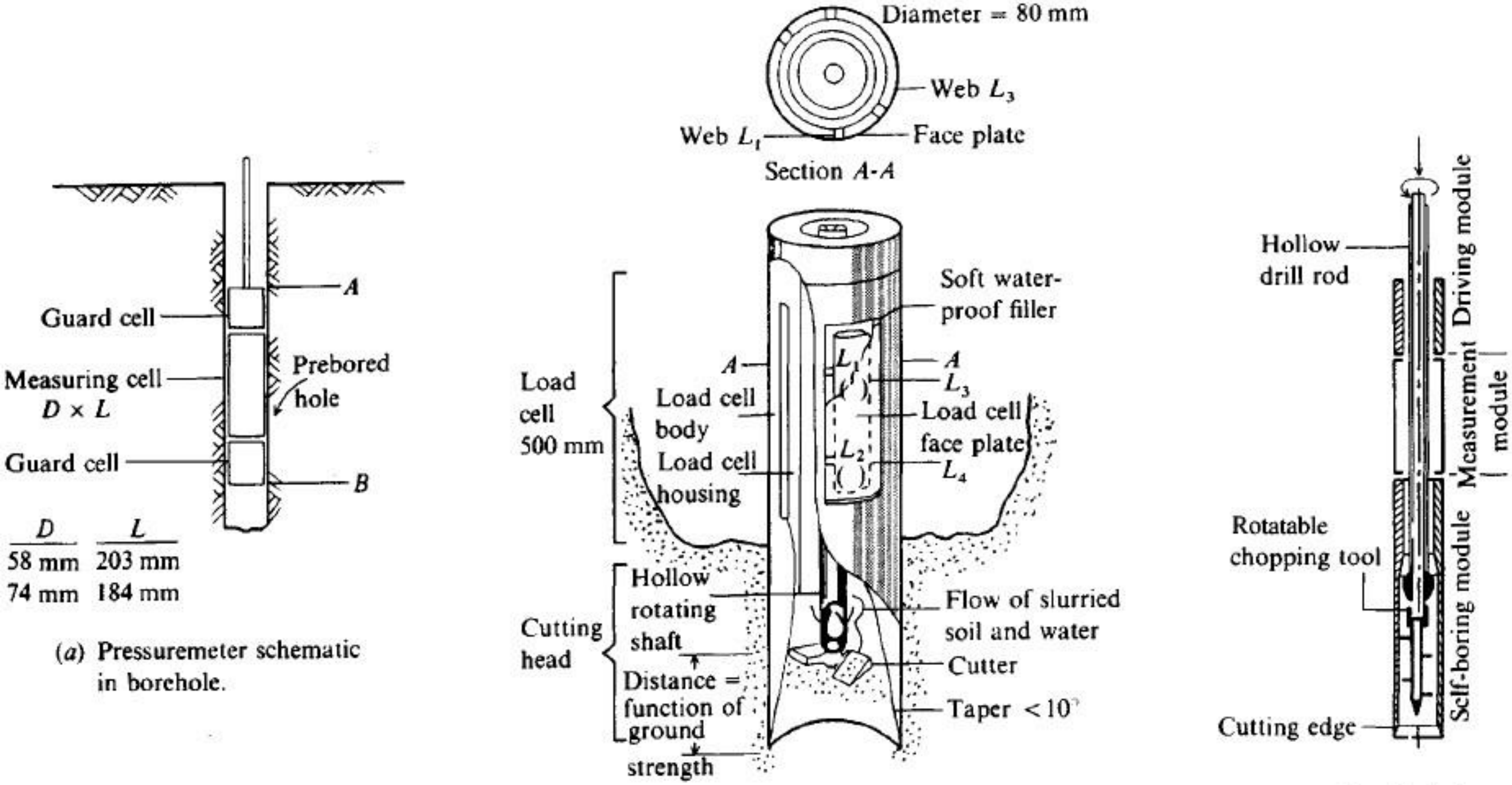
OVERBURDEN CORRECTION

Peck, Hanson and Thornburn (1974)

$$N' = 0.77 \log_{10}\left(\frac{2000}{\sigma_0'}\right) . \mathrm{N}$$

IS:2131-1981 recommend that the correction due to overburden shall be applied

PRESSUREMETER



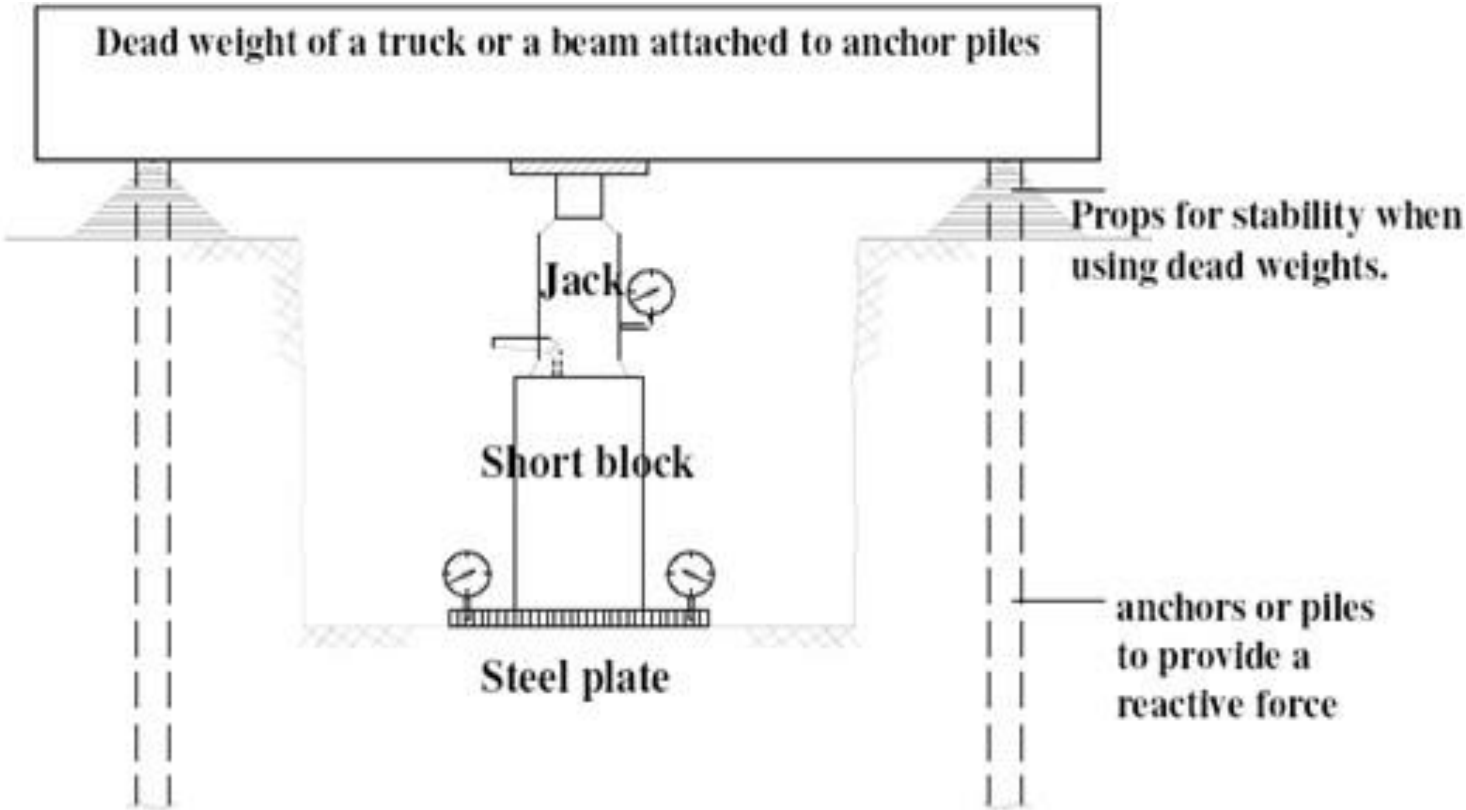
(a) Pressuremeter schematic in borehole.

(b) Camkometer.

(c) French device.

...sting
...panded to and
...d-condition effects on the
...ersus cell pressure relationship used in
data reduction.

THE PLATE LOAD TEST (PLT)



Several dial gauges attached to an independent suspension system to record plate settlements with each increment of the jack load.

REQUIRED INFORMATION

- Drilling & Sampling Depths & Methods
- Field Test Data
- Drilling Notes
- Soil appearance, stratification
- Acomplete record
- Pass/Fail

PREPARATION OF BORING LOGS

1. Name and address of the drilling company
2. Driller‘s name
3. Job description and number
4. Number, type, and location of boring
5. Date of boring
6. Subsurface stratification, which can he obtained by visual observation of the soil brought out by auger, split-spoon sampler, and thin-walled Shelby tube sampler
7. Elevation of water table and date observed, use of casing and mud losses, and so on
8. Standard penetration resistance and the depth of SPT
9. Number, type, and depth of soil sample collected
10. In case of rock coring, type of core barrel used and, for each

Boring Log

Name of the Project Two-story apartment building

Location Johnson & Olive St. Date of Boring March 2, 1982

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

Soil description	Depth (m)	Soil sample type and number	N	w_n (%)	Comments
Light brown clay (fill)					
Silty sand (SM)	1				
	2	SS-1	9	8.2	
*G.W.T. 3.5 m	3	SS-2	12	17.6	$LL = 38$ $PI = 11$
Light gray silty clay (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)	7				
End of boring @ 8 m	8	SS-4	27	9	

N = standard penetration number (below/304.8 mm)
w_n = natural moisture content
LL = liquid limit; PI = plasticity index
q_u = unconfined compression strength
SS = split-spoon sample; ST = Shelby tube sample

*Ground water table observed after one week of drilling

UNIT-II
SLOPE STABILITY

CLOs	Course Learning Outcome
CLO8	Understand basic concepts of earth slopes
CLO9	Analyse failure of infinite slopes
CLO10	Analyse types of failures for finite slopes
CLO11	Learn how to find Stability of slopes by Swedish arc Method

CLOs	Course Learning Outcome
CLO12	Learn how to find Stability of slopes by Method of Slices for slopes
CLO13	Find Stability of slopes by Taylor’s Stability number
CLO14	Understand basic concepts of Stability of slopes of earth dam under different conditions

THE AIMS OF SLOPE STABILITY ANALYSIS

- To understand the development and form of natural slopes and the processes responsible for different natural features.
- To assess the stability of slopes under short-term (often during construction) and long-term conditions.
- To assess the possibility of landslides involving natural or existing engineered slopes.
- To analyze landslides and to understand failure mechanisms and the influence of environmental factors.
- To enable the redesign of failed slopes and the planning and
  - design of preventive and remedial measures, where necessary.
- To study the effect of seismic loadings on slopes and embankments.

SLOPES OF EARTH ARE OF TWO TYPES

- **Natural slopes:** slopes exist in hilly areas
- **Man made slopes:**
  - The sides of cuttings
  - The slopes of embankments constructed for roads railway lines, canals etc.
  - The slopes of earth dams constructed for storing water.

THE SLOPES WHETHER NATURAL OR ARTIFICIAL MAY BE

Infinite slopes

The term infinite slope is used to designate a

- constant slope of infinite extent.
- The long slope of the face of a mountain

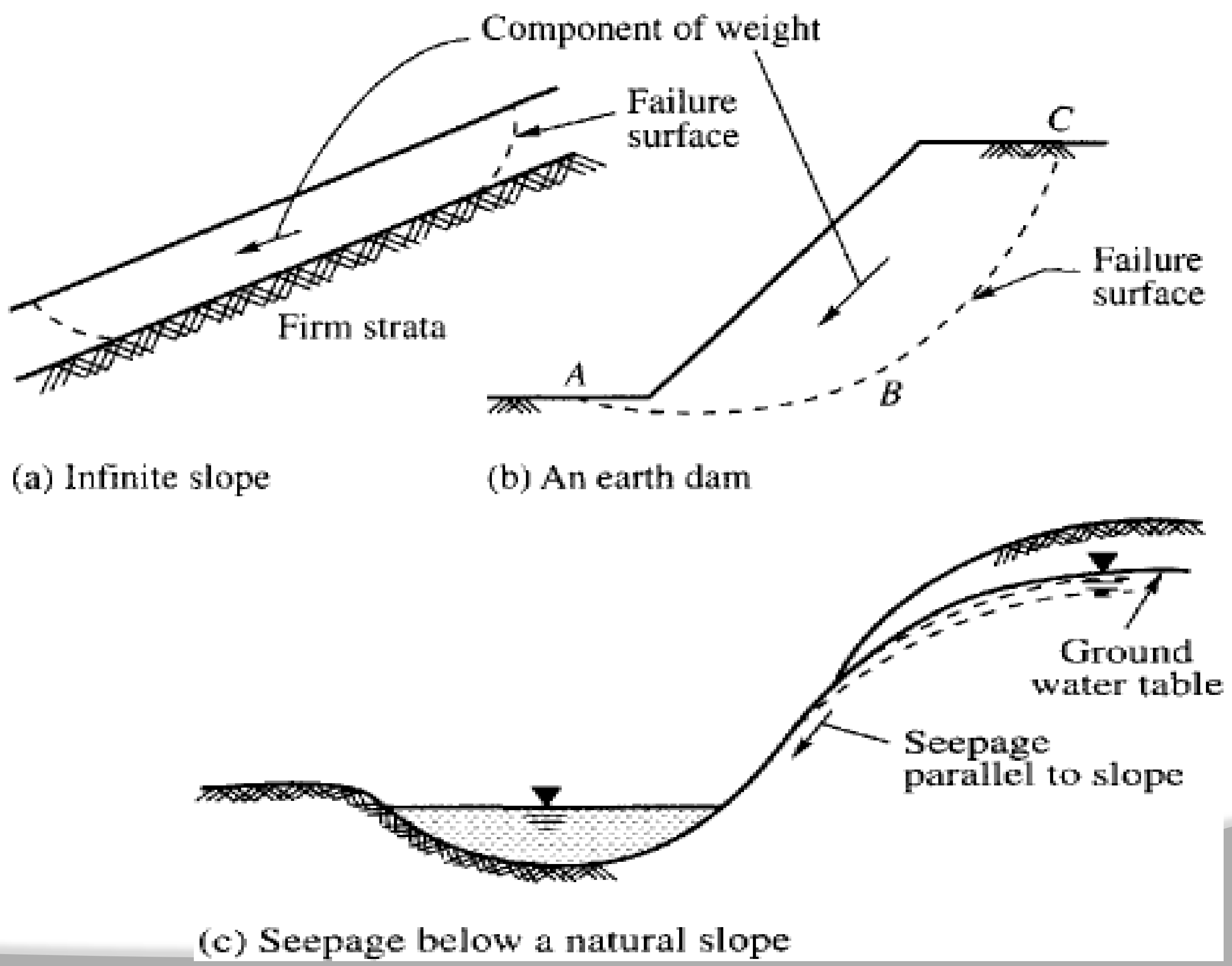
Finite slopes

- **Finite slopes are limited in extent.**
- The slopes of embankments and earth dams are examples of finite slopes

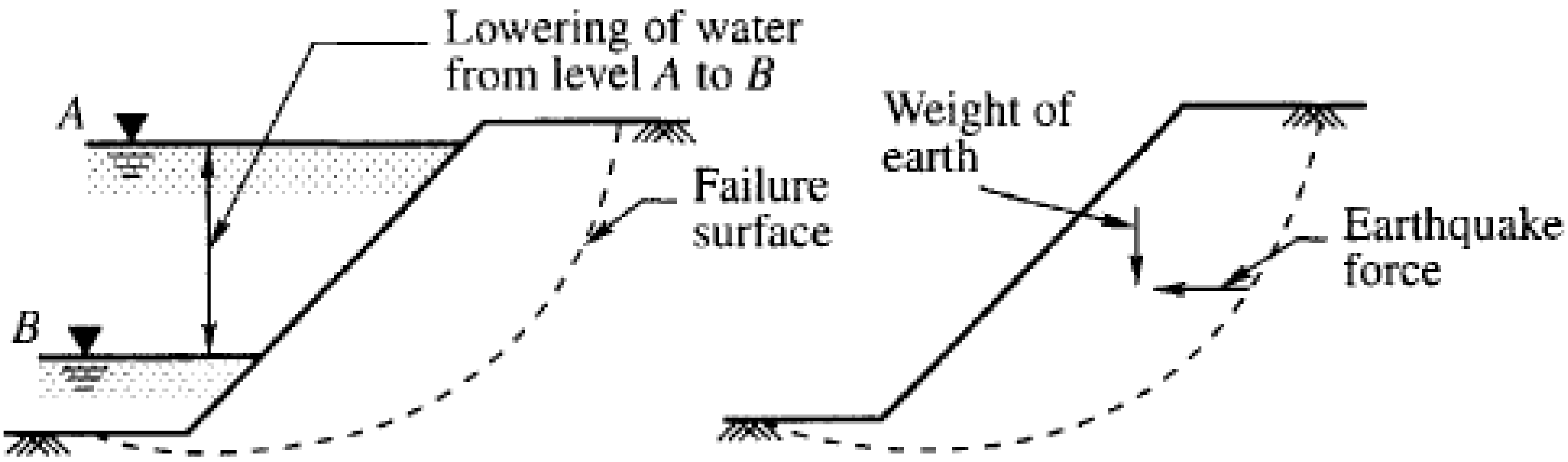
CAUSES OF FAILURE OF SLOPES

- Gravitational force
- Force due to seepage water
- Erosion of the surface of slopes due to flowing water
- The sudden lowering of water adjacent to a slope
- Forces due to earthquakes

FORCES THAT ACT ON EARTH SLOPES



FORCES THAT ACT ON EARTH SLOPES



(d) Sudden drawdown condition

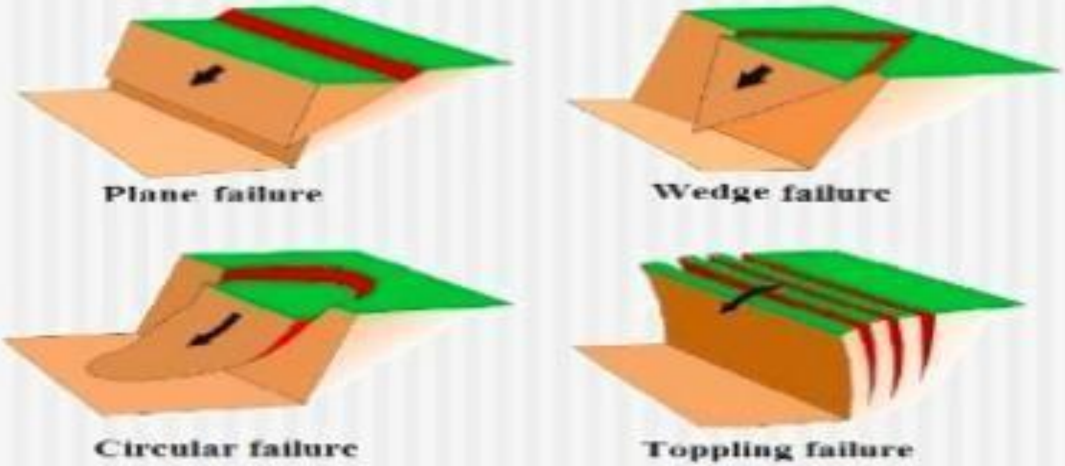
(e) Failure due to earthquake

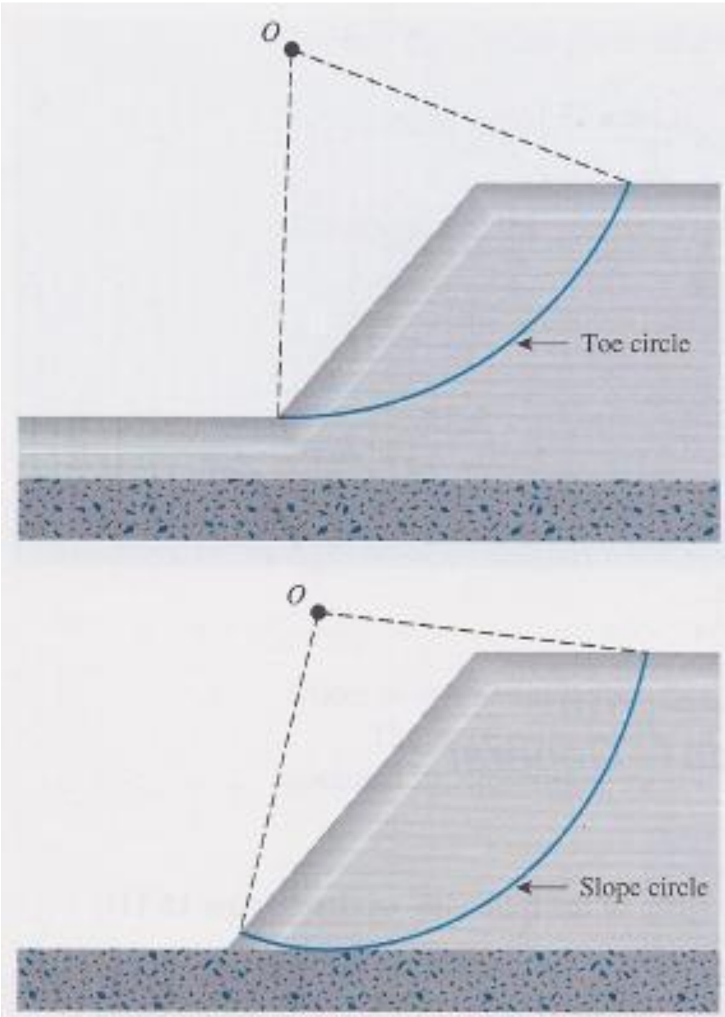
TYPES OF SLOPE FAILURES

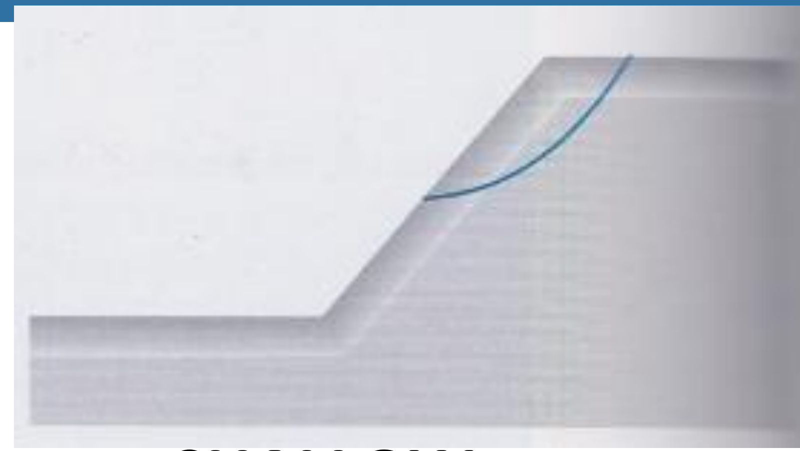
TYPES OF SLOPE FAILURES

- There are main four types of failures which are:

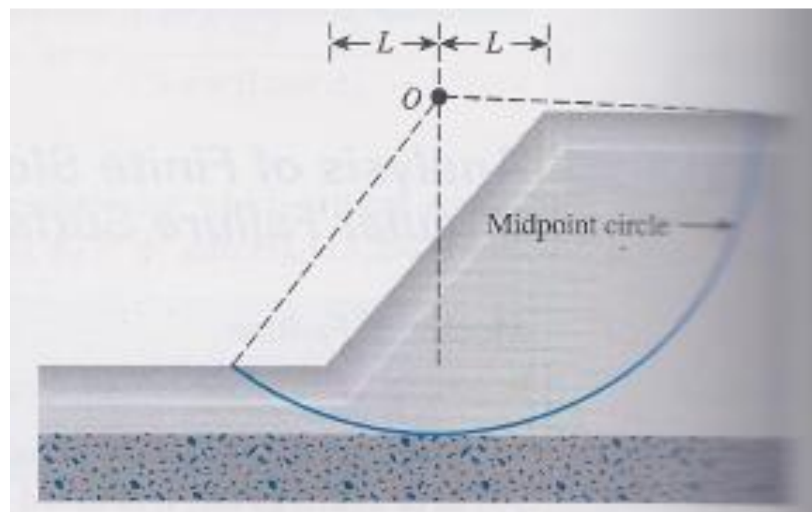
1. Plane failure
2. Wedge failure
3. Toppling failure
4. Rotational failure







SHALLOW
SLOPE FAILURE



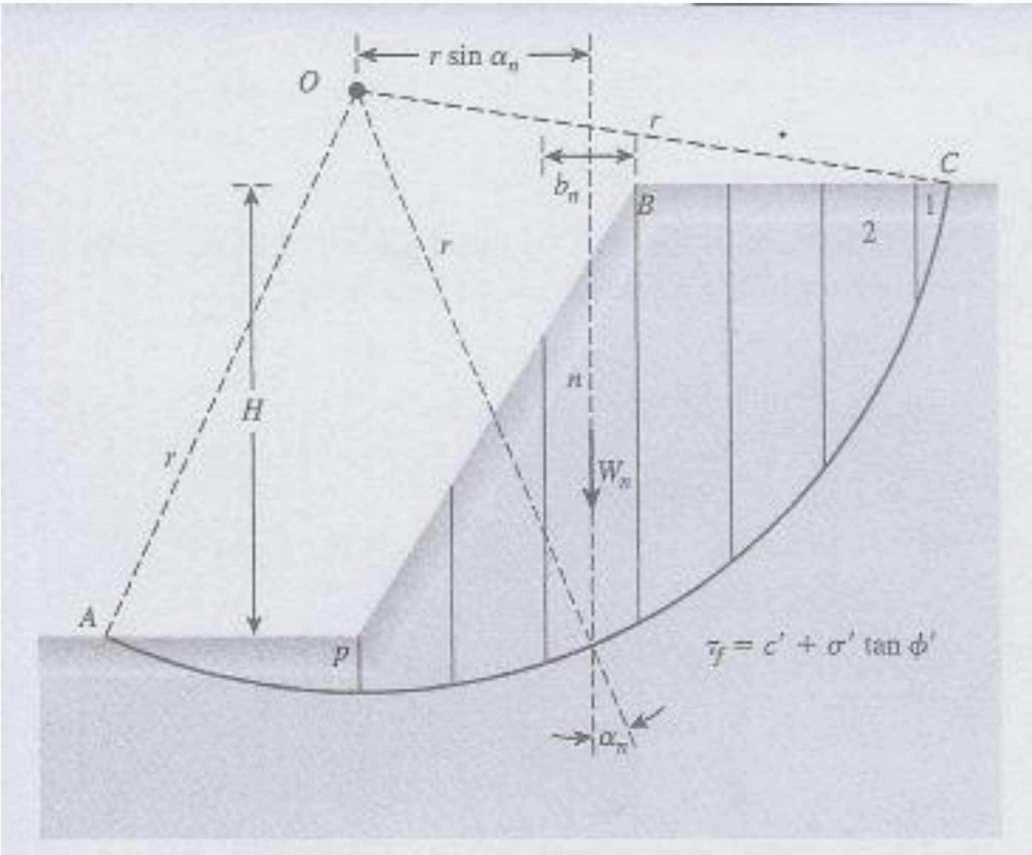
Toe circle / slope circle

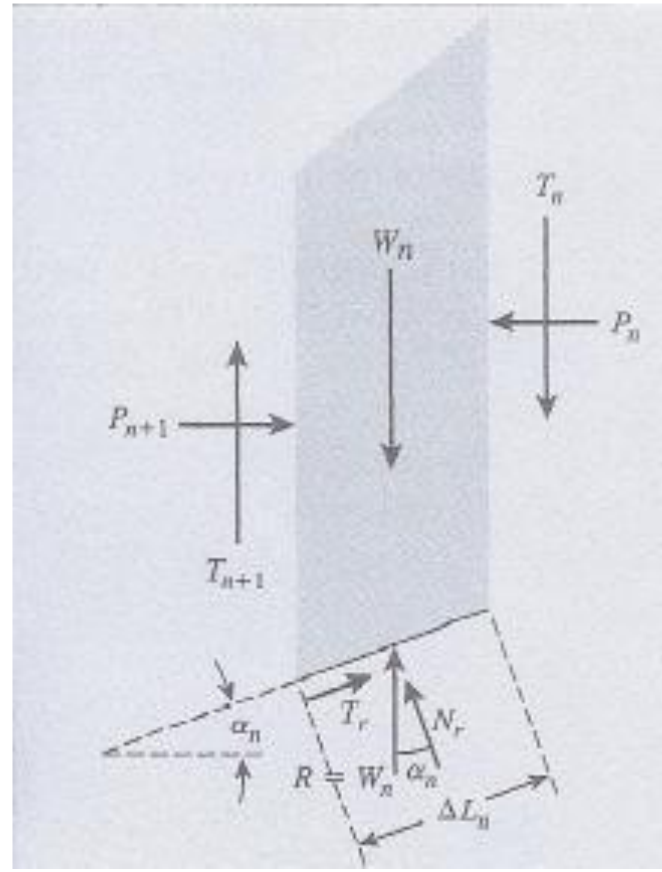
Base failure

Modes of failure of finite slope

ORDINARY METHOD OF SLICES

- The soil above the surface of sliding is **divided into a number of vertical parallel slices**. The stability of each slice is calculated separately. This is a versatile technique (no homogeneity, pwp).





