GEOTECHNICAL ENGINEERING - II

Soil Exploration – What is it and Why???

- The field and laboratory investigations required to obtain necessary data regarding the soil, for proper design and successful construction of any structure at the site are collectively called soil exploration.

- Exploration in soil- involves a site visit, quick visual inspection and detailed tests to determine the behaviour

Objectives of Soil Exploration

- Determination of the nature of the deposits of soil.
- Determination of the depth and thickness of the various soil strata and their extent in the horizontal direction.
- The location of groundwater and fluctuations in GWT.
- Obtaining soil and rock samples from the various strata.
- The determination of the engineering properties of the soil and rock strata that affect the performance of the structure
- Determination of the *in-situ* properties by performing field tests.

Need for Soil Exploration

- To determine the type of foundation required for the proposed project at the site, i.e. shallow foundation or deep foundation.

- Estimation of the probable settlement of a structure.

- Determination of potential foundation problems (for example, expansive soil, collapsible soil)

- Establishment of ground water table.

- Prediction of soil pressure for structures like retaining walls

- Establishment of construction methods for changing subsoil conditions.

Phases of a Soils Investigation

The soil investigation is conducted in phases. Each preceding phase affects the extent of the next phase. The various phases of a soil investigation are given below:

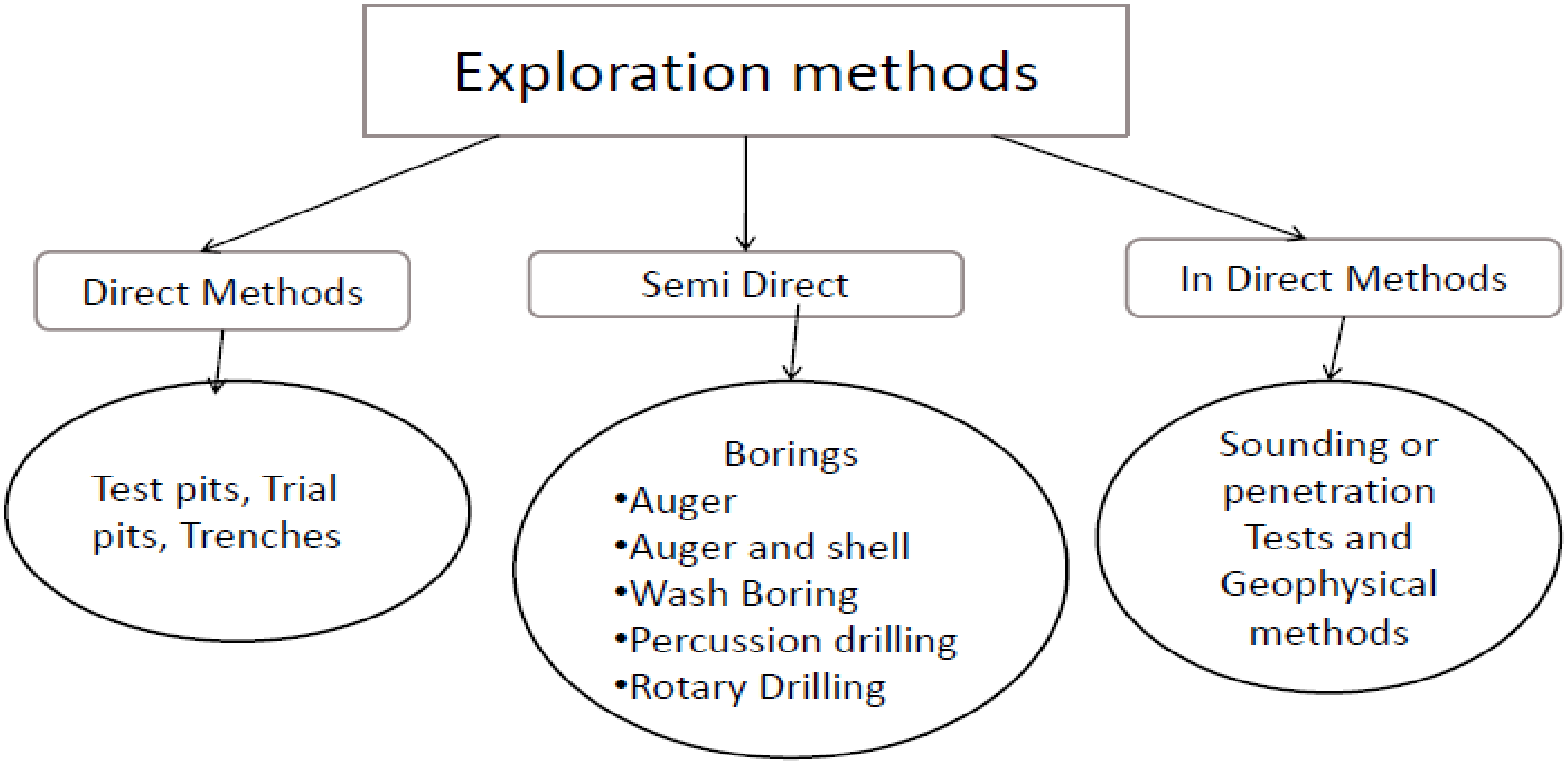
Phase I. Collection of available information such as a site plan, type, size, and importance of the structure, loading conditions, previous geotechnical reports, topographic maps, air photographs, geologic maps, hydrological information and newspaper clippings.

Phase II. Preliminary reconnaissance or a site visit to provide a general picture of the topography and geology of the site. It is necessary that you take with you on the site visit all the information gathered in Phase I to compare with the current conditions of the site. Here visual inspection is done to gather information on topography, soil stratification, vegetation, water marks, ground water level, and type of construction nearby.

Phase II. Detailed soils exploration. Here we make a detailed planning for soil exploration in the form, trial pits or borings, their spacing and depth. Accordingly, the soil exploration is carried out. The details of the soils encountered, the type of field tests adopted and the type of sampling done, presence of water table if met with are recorded in the form of bore log. The soil samples are properly labeled and sent to laboratory for evaluation of their physical and engineering properties.

Phase IV. Write a report. The report must contain a clear description of the soils at the site, methods of exploration, soil profile, test methods and results, and the location of the groundwater. This should include information and/or explanations of any unusual soil, water bearing stratum, and soil and groundwater condition that may be troublesome during construction.

Methods of soil Exploration



Test pits

- Depth upto 3m
- Uneconomical at greater depths.
- Supports are required at greater depths. Especially in case of weak strata
- Problems with GWT and the same should be lowered
- Open type Exploration
- Soils are investigated in natural condition
- Soil samples are collected for determining strength and Engineering properties

Excavated test pit

Boring :-

Making or drilling boreholes into the ground with a view to obtain soil or rock samples from specified or known depths is called 'boring'.

The common methods of advancing bore holes are :-

1. Auger boring.
2. Auger and shell boring.
3. Wash boring
4. Percussion drilling.
5. Rotary drilling.

} More commonly employed for sampling in Rock Strata. (items 4 and 5)

SOIL SAMPLES

- Disturbed
  - ✓ In situ structure not retained
  - ✓ Water content, classification, compaction
- Undisturbed
  - ✓ Less disturbed
  - ✓ Shear strength, consolidation, permeability

Auger Boring

- Drilling is made using a device called Soil Auger
- Power driven (upto 3 to 5m) and Hand operated (Greater than 5m)
- Advancement is made by drilling the auger by simultaneous rotating and pressing it into the soil
- Dry and unsupported bore holes
- When the auger gets filled with soil same, it is taken out and the soil sample collected

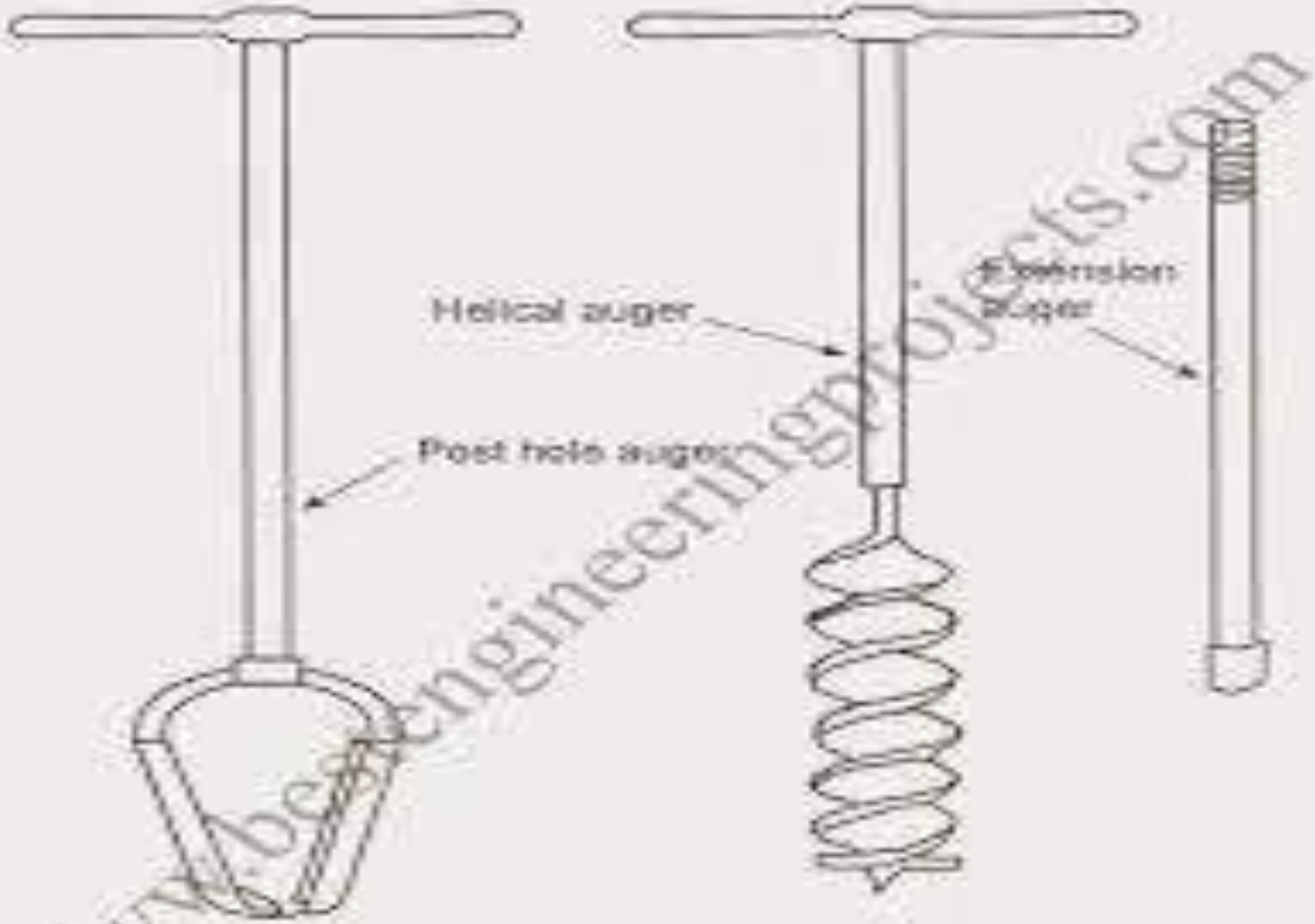
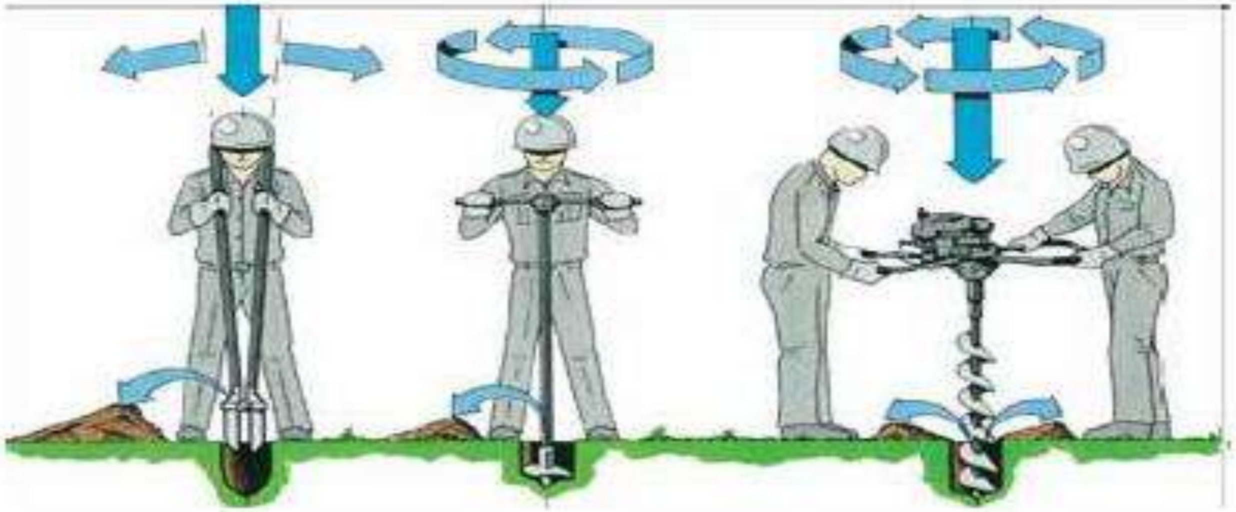
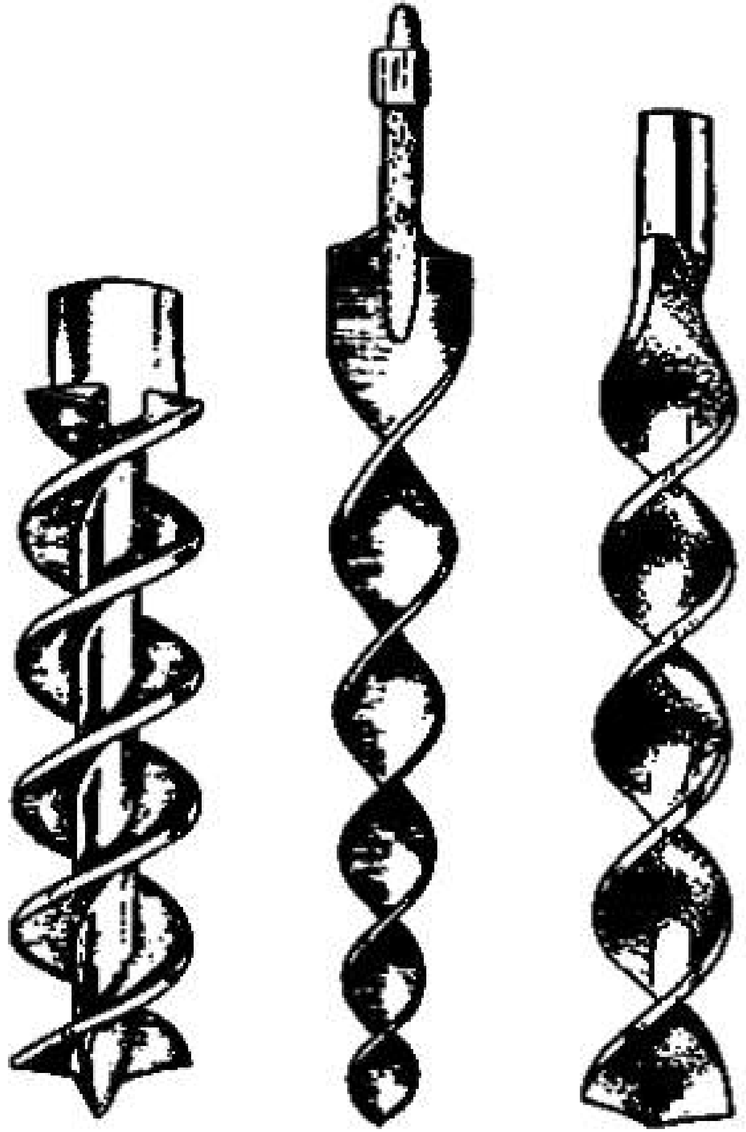


Fig 1: Hand Augers

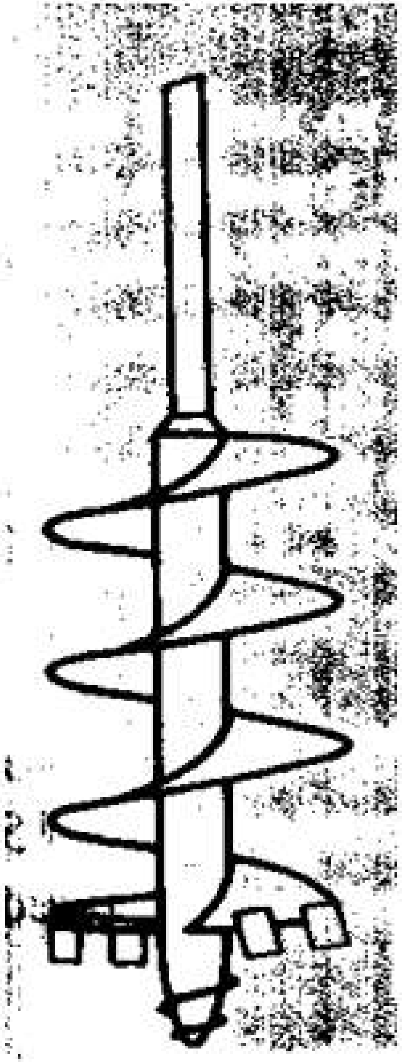
Hand operated augers



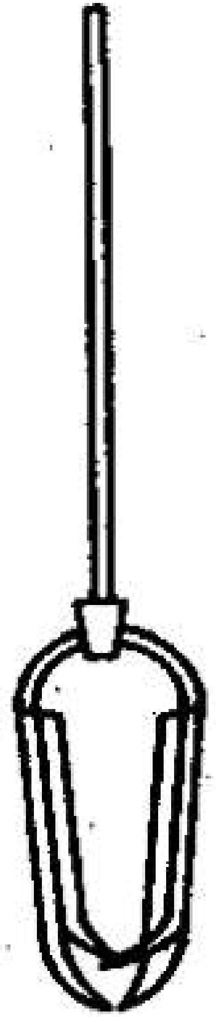
Power driven augers



Helical (Worm) Augers



Short Flight Auger



Iwan Auger

Auger and Shell Boring

- Casing is provided in case of weak strata
- First the casing is driven and then the auger
- Whenever the casing is to be extended, the auger has to be withdrawn which hinders the quick progress of the work.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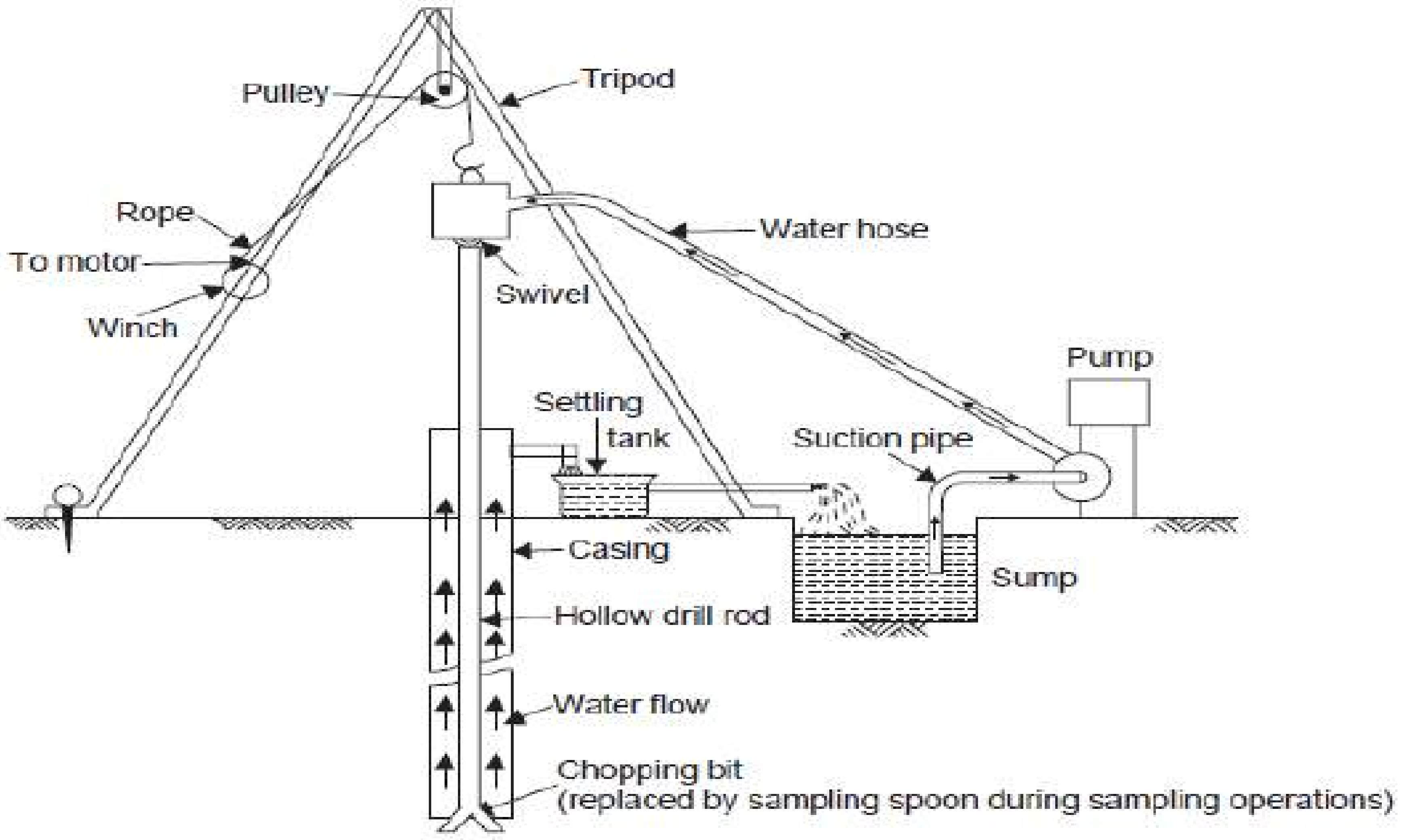
Wash Boring