Stability analysis by ordinary method of slices (Das, 2002)

Procedure

- Draw to scale a cross section of the slope.
- A trial curved surface along which sliding is assumed to take place is drawn.
- The trial surface is normally approximately circular.
- Soil between the trial surface and the slope is then divided into a number of vertical slices of equal width.

Weight of soil within each slice = Slice volume x Soil unit weight

- Noted as W.
- Can be resolved into two component ;

  i) component of W normal to the base, W_n and

  ii) component of W parallel to the base, W_p
- It is the parallel component that tends to cause sliding.
- Resisting to sliding is afforded by the soil's cohesion and internal friction.

Cohesion force = Soil cohesion x Length of the slice

Friction force = W_n x Friction coefficient, tan ϕ

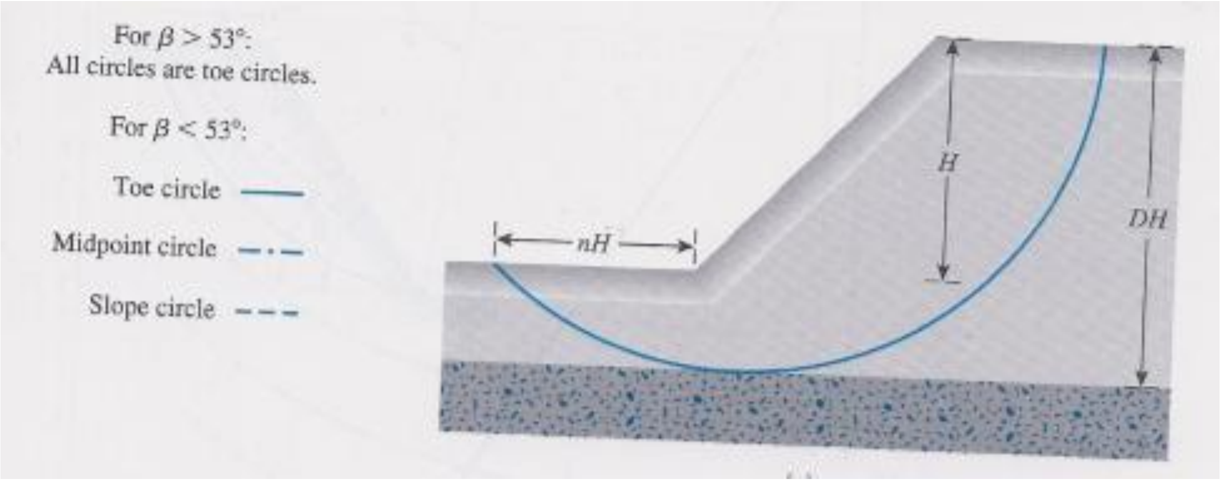
APPLY VALUE INTO FORMULA

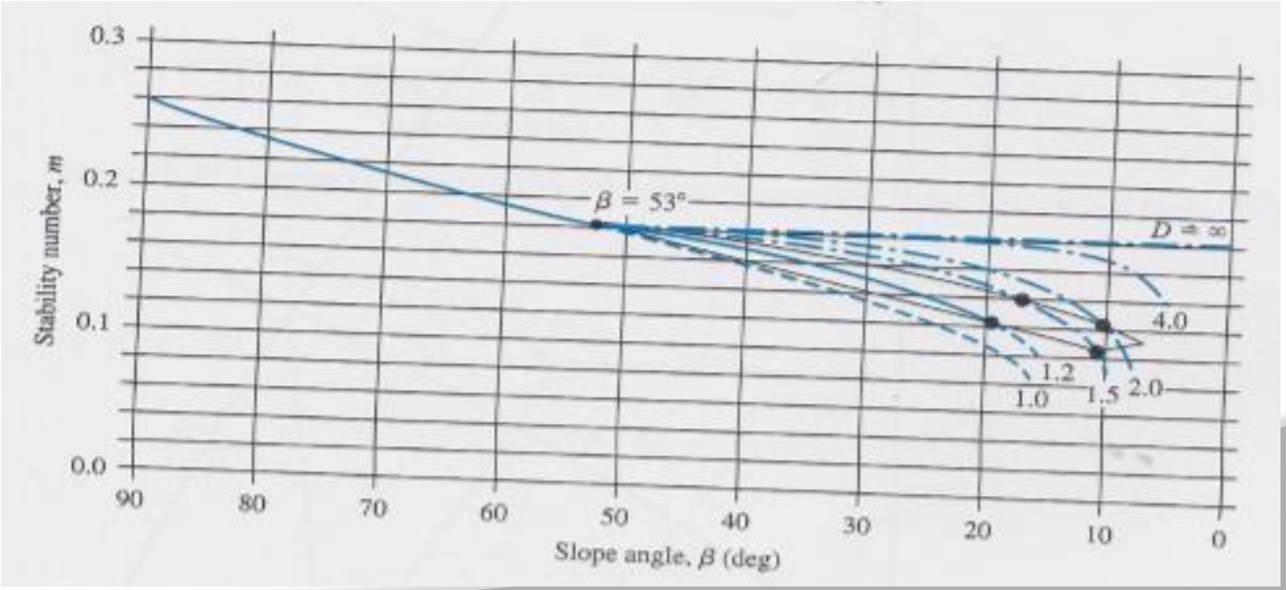
$$F = \frac{\sum_{n=1} \left(c\Delta L_n + W_n \cos\alpha_n \tan\phi \right)}{\sum_{n=1} W_n \sin\alpha_n}$$

- The corresponding circle that gives the lowest FOS value is called critical circle.

- Fellenius (1927(and Taylor (1937) suggested that (for critical circle case) :

$$\frac{\tau}{c} = \frac{c_u}{c}$$


For $\beta > 53°$:
All circles are toe circles.
For $\beta < 53°$:
Toe circle
Midpoint circle
Slope circle
nH
H
DH


0.3
0.2
0.1
0.0
Stability number, m
β = 53°
D = ∞
4.0
1.2
1.0
1.5
2.0
90
80
70
60
50
40
30
20
10
0
Slope angle, β (deg)

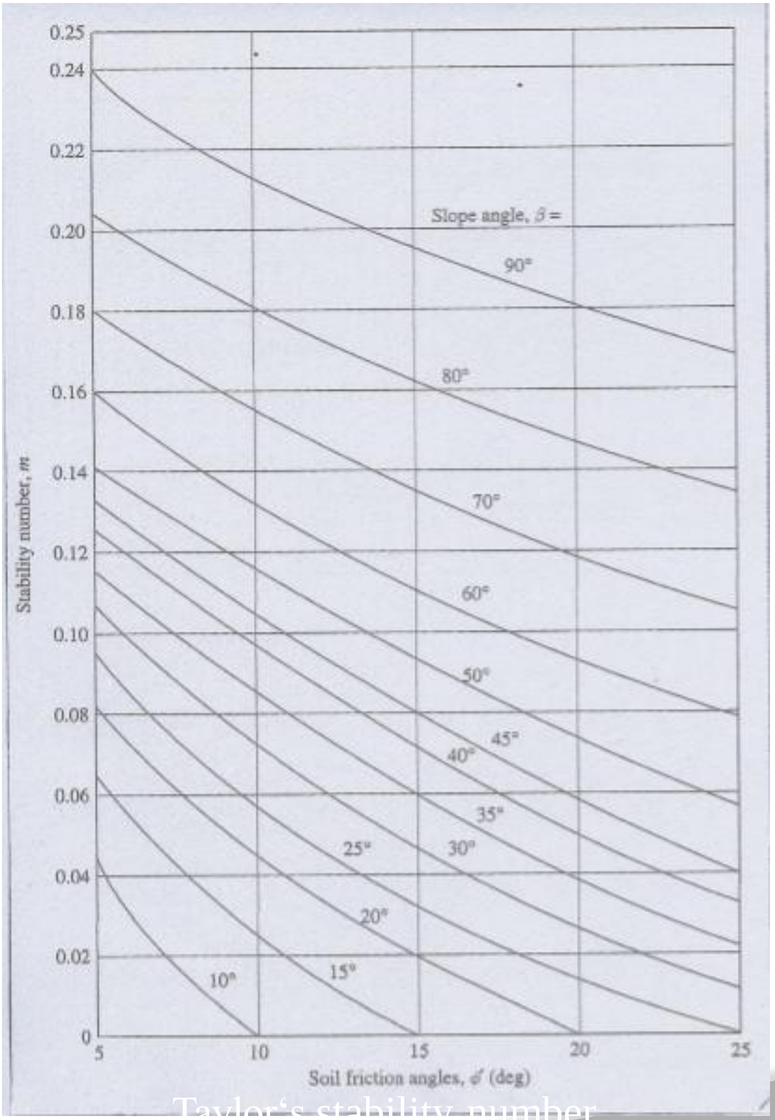
TAYLOR'S STABILITY CHART

- Taylor proposed stability coefficients for the analysis of homogeneous slopes.

$$f\,(\alpha, \beta, \theta, \phi') = \frac{c'}{\gamma H_{cr}}$$

stability number, m can be obtain from stability charts – Taylor's stability number.

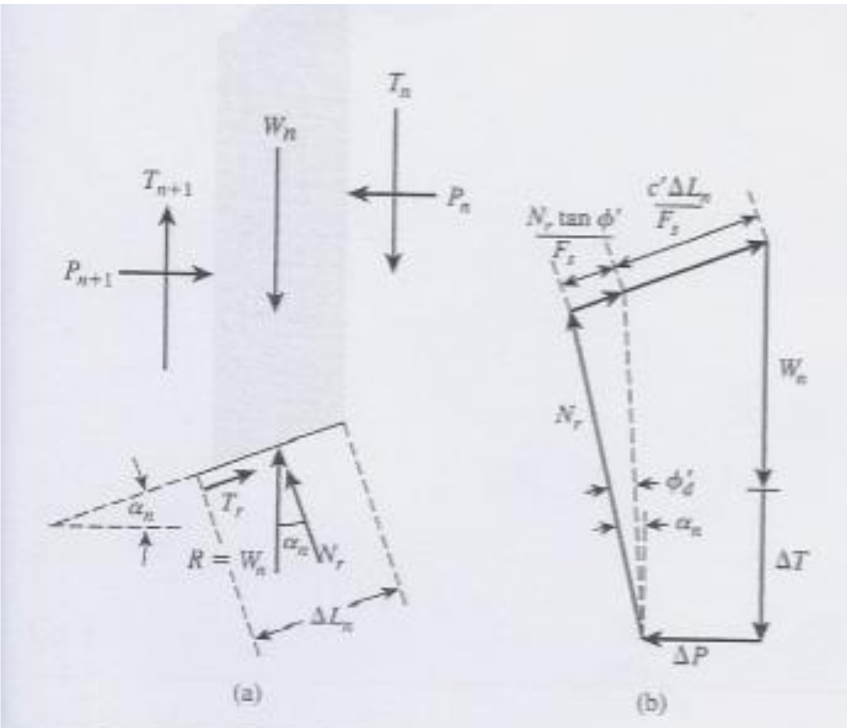
$$F_s = F_c = F_{\phi}$$



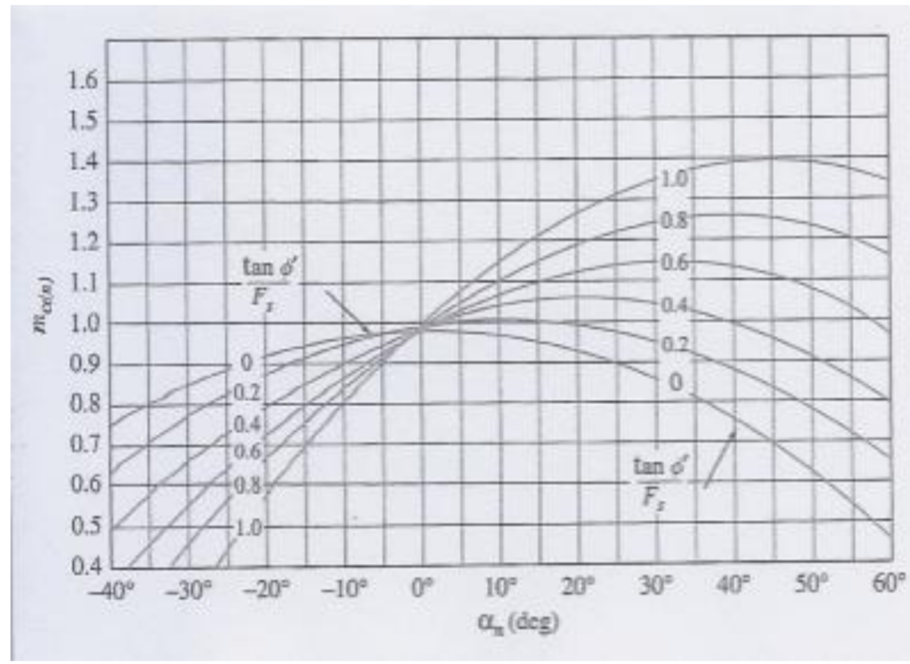
Taylor's stability number

BISHOP METHOD OF SLICES

- Bishop proposes a more refined solution to previous method where the effect of forces on the sides of each slice are accounted.



T_n
W_n
T_{n+1}
P_n
P_{n+1}
α_n
T_r
$R = W_n$
α_n
N_r
ΔL_n
(a)
$\frac{N_r \tan \phi'}{F_s}$
$\frac{c' \Delta L_n}{F_s}$
W_n
N_r
ϕ'_d
α_n
ΔT
ΔP
(b)



$m_{\alpha(n)}$
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
$\frac{\tan \phi'}{F_s}$
1.0
0.8
0.6
0.4
0.2
0
−40°
−30°
−20°
−10°
0°
10°
20°
30°
40°
50°
60°
α_n (deg)

- Several number of failure surface must be investigated so that the critical surface(yields the lowest FOS value) can be found.

  This method gives satisfactory results in most cases when programs Widely used.

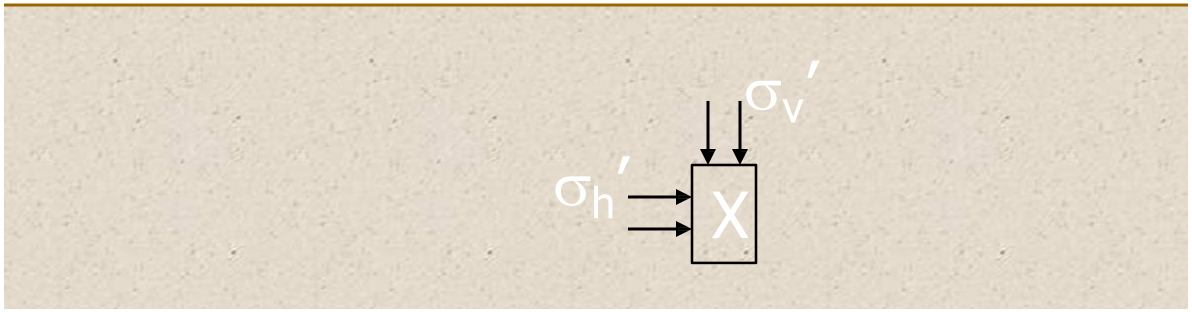
  .

UNIT-III
EARTH PRESSURE THEORIES

CLOs	Course Learning Outcome
CLO15	Understand concepts of earth pressure theories for stability of Retaining walls
CLO16	Calculate active and passive earth pressures from Rankine's earth pressure theories
CLO17	Calculate active and passive earth pressures from Coulomb's &Culmann's Method
CLO18	Asses the stability of retaining wall against overturning, sliding, bearing capacity

EARTH PRESSURE AT REST

- In a homogeneous natural soil deposit,



- Ratio σ_h'/σ_v' is a constant known as coefficient of earth pressure at rest (K_0).

- Importantly, at K_0 state, there are no lateral strains.

ESTIMATING K_0

normally consolidated clays and granular soils,

$$K_0 = 1 – \sin ?'$$

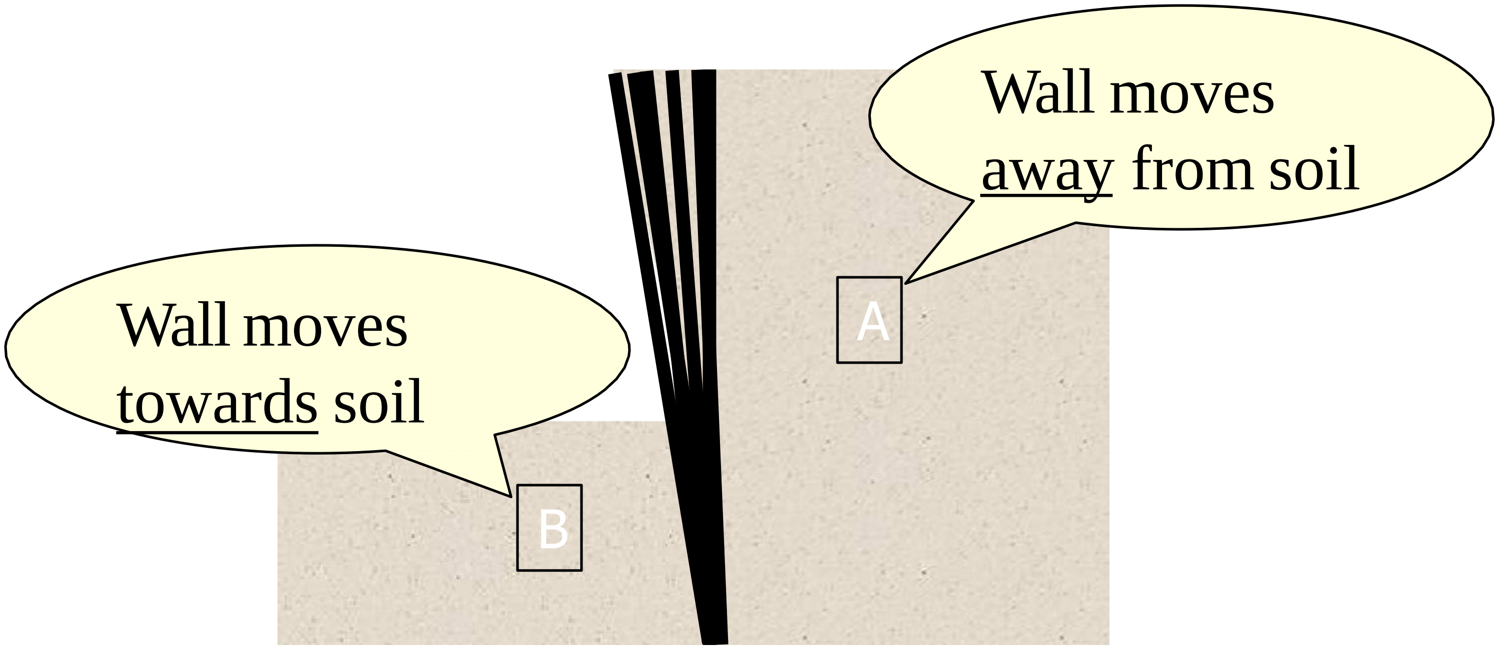
For overconsolidated clays,

K0,overconsolidated = K0,normally consolidated $OCR^{0.5}$

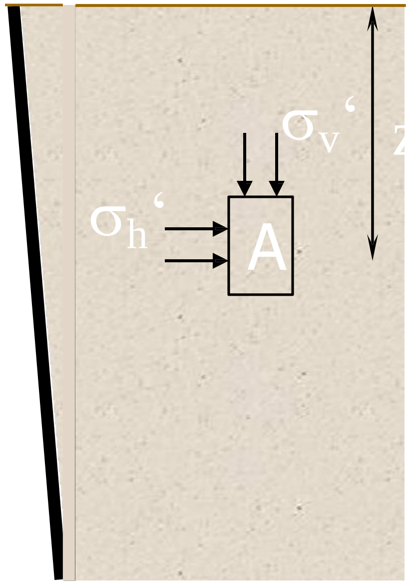
From elastic analysis

ACTIVE/PASSIVE EARTH PRESSURES

in **granular** soils



in **granular** soils



Initially, there is no lateral movement.

$\therefore \sigma_h' = K_0 \sigma_v' = K_0 \gamma z$

As the wall moves away from the soil,

σ_v' remains the same; and

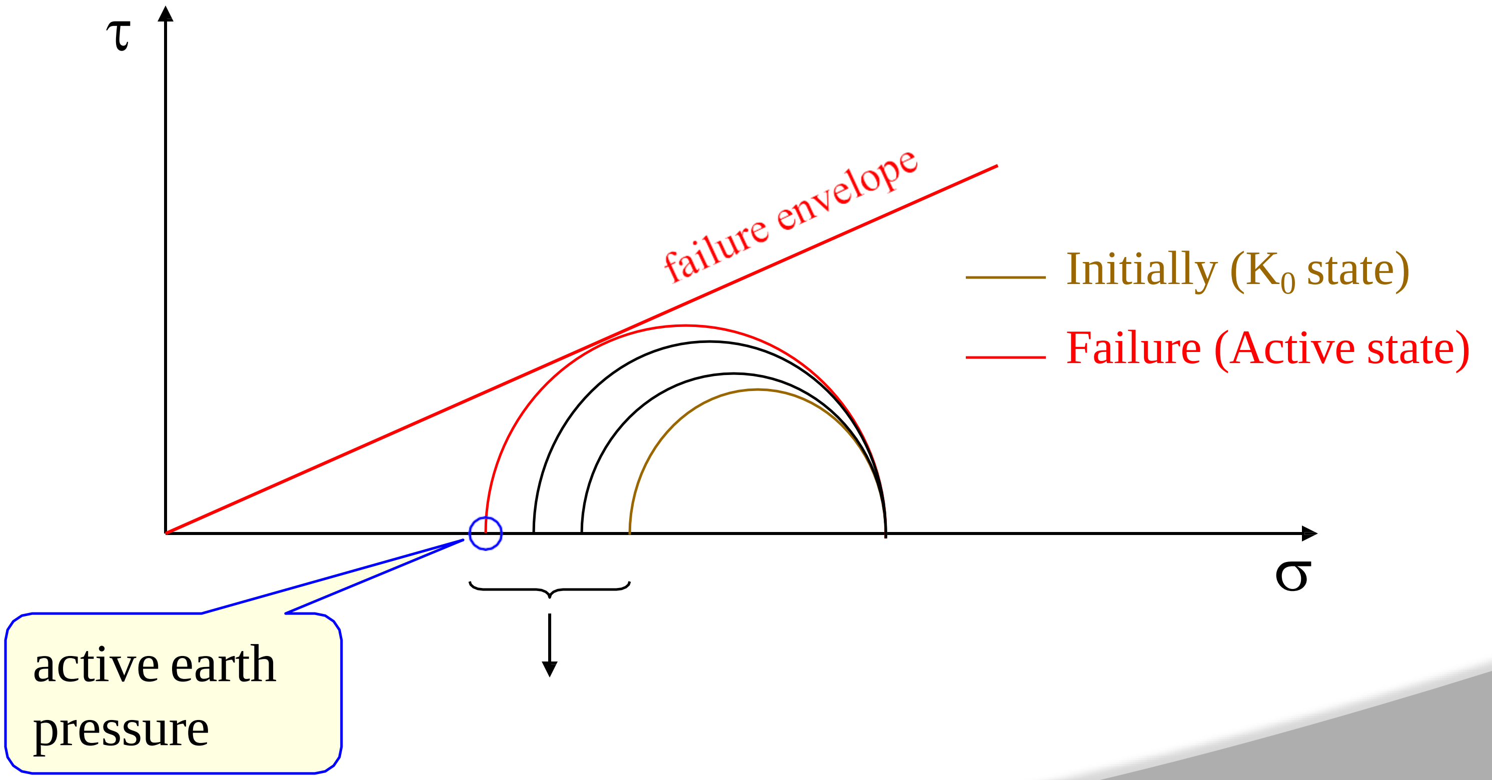
σ_h' decreases till failure occurs.

Active state

ACTIVE EARTH PRESSURE

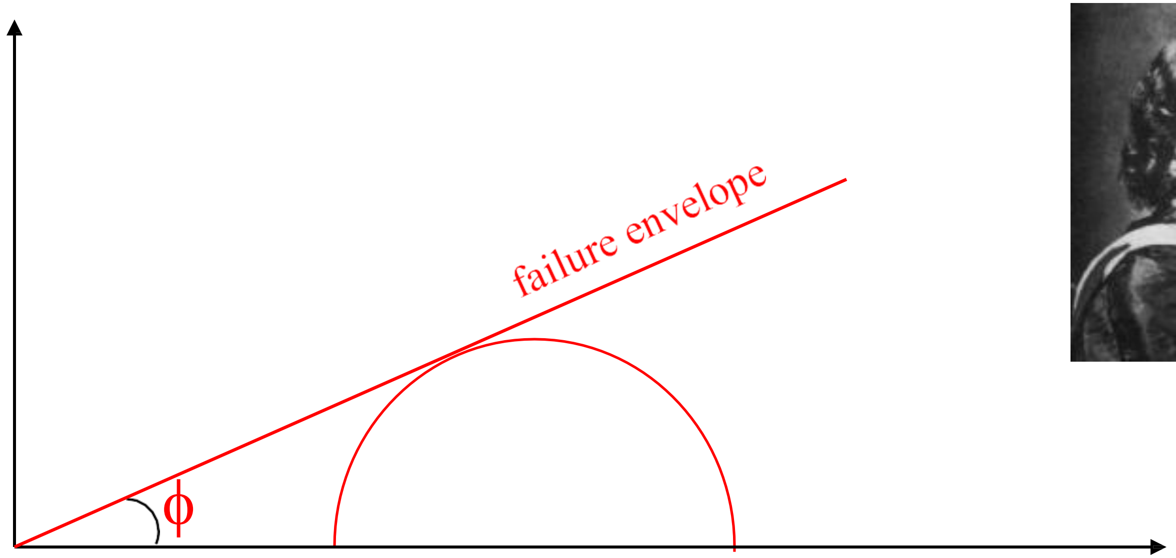
- in **granular** soils

As the wall moves away from the soil,



ACTIVE EARTH PRESSURE

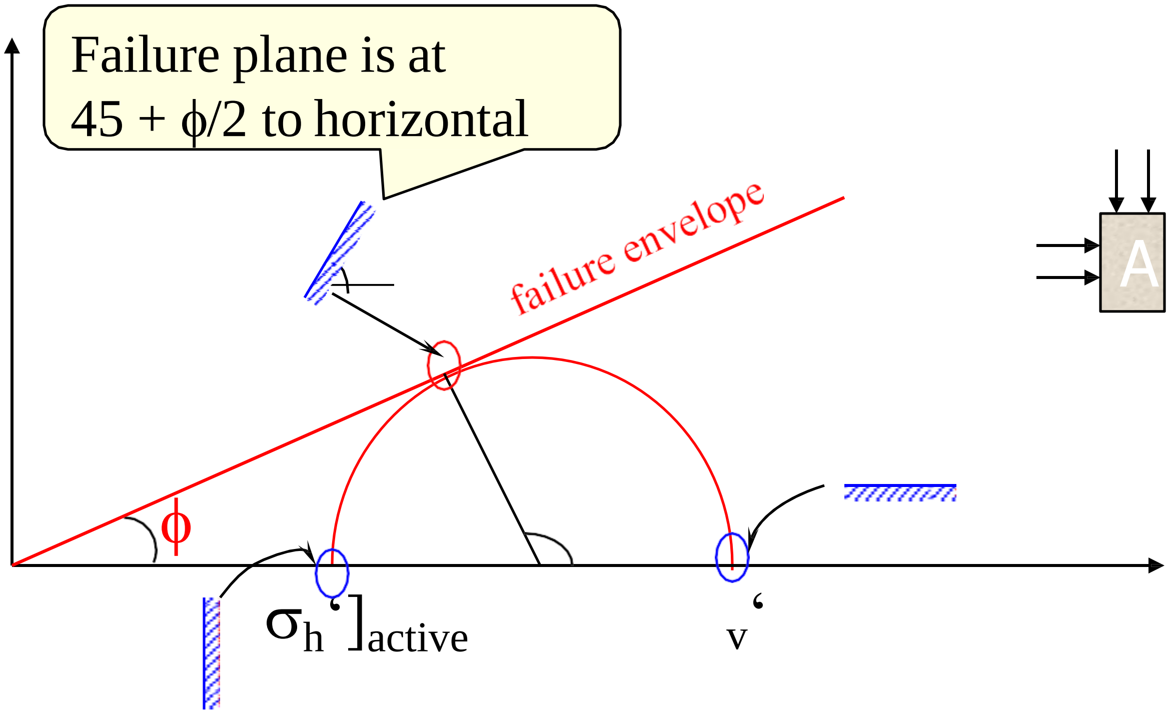
granular



Rankine's coefficient of active earth pressure

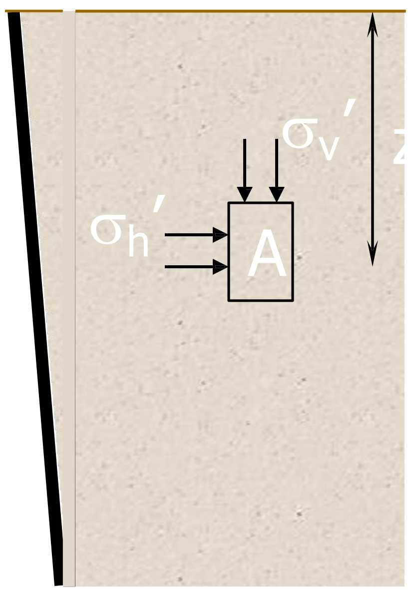
Active Earth Pressure

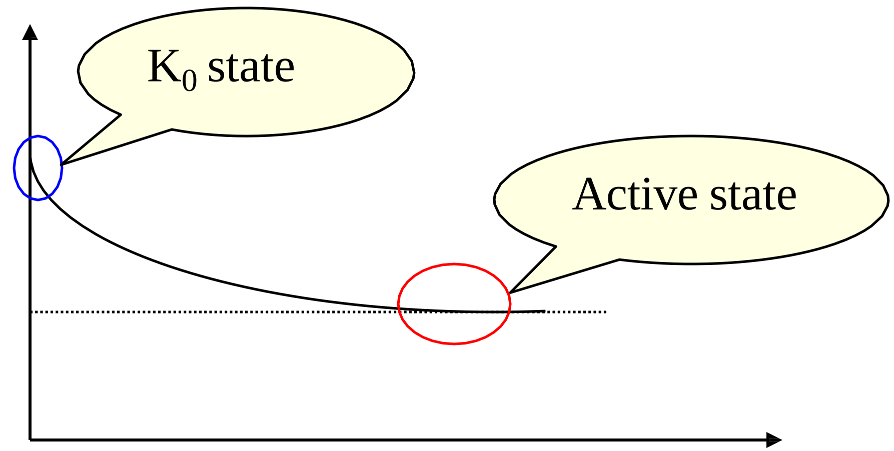
granular soils



in **granular** soils As the wall moves away from the soil,

σ_h' decreases till failure occurs.





ACTIVE EARTH PRESSURE

- in **cohesive** soils

Follow the same steps as for granular soils. Only difference is that c $\neq$ 0.

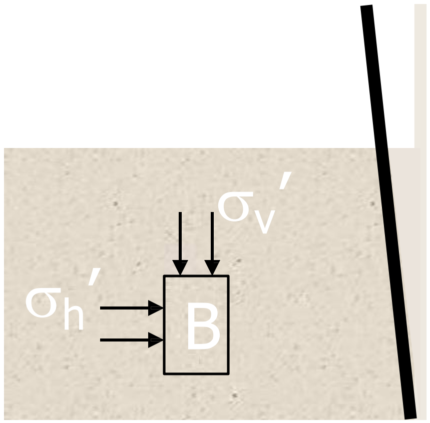
$$[\sigma_h']_{active} = K_A \sigma_v' - 2c\sqrt{K_A}$$

Everything else the same as for granular soils.

PASSIVE EARTH PRESSURE

in **granular** soils Initially, soil is in K_0 state.

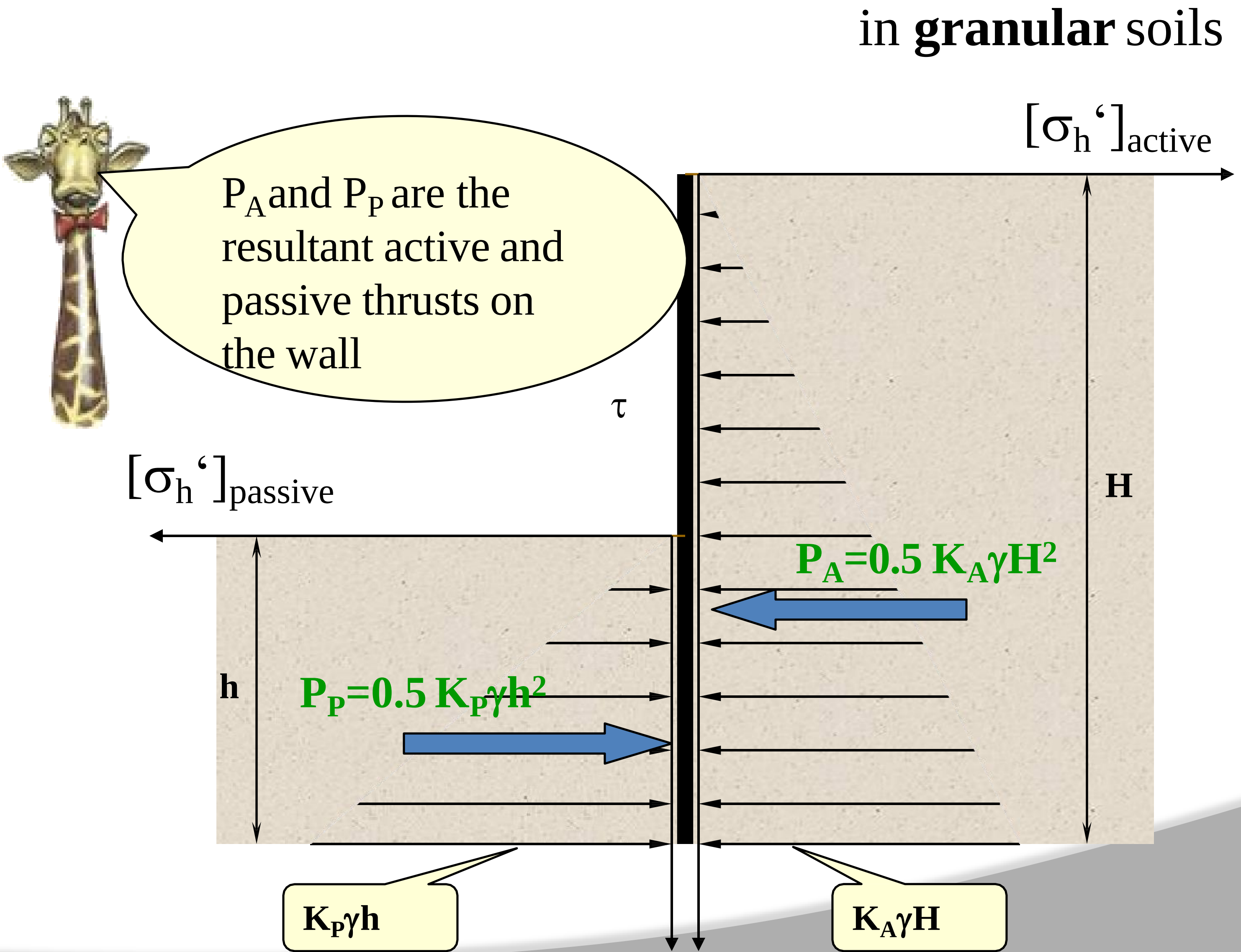
As the wall moves towards the soil, σ_v' remains the same.

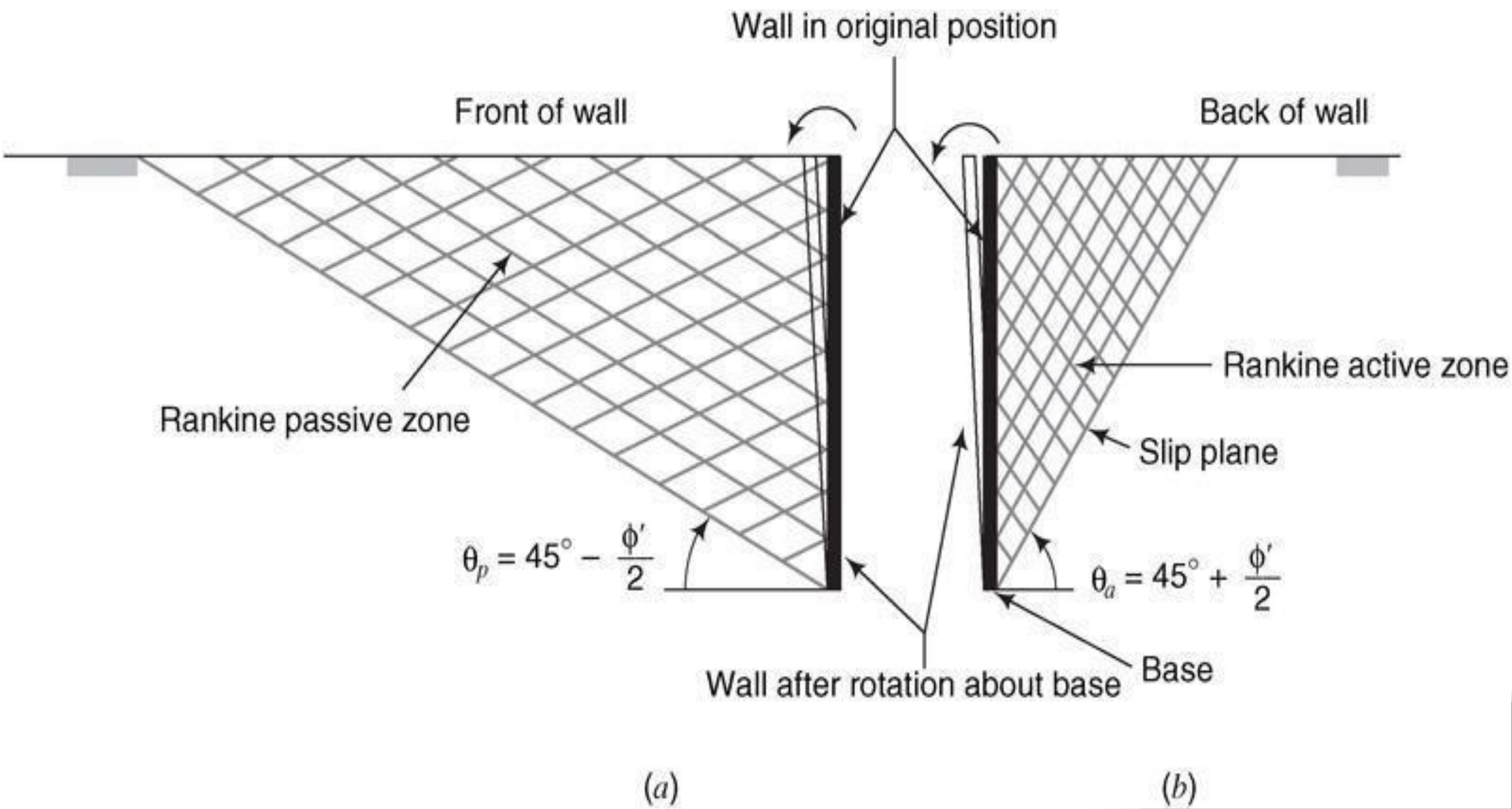


σ ' increases till failure occurs.

Passive state

Earth Pressure Distribution



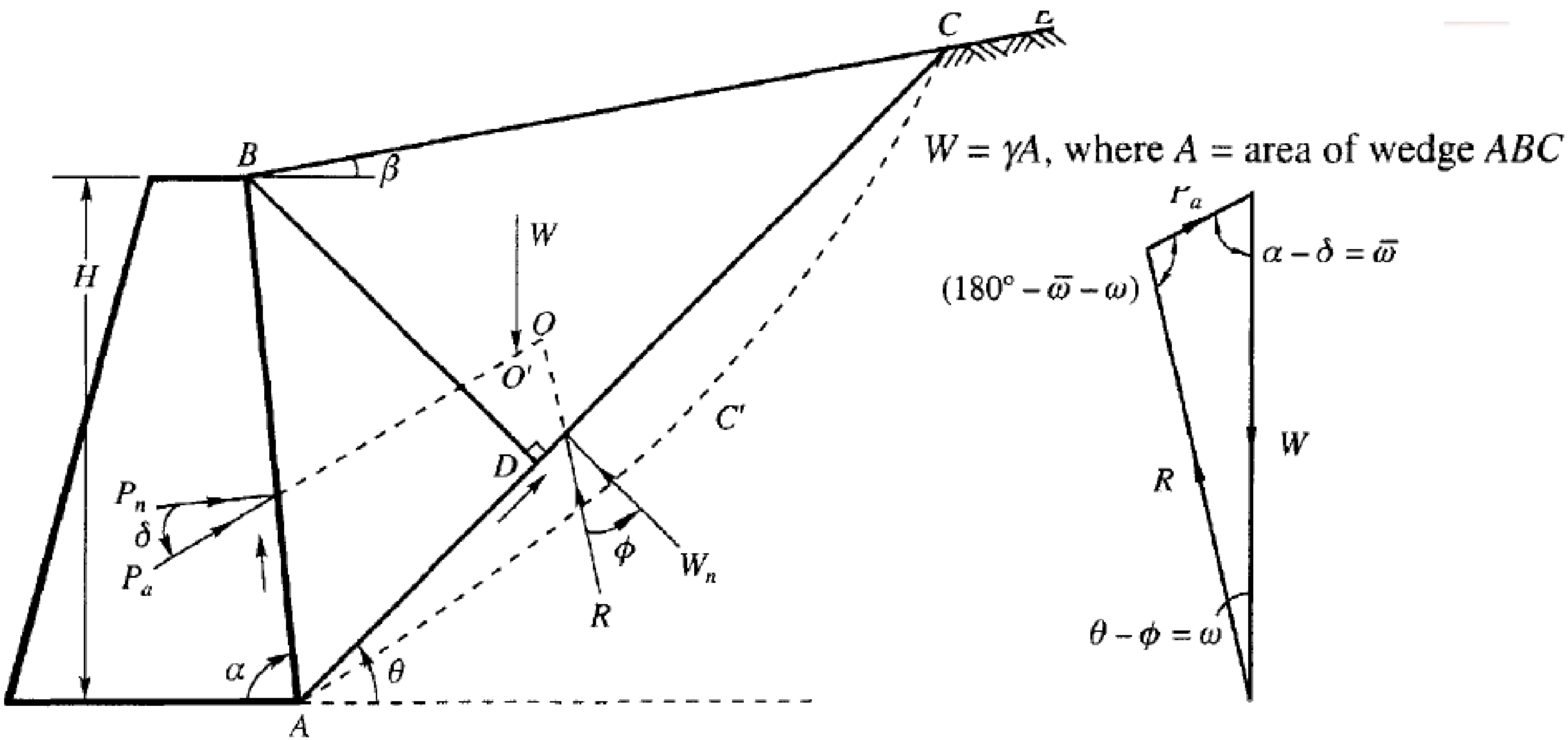

Wall in original position
Front of wall
Back of wall
Rankine passive zone
Rankine active zone
Slip plane
$\theta_p = 45^\circ - \frac{\phi'}{2}$
$\theta_a = 45^\circ + \frac{\phi'}{2}$
Base
Wall after rotation about base
(a)
(b)

RANKINE'S EARTH PRESSURE THEORY

$$\sigma_h']_{active} = K_A \sigma_v' - 2c\sqrt{K_A}$$

$$[\sigma_h']_{passive} = K_P \sigma_v' + 2c\sqrt{K_P}$$

- Assumes smooth wall
- Applicable only on vertical walls



(a) Retaining wall

(b) Polygon of forces

Figure 11.17 Conditions for failure under active conditions

Area of wedge $ABC = A = 1/2\, AC \times BD$

where BD is drawn perpendicular to AC.

From the law of sines, we have

$$AC = AB\frac{\sin(\alpha+\beta)}{\sin(\theta-\beta)}, \quad BD = AB\sin(\alpha+\theta), \quad AB = \frac{H}{\sin\alpha}$$

Making the substitution and simplifying we have,

$$W = \gamma A = \frac{\gamma H^2}{2\sin^2\alpha}\sin(\alpha+\theta)\frac{\sin(\alpha+\beta)}{\sin(\theta-\beta)}$$

From the polygon of forces, we may write

$$\frac{P_a}{\sin(\theta-\phi)} = \frac{W}{\sin(180^\circ-\alpha-\theta+\phi+\delta)}$$

$$\text{or} \quad P_a = \frac{W\sin(\theta-\phi)}{\sin(180^\circ-\alpha-\theta+\phi+\delta)}$$

$$P_a = \frac{\gamma H^2}{2\sin^2\alpha} \frac{\sin(\theta-\phi)}{\sin(180° - \alpha - \theta + \phi + \delta)} \left(\sin(\alpha+\phi) \frac{\sin(\alpha+\beta)}{\sin(\theta-\beta)} \right) \tag{11.49}$$

The maximum value for P_a is obtained by differentiating Eq. (11.49) with respect to θ and equating the derivative to zero, i.e.

$$\frac{dP_a}{d\theta} = 0$$

The maximum value of P_a so obtained may be written as

$$P_a = \frac{1}{2}\gamma H^2 K_A \tag{11.50}$$

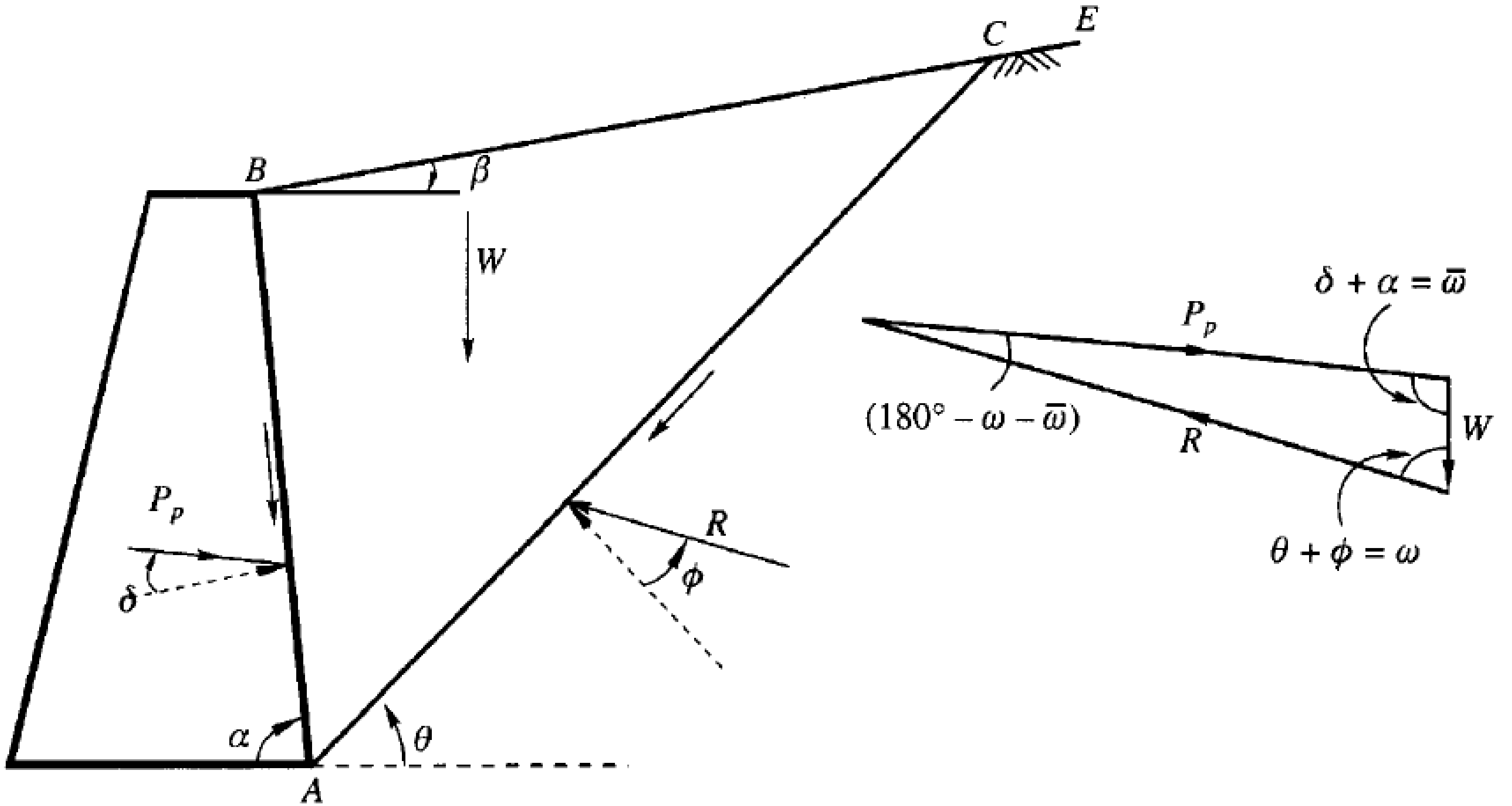
where K_A is the active earth pressure coefficient.

$$K_A = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha \sin(\alpha-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

The total normal component P_n of the earth pressure on the back of the wall is

$$P_n = P_a \cos\delta = \frac{1}{2}\gamma H^2 K_A \cos\delta$$

COLOUMB EARTH PRESSURE THEORY FOR SAND



(a) Forces on the sliding wedge

(b) Polygon of forces

Figure 11.18 Conditions for failure under passive state

PASSIVE STATE

$$P_p = \frac{1}{2}\gamma H^2 K_p$$

where K_p is called the *passive earth pressure coefficient*.

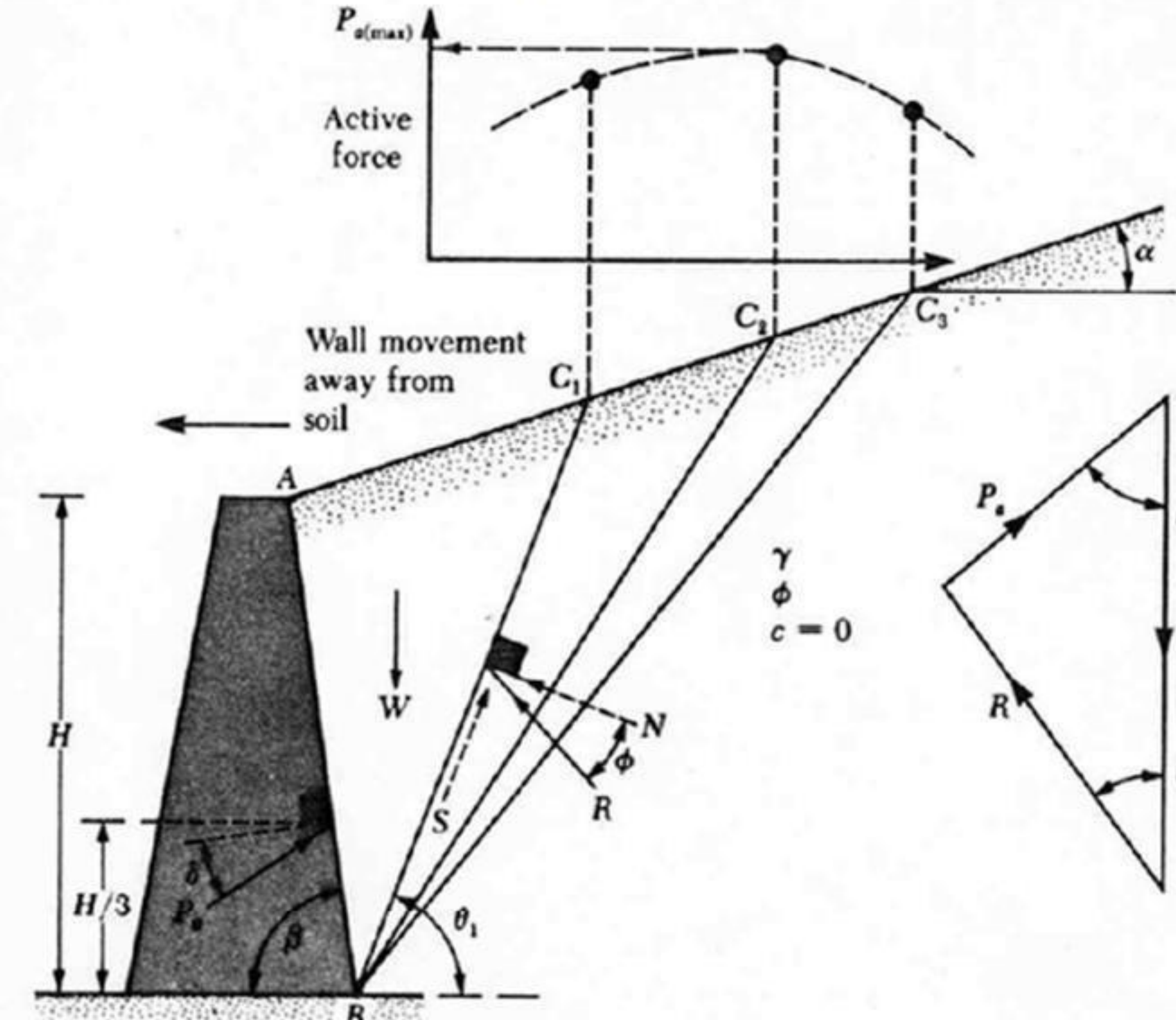
$$K_p = \frac{\sin^2(\alpha - \phi)}{\sin^2\alpha \sin(\alpha + \delta)\left[1 - \sqrt{\frac{\sin(\phi + \delta)\sin(\phi + \beta)}{\sin(\alpha + \delta)\sin(\alpha + \beta)}}\right]^2}$$

Note that $\mathbf{K_{pC} \neq 1/K_{aC}}$.