- Below GWT. May not be used for soils mixed with gravel and boulders
- Initially, the hole is advanced for a short depth by using an auger.
- Then a casing pipe is pushed in and driven with a drop weight. The driving may be with the aid of power.
- A hollow drill bit is screwed to a hollow drill rod connected to a rope passing over a pulley and supported by a tripod.
- Water jet under pressure is forced through the rod and the bit into the hole.
- This loosens the soil at the lower end and forces the soil-water suspension upwards along the annular surface between the rod and the side of the hole

- This suspension is collected in a settling tank.
- Soil particles are allowed to settle down and water is allowed to overflow into a sump which is then recirculated
- Very disturbed sample is obtained. Hence cannot be used for determining engineering properties.
- whenever a soil sample is required, the chopping bit is to be replaced by a sampler.
- The change of the rate of progress and change of colour of wash water indicate changes in soil strata.

Typical set up for Wash boring



Percussion Drilling

- A heavy drill bit called 'churn bit' is suspended from a drill rod or a cable and is driven by repeated blows.
- Water is added to facilitate the breaking of stiff soil or rock.
- The slurry of the pulverised material is bailed out at intervals.

Disadvantages

- Cannot be used in loose sand and is slow in plastic clay.
- The formation gets badly disturbed by impact.

Rotary Drilling

- Suitable for rock formations.
- A drill bit, fixed to the lower end of a drill rod, is rotated
- by power while being kept in firm contact with the hole.
- Drilling fluid or bentonite slurry is used under pressure which brings up the cuttings to the surface.
- Even rock cores may be obtained by using suitable diamond drill bits.

Disadvantage

- Not used in porous deposits as the consumption of drilling fluid would be high.

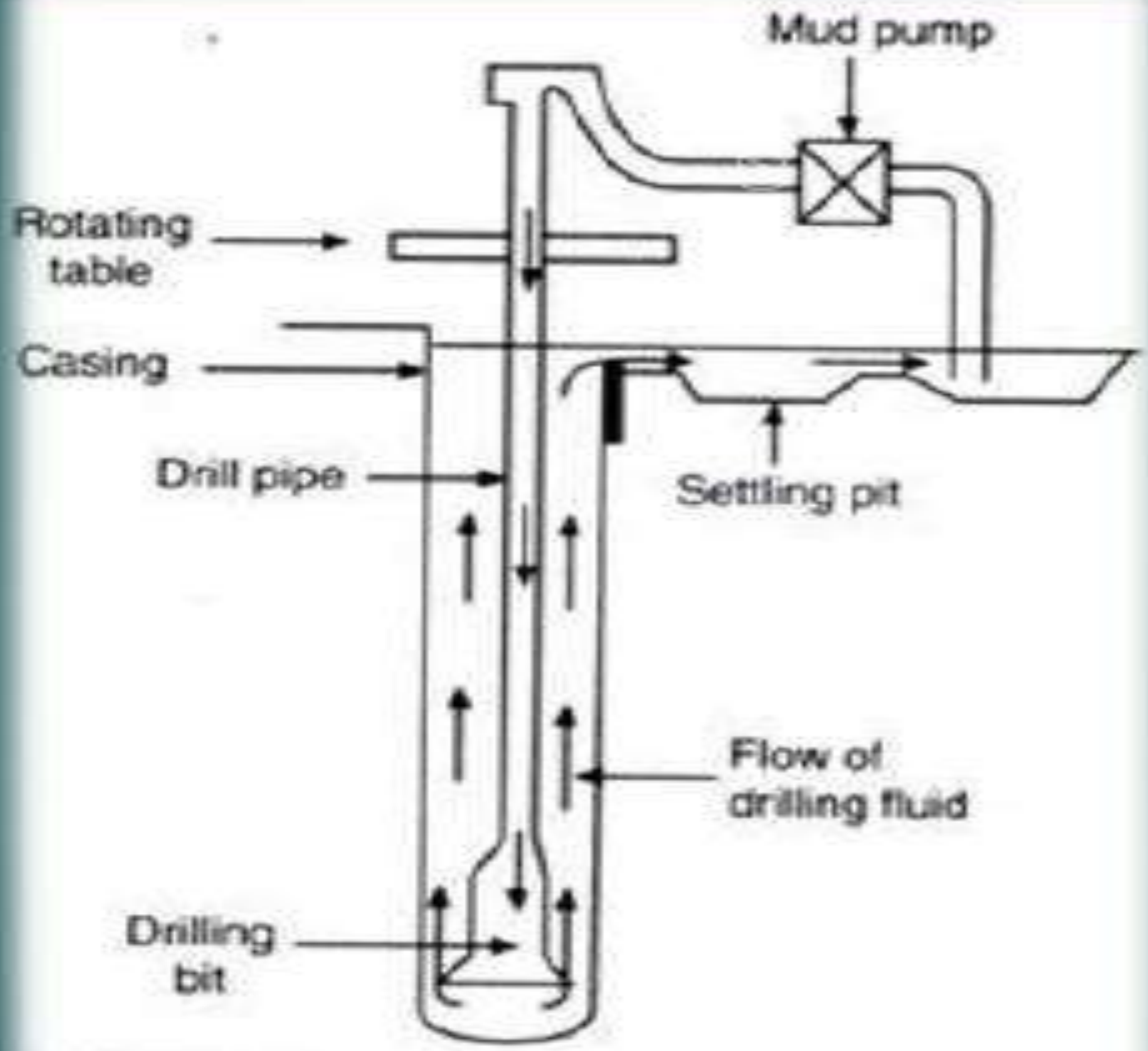


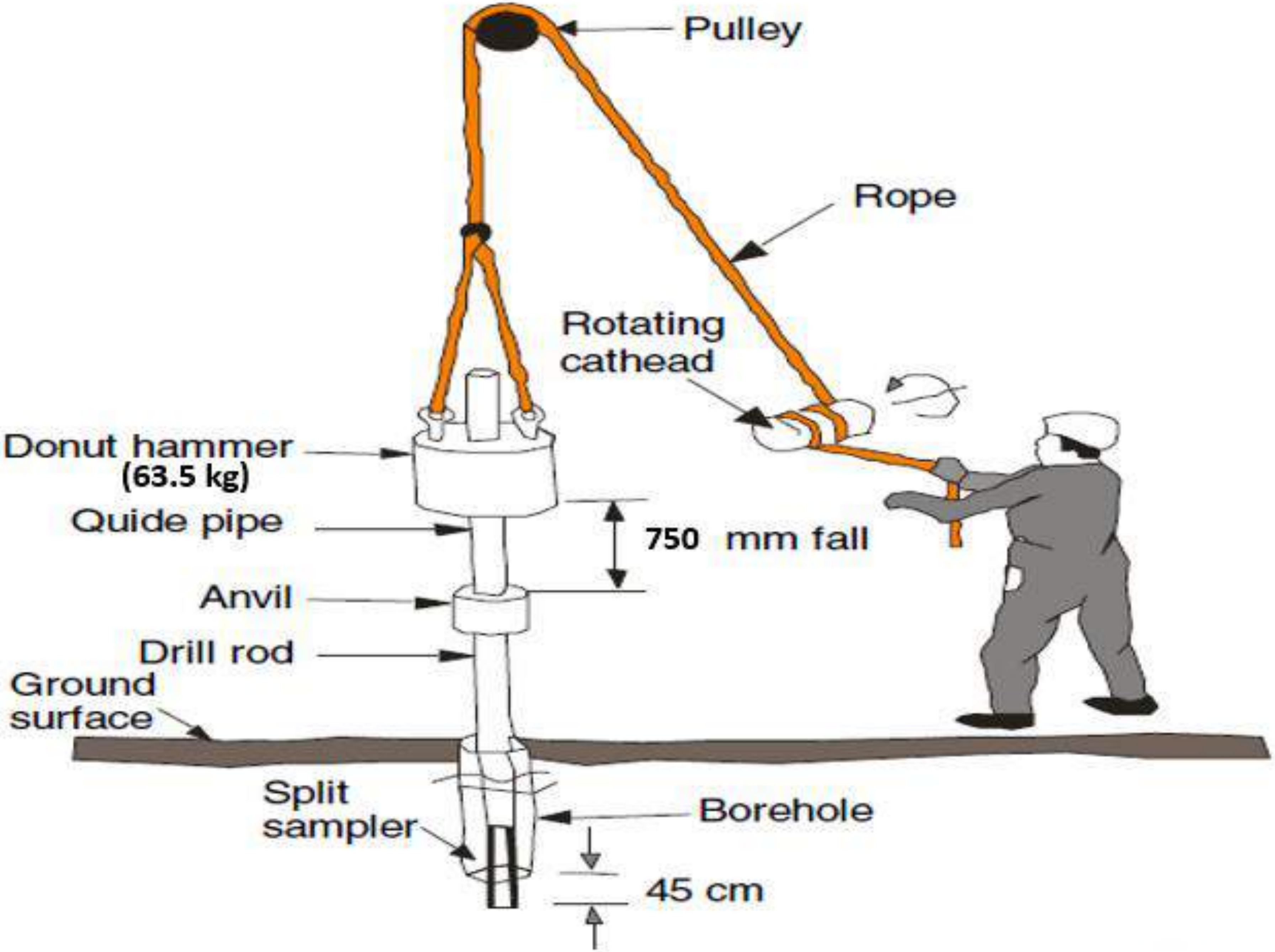
Fig. 18.8. Schematic diagram of rotary method

Indirect methods

Standard Penetration Test”.

- Generally used for cohesionless soils
- To determine relative density , angle of shearing resistance, UCC
- A bore hole is made using drilling tools and a hammer of weight 63.5 kg falling from the height of 750 mm at the rate of 30 blows/minute
- After reaching the specified depth, the drilling tool is replaced by a split spoon sampler to collect soil sample.

- First 150 mm penetration is taken as seating drive and the no. of blows required for that penetration is discarded
- No of blows required for next 300mm penetration after seating drive is taken as standard penetration number (N)
- No of blows greater than 50 are taken as refusal and the test is discontinued
- Corrections are applied to the observed N value



Pulley
Rope
Rotating
cathead
Donut hammer
(63.5 kg)
Quide pipe
750 mm fall
Anvil
Drill rod
Ground
surface
Split
sampler
Borehole
45 cm

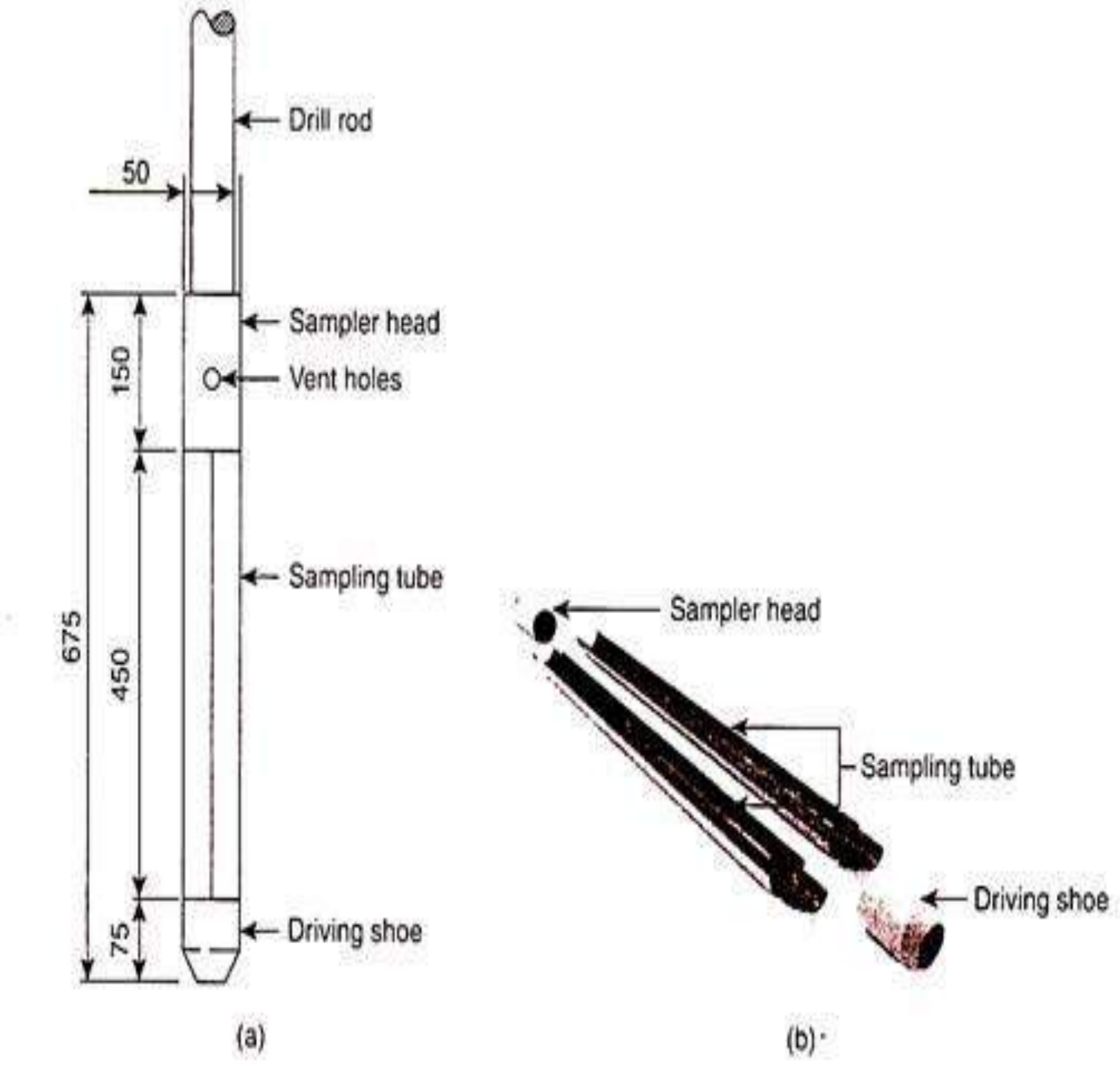


Figure 14.12 Standard split spoon sampler: (a) Schematic diagram and (b) typical sampler.