The inclination of the slip plane to the horizontal is

$$\boxed{\tan\theta = \frac{1}{\cos\phi'}\sqrt{\frac{\sin\phi'\cos\delta}{\sin(\phi' + \delta)}} \pm \tan\phi'}$$

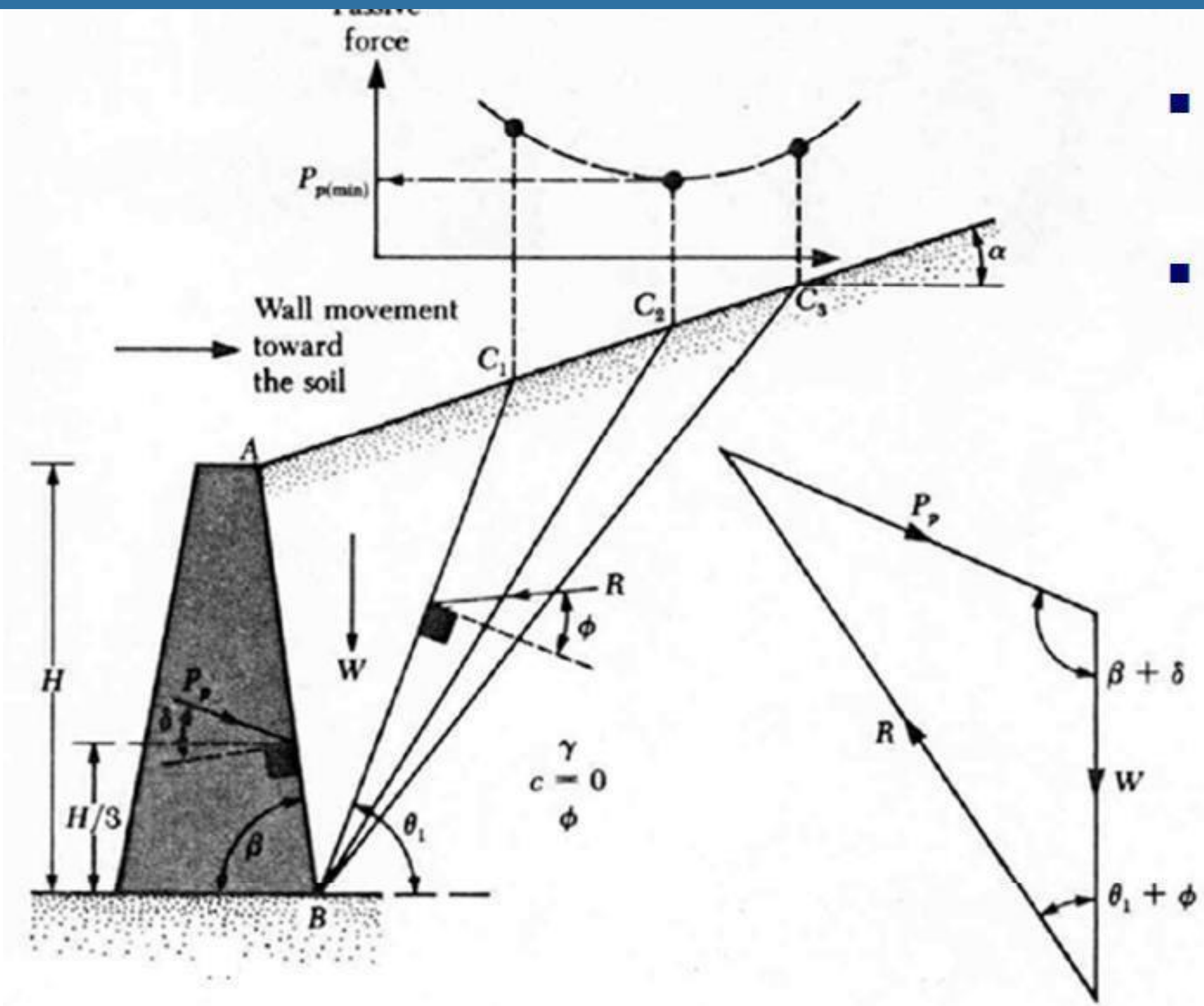
where the positive sign refers to the active state (θ_a) and the negative sign refers to the passive state (θ_p).



- *Wall Friction:*

  $\frac{1}{2}\phi < \delta < 2/3\phi$

- *Coulomb's theory underestimates Active EP*



- *Wall Friction:*

  $\frac{1}{2}\phi < \delta < 2/3\phi$

- *Coulomb's theory overestimates Passive EP*

COULOMB WEDGE ANALYSIS IN C_U, F_U=0 SOIL

$$z_o = (2c_u / \gamma)\sqrt{1 + c_w / c_u}$$

$$P_a = \gamma H(H - z_o)/2 - c_u(H - z_o)\sqrt{1 + c_w / c_u}$$

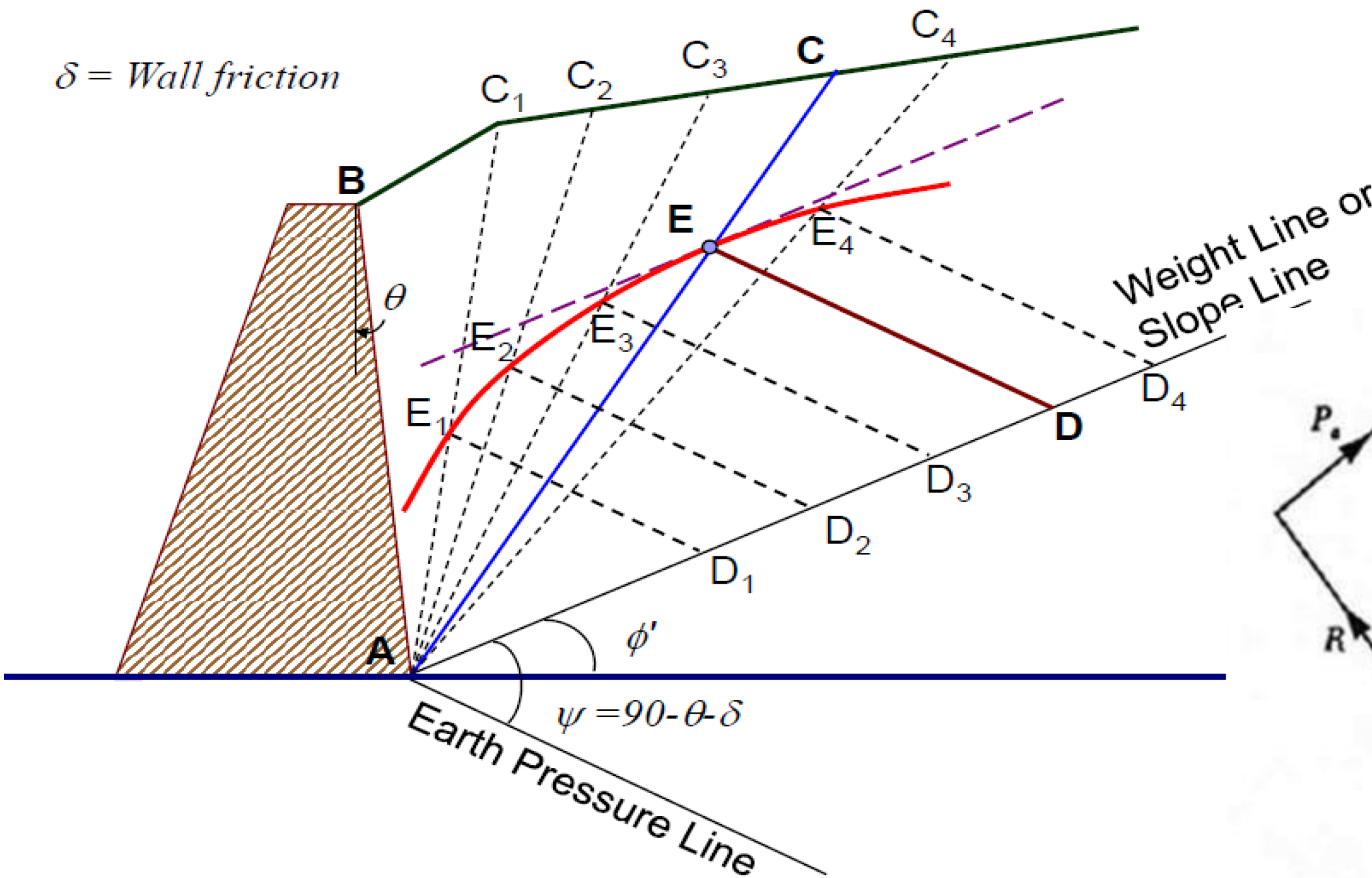
For $q > 0$, and $p_a < 0$ at $z = 0$, the equivalent linear lateral earth pressure, depth of tension crack and total active thrust are given by:

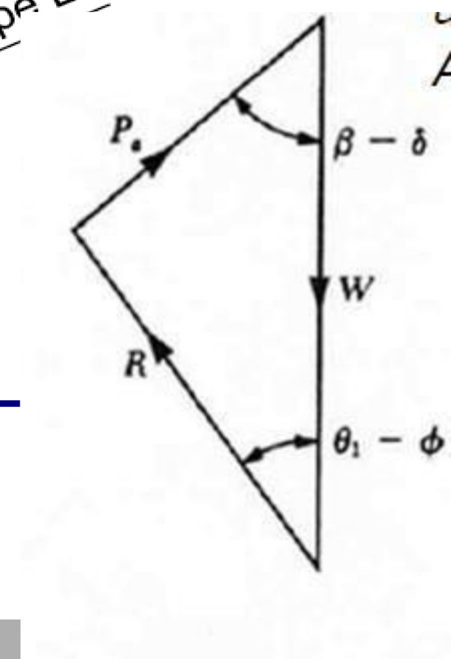
$$p_a = \gamma z + q - 2c_u\sqrt{1 + c_w / c_u}$$

$$z_o = (2c_u / \gamma)\sqrt{1 + c_w / c_u} - q / \gamma$$

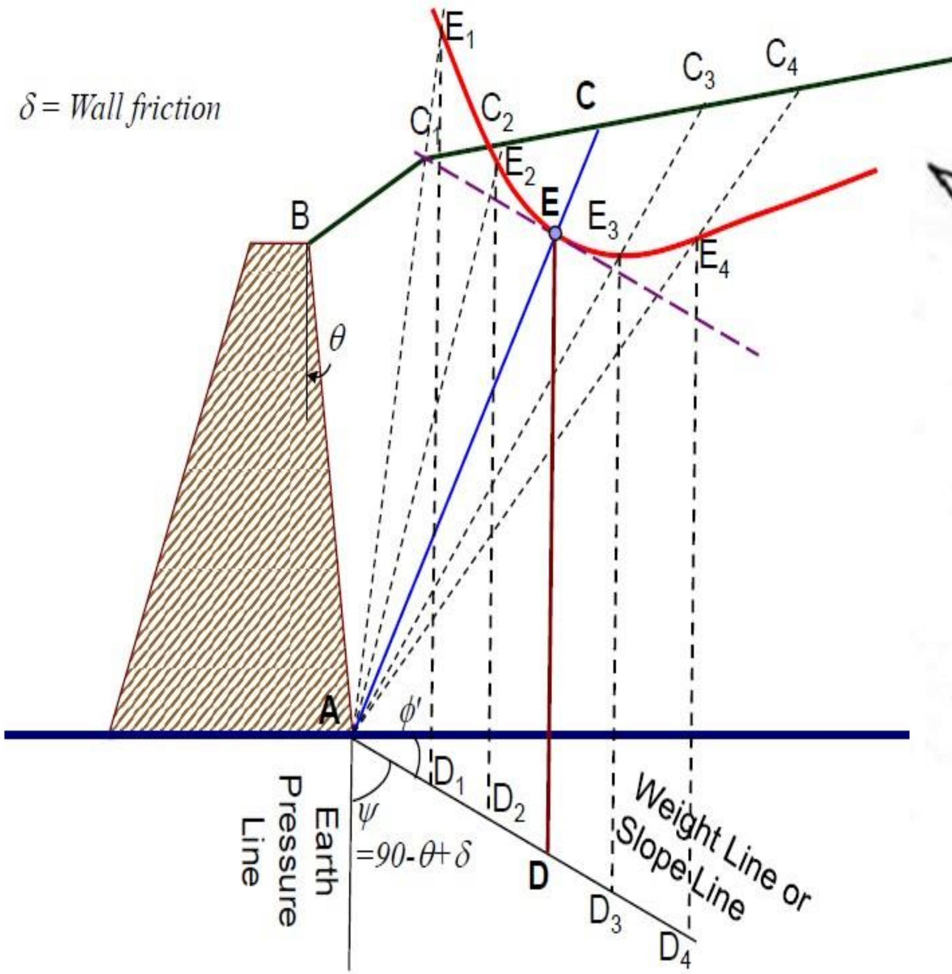
$$P_a = \gamma H(H - z_o)/2 + q(H - z_o) - c_u(H - z_o)\sqrt{1 + c_w / c_u}$$

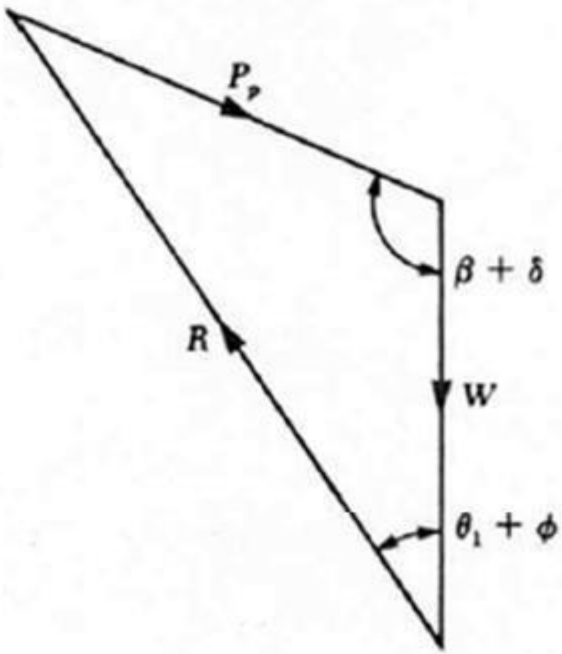
Culmann's Graphical Method: Active EP



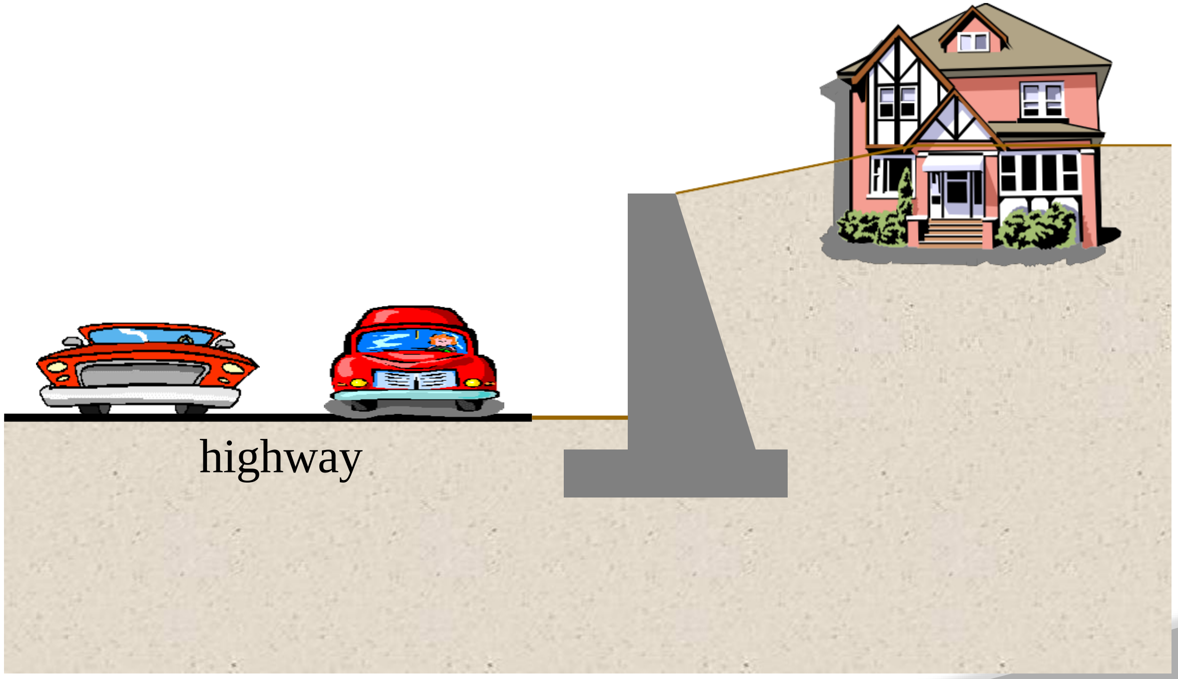


Culmann's Graphical Method: Passive EP

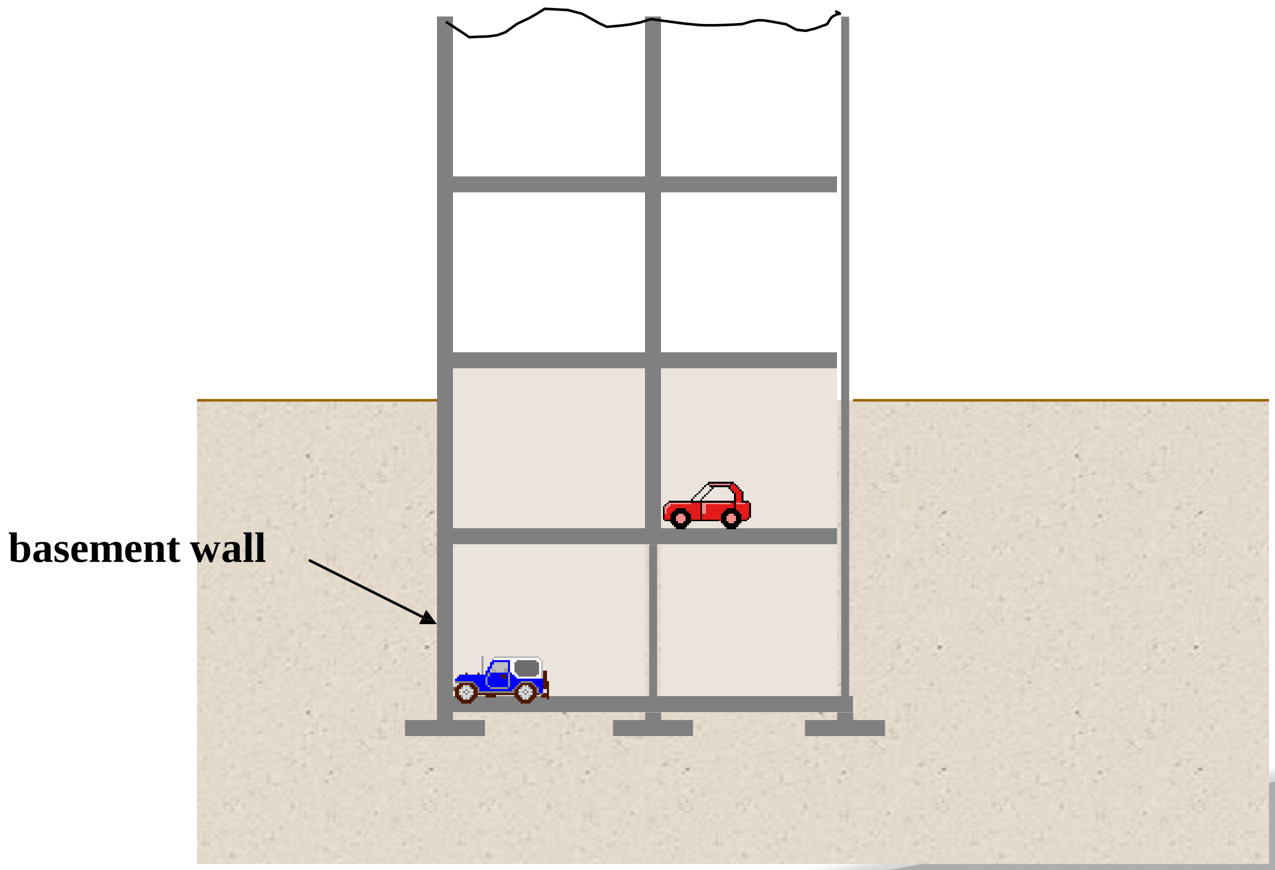




RETAINING WALLS - APPLICATIONS

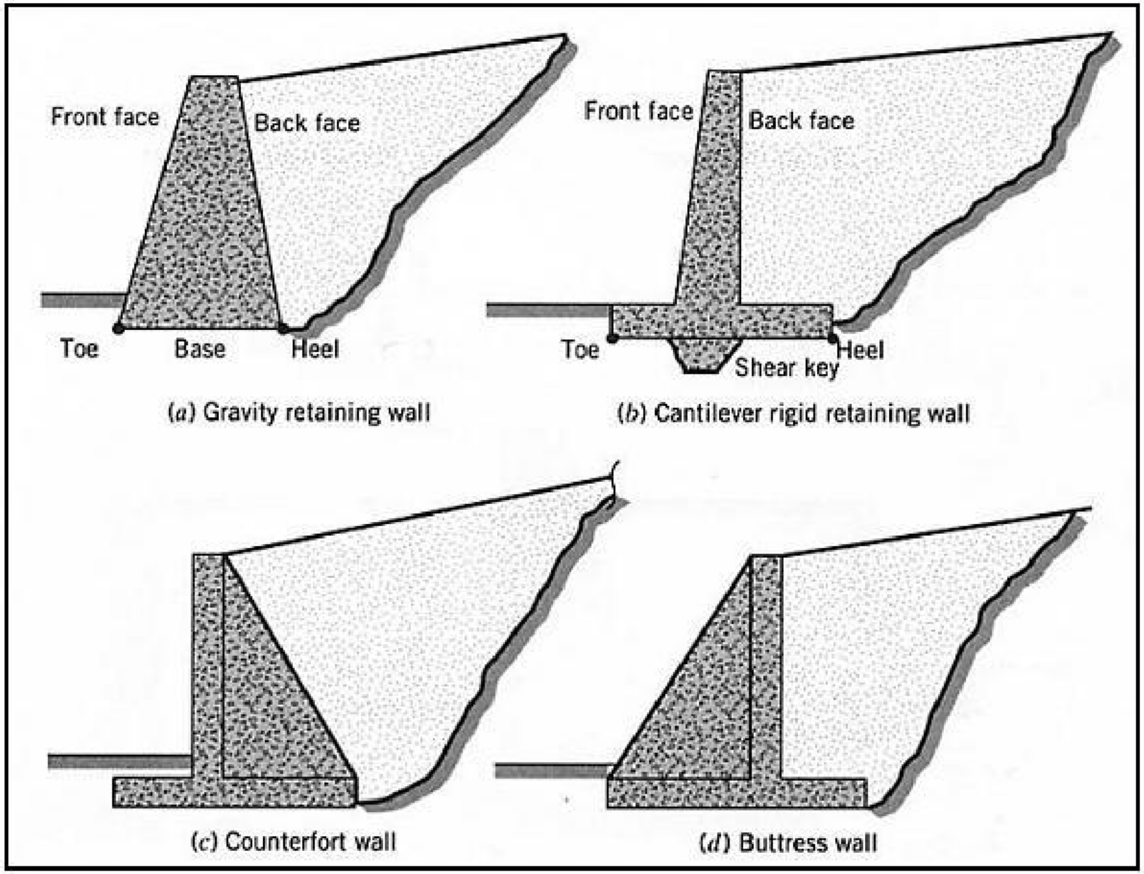


RETAINING WALLS - APPLICATIONS



Types of retaining walls

- There are two general classes of retaining walls – **rigid** and **flexible**.



(*a*) Gravity retaining wall

(*b*) Cantilever rigid retaining wall

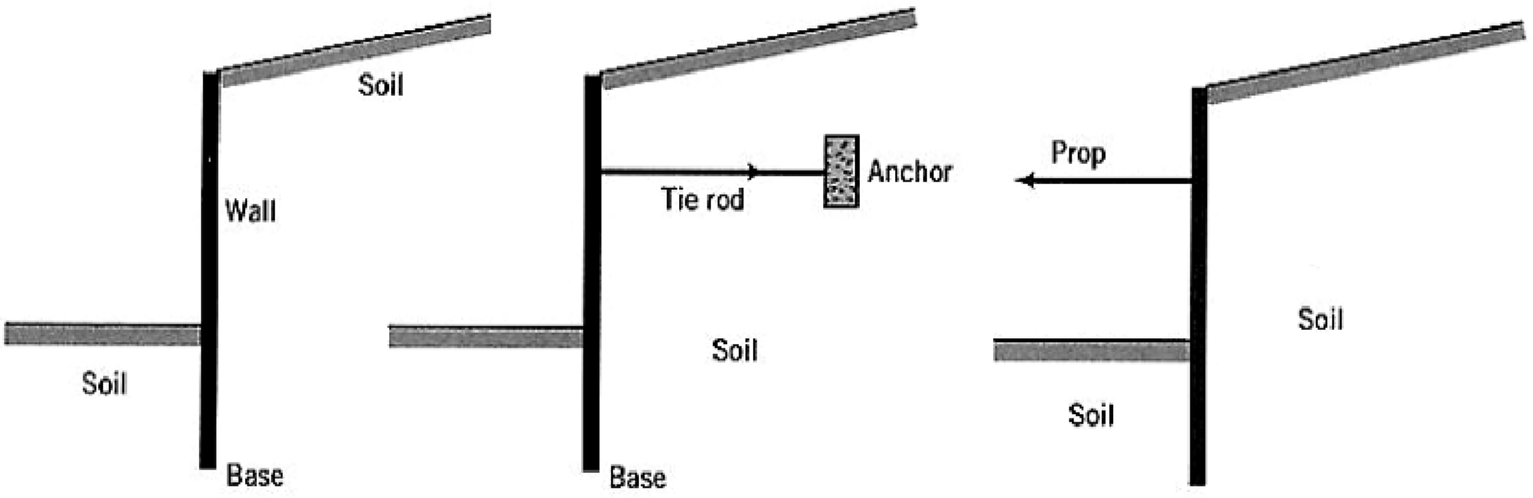
(*c*) Counterfort wall

(*d*) Buttress wall

- Rigid retaining walls consist of concrete walls that rely on **gravity** for their stability.
- Four different rigid retaining walls are shown in the figure on the left.