SPT correlations for cohesionless soil

S. No.	Condition	N	D_r	ϕ
1.	Very loose	0 – 4	0 – 15%	Less than 28°
2.	Loose	4 – 10	15 – 35%	28° – 30°
3.	Medium	10 – 30	35 – 65%	30° – 36°
4.	Dense	30 – 50	65 – 85%	36° – 42°
5.	Very dense	Greater than 50	Greater than 85%	Greater than 42°

SPT correlations for Clays

S. No.	*Consistency*	*N*	q_u (kN/m^2)
1.	Very soft	0 – 2	Less than 25
2.	Soft	2 – 4	25 – 50
3.	Medium	4 – 8	50 – 100
4.	Stiff	8 – 15	100 – 200
5.	Very stiff	15 – 30	200 – 400
6.	Hard	Greater than 30	Greater than 400

Correction to N value

- Dilatancy Correction
- Overburden correction

  Of these, overburden correction is applied first and to that corrected value, dilatancy Correction is applied

Over burden correction

- Soils having the same relative density will show higher N value at greater depth due to presence of over burden.
- Cohesionless soils are greatly affected by confining pressure. Hence N value is corrected .σ <=280 kN/m^2

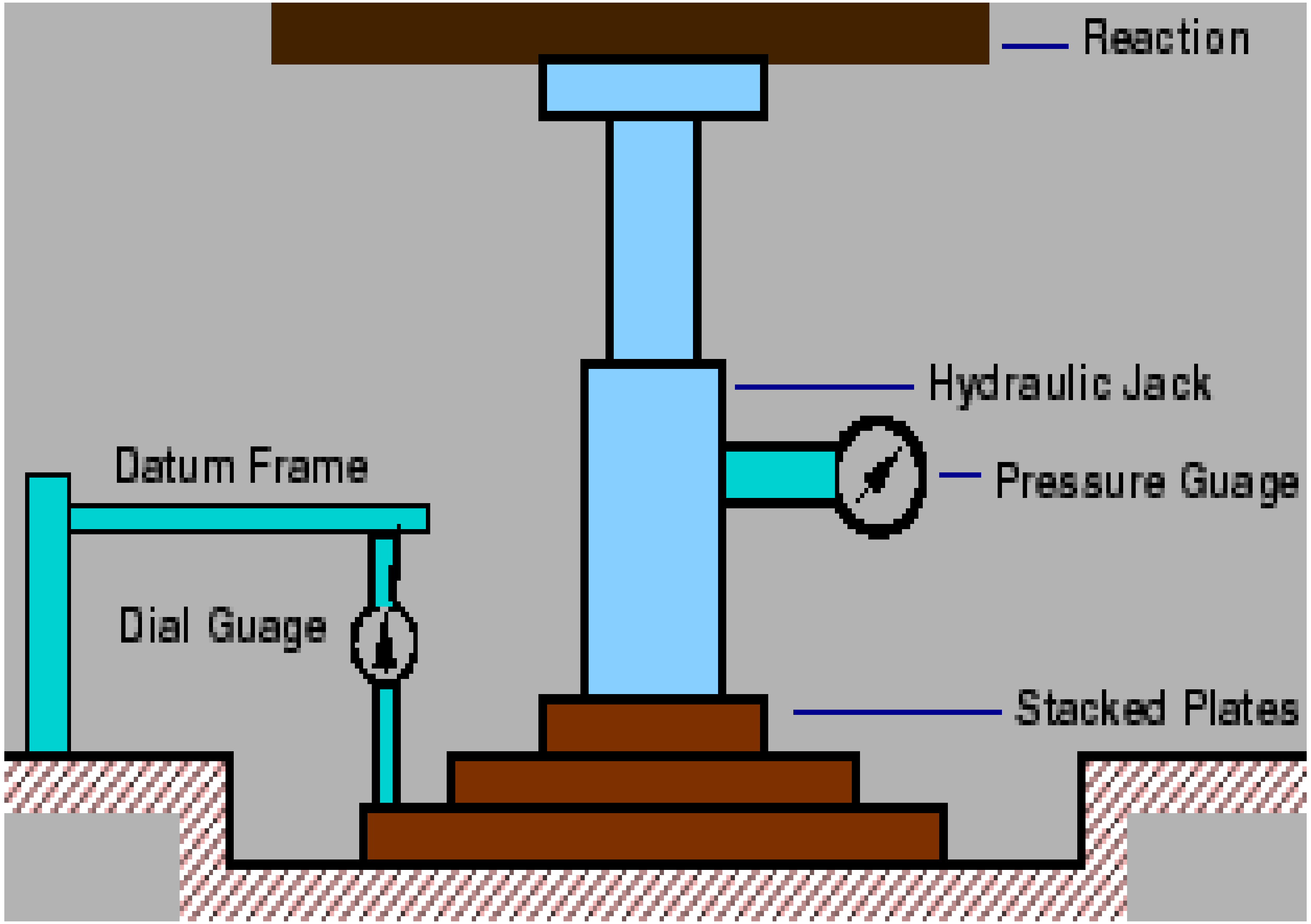
$$N = N' \cdot \frac{350}{(\sigma + 70)}$$

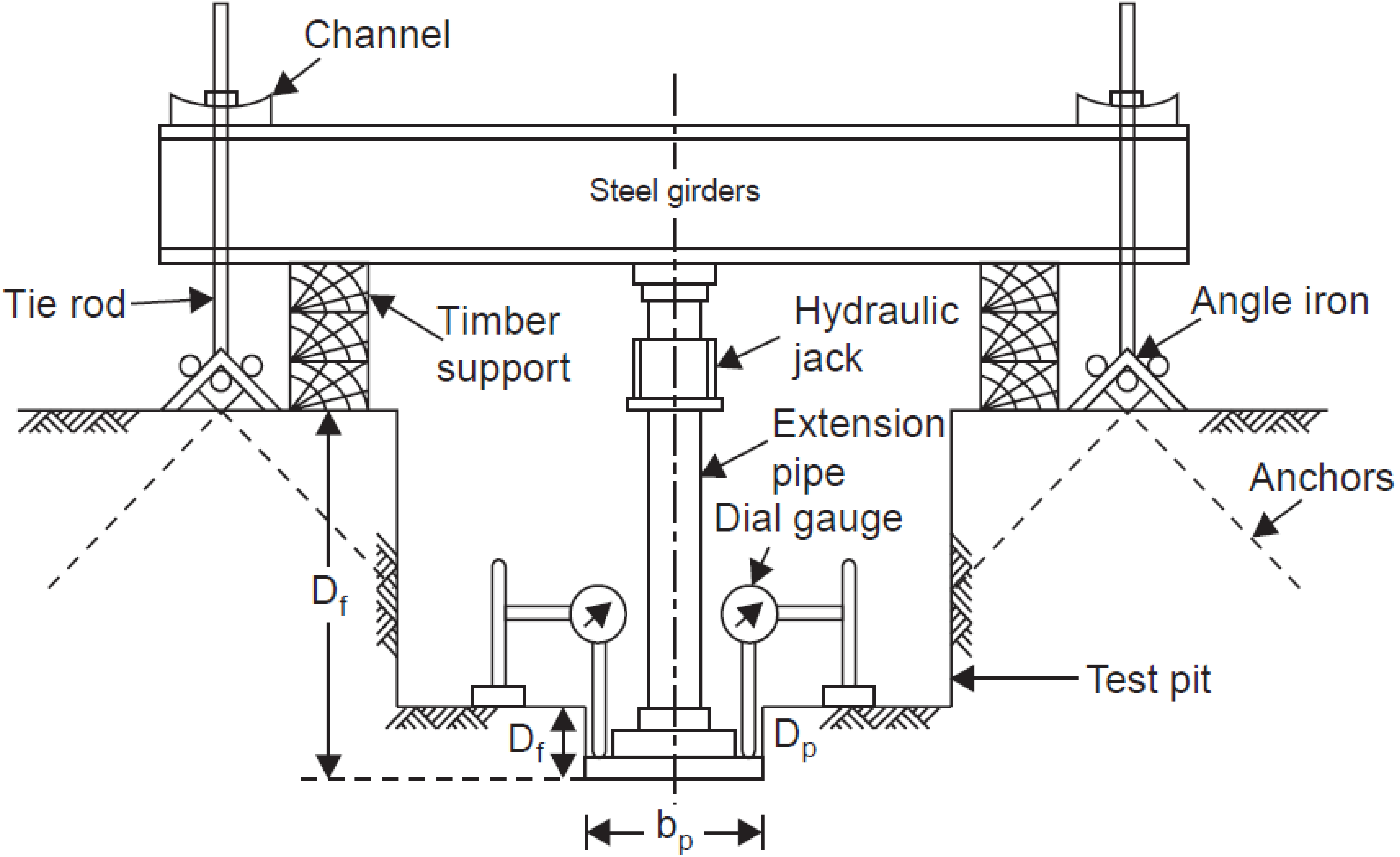
Dilatancy Correction

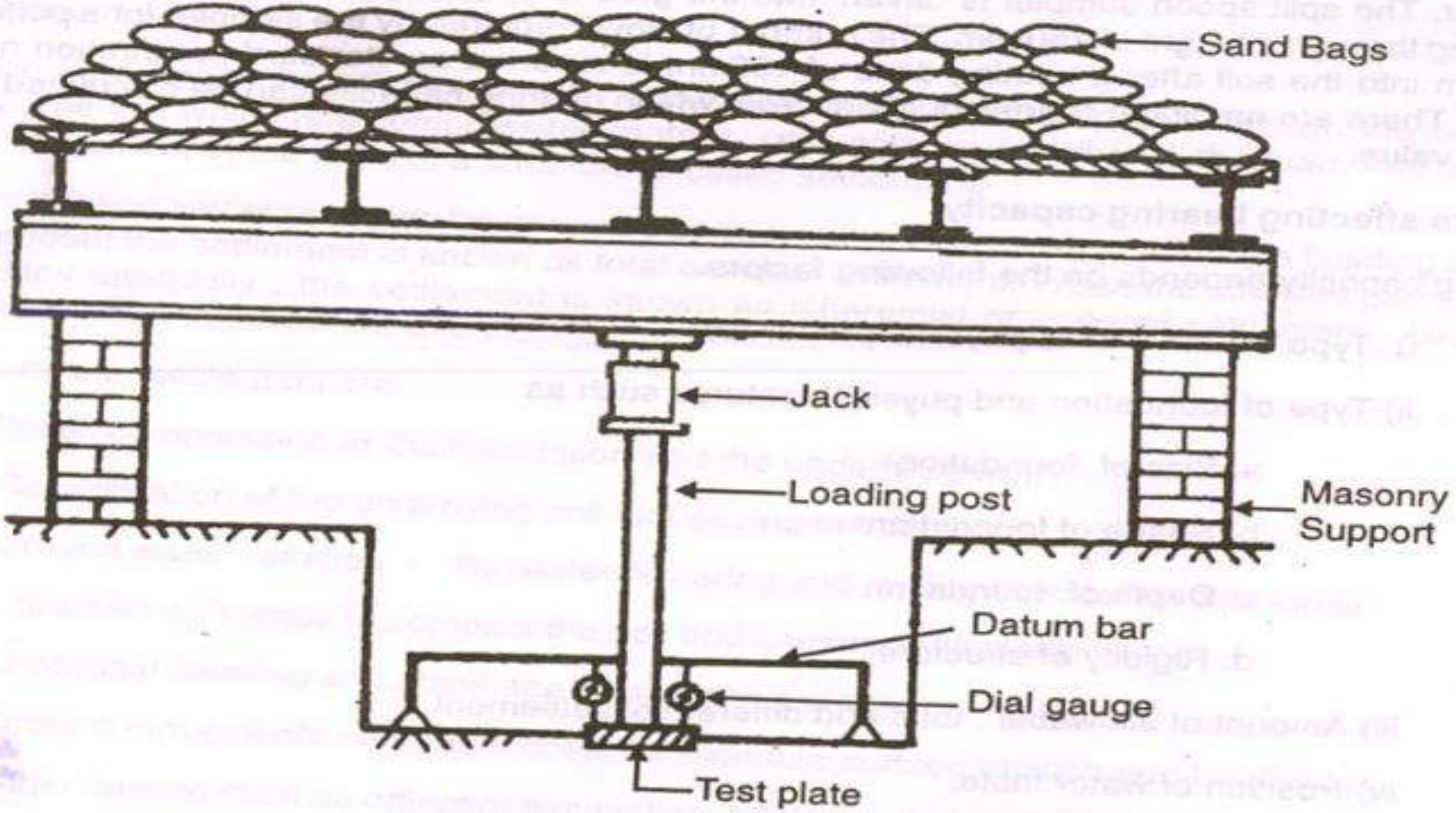
Due to the presence of fine sand and silt below the water table, negative pore pressure develops which increases, the observed N value. Hence correction is applied. (If N’<15 or N=15 , N’ = N)

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

PLATE LOAD TEST




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


Sand Bags
Jack
Loading post
Masonry Support
Datum bar
Dial gauge
Test plate

- It is a field test used to determine the ultimate bearing capacity of soil
- A pit is dug up to the foundation level
- A square plate of 300mm x 300mm & 25 mm is placed at the centre of the pit
- A dial gauge is connected to the test plate
- Now weights in the form of sand bags are placed on the platforms in equal increments.
- The test is continued till the failure occurs or the plates settled by 25 mm whichever occurs earlier
- The load settlement curve is then recorded.

- The test pit should be at least five times as wide as the test plate and the bottom of the test plate should correspond to the proposed foundation level.
- At the centre of the pit, a small square hole is made the size being that of the test plate and the depth being such that,

$$\frac{D_p}{b_p} = \frac{D_f}{b}$$

(*i*) After excavating the pit of required size and levelling the base, the test plate is seated over the ground.

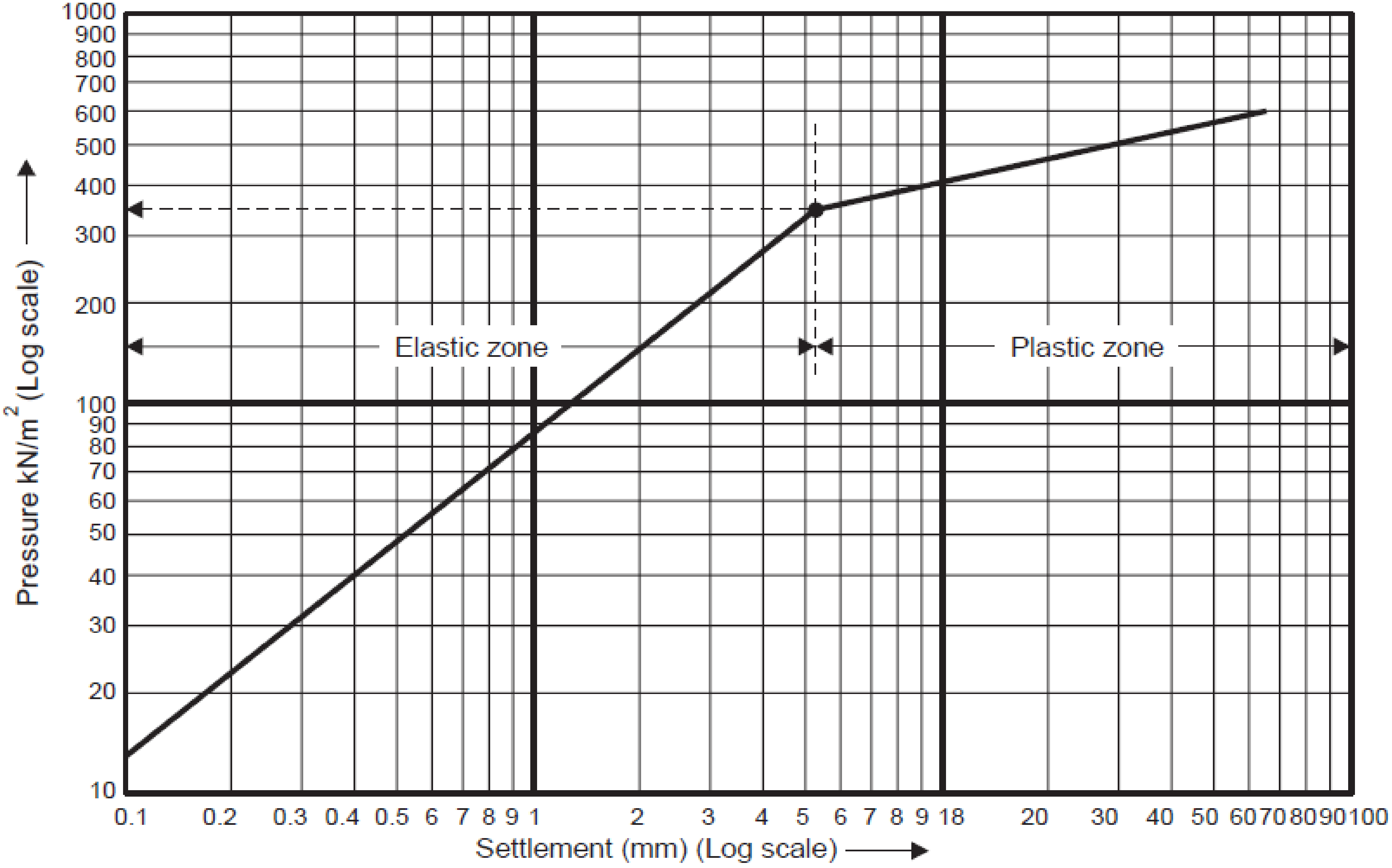
(*ii*) A seating pressure of 7.0 kN/m2 (70 g/cm2) is applied and released before actual loading is commenced.

(*iii*) The first increment of load, say about one-tenth of the anticipated ultimate load, is applied. Settlements are recorded with the aid of the dial gauges after 1 min., 4 min., 10 min., 20 min., 40 min., and 60 min., and later on at hourly intervals until the rate of settlement is less than 0.02 mm/hour, or at least for 24 hours.

(*iv*) The test is continued until a load of about 1.5 times the anticipated ultimate load is applied. According to another school of thought, a settlement at which failure occurs

or at least 2.5 cm should be reached.

(*v*) From the results of the test, a plot should be made between pressure and settlement, which is usually referred to as the "load-settlement curve",. The bearing capacity is determined from this plot

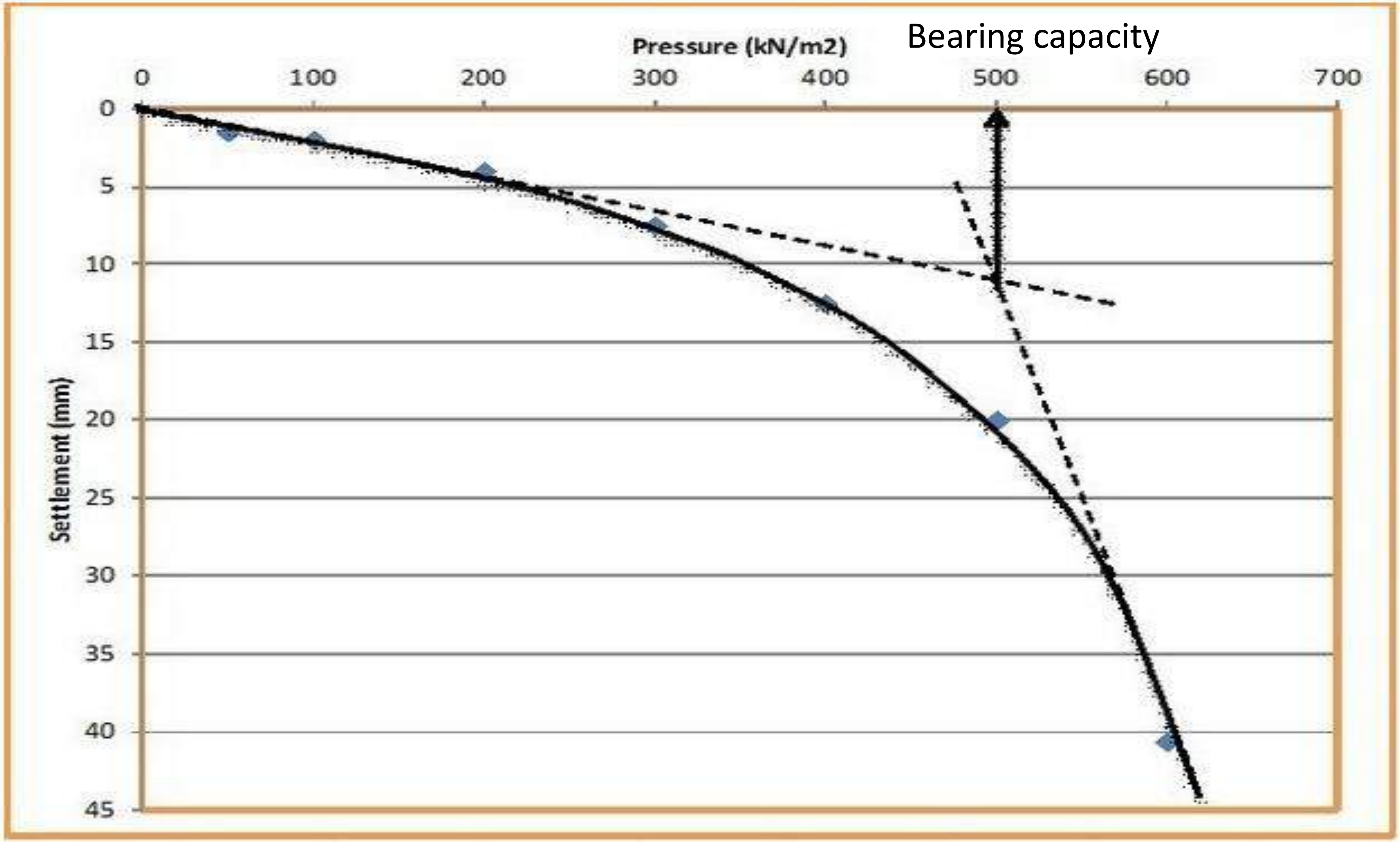


Pressure kN/m² (Log scale)
1000
900
800
700
600
500
400
300
200
100
90
80
70
60
50
40
30
20
10
Elastic zone
Plastic zone
0.1 0.2 0.3 0.4 0.5 6 7 8 9 1 2 3 4 5 6 7 8 9 18 20 30 40 50 60 70 80 90 100
Settlement (mm) (Log scale)

- The plot between pressure and settlement usually consists of two straight lines as shown in Figure. The point corresponding to the break gives the failure point and the pressure corresponding to it is taken as the bearing capacity.