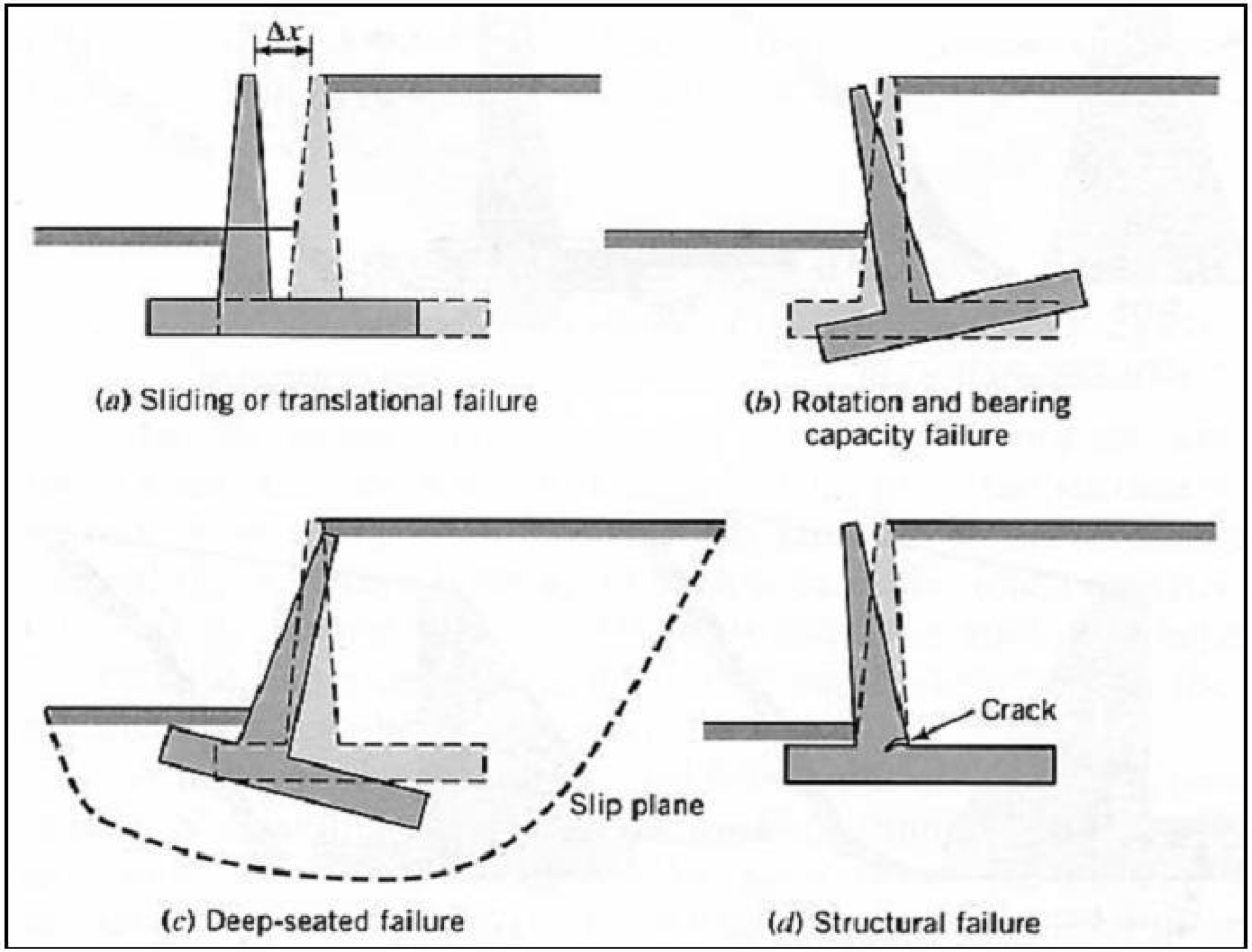
Flexible retaining walls

- Flexible retaining walls consist of slender members of either steel or concrete or wood and rely on **passive soil resistance**, **props** or **anchors** for stability.
- Three different types of flexible retaining walls are shown in the figure below.



(*a*) Cantilever (*b*) Anchored or tie-back (*c*) Propped

Failure modes for rigid retaining walls



Rigid retaining walls – Other failure modes

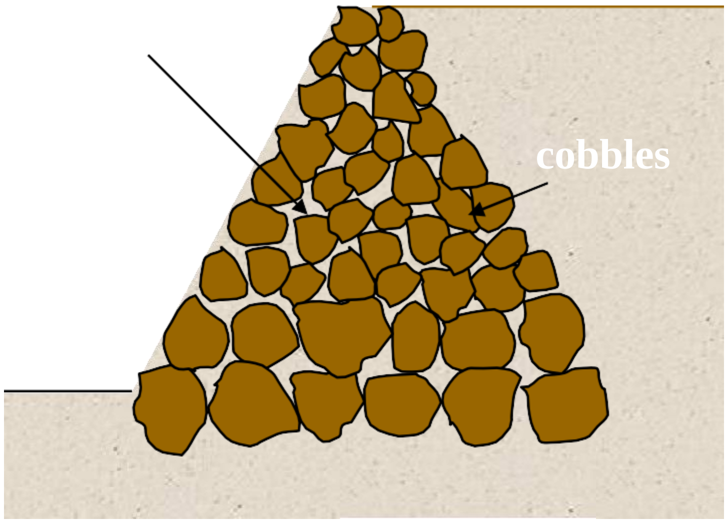
[Please fill in the equations during the lecture.]

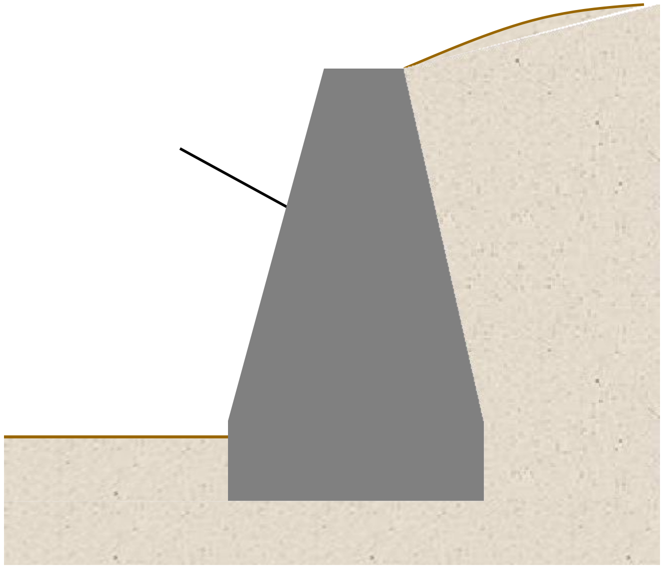
- A rigid retaining wall must have a sufficient margin of safety against soil **bearing capacity failure**.
- The maximum pressure imposed on the soil at the base of the wall must not exceed the allowable soil bearing capacity; that is

  where σ_{max} is the maximum vertical stress imposed by the wall and q_a is the allowable soil bearing capacity.

- A rigid retaining wall must not fail by **deep-seated failure** whereby the sliding mass includes both the wall and the soil.
- The stability against a deep-seated failure can be checked by using one of the many **slope stability methods** discussed in the previous lectures.
- **Seepage forces**, if left unchecked, can alter the stability of a rigid retaining wall significantly.
- The hydraulic gradient should generally be less than the **critical** hydraulic gradient:

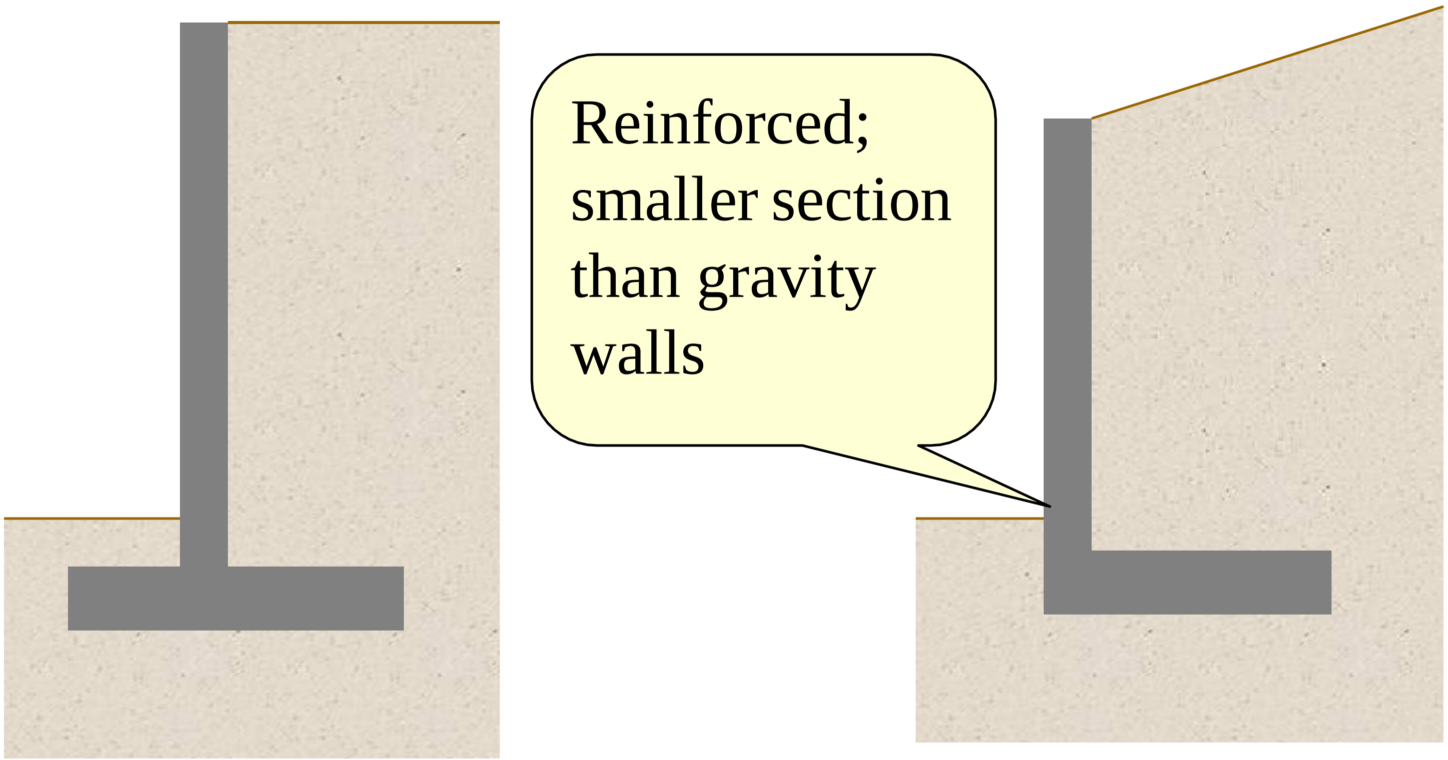
GRAVITY RETAINING WALLS





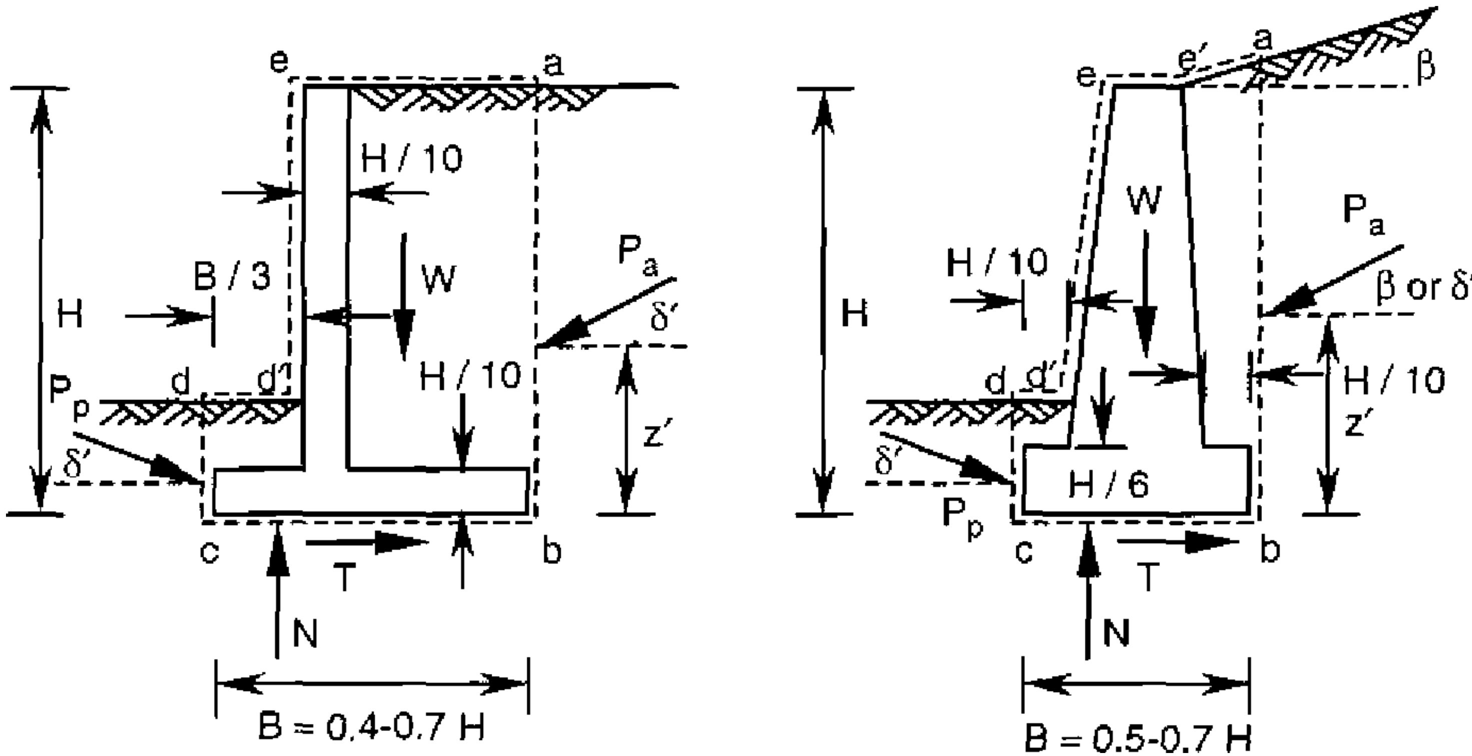
They rely on their self weight to support the backfill

CANTILEVER RETAINING WALLS



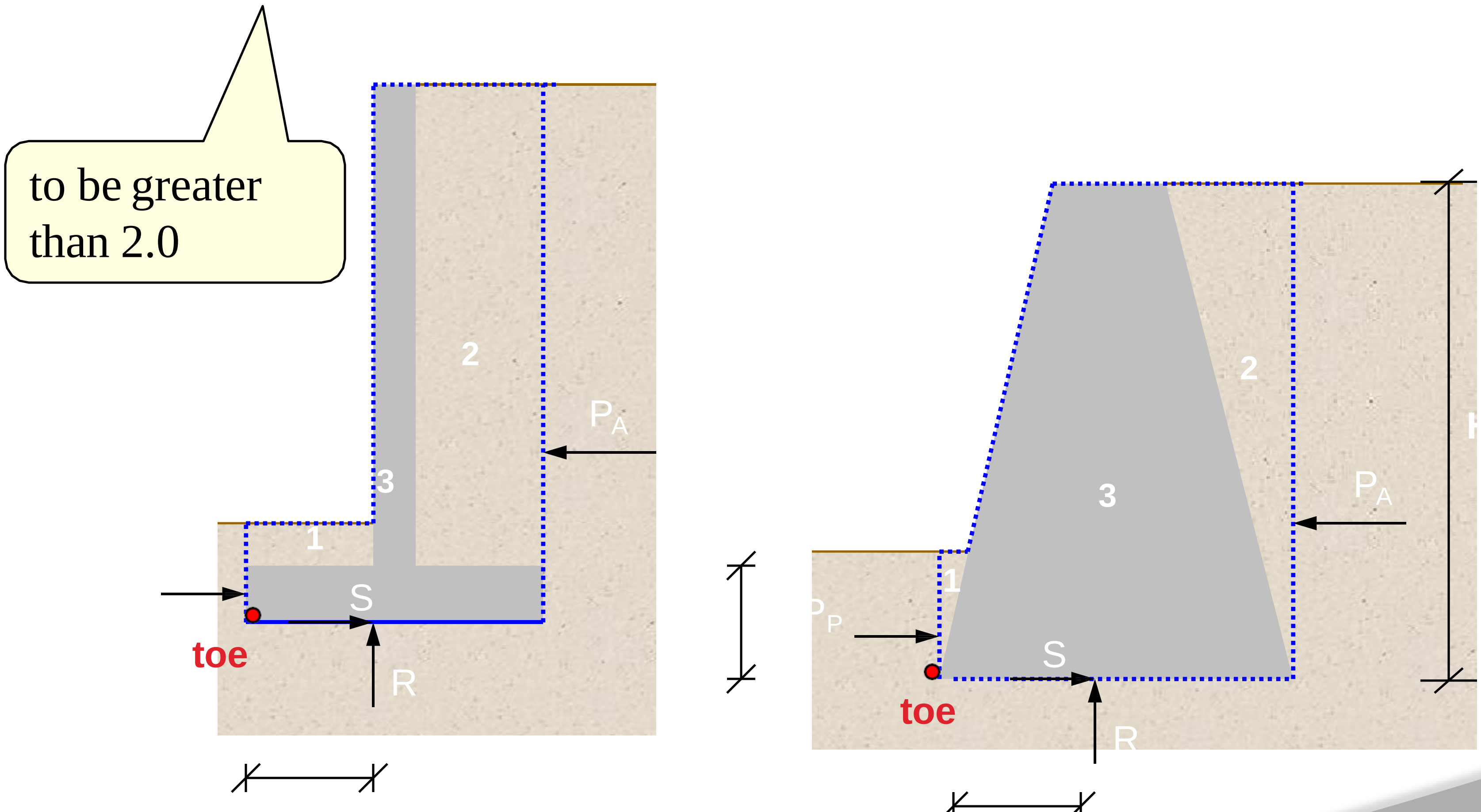
They act like vertical cantilever, fixed to the ground

DESIGN OF RETAINING WALLS



Sections and free-body diagrams of typical cantilever and gravity retaining walls.

Safety against overturning about toe

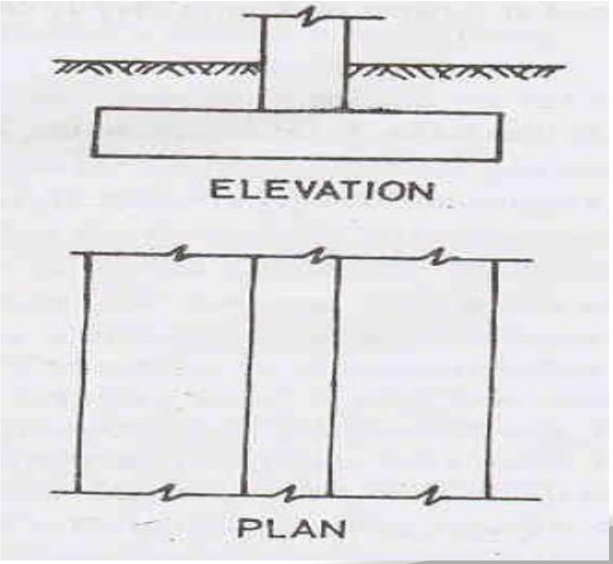


UNIT-IV
SHALLOW AND DEEP FOUNDATIONS

CLOs	Course Learning Outcome
CLO19	Understand the concepts of safe bearing capacity, ultimate bearing capacity etc.,
CLO20	Calculate the bearing capacity of shallow foundation using Terzaghi, Meyerhof, Skempton and IS Methods.
CLO21	Calculate the load carrying capacity of pile using static, dynamic pile formula and pile load test
CLO22	Calculate load carrying capacity of pile group in sands and clay & settlement of pile group

FOUNDATION

A Foundation is a integral part of the structure which transfer the load of the superstructure to the soil. A foundation is that member which provides support for the structure and it's loads. It includes the soil and rock of earth's crust and any special part of structure that serves to transmit the load into the rock or soil.



DIFFERENT TYPES OF FOOTINGS

SHALLOW FOUNDATIONS($D_f \le B$)

- STRIP FOOTING
- SPREAD FOOTING
- COMBINE FOOTING
- ISOLATED FOOTING
- RAFT FOOTING

DEEP FOUNDATION ($D_f > B$)

- PILE FOUNDATION
- WELL FOUNDATION

BEARING CAPACITY

- **Ultimate Bearing capacity: q_u**

Maximum gross intensity of loading that the soil can support against shear failure is called ultimate bearing capacity.

- **Net Ultimate Bearing Capacity: q_{nu}**

Maximum net intensity of loading that the soil can support at the level of foundation.

$$q_{nu} = q_u - \gamma D_f$$

- **Net Safe Bearing capacity: q_{ns}**

Maximum net intensity of loading that the soil can safely support without the risk of shear failure.

$$q_{ns} = q_{nu} / FOS$$

Gross Safe Bearing capacity:

Maximum gross intensity of loading that the soil can safely support without the risk of shear failure

$$q_{gs} = q_{ns} + \gamma D$$

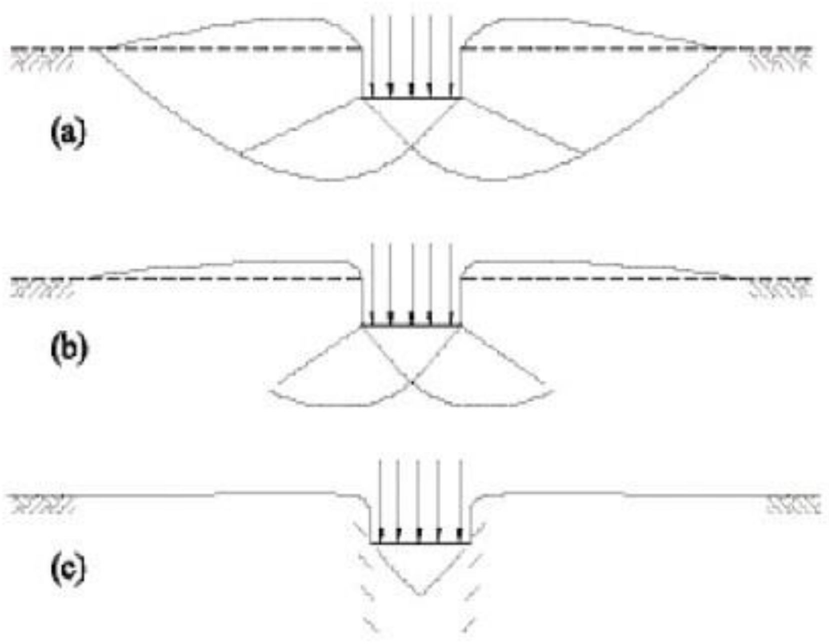
Safe Bearing Pressure:

Maximum net intensity of loading that can be allowed on the soil without settlement exceeding the permissible limit.

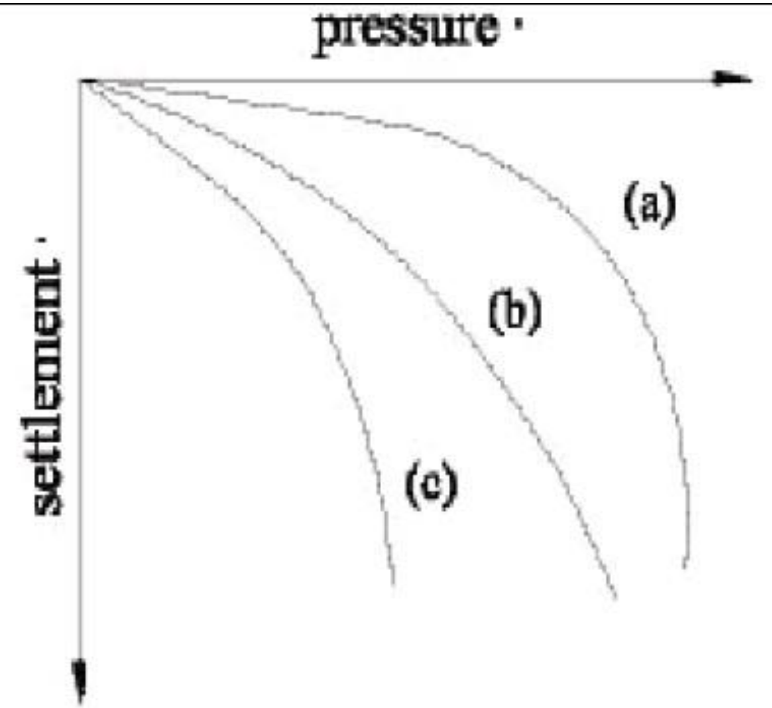
Allowable Bearing Pressure:

Maximum net intensity of loading that can be allowed on the soil with no possibility of Minimum of capacity and shear failure or settlement exceeding the permissible limit.

TYPES OF FAILURES



Shear failure in foundation soil



P – Δ curve in different foundation soils

Fig. 7. 1 : Footing on ground that experiences a) General shear failure, b) Local shear failure and c) Punching shear failure

GENERAL SHEAR FAILURE

Experiments have shown that foundations on dense sand with RD greater than 70 percent fail suddenly with pronounced peak when settlement reaches about 7 percent of foundation width. This type of failure is seen in dense and stiff soil. The following are some characteristics of general shear failure. Continuous, well defined and distinct failure surface develops between the edge of footing and ground surface. Dense or stiff soil that undergoes low compressibility experiences this failure. Continuous bulging of shear mass adjacent to footing is visible. Failure is accompanied by tilting of footing.

The length of disturbance beyond the edge of footing is large State of plastic equilibrium is reached initially at the footing edge and spreads gradually downwards and outwards. General shear failure is accompanied by low strain (<5%) in a soil with considerable ϕ ($\phi > 36^o$) and large N (N > 30) having high relative density(ID > 70%).

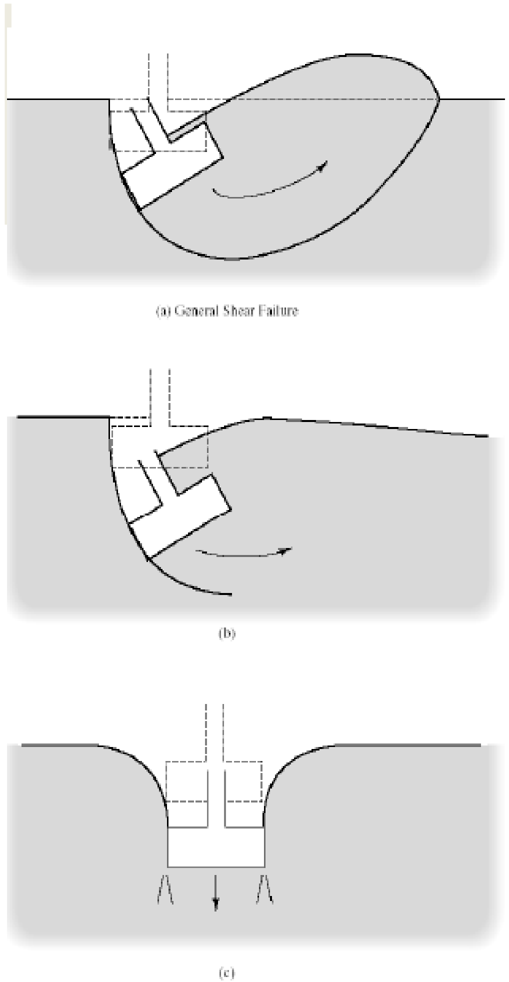
ID- Density index or relative density)

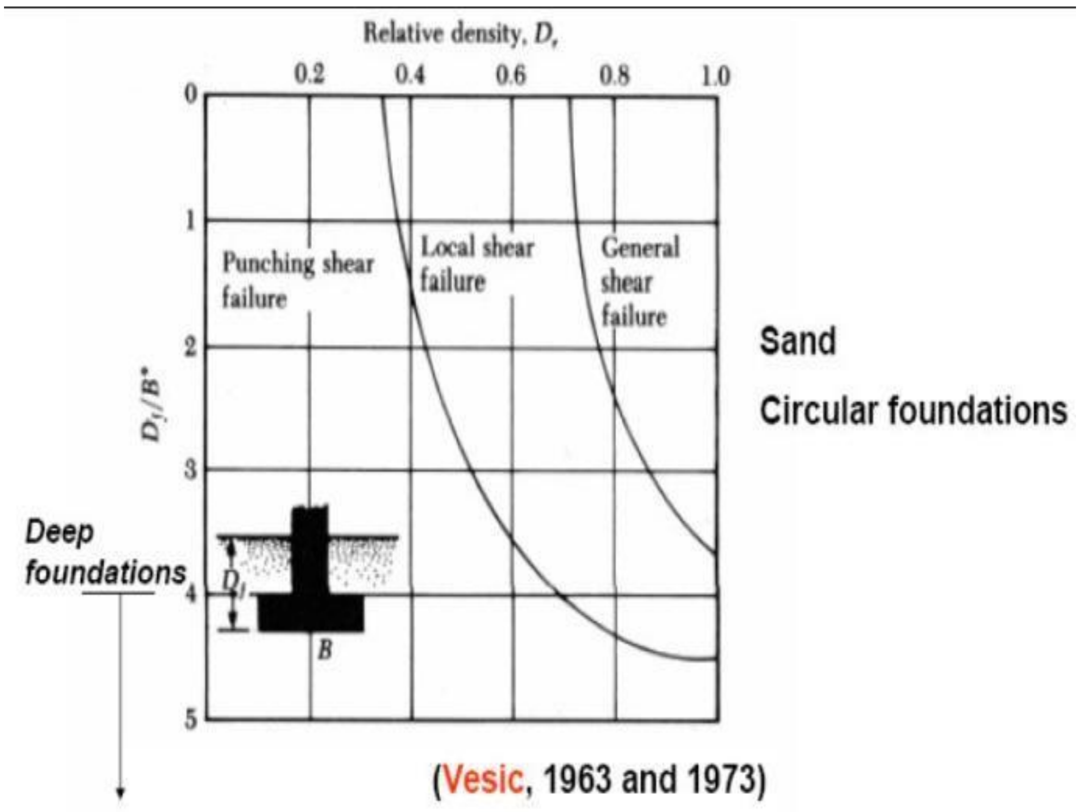
N- Standard penetration test N value

PUNCHING SHEAR FAILURE

This type of failure is seen in loose and soft soil and at deeper elevations. The following are some characteristics of general shear failure.

1.This type of failure occurs in a soil of very high compressibility.
2. Failure pattern is not observed.
3. Bulging of soil around the footing is absent.
4. Failure is characterized by very large settlement.
5.Continuous settlement with no increase P is observed in P – curve.


(a) General Shear Failure
(b)
(c)

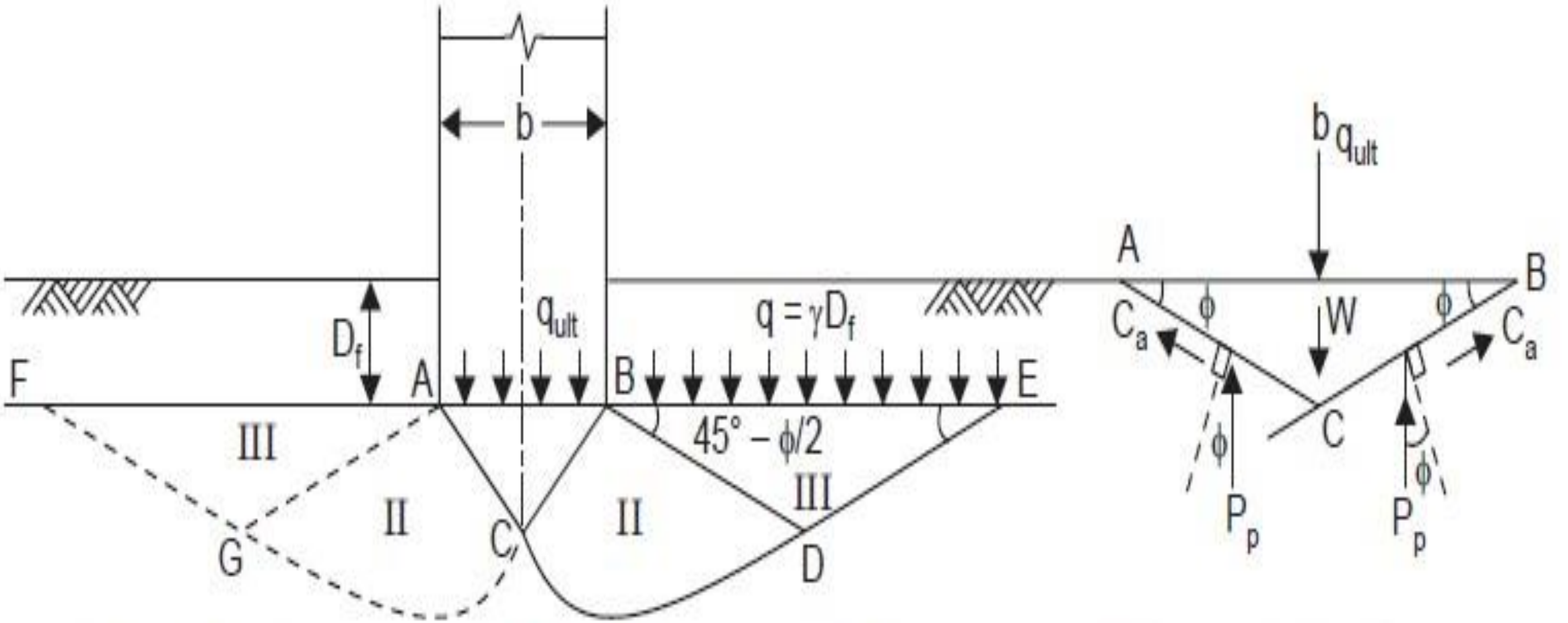

Relative density, D_r
0.2
0.4
0.6
0.8
1.0
0
1
2
3
4
5
D_f/B^*
Punching shear failure
Local shear failure
General shear failure
D_f
B
Deep foundations
Sand
Circular foundations
(Vesic, 1963 and 1973)

TERZAGHI BEARING CAPACITY

Terzaghi (1943) was the first to propose a comprehensive theory for evaluating the safe bearing capacity of shallow foundation with rough base. He extended the theory of Prandtl

Assumptions

1. Soil is semi infinite, homogeneous and Isotropic.
2. The shear strength of soil is represented by Mohr Coulombs Criteria.
3. The footing is of strip footing type with rough base. It is essentially a two dimensional plane strain problem.



(a) Terzaghi system for ideal soil, rough base and surcharge (b) Forces on the elastic wedge

1.The inclination of sides *ac* and *bc* of the wedge with the horizontal is ϕ(soil friction angle).

2.Zone *bcf*. This zone is the Prandtl's radial shear zone.

3. Zone*bfg*. This zone is the *Rankine passive zone*. The *slip lines* in this zone make angles of $(45 - \phi/2)$ with the horizontal.

RELATIONSHIP FOR P_{pq} ($\phi \neq 0, \gamma = 0, q \neq 0, c = 0$)

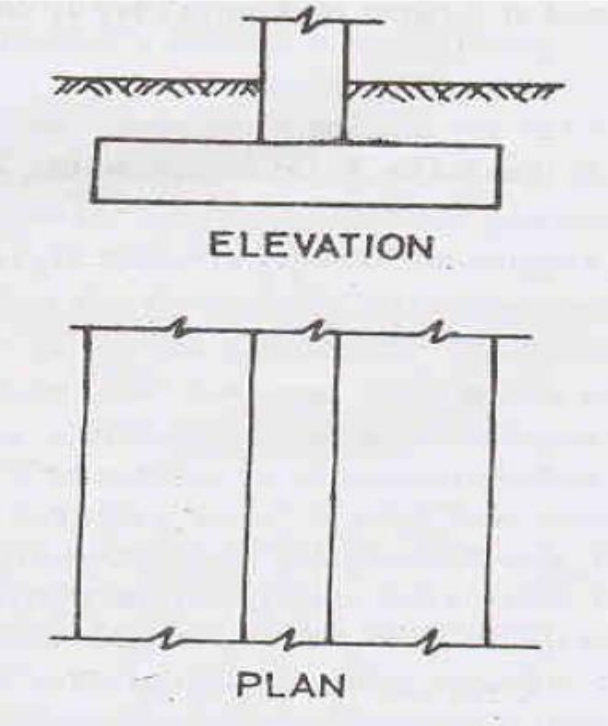
RELATIONSHIP FOR P_{pc} ($\phi \neq 0, \gamma = 0, q = 0, c \neq 0$)

RELATIONSHIP FOR $P_{p\gamma}$ ($\phi \neq 0, \gamma \neq 0, q = 0, c = 0$)

$$q_u = q_q + q_c + q_\gamma$$

$$q_u = cN_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

N_c, N_q, and N_γ = bearing capacity factors, and



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

$$N_\gamma = \frac{1}{2}\tan\phi'\left[\frac{K_{P\gamma}}{\cos^2\phi'}-1\right]$$

FACTORS AFFECTING BEARING CAPACITY

Table 5.1 Bearing capacity factors of Terzaghi

ϕ^o	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	0.14
10	9.6	2.7	1.2
15	12.9	4.4	1.8
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	360.0
50	347.5	415.1	1072.8

FACTORS AFFECTING BEARING CAPACITY

Bearing capacity of soil depends on many factors. The following are some important ones.

1. Type of soil
2. Unit weight of soil
3. Surcharge load
4. Depth of foundation
5. Mode of failure
6. Size of footing
7. Shape of footing
8. Depth of water table
9. Eccentricity in footing load
10. Inclination of footing load
11. Inclination of ground
12. Inclination of base of foundation

EFFECT OF SHAPE

7.4.1 Circular footing

$$q_f = 1.3cN_c + \gamma DN_q + 0.3\gamma BN_\gamma$$

7.4.2 Square footing

$$q_f = 1.3cN_c + \gamma DN_q + 0.4\gamma BN_\gamma$$

7.4.3 Rectangular footing

$$q_f = (1 + 0.3\frac{B}{L})cN_c + \gamma DN_q + (1 - 0.2\frac{B}{L})0.5\gamma BN_\gamma$$

EFFECT OF WATER TABLE

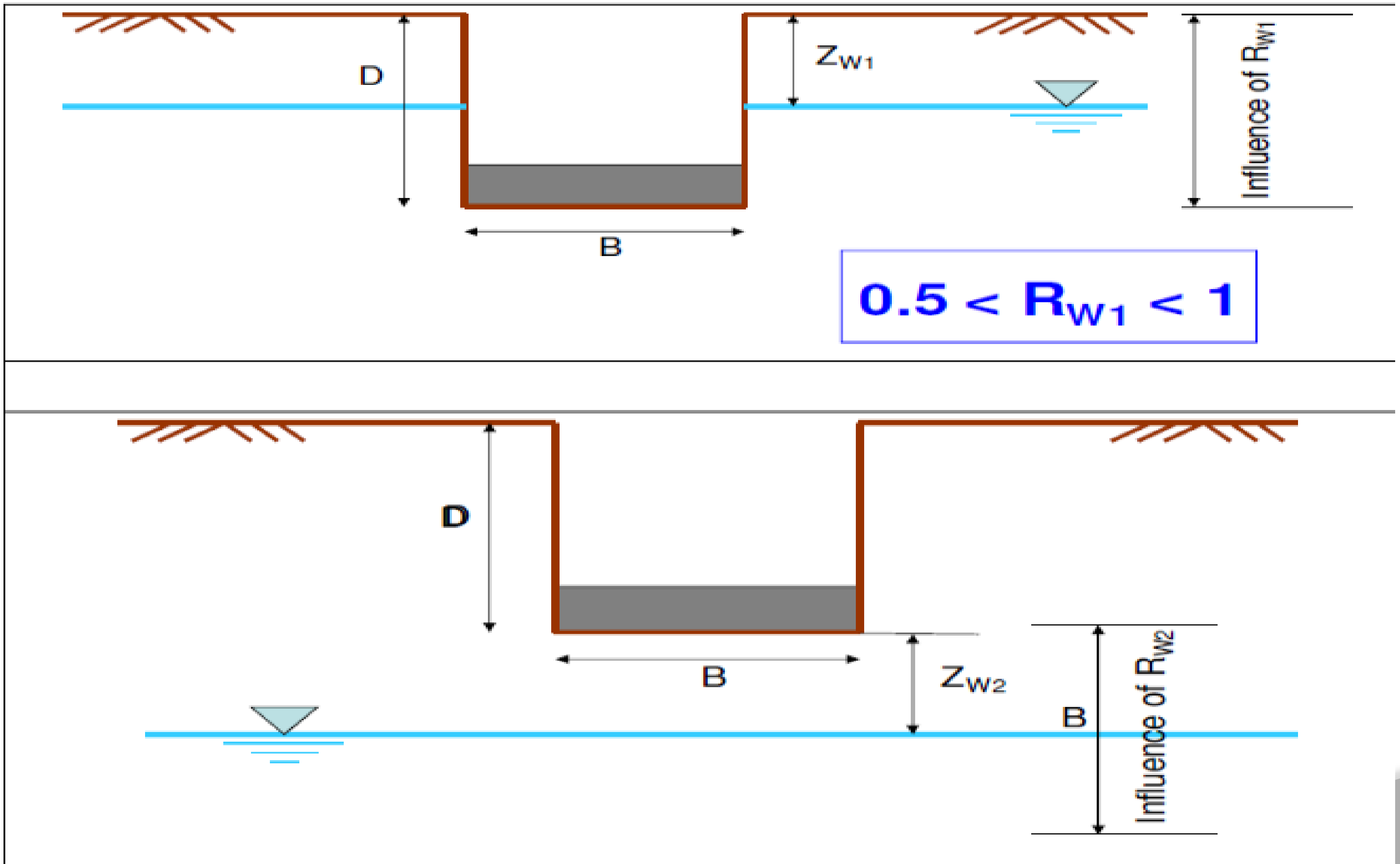


Fig. 7.5 : Effect of water table on bearing capacity

EFFECT OF WATER TABLE

Ultimate bearing capacity with the effect of water table is given by,

$$q_f = cN_c + \gamma DN_q R_{w1} + 0.5\gamma BN_\gamma R_{w2}$$

Here, $R_{w1} = \frac{1}{2}\left[1 + \frac{Z_{w1}}{D}\right]$

where Z_{W1} is the depth of water table from ground level.

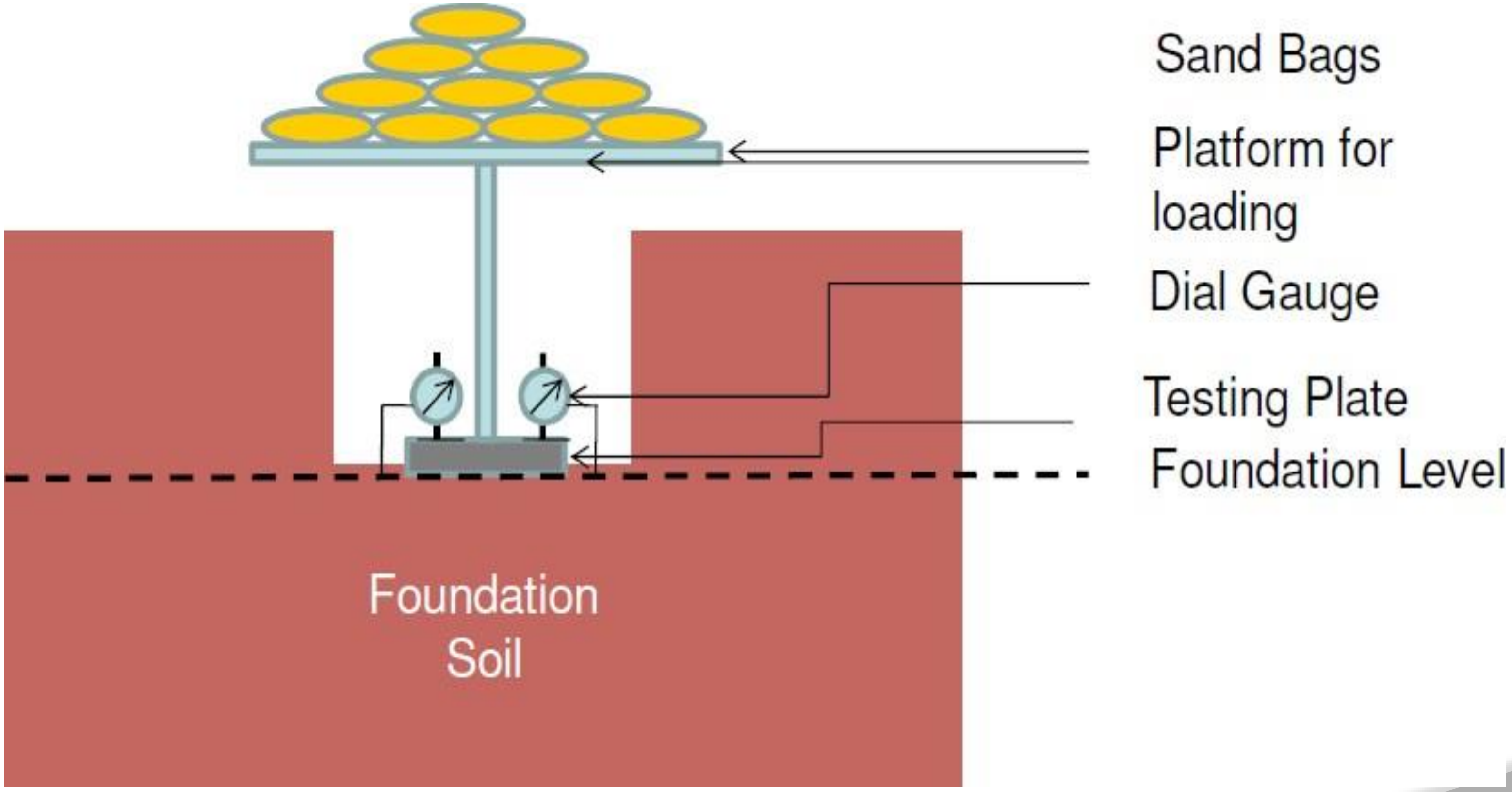
1. $0.5 < R_{w1} < 1$
2. When water table is at the ground level ($Z_{w1} = 0$), $R_{w1} = 0.5$
3. When water table is at the base of foundation ($Z_{w1} = D$), $R_{w1} = 1$
4. At any other intermediate level, R_{w1} lies between 0.5 and 1

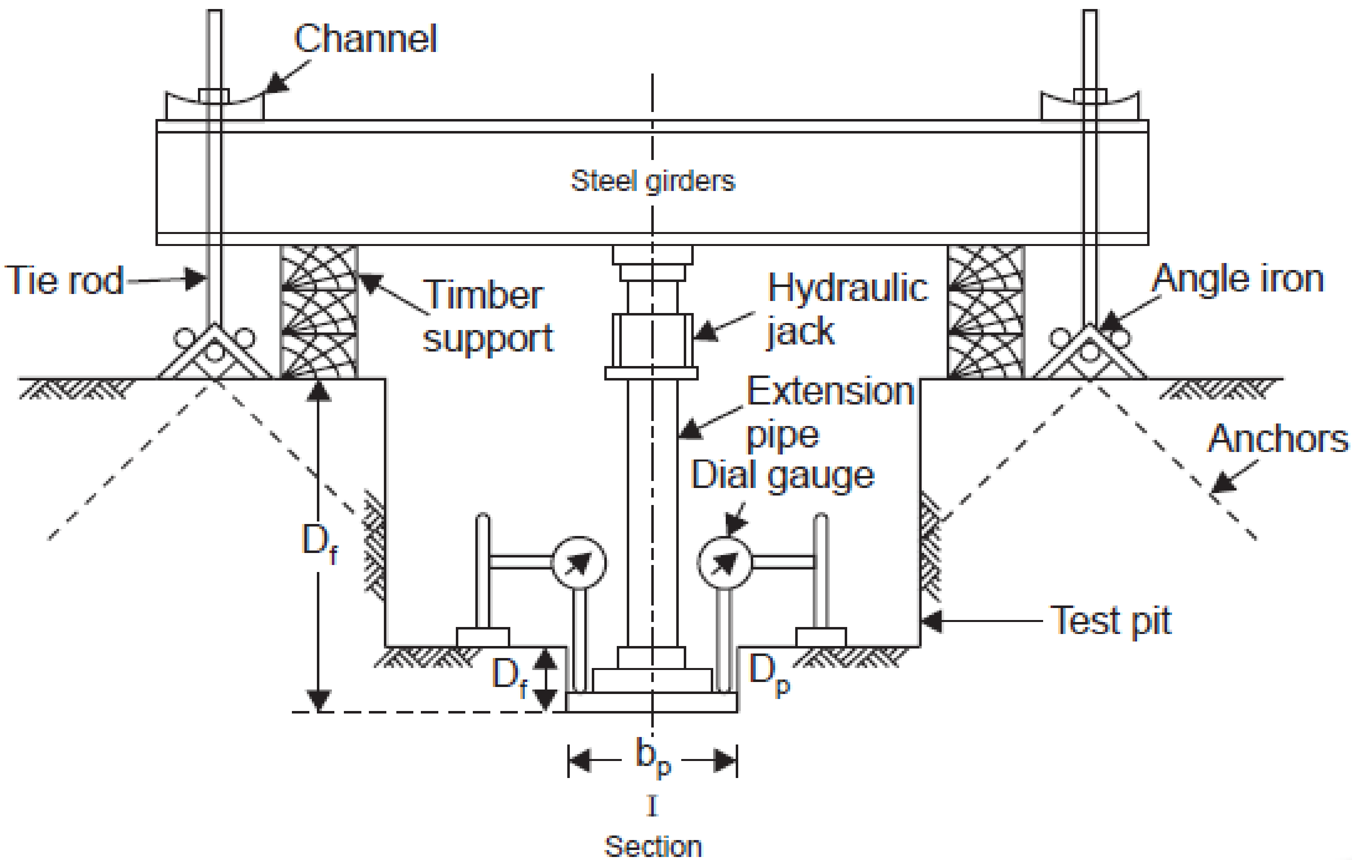
Here, $R_{w2} = \frac{1}{2}\left[1 + \frac{Z_{w2}}{B}\right]$

where Z_{W2} is the depth of water table from foundation level.

1. $0.5 < R_{w2} < 1$
2. When water table is at the base of foundation ($Z_{w2} = 0$), $R_{w2} = 0.5$
3. When water table is at a depth B and beyond from the base of foundation ($Z_{w2} >= B$), $R_{w2} = 1$
4. At any other intermediate level, R_{w2} lies between 0.5 and 1

EFFECT OF WATER TABLE





Channel
Steel girders
Tie rod
Timber support
Hydraulic jack
Angle iron
Extension pipe
Anchors
Dial gauge
D_f
Test pit
D_f
D_p
b_p
I
Section

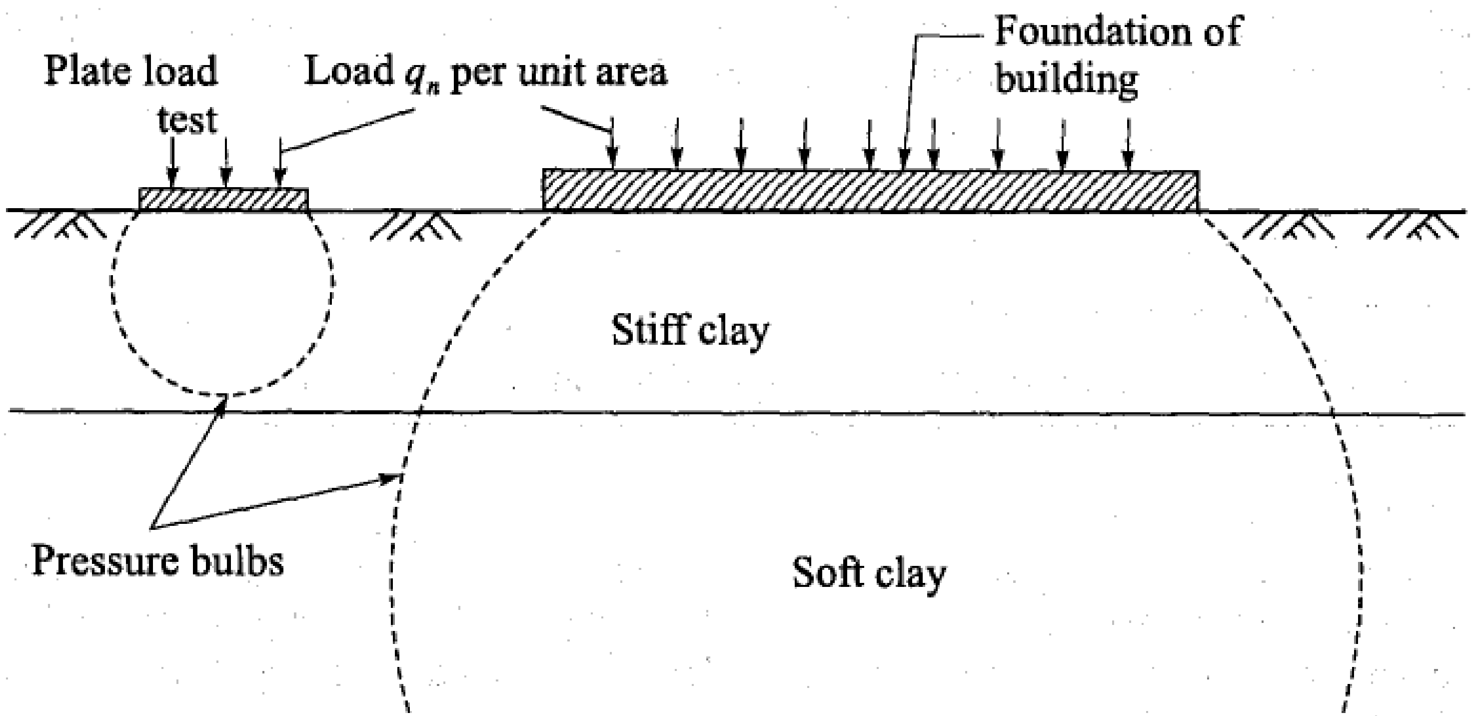


Fig. 6.2 (c) Plate load test on non-homogeneous soil

Since a load test is of short duration, consolidation settlements cannot be predicted. The test gives the value of immediate settlement only. If the underlying soil is sandy in nature immediate settlement may be taken as the total settlement. If the soil is a clayey type, the immediate settlement is only a fraction of the total settlement. Load tests, therefore, do not have much significance in clayey soils to determine allowable pressure on the basis of a settlement criterion.

Plate load tests should be used with caution and the present practice is not to rely too much on this test. If the soil is not homogeneous to a great depth, plate load tests give very misleading results.

Assume, as shown in Fig. 6.2 (c), two layers of soil. The top layer is stiff clay whereas the bottom layer is soft clay. The load test conducted near the surface of the ground measures the characteristics of the stiff clay but does not indicate the nature of the soft clay soil which is below. The actual foundation of a building however has a bulb of pressure which extends to a great depth into the poor soil which is highly compressible. Here the soil tested by the plate load test gives results which are highly on the unsafe side.

A plate load test is not recommended in soils which are not homogeneous at least to a depth equal to 1½ to 2 times the width of the prototype foundation.