- IS: 1888–1971 also recommends this method for use with plate load tests.

ALTERNATE METHOD FOR DETERMINATION OF BEARING CAPACITY



Load settlement curve

Settlement of original foundation (S)

Sandy soils

$$\frac{S_f}{S_p} = \left[\frac{b_f(b_p + 0.3)}{b_p(b_f + 0.3)}\right]^2$$

Clayey soils

$$\frac{S_f}{S_p} = \frac{b_f}{b_p}$$

where S_f = settlement of the proposed foundation (mm),
S_p = settlement of the test plate (mm),
(same units)
b_f = size of the proposed foundation (m), and
b_p = size of the test plate (m).

Ultimate bearing capacity (qu) for foundation

$$q_{u(f)} = q_{u(p)} \; for \; Clay$$

$$q_{u(f)} = q_{u(p)} \frac{b_f}{b_p} \; for \; sady \; layer$$

where

$q_{u(f)}$ = Ultimate Bearing Capacity for the Proposed Foundation

$q_{u(p)}$ = ultimate Bearing capacity of the Test Plate

Limitations of Plate load test

- Size effects are very important. Since the size of the test plate and the size of the prototype foundation are very different, the results of a plate load test do not directly reflect the bearing capacity of the foundation.

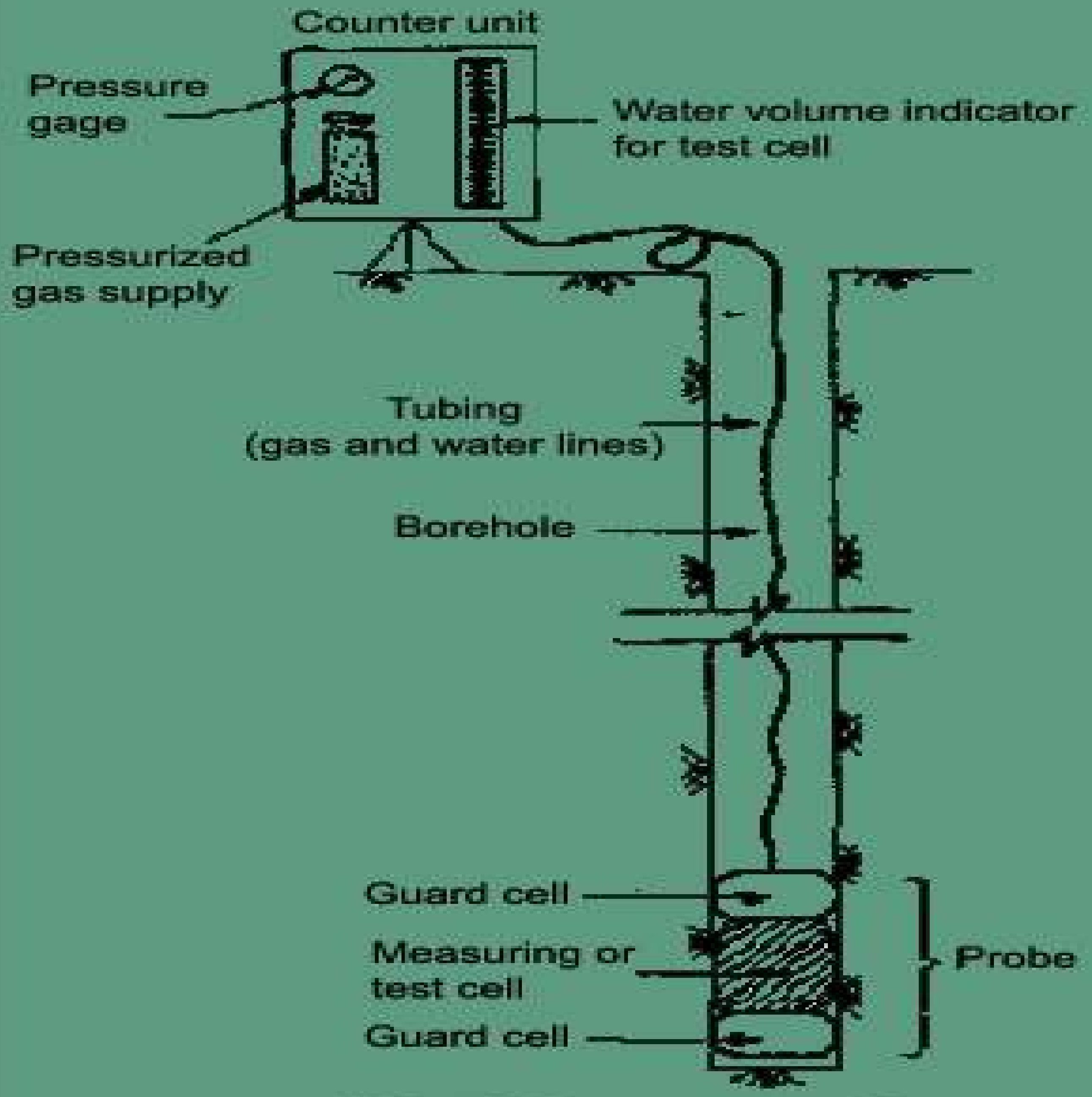
- Consolidation settlements in cohesive soils, which may take years, cannot be predicted, as the plate load test is essentially a short-term test.

- Results from plate load test are not recommended to be used for the design of strip footings, since the test is conducted on a square or circular plate and shape effects enter.

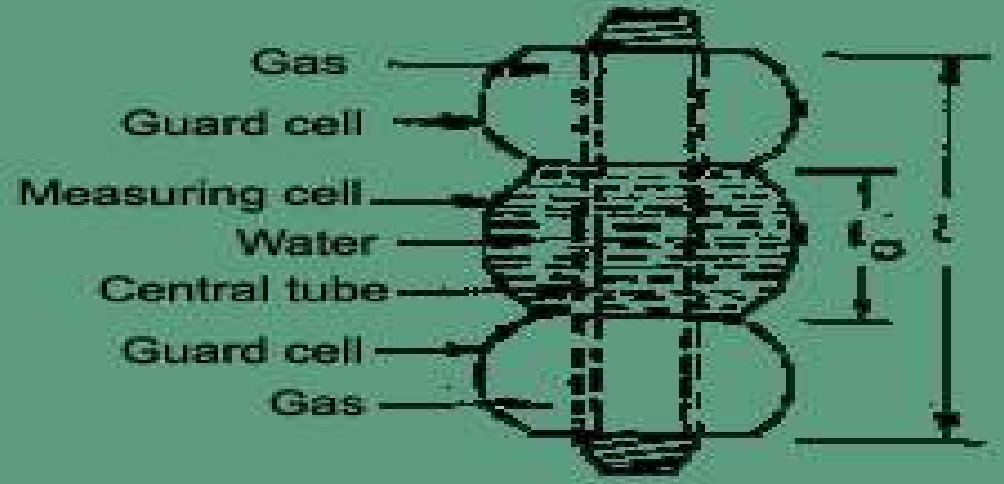
- The load test results reflect the characteristics of the soil located only within a depth of about twice the width of the plate. This zone of influence in the case of a prototype footing will be much larger and unless the soil is essentially homogeneous for such a depth and more, the results could be terribly misleading.

PRESSUREMETER TEST

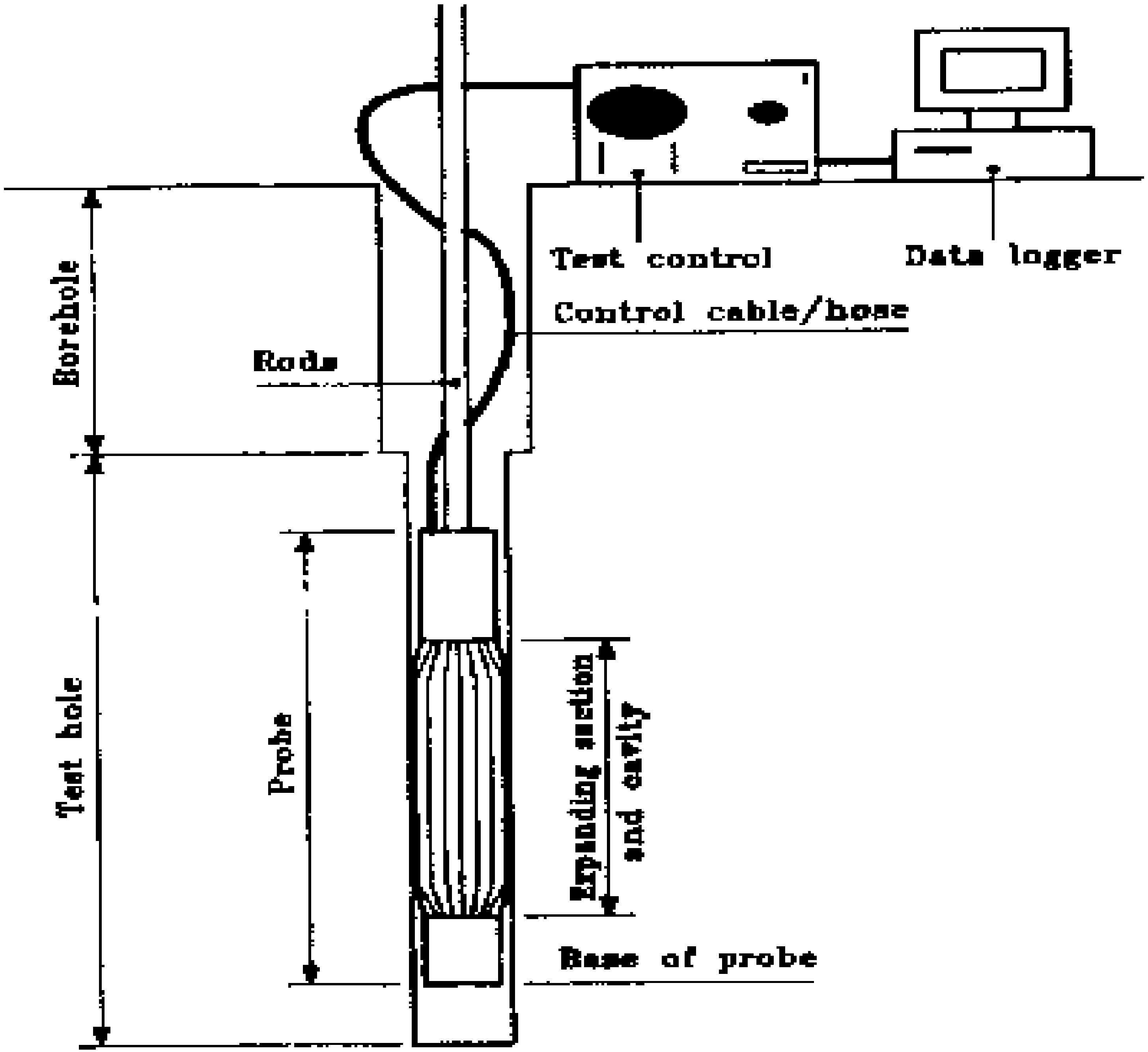
- The pressure meter consists of two parts, the read-out unit, which rests on the ground surface, and the probe that is inserted into the borehole.
- The probe consists of three independent cells, a measuring cell and two guard cells.
- The probe can be installed by pre-drilling a hole using hollow stem auger or hand auger, or forcing the probe into the ground and displacing the soil by driving, jacking, or vibrating.
- Once the probe is at the test depth, the guard cells are inflated to brace the probe in place. Then the measuring cell is pressurized with water, inflating its flexible rubber bladder, which exerts a pressure on the borehole wall.
- As the pressure in the measuring cell increases, the borehole walls deform. The pressure within the measuring cell is held constant for approximately 60 seconds, and the increase in volume required to maintain the constant pressure is recorded.
- A load-deformation diagram, as shown in Figure



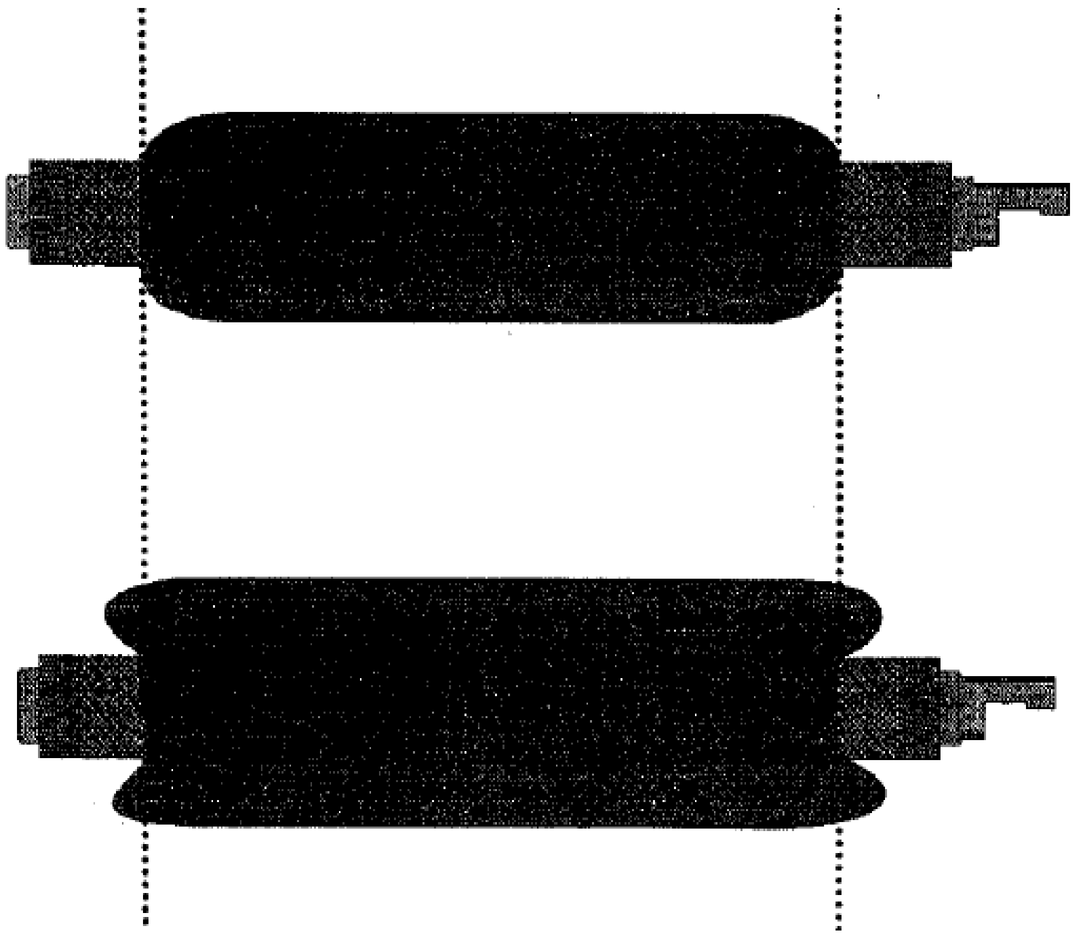
(a) Basic Components of Pressurement



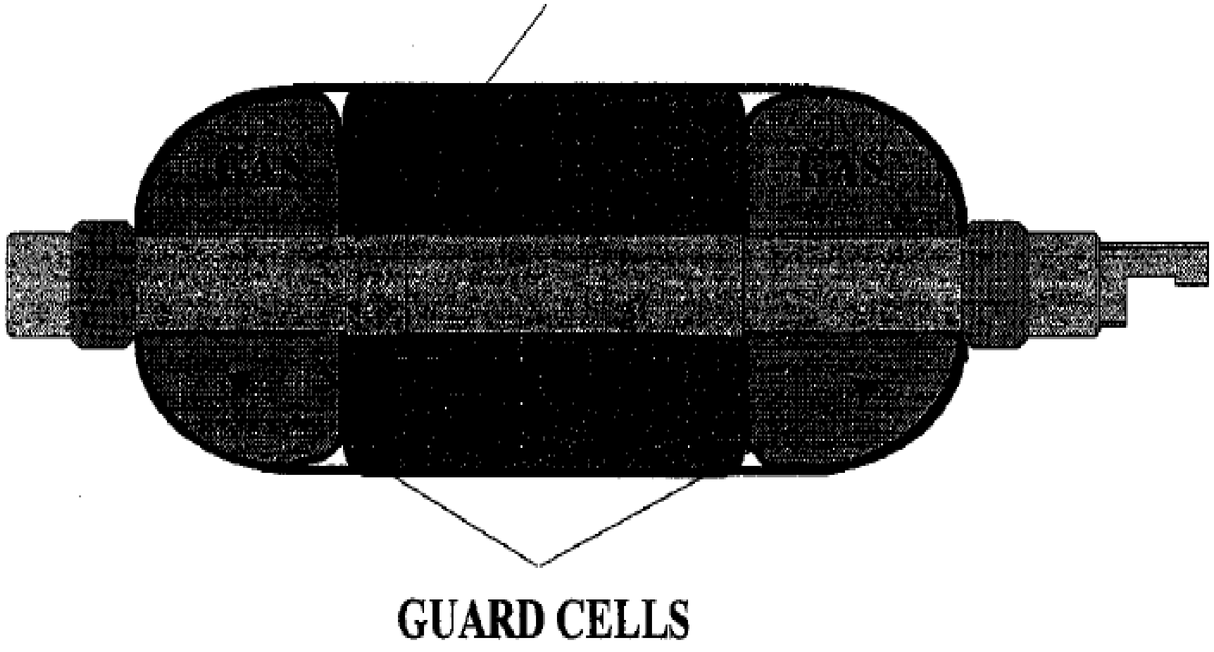
(b) Illustration of the Probe



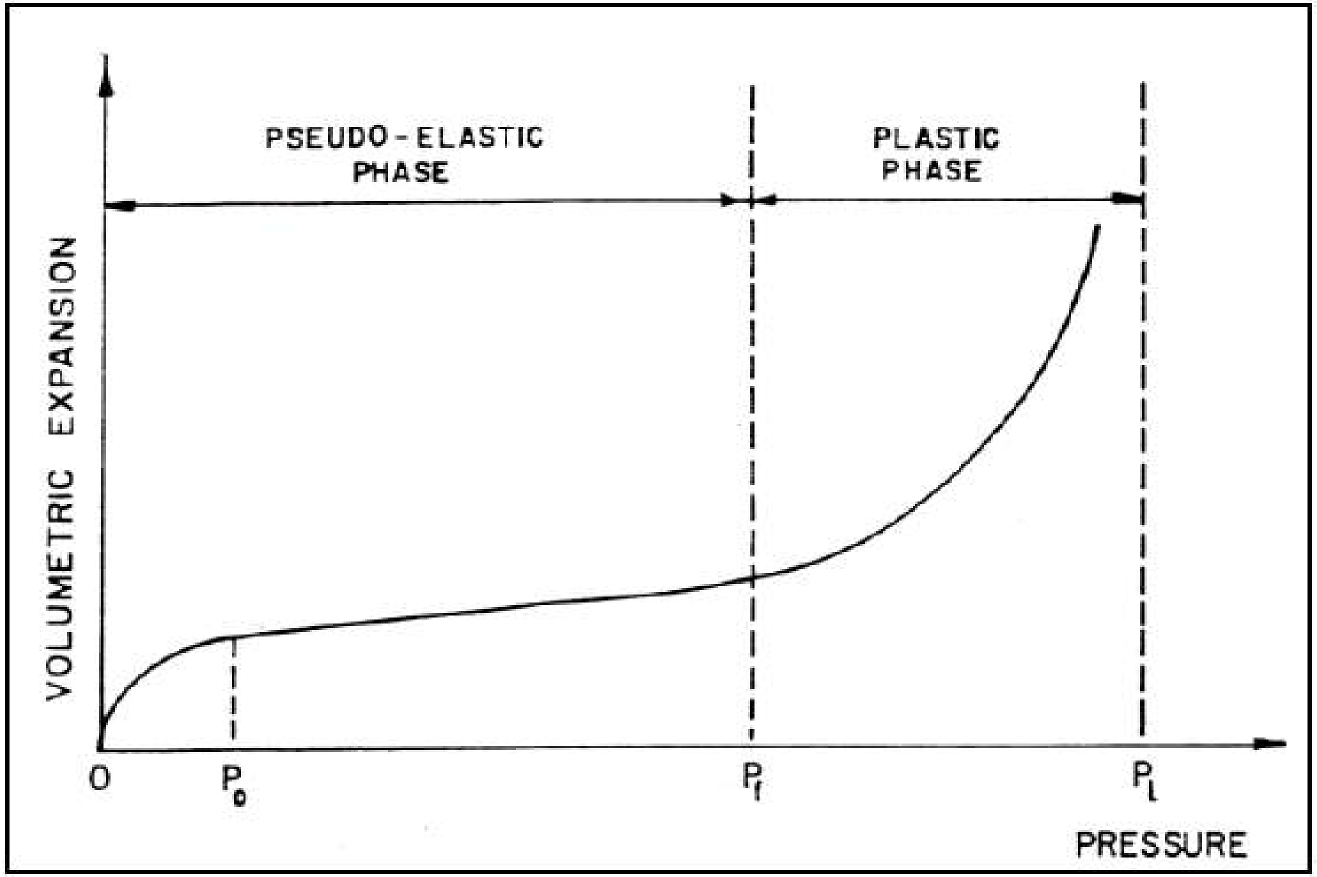
Test control
Data logger
Control cable/hose
Rods
Borehole
Test hole
Probe
Expanding section and cavity
Base of probe



END EFFECTS



- The pressure-volume data is plotted to determine the limit pressure (PL), cohesion and the pressure meter deformation modulus (E). These values are used for foundation design.



Typical pressuremeter test curve

The curve can be divided into three parts:

i) From $P = 0$ to $P = P_0$

This portion of the curve corresponds to the probe seating against the borehole wall. The wall disturbance induced by drilling or driving the probe into place has considerable influence on this segment of the curve. The difference in borehole and probe diameters also affects this segment.

ii) From $P = P_0$ to $P = P_f$

This segment represents the pseudo-elastic behavior of the tested material. The probe is in contact with the borehole walls. The loading is uniform along the probe length. This segment is linear and defines E, the deformation modulus of the tested material. E, in turn is used to evaluate settlement. Should the probe be in contact with the borehole walls before applying pressure, the mass would exhibit pseudo-elastic behavior from the outset.

iii) From $P = P_f$ to $P = P_L$

P_f by definition is the pressure at which the mass enters a plastic state. Above P_f, the loaded mass' deformation accelerates up to the complete failure point. The pressure that defines failure is the limit pressure P_L. This fundamental mechanical characteristic of the mass is used to evaluate the stability of foundations in accordance with pressuremeter methods.

- The test is conducted in a predrilled borehole normally at intervals of 1m.

- The pressure of water in the measuring cell is increased in increments until the soil fails.

- Usually failure is considered to have reached when the total expanded volume of the test zone reaches twice the volume of the original cavity.

- The asymptotic value of pressure corresponding to final point where failure occurs is known as limit pressure (PL).

- Ultimate bearing capacity is directly proportional to the limit pressure

$$q_{ult} = k \times PL$$

K is bearing capacity factor depending on type of foundation, depth and soil type (0.8-0.9)

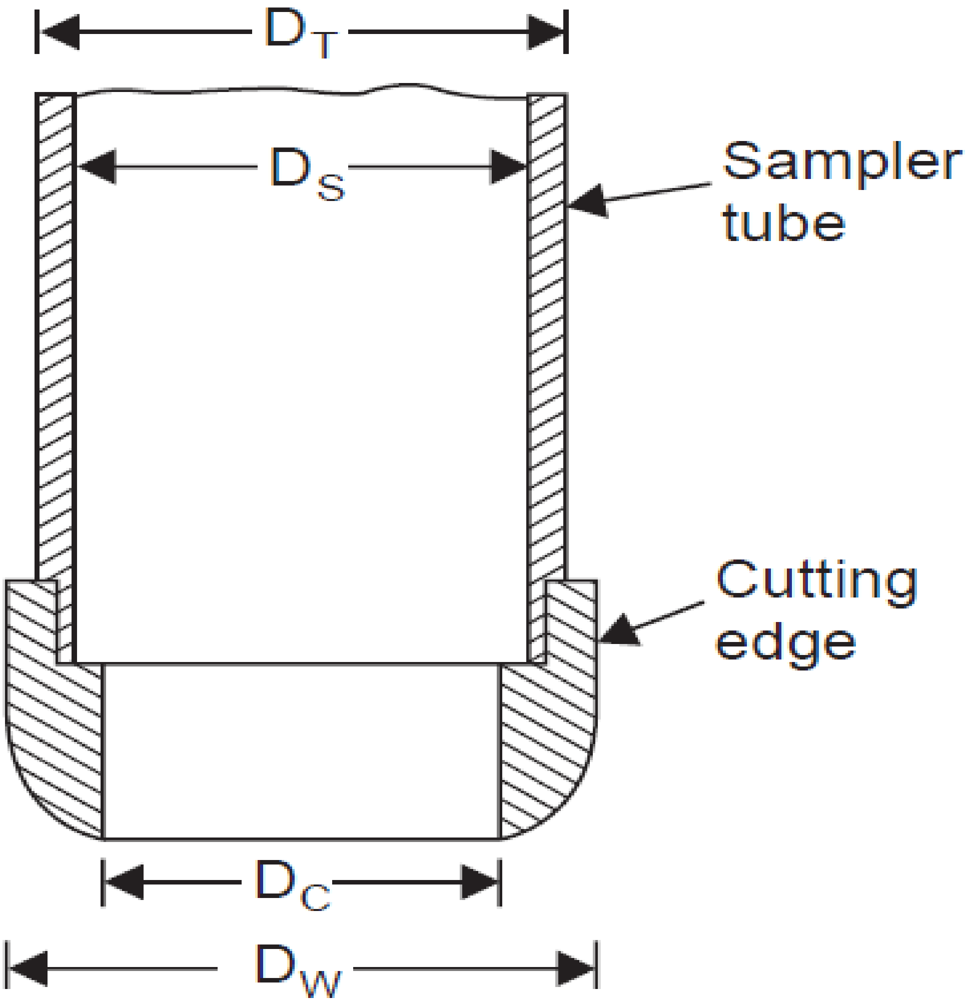
- Cohesion C= PL/9

- Pressure meter modulus is calculated as:

$$E_{PMT} = 2(1+\nu_s)V\frac{\Delta p}{\Delta v}$$

ν_s = Poisson's ratio, typically = 0.33;
V = cavity volume during the pseudo-elastic phase
V_0 = initial or at-rest volume of the measuring cell
$\Delta p/\Delta v$ = slope of the pseudo-elastic phase.

SAMPLE DISTURBANCE



D_C : Inner diameter of cutting edge
D_W : Outer diameter of cutting edge
D_S : Inner diameter of sampling tube
D_T : Outer diameter of sampling tube

Factors affecting soil disturbance while sampling

Area Ratio, $A_r = \frac{(D_w^2 - D_c^2)}{D_c^2} \times 100\%$ $\leq$ **10%** ***for undisturbed samples***

Inside clearance, $C_I = \frac{(D_s - D_c)}{D_c} \times 100\%$ **= 0.5 to 3%** ***for undisturbed samples***

Outside clearance, $C_o = \frac{(D_w - D_T)}{D_T} \times 100\%$ **= 0 to 2 %** ***for undisturbed samples***

recovery ratio $R_r = L/H$ **= 96 to 98 %** ***for undisturbed samples***

L is the length of the sample obtained from the sampler and H is the penetration depth

BORING LOG/ BORE HOLE LOG

- Information on subsurface conditions obtained from the boring operation is typically presented in the form of a boring record, commonly known as "boring log".

- A continuous record of the various strata identified at various depths of the boring is presented.

- Description or classification of the various soil and rock types encountered, and data regarding ground water level have to be necessarily given in a pictorial manner on the log.

Borehole Log

Location :

Project :

Boring method : Shell and auger

Diameter : 150 mm

BH No. 1

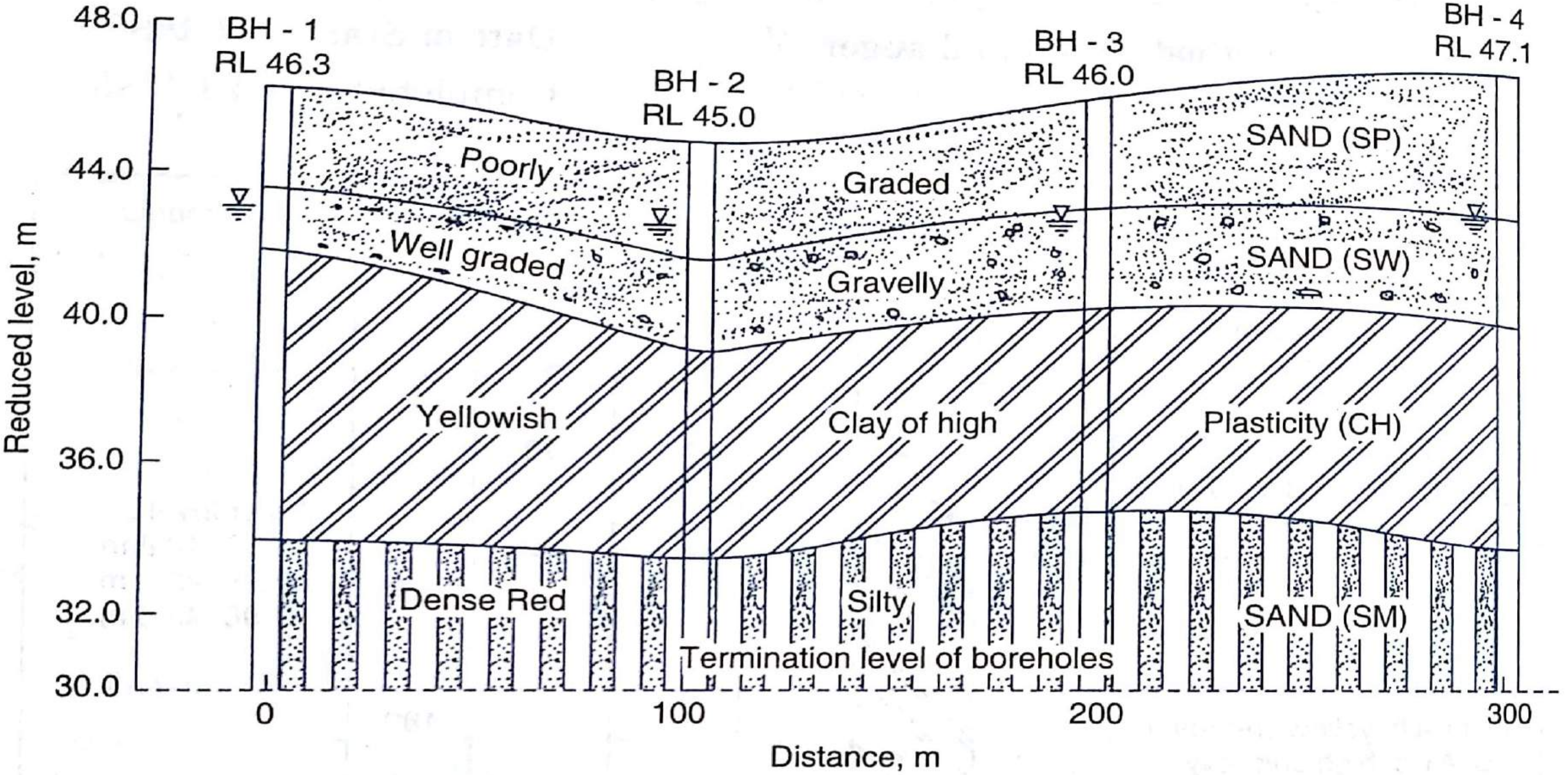
Ground RL : 46.3 m

Date of Start : 1.1.1998

Completed on : 4.1.1998

Description of start	R.L.	Legend	Depth	Samples	N	q_u kN/m^2	Remarks
Loose, light brown SAND (SP)	43.7		2.6	R	6		
Medium dense brown gravelly SAND (SW) ▽ W.T.	42.5 41.9		4.4	R R	18 20		Water level : 1.1.98 42.8 m 2.1.98 42.5 m 3.1.98 42.5 m
Firm to stiff, yellowish-brown, CLAY of high palsticity (CH)	34.0		12.3	U U U U		160 180 200 210	
Very dense, red, silty SAND (SM)	30.0		16.3	R R	50 62 50 for 150 mm		Refusal Termination level of borehole 30.0 m

U : Undisturbed R : Representative sample



Subsoil profile from Bore log

SOIL EXPLORATION REPORT

A report is the final document of the whole exercise of soil exploration. A report should be comprehensive, clear and to the point.

1. Introduction, which includes the scope of the investigation.

2. Description of the proposed structure, the location and the geological conditions at the site.

3. Details of the field exploration programme, indicating the number of borings, their location and depths.

4. Details of the methods of exploration

5. General description of the sub-soil conditions as obtained from in-situ tests, such as standard penetration test and cone penetration test.

6. Details of the laboratory tests conducted on the soil samples collected and the results obtained.

7. Depth of the ground water table and the changes in water levels.

8. Analysis and discussion of the test results.

9. Recommendations about the allowable bearing pressure, the type of foundation of structure.

10. Calculations for determining safe bearing pressure, pile loads, etc.

11. Tables containing bore logs, and other field and laboratory test results.

12. Drawings which include site-plan, test results plotted in the form of charts and graphs, soil profiles, etc.

Geotechnical Engineering –II

Assignment No. 1

(Last date of submission- 19/01/2019)

1. Explain Standard Penetration test. Discuss the corrections applied to the observed N-values.

2. Explain the pressure meter test with a neat sketch. Also write the limitations of this test.

3. With a neat sketch, explain the procedure for conducting a Plate load test. How do you use the results of this test in designing foundations?

4. List the objectives of soil exploration. Describe the salient features of soil investigation report. Explain with the neat diagram of a borelog.

5. With neat sketches explain the different boring methods.

6. Explain the factors affecting soil disturbance while sampling. During a soil exploration programme, a soil sample of length 550mm was recovered using a split spoon sampler. The penetration length of the sample was 610mm. Dimensions of the sampler is given below:
   Inside and outside diameter of the sample tube = 5 and 38mm respectively
   Inside and outside diameter of the driving shoe = 35 and 51mm respectively
   Determine inside clearance, outside clearance, area ratio and recovery ratio and make comment about the degree of disturbance of the soil sample.