Plate load tests should not be relied on to determine the ultimate bearing capacity of sandy soils as the scale effect gives very misleading results. However, when the tests are carried on clay soils, the ultimate bearing capacity as determined by the test may be taken as equal to that of the foundation since the bearing capacity of clay is essentially independent of the footing size.

Determination of bearing capacity from plate load test

The size effect has been empirically evolved in the form of the following equation (Terzaghi and Peck, 1948):

$$\frac{S}{S_p} = \left[\frac{b(b_p + 0.3)}{b_p(b + 0.3)}\right]^2 \qquad \text{...(Eq. 14.110)}$$

where S = settlement of the proposed foundation (mm), S_p = settlement of the test plate (mm), (same units)

b = size of the proposed foundation (m), and

b_p = size of the test plate (m).

This is applicable for sands.

However, the relationship is simpler for clays, since the modulus value E_s, for clays is reasonably constant:

$$\frac{S}{S_p} = \frac{b}{b_p} \qquad \text{...(Eq. 14.111)}$$

ALLOWABLE BEARING CAPACITY

The allowable bearing capacity shall be taken as either of the following, whichever is less:

- Net ultimate bearing capacity divided by suitable factor of safety, that is, net safe bearing capacity
- The net soil pressure that can be imposed on the base without the settlement exceeding the permissible values as given in IS:1904-1978 to be determined for each structure and type of soil, that is, safe bearing pressure

SETTLEMENT

Secondary consolidation (Ss)
Total Settlemt – Si+ Sc + Ss

(iii)Ground water lowering, especially repeated lowering and raising of ground water level in loose granular soils and drainage without adequate filter protection,

(iv)Vibration due to pile driving, blasting and oscillating machinery in granular soils,

(v) Seasonal swelling and shrinkage of expansive clays,

(vi) Surface erosion, creep or landslides in earth slopes,

(vii)Miscellaneous sources such as adjacent excavation, mining subsidence and underground erosion.

METHODS TO COMPUTE SETTLEMENT

Elastic settlement

- Theory of elasticity
- Jambu et al
- Schmertmann‘s method
- Pressure meter method

Consolidation settlement

- e- log p curve from oedometer test

CONSTRUCTION PRACTICES TO AVOID

Design of the structure and foundation ... desired degree of flexibility of the various component parts of a large structure may be introduced in the construction.

(i)Choice of a suitable type of foundation for the structure and the foundation soil conditions...*e.g.*, large, heavily loaded structures on relatively weak and non-uniform soils may be founded on ‗mat' or ‗raft' foundations. Sometimes, piles and pile foundations may be used to bypass weak strata.

(ii)Treatment of the foundation soil...to encourage the occurrence of settlement even before the construction of the structure, *e.g.,* (*a*) Dewatering and drainage, (*b*) Sand drains and (*c*) Preloading.

(iii)Provision of plinth beams and lintel beams at plinth level and lintel level in the case of residential buildings to be founded on weak and compressible strata.

ALLOWABLE ETTLEMENT

- The differential settlement should not exceed 75% of the maximum settlement
- Maximum settlement range from 20 mm to 300 mm
- ρ> 150 mm damages the utilities
- IS 1904 (1966)- Permissible settlement
- Isolated footing

  On sand -40mm

  On clay -65 mm
- Raft

  On sand -40mm to 65mm

  On clay – 65 to 100 mm

Table 23.9. Maximum and Differential Settlements (IS : 1904—1978)

	Sand and hard clay			Plastic clay		
	Max-Settlement	*Diff-Settlement*	*Angular Distortion*	*Max-Settlement*	*Diff-Settlement*	*Angular Distortion*
(a) Isolated Foundations						
(i) Steel structure	50 mm	0.0033 L	1/300	50 mm	0.0033 L	1/300
(ii) R.C.C. structures	50 mm	0.0015 L	1/666	75 mm	0.0015 L	1/666
(b) Raft Foundations						
(i) Steel Structures	75 mm	0.0033 L	1/300	100 mm	0.0033 L	1/300
(ii) R.C.C. Structures	75 mm	0.002 L	1/500	100 mm	0.002 L	1/500

SETTLEMENT CALCULATION

$$s_i = qB\left(\frac{1-\mu^2}{E_s}\right)I$$

The value of E_s can be determined from the standard penetration number (N) using the following equations given by Schmertmann (1970).

$$E_s = 766\,N \text{ (kN/m}^2\text{)} \qquad \text{...(23.70)}$$

Alternatively, it can be estimated from the static cone penetration resistance (q_c) as

$$E_s = 2\,q_c \qquad \text{...(23.71)}$$

Table 6.3 Typical range of values for Poisson's ratio (Bowles, 1996)

Type of soil	*m*
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand (dense)	0.2–0.4
Coarse (void ratio 0.4 to 0.7)	0.15
Fine grained (void ratio = 0.4 to 0.7)	0.25
Rock	0.1–0.4

Settlement calculation from e-log p curves

A general equation for computing oedometer consolidation settlement may be written as follows.

Normally, consolidated clays

$$S_c = H \frac{C_c}{1+e_0} \log \frac{p_0 + \Delta p}{p_0} \tag{6.41}$$

Overconsolidated clays

for $p_0 + \Delta p < p_c$

$$S_c = H \frac{C_s}{1+e_0} \log \frac{p_0 + \Delta p}{p_0} \tag{6.42}$$

for $p_0 < p_c < p_0 + \Delta p$

$$S_c = \frac{H}{1+e_0} \left(C_s \log \frac{p_c}{p_0} + C_c \log \frac{p_0 + \Delta p}{p_c} \right) \tag{6.43}$$

where C_s = swell index, and C_c = compression index.

If the thickness of the clay stratum is more than 3 m the stratum has to be divided into layers of thickness less than 3 m. Further, e_0 is the initial void ratio and p_0, the effective overburden pressure corresponding to the particular layer; Δp is the increase in the effective stress at the middle of the layer due to foundation loading which is calculated by elastic theory. The compression index, and the swell index may be the same for the entire depth or may vary from layer to layer.

Settlement calculation from e-p curve

Equation (6.43) can be expressed in a different form as follows:

$$S_c = \sum H m_v \Delta p \tag{6.44}$$

where m_v = coefficient of volume compressibility.

(*a*) For undisturbed soils, $C_c = 0.009\,(w_L - 10)$
(*b*) For remoulded soils, $C_c = 0.007\,(w_L - 10)$
where w_L = liquit limit (%).

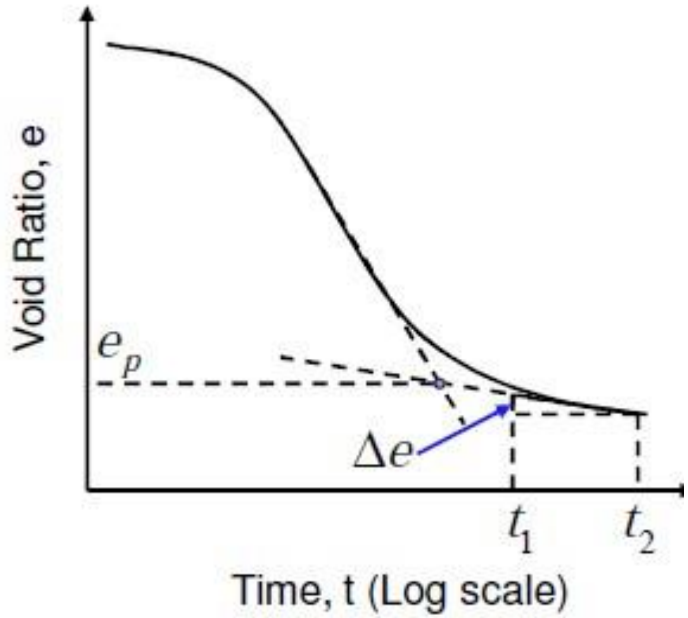
Settlement Due to Secondary Consolidation

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

$C_\alpha =$ Secondary Compression Index $= \dfrac{\Delta e}{\log(t_2/t_1)}$

$e_p =$ Void ratio at the end of primary consolidation

$H_c =$ Thickness of Clay Layer



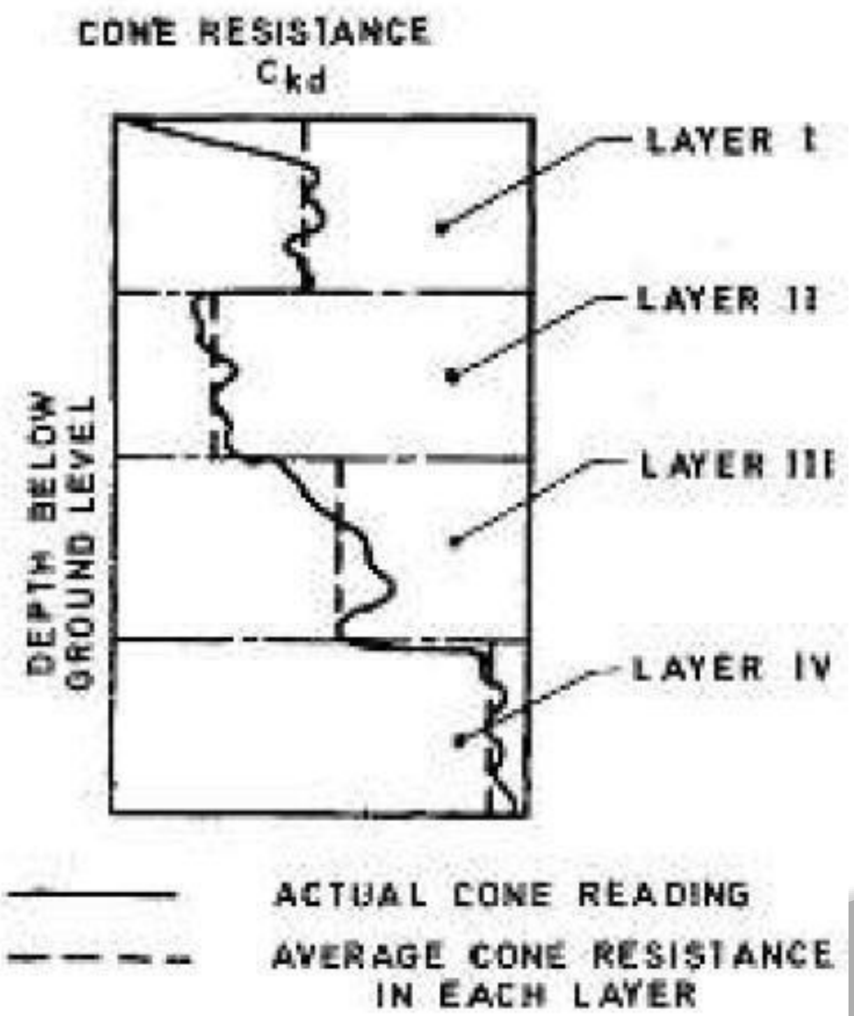
Secondary consolidation settlement is more important in the case of organic and highly-compressible inorganic clays

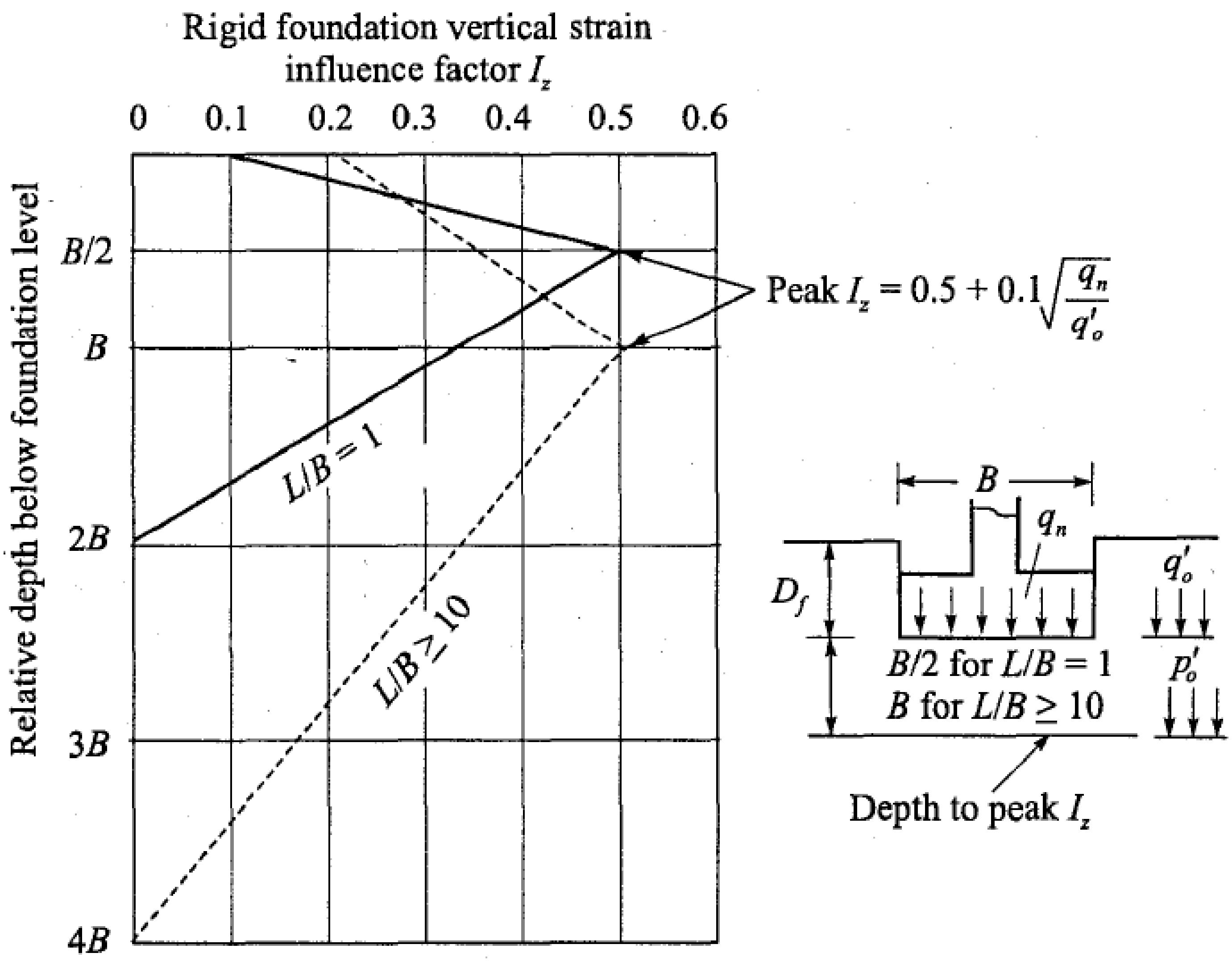
Total Settlement from CPT Data for Cohesionless soil

$$S_t = \frac{H_t}{C} \ln\left[\frac{\sigma_o + \Delta\sigma}{\sigma_o}\right]$$

$$C = \frac{3}{2}\left(\frac{q_c}{\sigma_o}\right)$$

- Depth profile of cone resistance can be divided in several segments of average cone resistance
- Average cone resistance can be used to calculate constant of compressibility.
- Settlement of each layer is calculated separately due to foundation loading and then added together





Rigid foundation vertical strain influence factor I_z
0 0.1 0.2 0.3 0.4 0.5 0.6
Relative depth below foundation level
B/2
B
2B
3B
4B
L/B = 1
L/B ≥ 10
Peak $I_z = 0.5 + 0.1\sqrt{\frac{q_n}{q'_o}}$
B
q_n
q'_o
D_f
B/2 for L/B = 1
B for L/B ≥ 10
p'_o
Depth to peak I_z

- Depth/Width ≥ 4
- Low Bearing Capacity of soil .
- Non availability of proper bearing stratum at shallow depths.
- Heavy loads from the super structure for which shallow foundation may not be economical or feasible.

- A timber, steel or reinforced concrete post usually vertical, used as a structural element for transferring the loads at the required depth in the deep foundations is called PILE.
- These are the long slender members either driven or cast-in-situ and may be subjected to vertical or lateral or vertical plus lateral loads.

USES OF PILS

1. End Bearing or compressive strength: To transfer the load through a soft soil to a suitable bearing stratum by means of end bearing of the piles.
2. Scour depth. To transfer the load through Water, for any hydraulic structure because in this case, we have to keep the foundation at the scour depth below the bed level. For **River Ravi** Scour depth is 30 to 35m below the bed. So if we go for the shallow foundation, we will have to make an open pit, coffer dam diversion of River etc. and it is highly uneconomical.
3. Tension or Uplift: For a very tall structure (tower), even if the Soil is very good, but here the overturning is the problem. So either make the base very large (Thick raft) or make deep foundation.

4) Vibration Control: if then to absorb the vibrations either make a massive block or the next choice is deep foundation, But Massive black is very expensive. e.g. At Terbela the shaft of Turbine is 2m and when it runs there area a lot of vibrations.
5) Compaction Piles: In order to compact the granular soils and to increase their bearing capacity, piles are used (compaction Piles).
6) Anchor Piles: To provide Anchorage against horizontal pull from sheet piling walls or other pulling forces.
7) Fender piles: To protect Water front structure against impact from ships or other floating objects.
8) Batter piles: To resist large horizontal or inclined forces.
9) Rapid Construction: Piles can

TYPES OF PILES

1. Mode of construction
2. Material of construction
3. Material of load
4. Function of pile
5. Shape
6. Size

CLASSIFICATION BASED ON CONSTRUCTION

Cast in-situ Piles(Bored Piles)

- Under sized Bore.(It is feasible because of less noise , under sized hole is dug and full size pile is driven, (NABWI MOSQUE PILES).
- By driving the piles, the soil is displaced so type is

a) High volume displaced piles (vol. almost equal to vol. of pile).

b) No volume displaced piles.

c) Low volume displaced piles.

CLASSIFICATION WITH RESPECT TO FUNCTION

1) Compression pile (To resist the comp. load)
2) Tension pile or Anchor pile
3) Compaction piles granular soil i.e. very loose sand can be compacted by driving the piles at one place, then are pulled out and driven at the next place, in this way sand is densified).
4) Fender piles (Used near sea-part to protect the Harbour, just to absorb the impact of floating objects)
5) Batter piles (Provided at an inclination their stability is more against overturning).
6) Sheet piles.

(To reduce seepage or to provide lateral stability).

CLASSIFICATION WITH RESPECT TO SHAPE

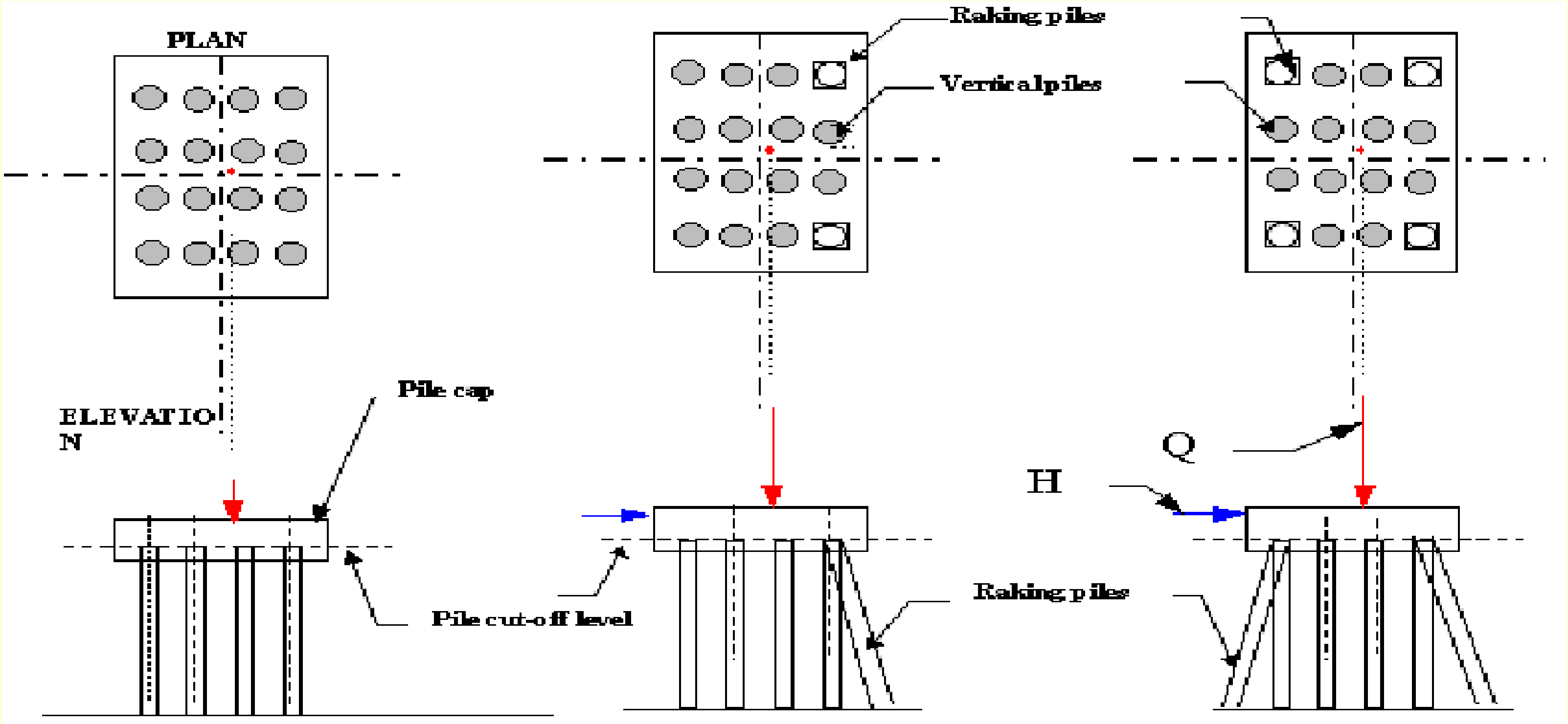
1. Round Piles
2. Square Piles
3. Octagonal Piles
4. I-Shaped Piles
5. Straight Piles
6. Tapered Piles
7. Bell-Bottom Piles
8. Screw Piles

CLASSIFICATION WITH RESPECT TO SIZE

1. Large Dia Pile:
2. Small Dia Pile:
3. Micro Dia Pile:

PILES GROUPS

Piles usually exist as groups which are all integrated by means of a pile cap as shown in the below schematic



LOAD CARRYING MECHANISAM OF PILES

The ultimate bearing capacity of a pile is the maximum load which it can carry without failure or excessive settlement of the ground. The bearing capacity of a pile depends primarily on 3 factors as given below, Type of soil through which pile is embedded.

mechanism such as Floating Piles or Friction Piles and End-bearing or Point bearing piles. capacity is significantly comprised of skin friction then the pile is classified as Friction piles.
Axial Load transfer with depth in a friction pile. Variation of friction around the Pile with depth.

SCHEMATICS SHOWING AXIAL LOAD CARRYING MECHANISM

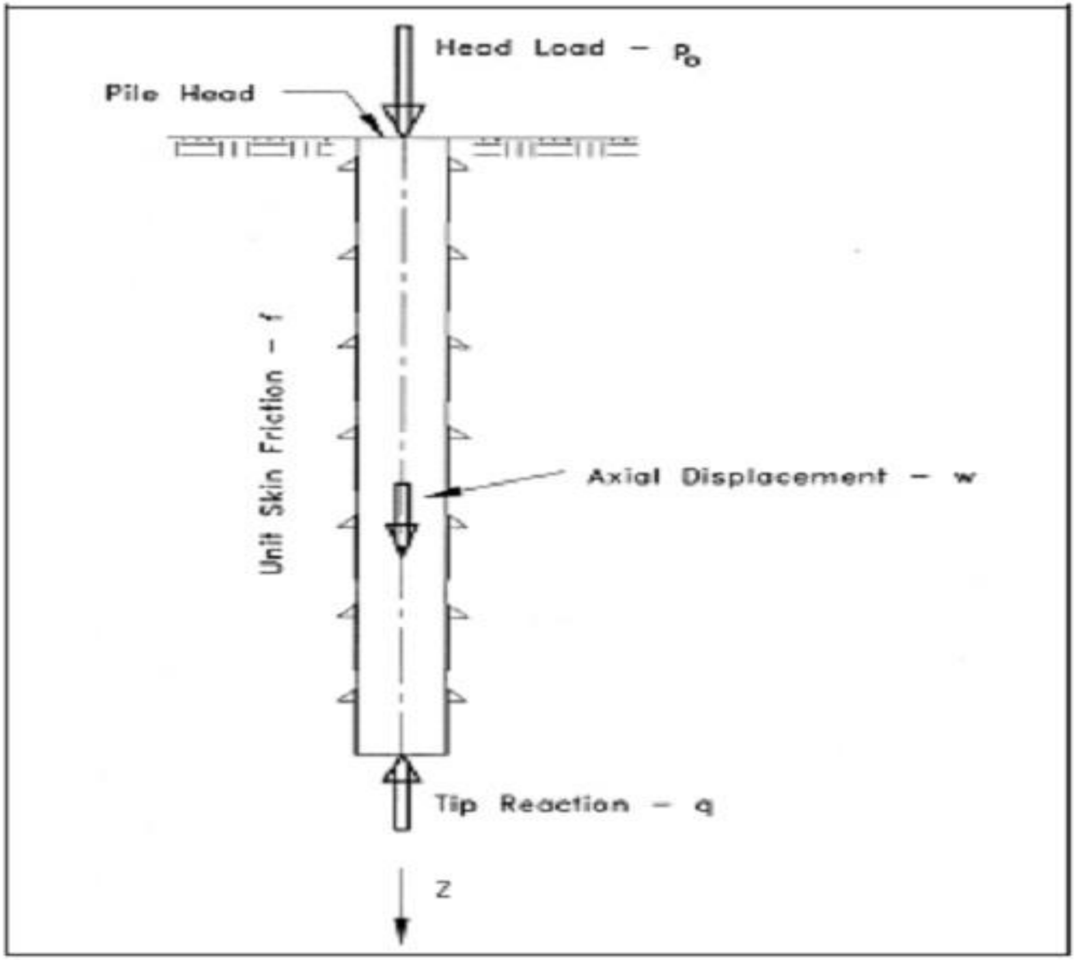


Figure 1. Axially loaded pile

PILES CAPACITIES

AXIAL CAPACITY

LATERAL CAPACITY

PULL OUT OR TENSION CAPACITY

Pile load tests are usually carried out for the following main reasons:

- To obtain back figured soil data that will enable other piles to be designed

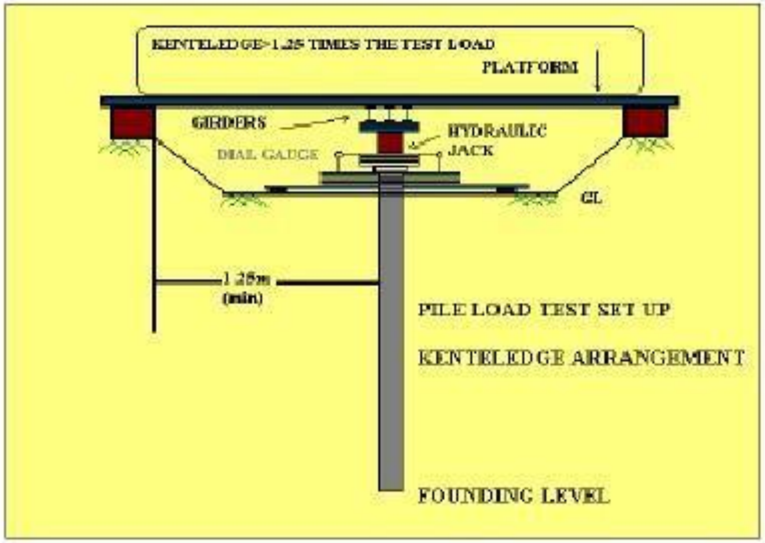
- To confirm pile lengths and hence contract costs before the client is committed to over all job costs

- To counter check results from geotechnical and pile driving formulae

- To determine the load-settlement behavior of the pile, especially in the region of anticipated working load, that the data can be used for prediction of group settlement.