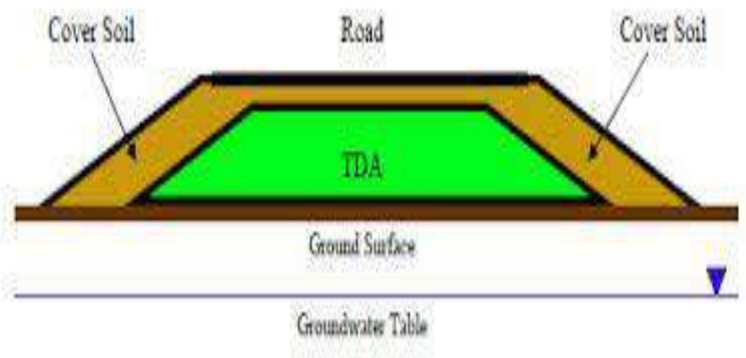
GEOTECHNICAL ENGINEERING- II

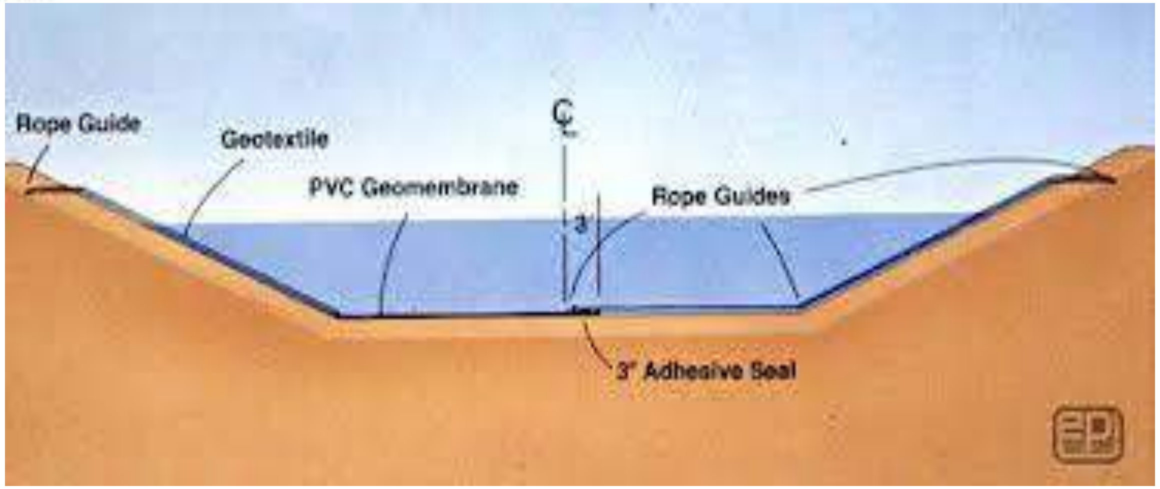
MODULE- II

STABILITY OF SLOPES

SLOPES

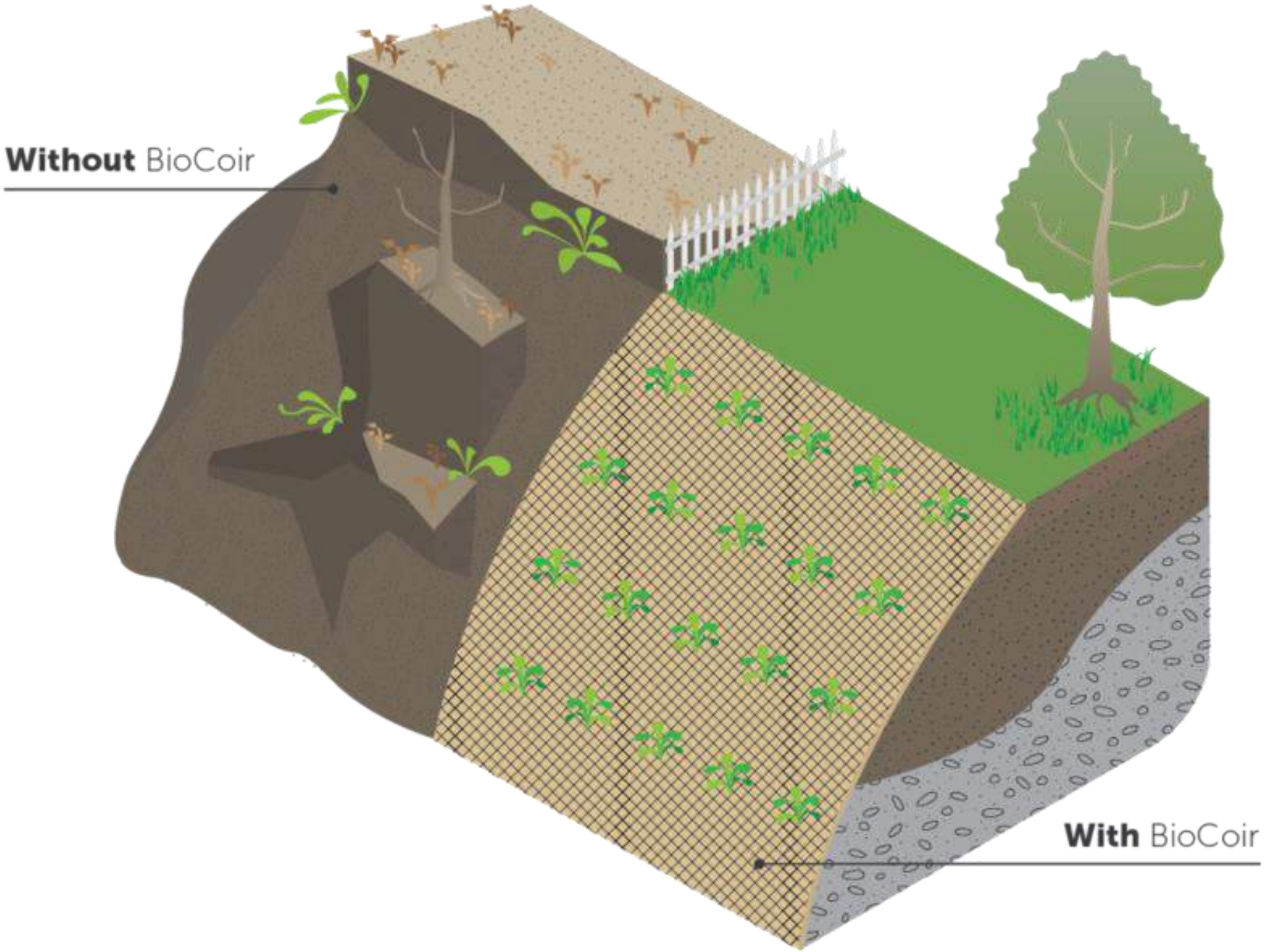
- **Earth slope-** an unsupported, inclined surface of a soil mass
- Formed for railway formations, highway embankments, earth dams, canal banks etc.

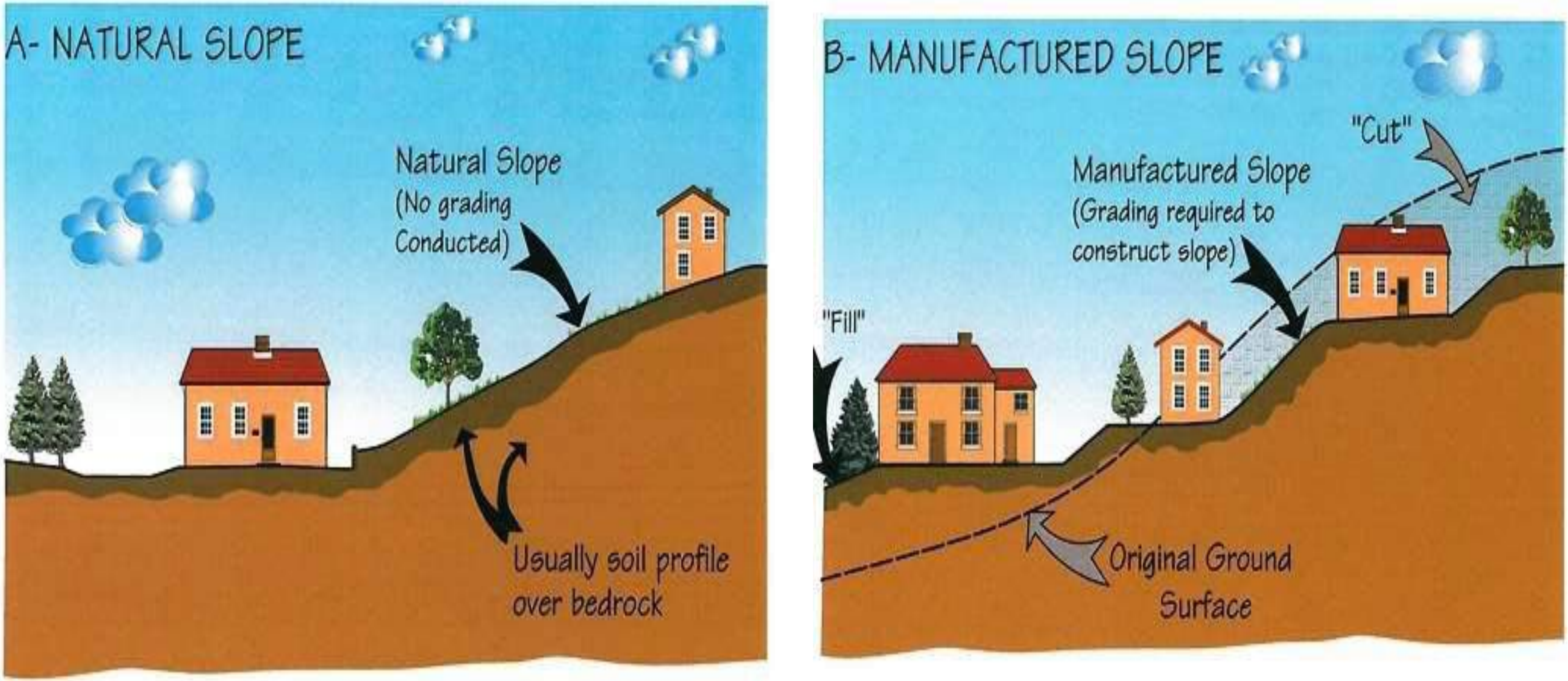




Slope Failures


© AP


Without BioCoir
With BioCoir



A- NATURAL SLOPE
Natural Slope
(No grading
Conducted)
Usually soil profile
over bedrock
B- MANUFACTURED SLOPE
"Cut"
Manufactured Slope
(Grading required to
construct slope)
"Fill"
Original Ground
Surface

NEED FOR STABILITY OF SLOPES

- Steepest section is the most economical section
- Very steep slopes are however not stable
- **For safety and economy-** slopes provided are neither too steep nor flat
- The steepest slopes which are stable and safe would be provided
- Failure of soil mass occurs along a plane or curved surface when a large mass of soil slides w.r.t. remaining soil mass
- A downward and outward movement of soil mass occurs during failure
- Failure occurs when forces causing failure are greater than the shearing resistance developed along a critical plane

SLOPES OF EARTH ARE OF TWO TYPES

1. **Natural slopes**

- slopes exist in hilly areas

2. **Man made slopes**

- 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

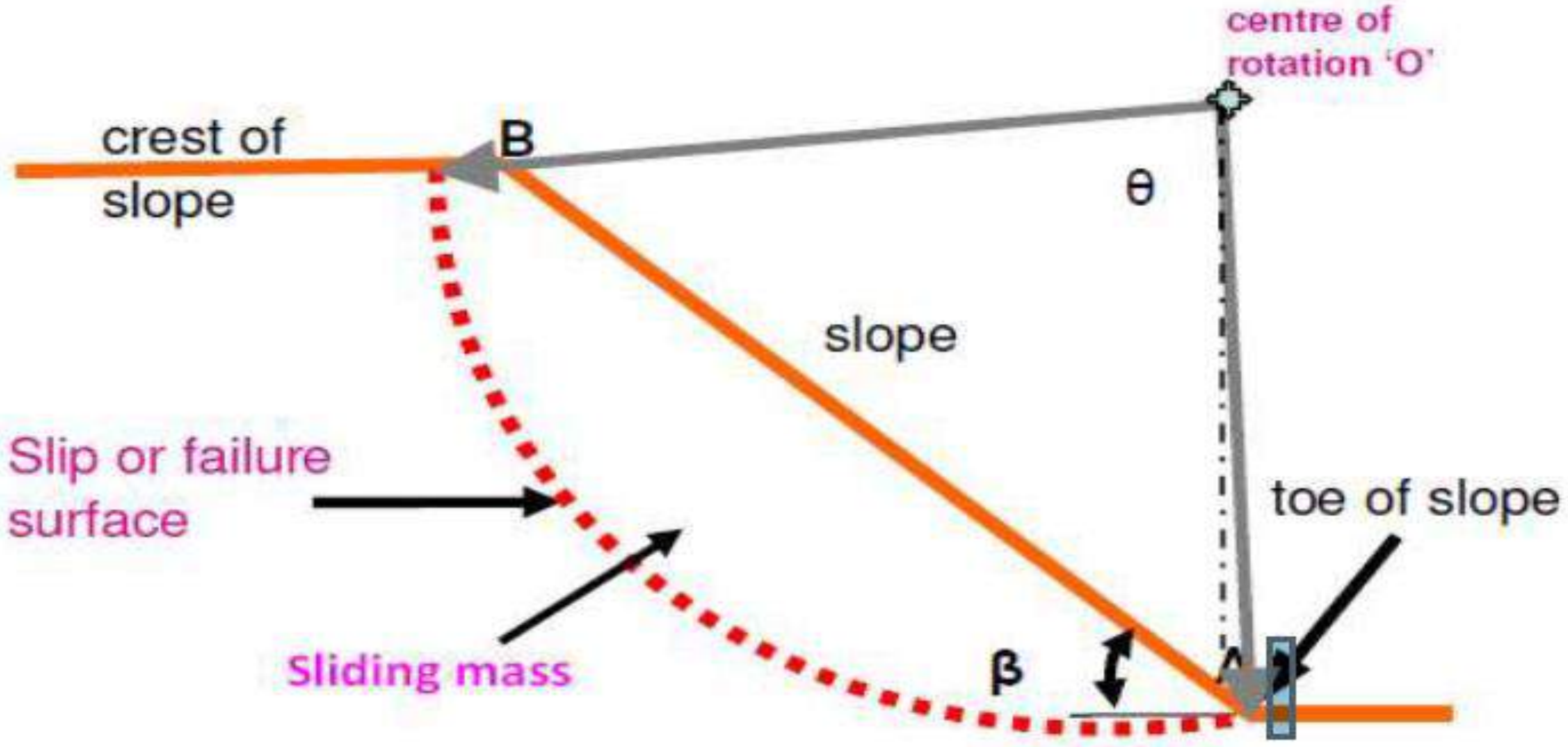
1. Infinite slopes

- The term infinite slope is used to designate a constant slope of infinite extent.
- Example-The long slope of the face of a mountain

2. Finite slopes

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

Definition of Key Terms



TYPES OF SLOPE FAILURE

Slope can fail due to one of the following methods

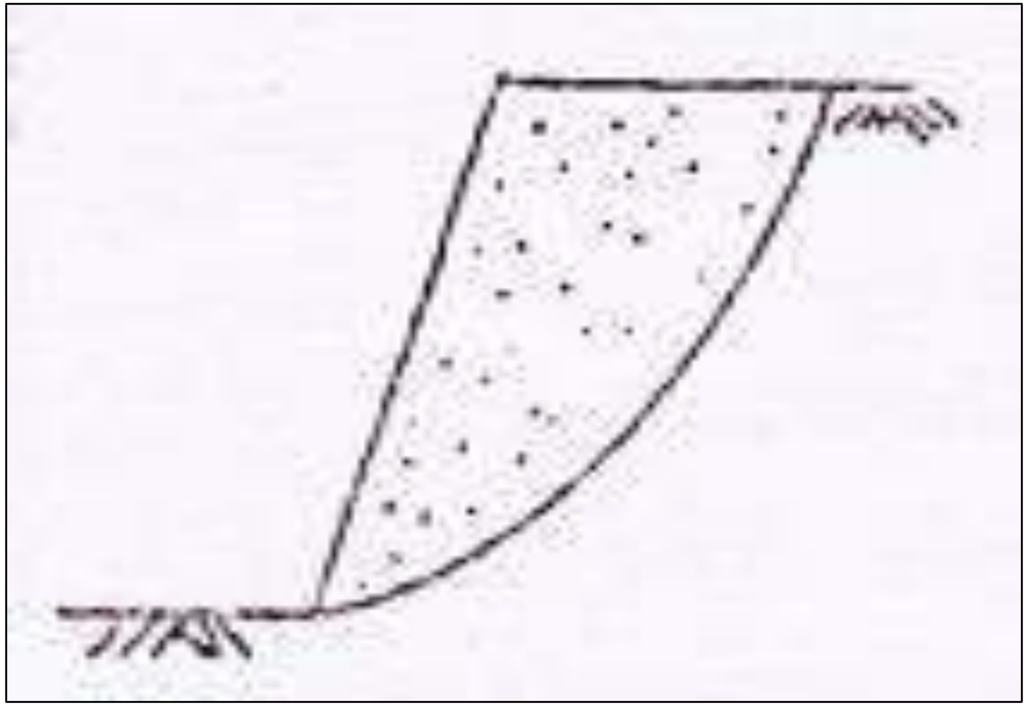
1. Rotational failures
2. Translational failures
3. Compound failures
4. Wedge failures
5. Miscellaneous failures

1. Rotational failures

- Occurs by rotation along a slip surface by downward and outward movement of soil mass
- Slip circle formed is circular for homogeneous soil and non-circular for non-homogeneous soils

a) Toe failure

- Occurs along surface that passes through the toe
- Most common failure
- occurs when the slope is steep and homogeneous.

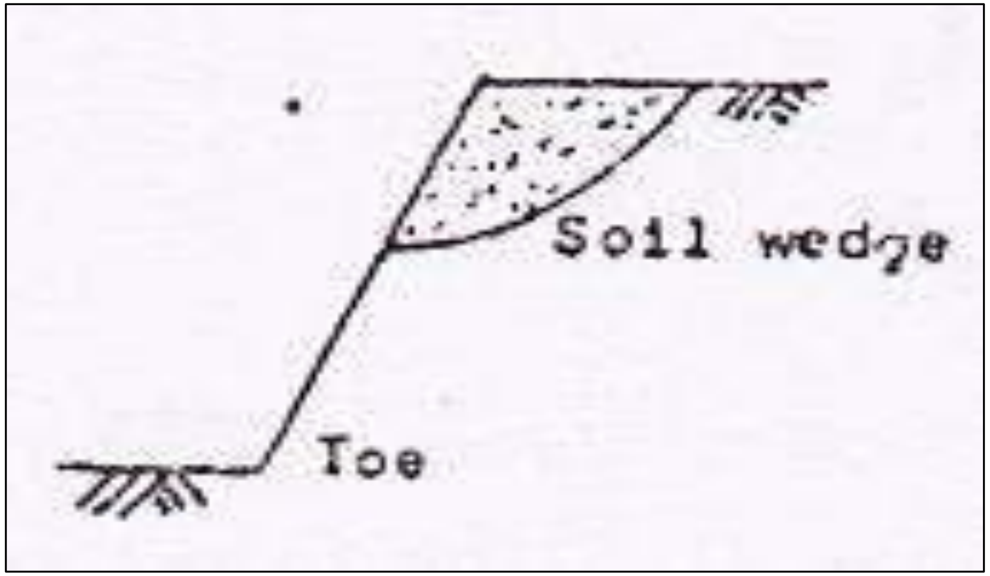


Toe failure

b) Slope failure

- Failure surface intersects the slope above the toe
- This type of failure occurs when the slope angle

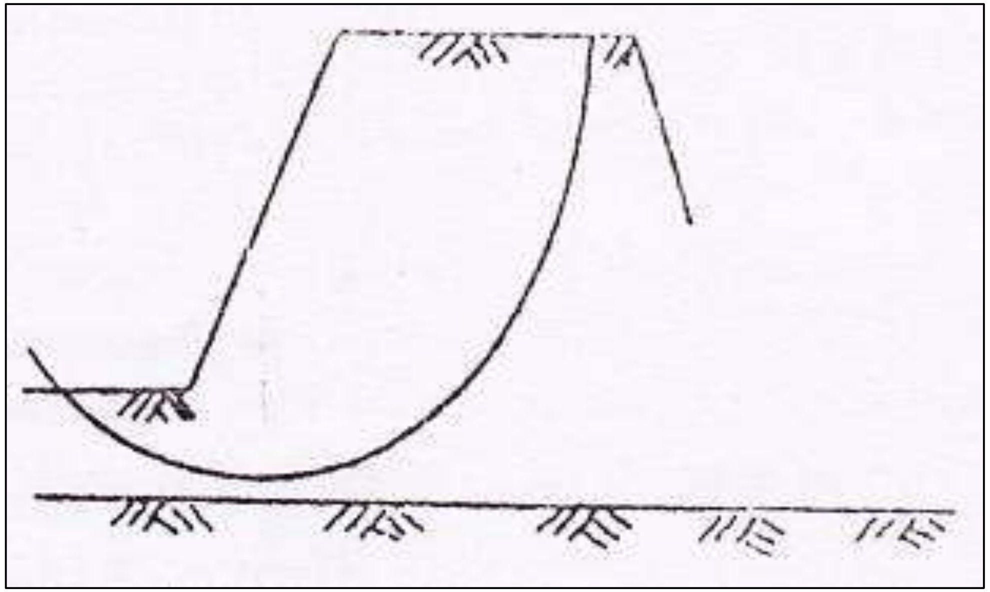
is large and when the soil at the toe portion is strong.



Slope failure

c)Base failure

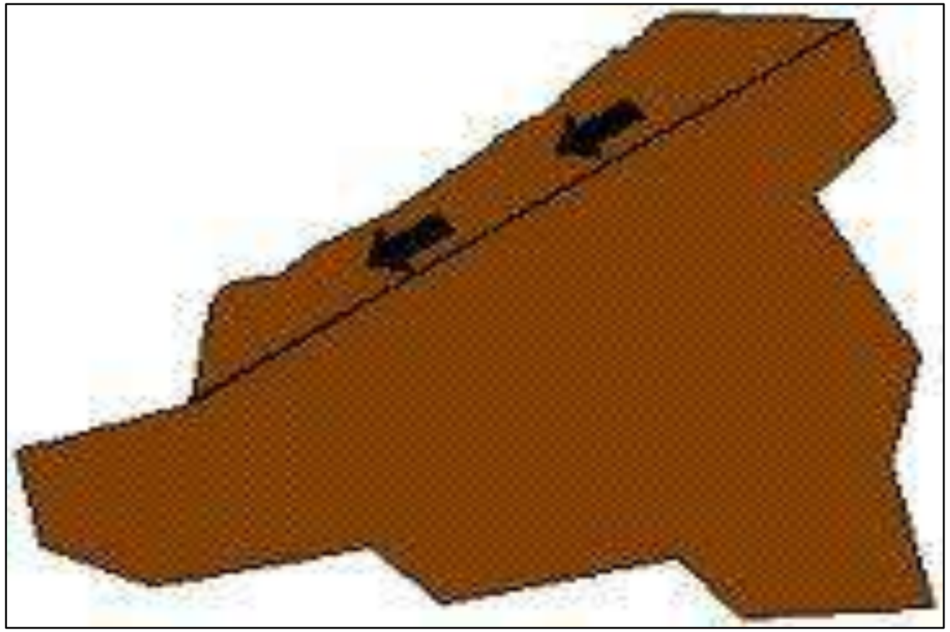
- Failure surface passes below the toe
- Occurs when weak stratum lies beneath the toe



Base failure

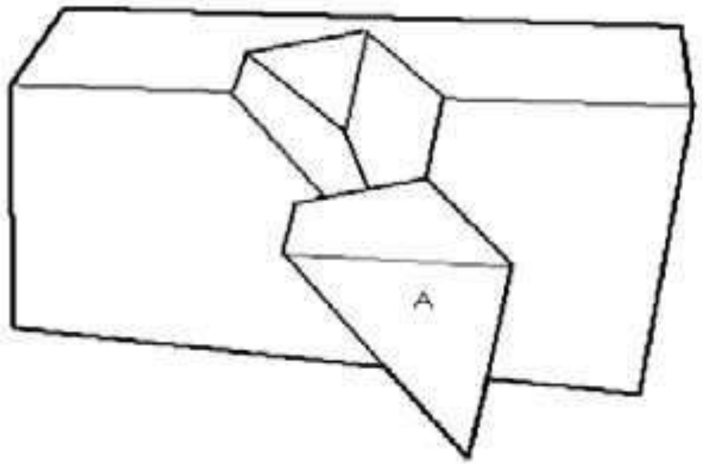
2. Translational Failure

- Occurs in infinite slopes along a long failure surface parallel to the slope
- Shape of failure surface influenced by presence of hard stratum at a shallow depth below slope surface
- Common in slopes of layered materials



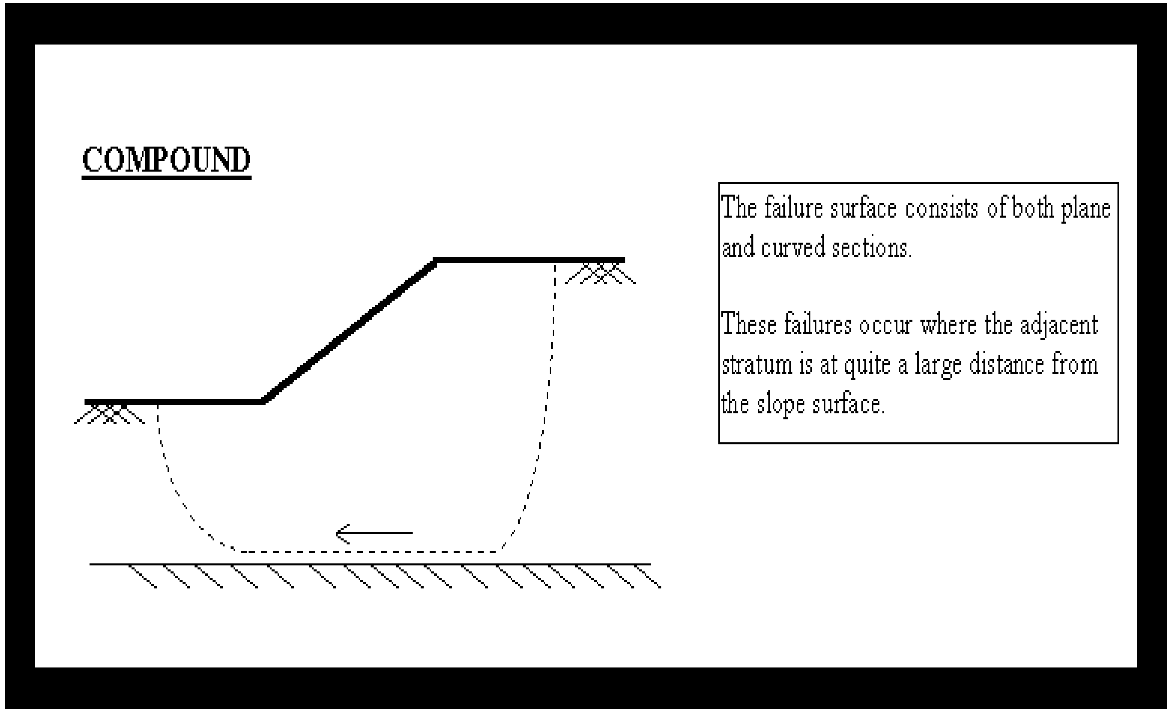
3. Wedge Failure

- Plane failure, wedge failure or block failure
- Failure along an inclined plane
- Occurs when distinct blocks and wedges of the soil mass become separated
- Similar to translational failure in many aspects
- Wedge failure can occur in finite slopes
  - Having two different materials
  - Homogeneous slopes with cracks, joints or any other specific plane of weakness



4. Compound Failures

- Combination of rotational and translational failures
- Failure surface is curved at both ends and is plane in the middle portion
- Occurs normally when a hard stratum is exists at a considerable depth below the toe



Infinite Slopes: Analysis

- Infinite slopes have dimensions that extended over great distances and the soil mass is inclined to the horizontal.

- Failure is assumed to occur along a plane parallel to the surface.

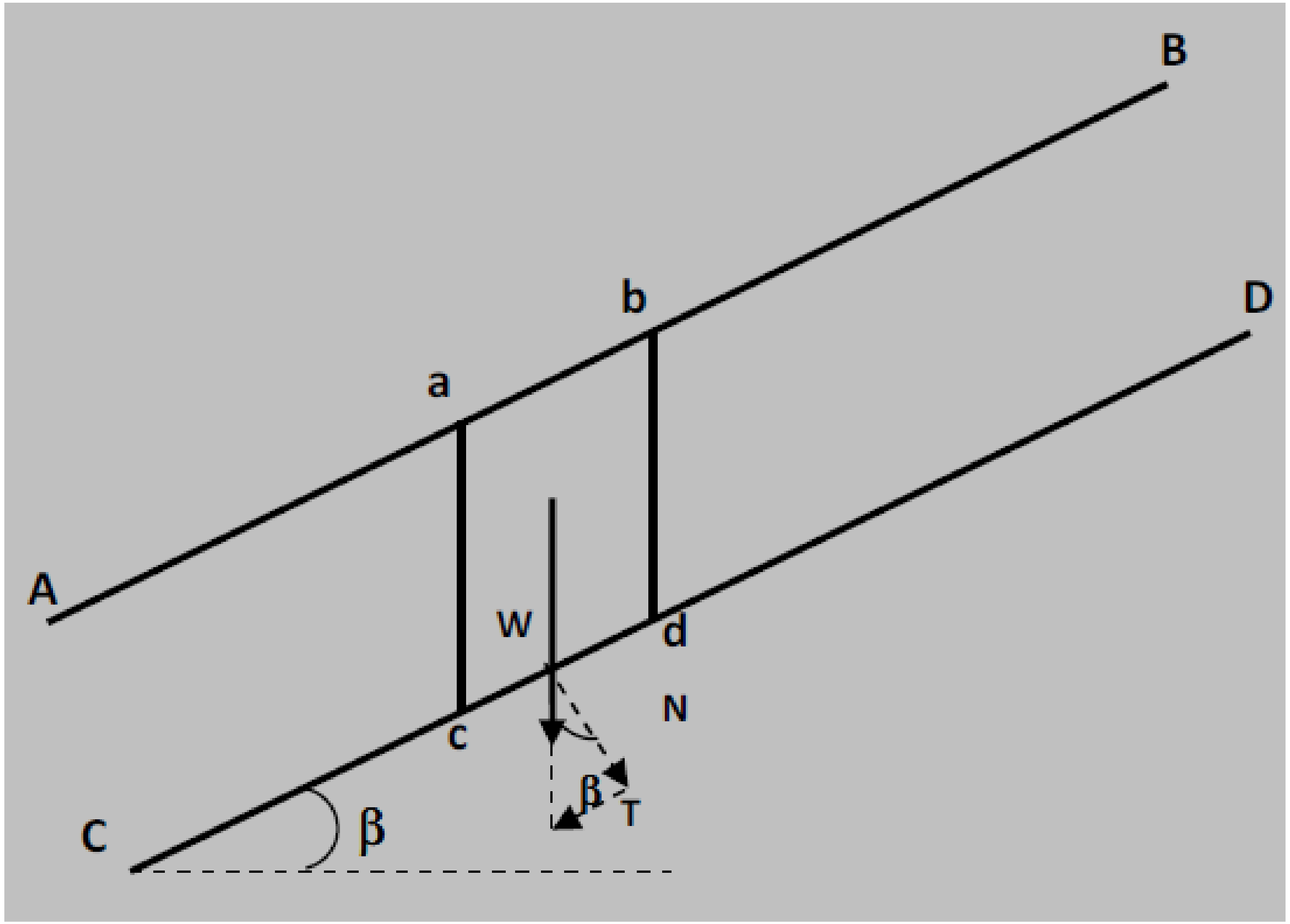
- Analysis cases

Case (i) Cohesionless soil

Case (ii) Cohesive soil

Case (iii) Cohesive-frictional soil

Infinite slopes in Cohesionless soils



Consider an infinite slope in a cohesionless soil inclined at an angle to the horizontal as shown.

Consider an element ‘abcd’ of the soil mass.

Let the weight of the element be W.

The component of W parallel to slope = T = W Sin β

The component of W perpendicular to slope = N = W Cos β

The force that causes slope to slide = T = W Sin β

The force that restrains the sliding of the slope = $\sigma \tan\phi$

$$= N \tan\phi = W \cos\beta \tan\phi$$

he factor of safety against sliding failure is

$$FS = \frac{\text{Restraining force}}{\text{Sliding force}} = \frac{W \cos\beta \tan\phi}{W \sin\beta}$$

$$FS = \frac{\tan\phi}{\tan\beta}$$

Find the factor of safety of a slope of infinite extent having a slope angle = 25°. The slope is made of cohesionless soil with $\phi = 30°$.

Solution

Factor of safety

$$F_s = \frac{\tan \phi'}{\tan \beta} = \frac{\tan 30°}{\tan 25°} = \frac{0.5774}{0.4663} = 1.238$$

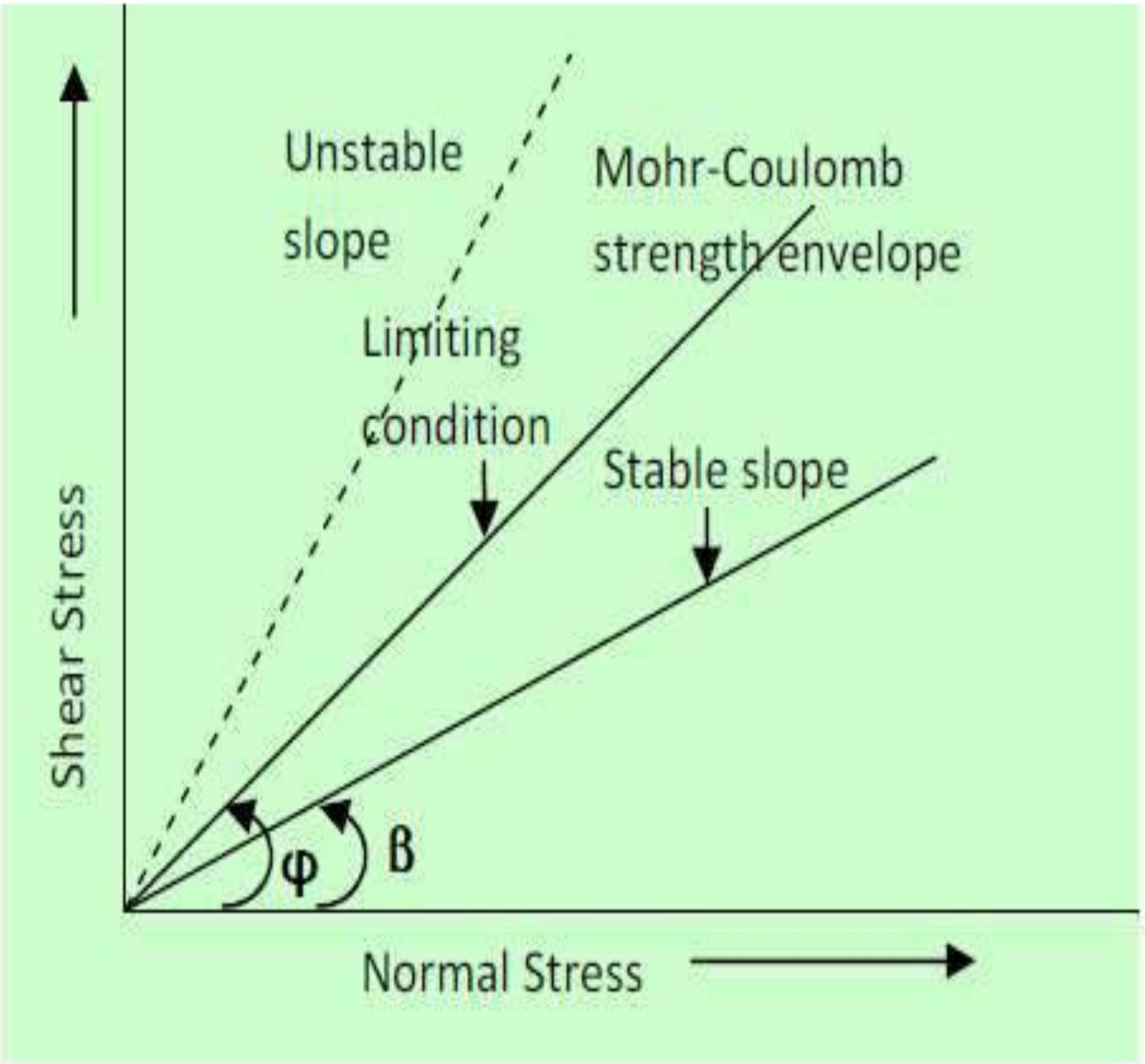
For limiting equilibrium ($F = 1$),

$$\tan \beta = \tan \phi$$

$$\beta = \phi.$$

"The maximum inclination of an infinite slope in a cohesionless soil for stability is equal to the angle of internal friction of the soil".

The limiting angle of inclination for stability of an infinite slope in cohesionless soil is as shown below.



$\beta < \phi$ - **Slope is stable**

$\beta = \phi$ - **Limiting condition**

$\beta > \phi$ - **Unstable**

Analysis of infinite slopes

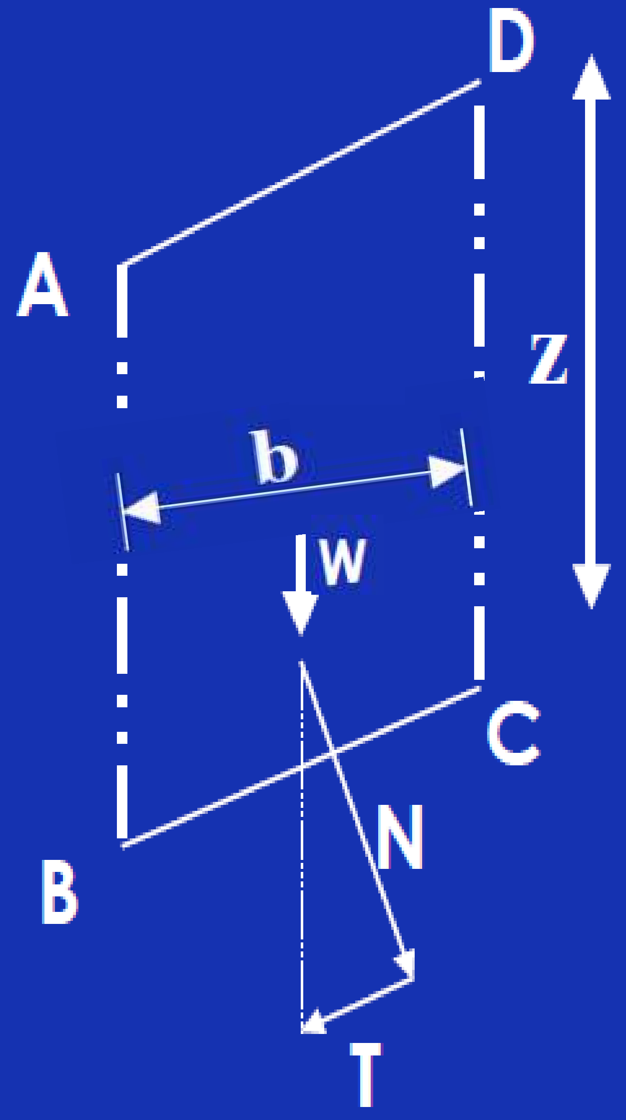
Weight of segment ABCD $W = \gamma z b(1)$

Tangential stress τ down the slope

$$\tau = \frac{\gamma z b \sin\beta}{b/\cos\beta} = \gamma z \sin\beta\cos\beta$$

Normal stress σ within the segment

$$\sigma = \frac{\gamma z b \cos\beta}{b/\cos\beta} = \gamma z \cos^2\beta$$



Infinite slope in pure cohesive soil

$$\tau_f = c + \sigma \tan\phi = c \quad \text{(cohesive soil)}$$

$$FS = \frac{\tau_f}{\tau_d} = \frac{c}{\gamma Z \, Sin\beta \, Cos\beta}$$

$$FS = \frac{c}{\gamma Z \, Sin\beta \, Cos\beta}$$

Infinite slope in cohesive frictional soil (C and φ)

$$FS = \frac{Shearing\,strength}{shearing\,stress} = \frac{c + \gamma Z \, Cos^2\beta \tan\phi}{\gamma Z \, Sin\beta \, Cos\beta}$$