- To verify the structural soundness of the pile.

Field setup for a Static Axial compressive load test on a single pile



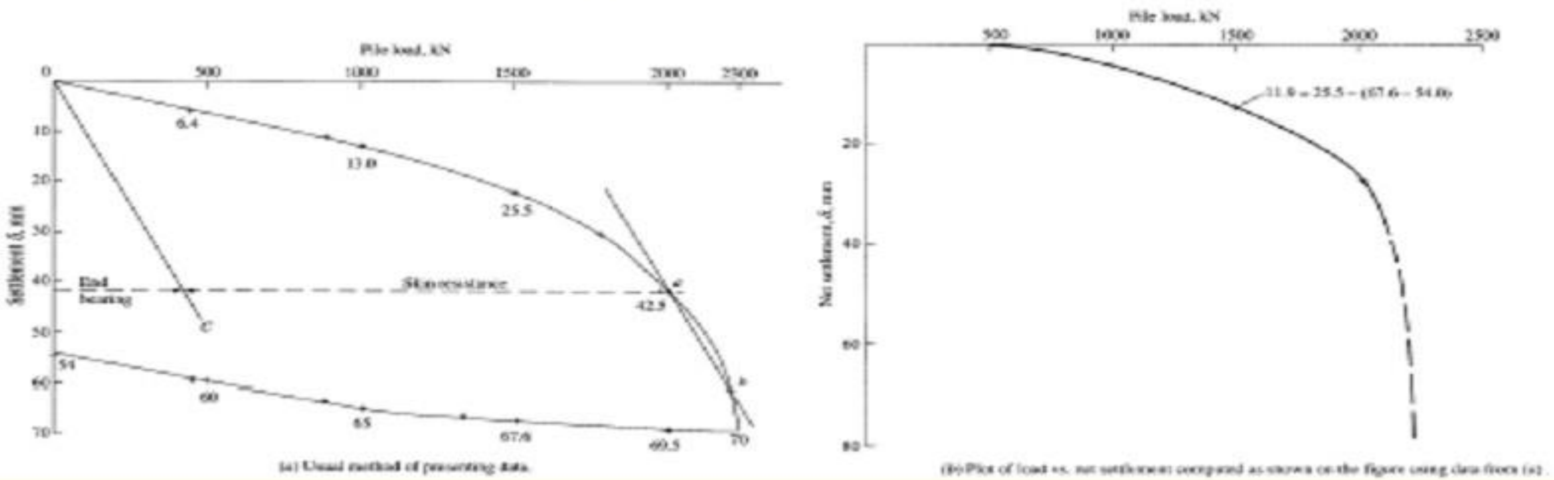
Test Schematic

Real time field set up

Determination of pile axial compressive capacity from static load test

The Pile load test data is presented as shown below

Figure 10



From the above plots the ultimate pile load is commonly taken as the load where the <u>load settlement curve approaches a vertical asymptote</u>

UNIT-V
WELL FOUNDATIONS

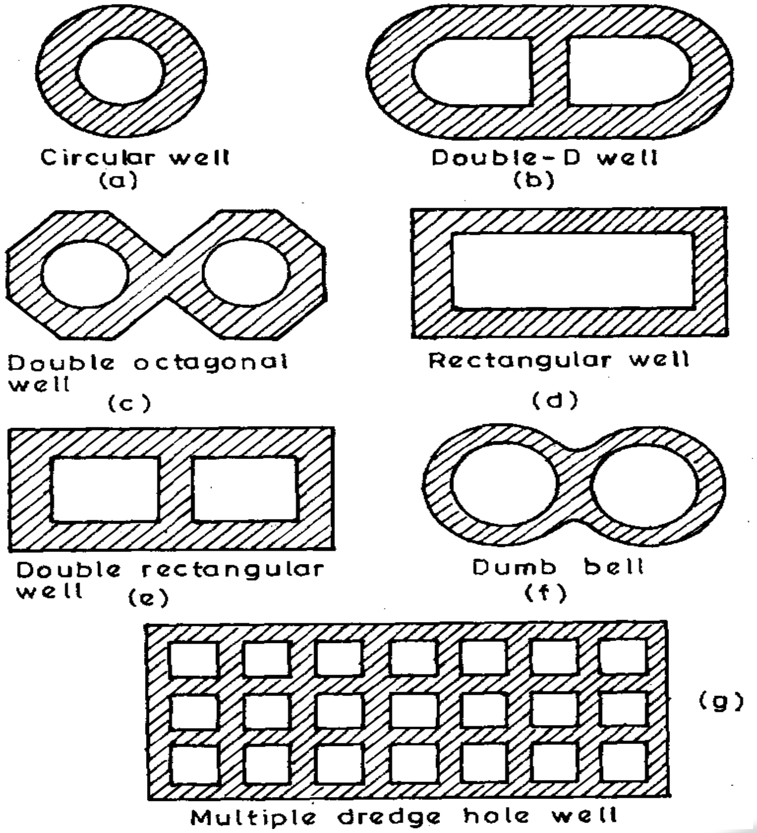
CLOs	Course Learning Outcome
CLO23	Learn different shapes of well & components of Well Foundation
CLO24	Understand the principle of analysis and design of wells, Seismic analysis and IRC guidelines

WELL FOUNDATION

- Well foundation is a box of timber, metal, reinforced concrete or masonry which open both at the top and bottom, and is used for building for building and bridge foundations.
- Well foundations are being used in India from very early days. Taj Mahal was built on such foundations.

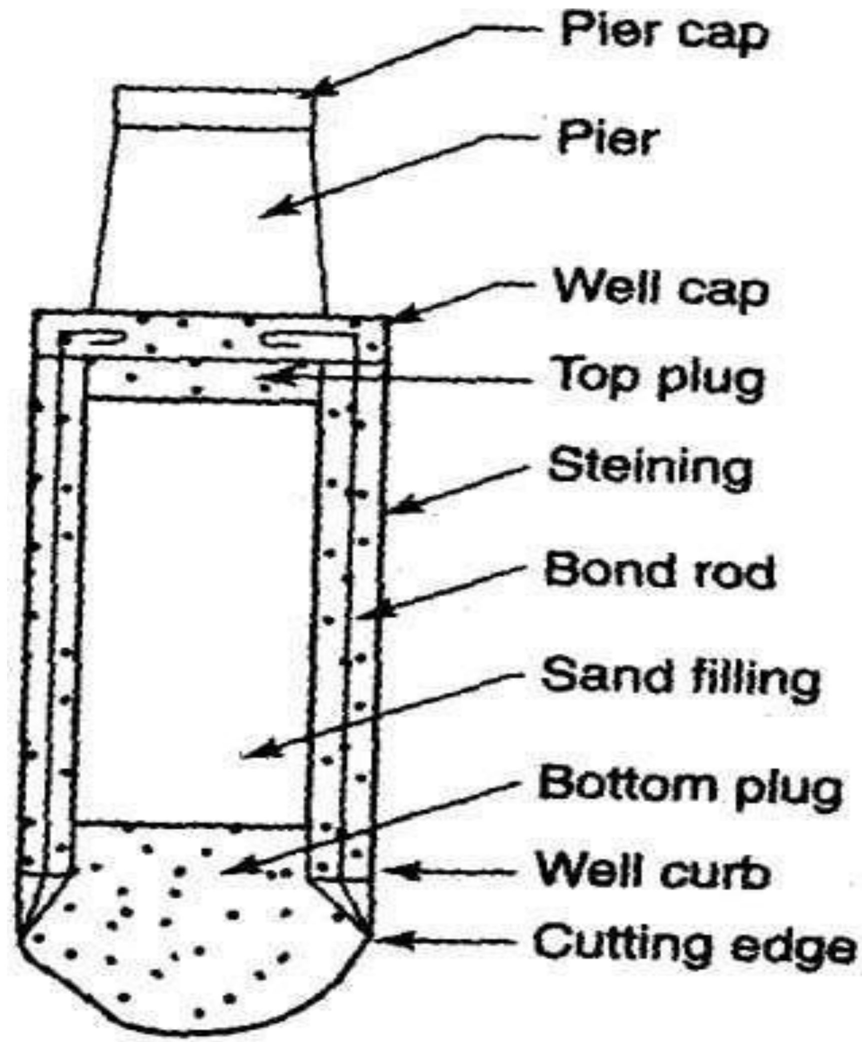
TYPES OF WELL SHAPES

- Circular well
- Rectangular well
- Double Rectangular well
- Double Octagonal well
- Double – D well
- Twin circular well



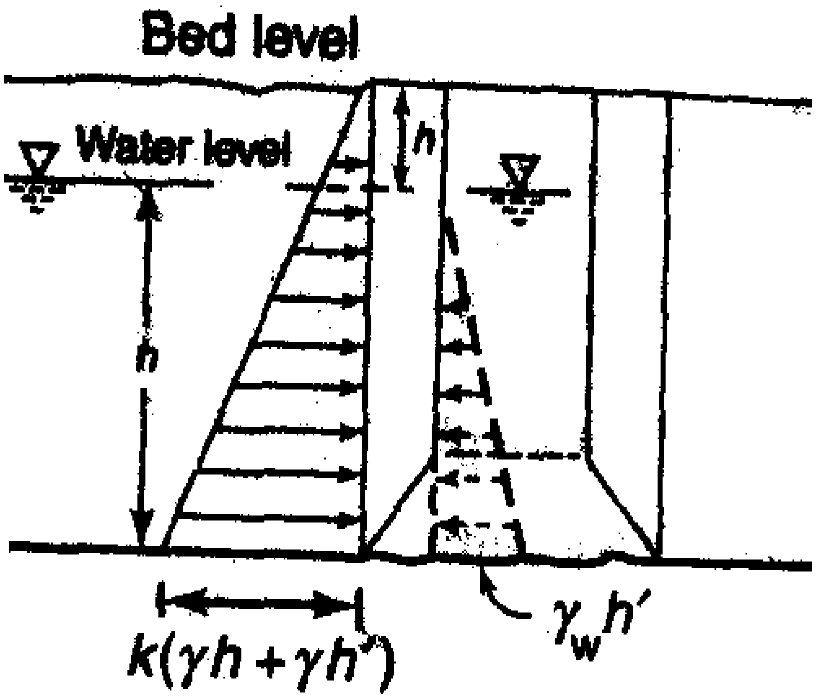
COMPONENTS OF WELL FOUNDATION

- Cutting Edge
- Well Curb
- Bottom Plug
- Steining
- Top Plug
- Well Cap



STEINING

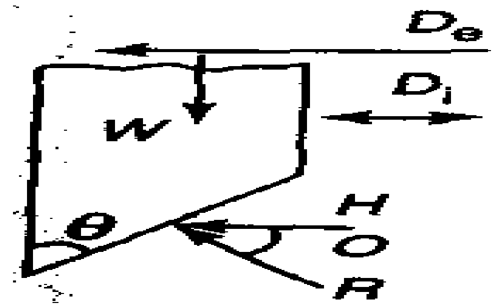
- Walls of the wells are known as **steining**
- Made of brick masonry, stone masonry, plain or reinforced concrete
- The design of steining reinforcement rely on skin friction & unit weight of well
- The thickness of steining is designed in such a manner that all platforms of well are sunk under its own weight



Pressure distribution on steining

CURB

- The curb of a well transfers all the superimposed loads to the soil through the cutting edge while sinking. The material used for curbs may be timber or RCC. The forces acting on well curb are shown in Fig. The total horizontal force on the well curb on both sides is



D_e — external diameter
D_i — internal diameter

Force acting on curb

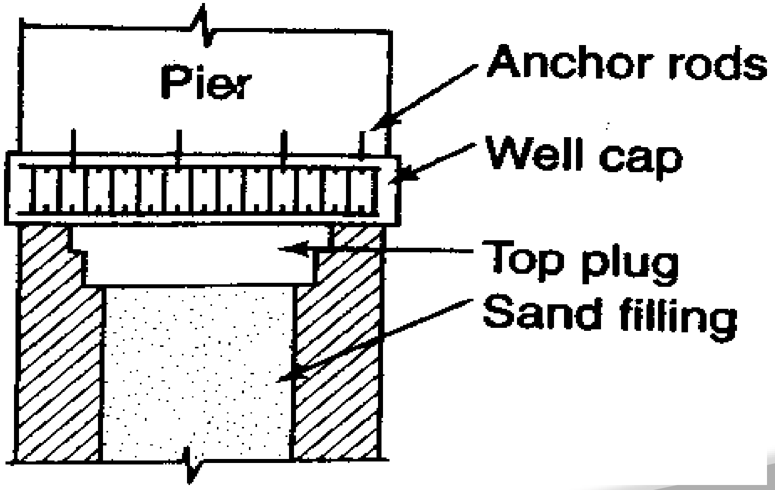
SAND FILLING

The bottom plug concrete is cured and after curing, the well is filled with sand in saturated condition. Sand filling provides

1. Stability to the bottom of the well.
2. Eliminate the tensile forces at the base

TOP PLUG

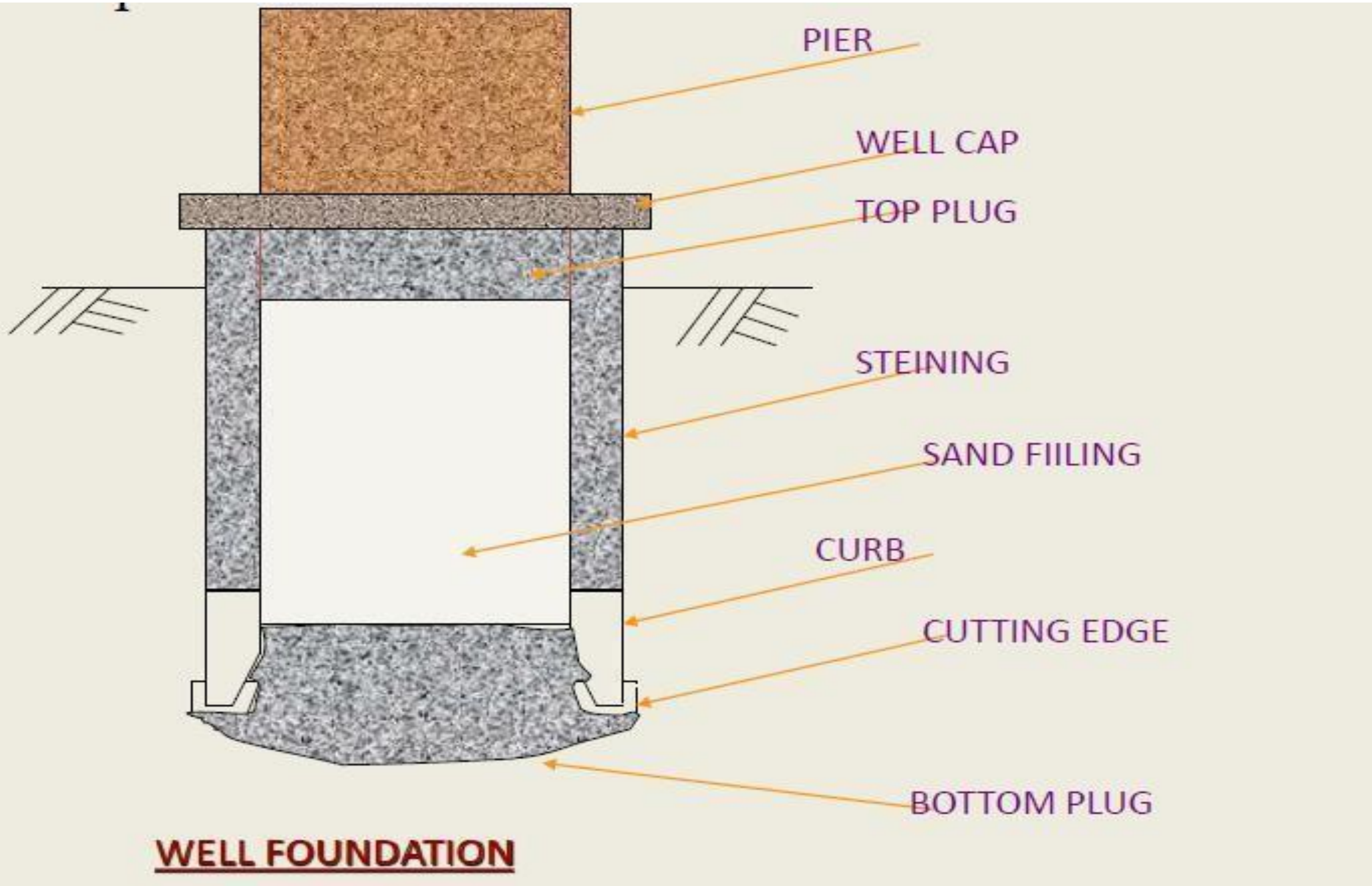
The top plug is provided after the filling is completed. Top plug helps in transferring the load of the pier and superstructure to the steining. The thickness of the top plug is generally kept greater than 50 % of the smaller dimension of the dredge hole. If sand filling is used, the top plug is simply constructed using PCC of 1:2:4 otherwise it is reinforced with steel bars and lean concrete of 1:3:6 is used.



WELL CAP

- Well cap is constructed as a slab resisting on the well it is used to transfer the load of pier to the well
- As the shape of the well pier and cap are different
- The well cap forms an interim layer to accommodate the pier.
- The well cap is so designed that the base of the pier is provided with a minimum all round offset.
- The centre of the well cap is made to coincide with that of the pier and not with that of the well.
- Such positioning nullifies the effect of the minor shifts which might have occurred during well sinking.

COMPONENTS OF WELL



FORCES ACTING ON WELL FOUNDATION

Dead loads:
it includes weight of superstructure (pier/abutment) + self weight of well.

Live loads:

Load caused due to tractive effect of vehicles on bridges and road, load due to human beings, furniture floors &other materials For road bridges, the live loads may be specified via standard specifications and code of practice for road bridges.

Impact loads:

The impact loads is the result of live load and shall be considered only during the design of a pier cap and the bridge seat on the abutment. However, for other components of the well this effect shall be neglected.

Wind loads:
The wind loads shall be seen only on the exposed are in elevation and hence acts laterally on the bridge

Laying of Curbs

In dry ground excavate up to 15 cm in river bed and place the cutting edge at the required position. If the curb is to be laid under water and depth of water is greater than 5 m, prepare Sand Island and lay the curb. If depth of water exceeds 5 m built curb in dry ground and float it to the site.

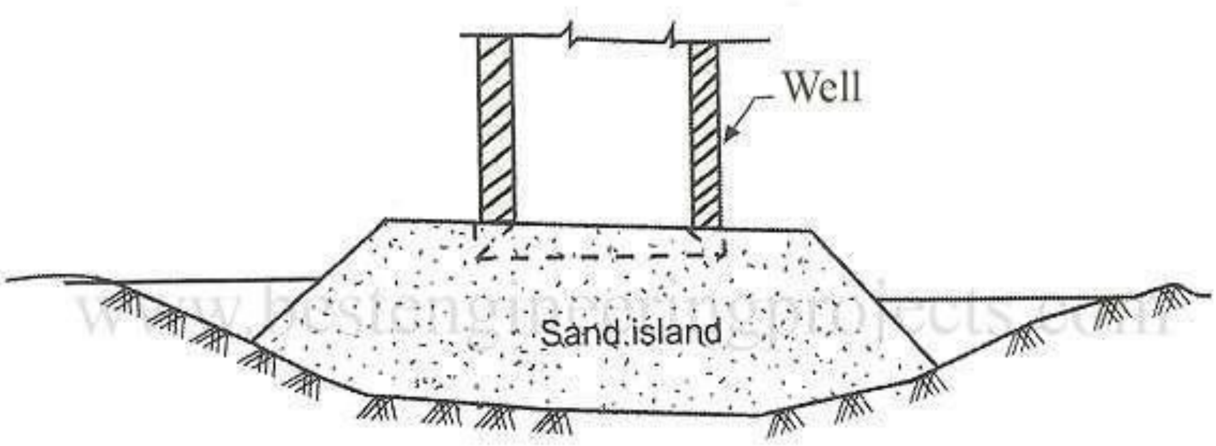


Fig 1 A Typical Sand Island

Construction of Well Steining:

The steining should be built in short height of 1.5 m initially and 3 m after a 6 m grip length is achieved. The verticality should be maintained. The aim of the well sinking is to sink the well vertically and at the correct position.

Precautions – The following precautions should be taken during well sinking.

Outer surface should be regular and smooth. Radius of the curb should be 2 to 4 cm larger than the radius of the steining.

Cutting edge should be of uniform thickness and sharpness.

Sinking Operation

Excavate material under the inside of well curb mechanically or manually Allow the well to remain vertical. Up to a depth of 1 m, excavation underwater can be made manually. When the depth of water exceeds 1 m excavate by Jh ms or grabs.

- When well goes on sinking skin friction increases and weight of well decreased due to buoyancy.
- When the well does not sink, sunk by applying kentledge. If this operation is not sufficient jet outside the well or grease the outside. A typical loading on steining by kentledge is shown in Fig.
- Go on adding sections of steining (2 to 5 m in length) up to the required founding strata.

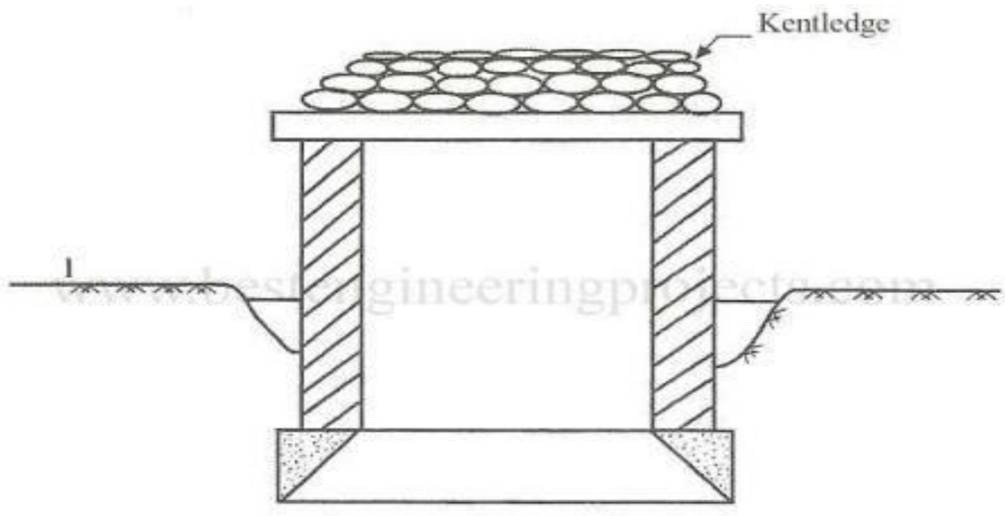


Fig 2 Loading Steining by Kentledge

TILT AND SHIFT

TILT

The well should be sunk vertical & at the right position through all kinds of soils IS 3955 – 1967 suggests that tilt should be restricted to 1 in 60

SHIFT

IS 3955 – 1967 suggests that shift be limited to 1% of depth sunk

RULE OF GRABBING

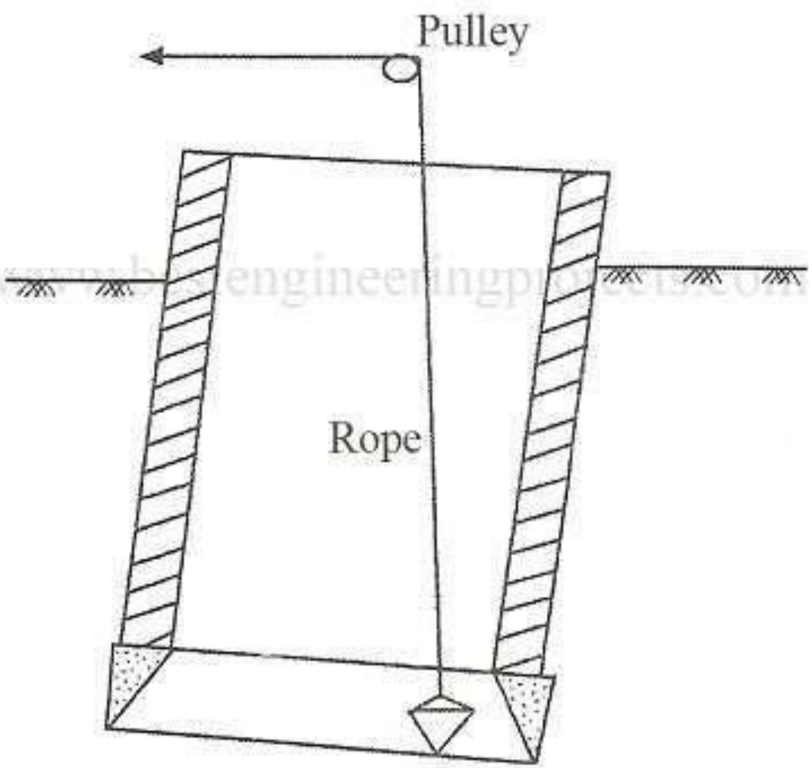


Fig 3 Dredging the Well

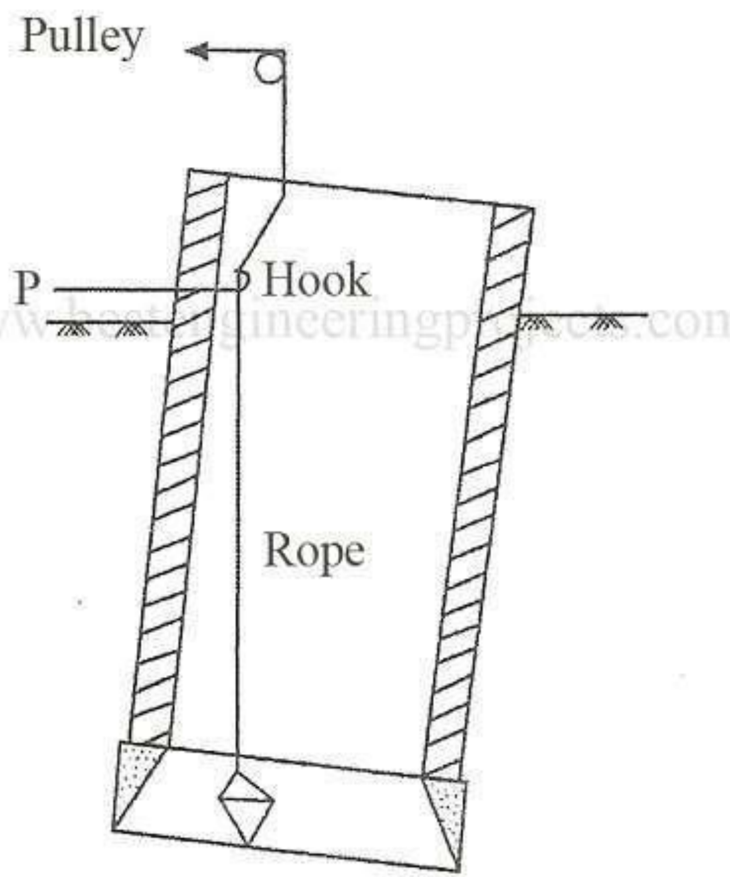


Fig 4 Dredging the well