Finite Slopes: Analysis

1. Swedish Circle/Arc Method/Method of slices/Standard method

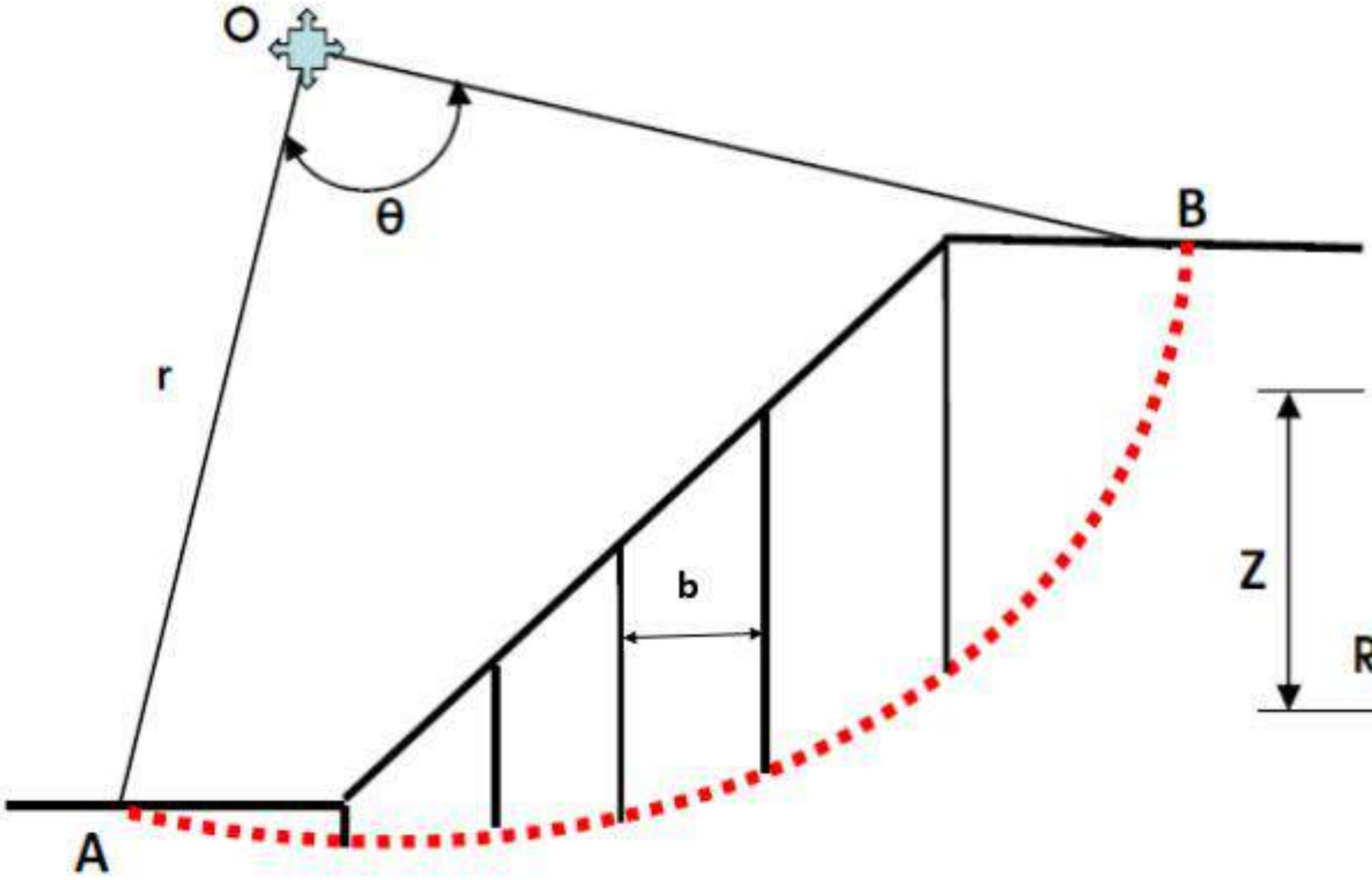
2. Bishop’s Simplified method

3. Taylor’s stability Number Method

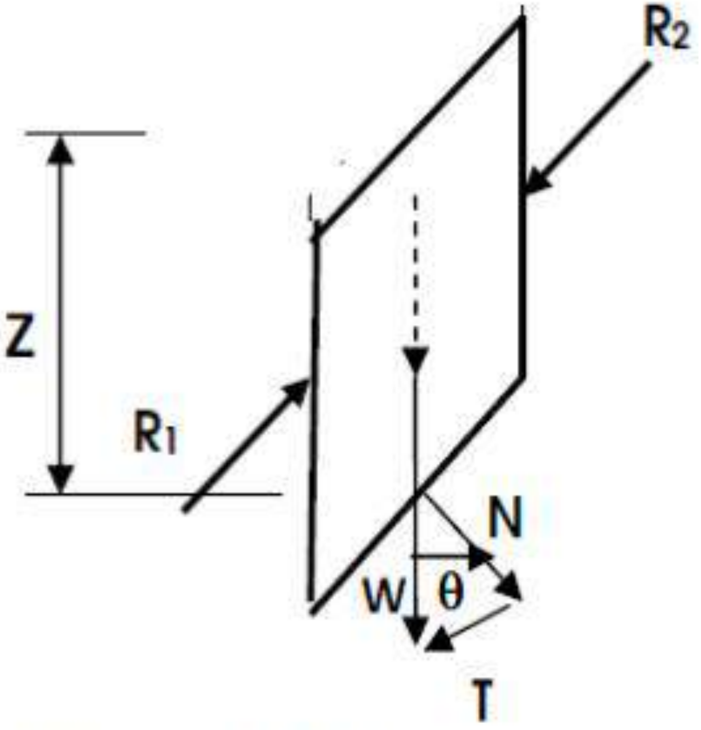
Swedish method of slices

- Adopted for a cohesive – frictional (c-Ø) soil

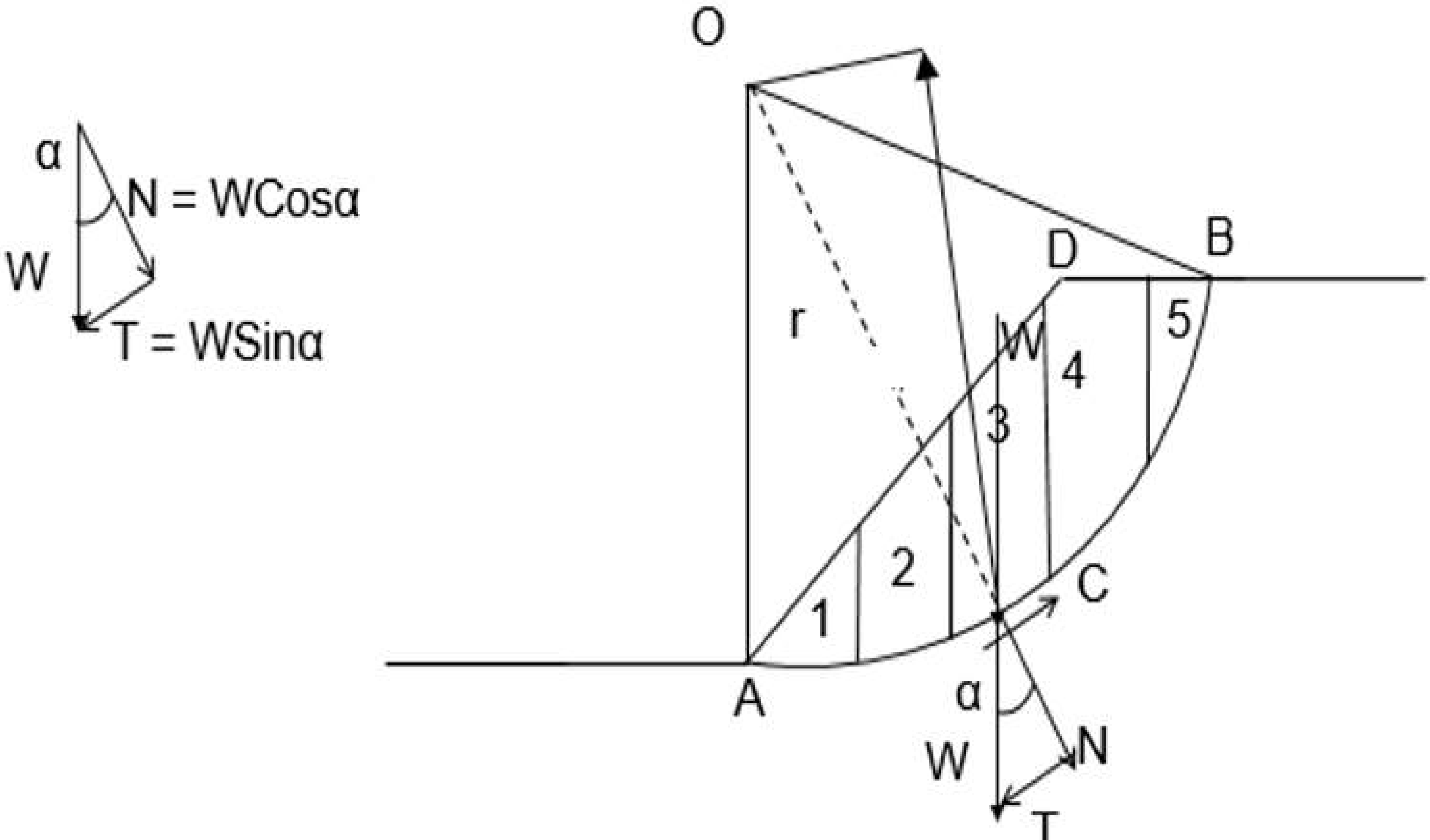
- The total stress analysis can be adopted.



Slip circle divided into slices



FBD on a typical slice



α
N = WCosα
W
T = WSinα
O
r
D
B
W
5
4
3
2
1
C
A
α
W
N
T

1. Draw the slope to scale
2. A trail slip circle such as AB with radius 'r' is drawn from the center of rotation O.
3. Divide the soil mass above the slip surface into convenient number of slices (more than 5 is preferred)
4. Determine the area of each slice A1, A2, -------, An

$$A = \text{width of the slice X mid height}$$

$$= b \times Z$$

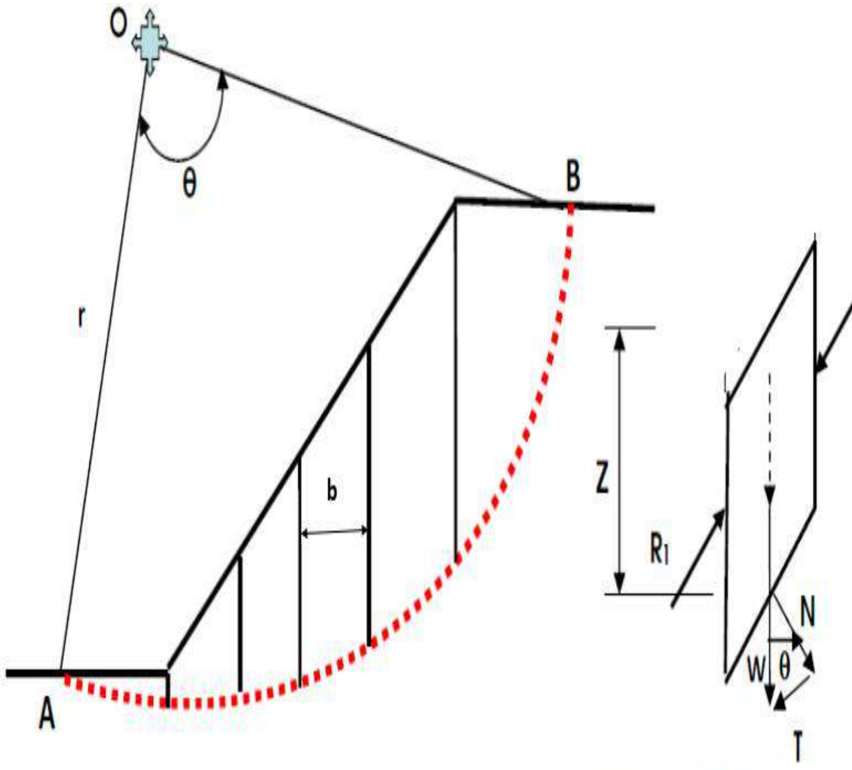
5. Determine the total weight W including external load if any as

$$W = \gamma\, b\, Z = \gamma\, A$$

Where, γ = unit weight

b = width of slice

Z = height of slice.



Slip circle divided into slices

FBD on a typical slice

The reactions R_1 and R_2 on the sides of the slice are assumed equal and therefore do not have any effect on stability.

6. The weight W of the slice is set –off at the base of the slice. The directions of its normal component 'N' and the tangential component 'T' are drawn to complete the vector triangle.

   $N = W \cos\delta, \; T = W \sin\delta$

7. The values of N and T are scaled off for each of the slices

8. The values of 'N' and 'T' are tabulated and summed up as shown in the following table.

9. The factor of safety is calculated as follows

$$Sliding\ moment = r \sum T\ (reckoned\ positive\ if\ clockwise)$$

$$Restoring\ moment = r\ (c\, r\, \theta + \sum N \tan\phi)\ (reckoned\ positive\ if\ counterclockwise)$$

$$Factor\ of\ safety, FS = \frac{(c\, r\, \theta + \sum N \tan\phi)}{\sum T}$$

Table 1: Normal and tangential components of various slices in the slope

Slice No.	Area m^2	Weight W (kN)	Normal component N (kN) $N = W \cos\delta$	Tangential components T (kN) $T = W \sin\delta$
1				
2				
3				
			Sum, ΣN = ___ kN	Sum, ΣT = ___ kN

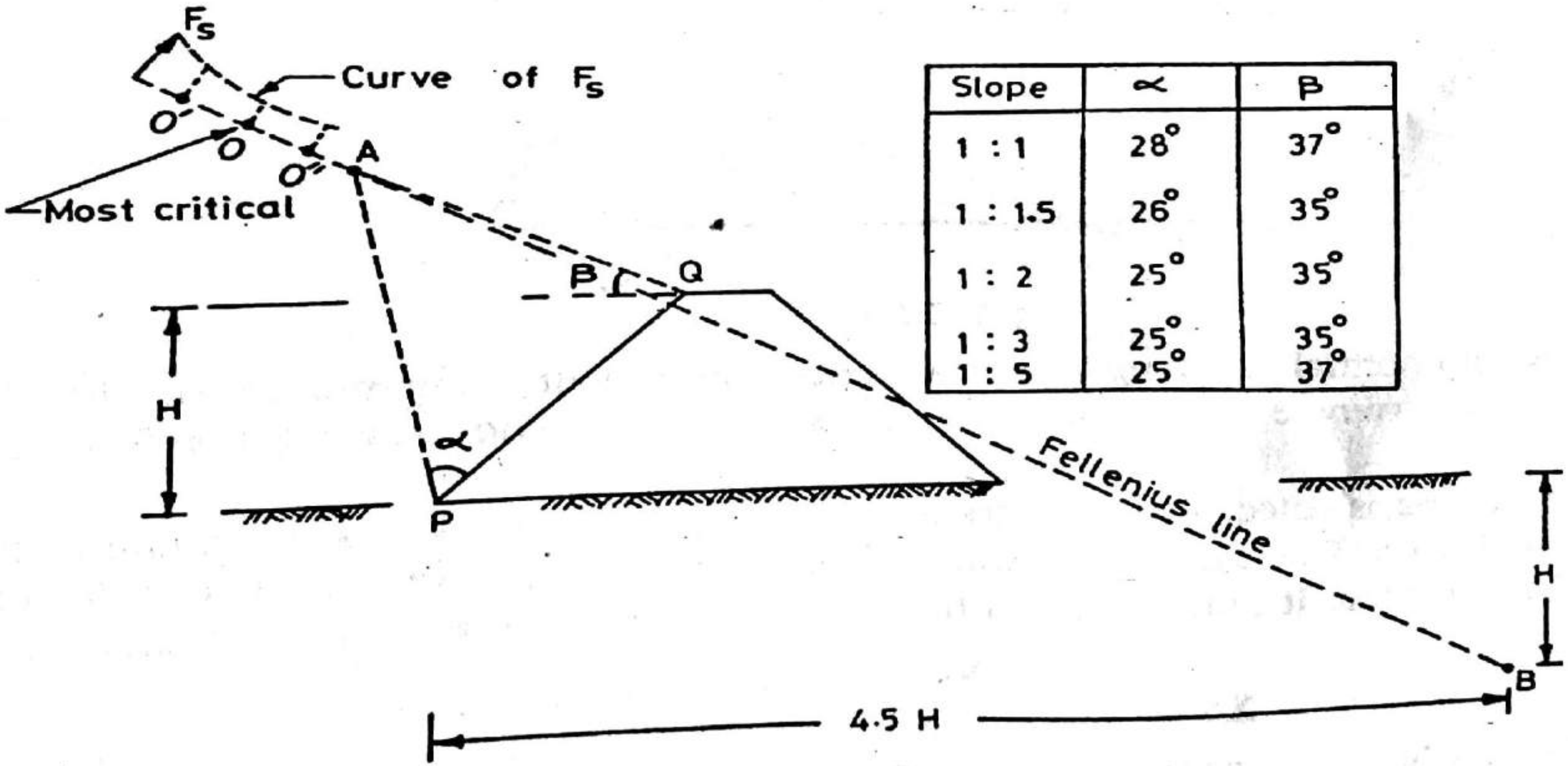
10. Repeat step 2 to 9 by considering various trial slip circles and calculate FS for each of these slip circles. The slip circle with a minimum FS is called critical slip circle.

... which gives the *minimum* factor of safety is the most critical circle.

Location of Most Critical Circle

In order to reduce the number of trials to locate the most critical circle, the Fellenius line *AB* can be drawn (Fig. 18.16). Fellenius has shown that the centre of the most critical circle lies on this line. For drawing the Fellenius line *AB*, the point *B* is located at a depth *H* and at a distance 4.5 *H* from point *P* at the toe of the slope, where *H* is the height of the slope. The point *A* is located by drawing two lines *PA* and *QA*, where *PA* makes angle α with the slope line *PQ* and *QA* makes angle β with the horizontal at *Q*. The angles α and β are obtained from the table given in Fig. 18.16. The angles depend upon the slope.

The centre of the most critical circle may lie anywhere on the line *AB* or its extension. The centres of



Slope	α	β
1 : 1	28°	37°
1 : 1.5	26°	35°
1 : 2	25°	35°
1 : 3	25°	35°
1 : 5	25°	37°

Fig. 18.16. Fellenius Line.

trial circles are taken on this line shown as O'', O and O'. The factors of safety obtained when these were the trial centres are generally plotted as normals to the line AB to obtain a curve of F_s. The centre corresponding to the minimum factor of safety indicates the most critical circle.

For a purely cohesive soil ($\phi = 0$), the point A itself represents the centre of most critical circle.

The Swedish circle method is a general method of slope stability analysis. It can be used for non-homogeneous soil masses, stratified deposits, fully submerged or partly submerged conditions. The method is also applicable when seepage occurs and pore pressure develops in the soil masses, as explained later.

However, the method is necessarily an approximate one, as it neglects the effect of forces acting on the sides of the vertical strips. Forunately, the method errs on the safe side, *i.e.* the factor of safety obtained is generally less than that obtained from the more accurate methods, such as Bishop's method, which also consider the forces on the sides of the vertical strips.

Example 9.3: An embankment 10 m high is inclined at an angle of 36° to the horizontal. A stability analysis by the method of slices gives the following forces per running meter:

$$\Sigma \text{ Shearing forces} = 450 \text{ kN}$$

$$\Sigma \text{ Normal forces} = 900 \text{ kN}$$

$$\Sigma \text{ Neutral forces} = 216 \text{ kN}$$

The length of the failure arc is 27 m. Laboratory tests on the soil indicate the effective values c' and ϕ' as 20 kN/m^2 and 18° respectively.

Determine the factor of safety of the slope with respect to (*a*) shearing strength and (*b*) cohesion.

(*a*) Factor of safety with respect to shearing strength

$$F_s = \frac{c' r\theta + \{\Sigma(N - U)\}\tan\phi'}{\Sigma T}$$

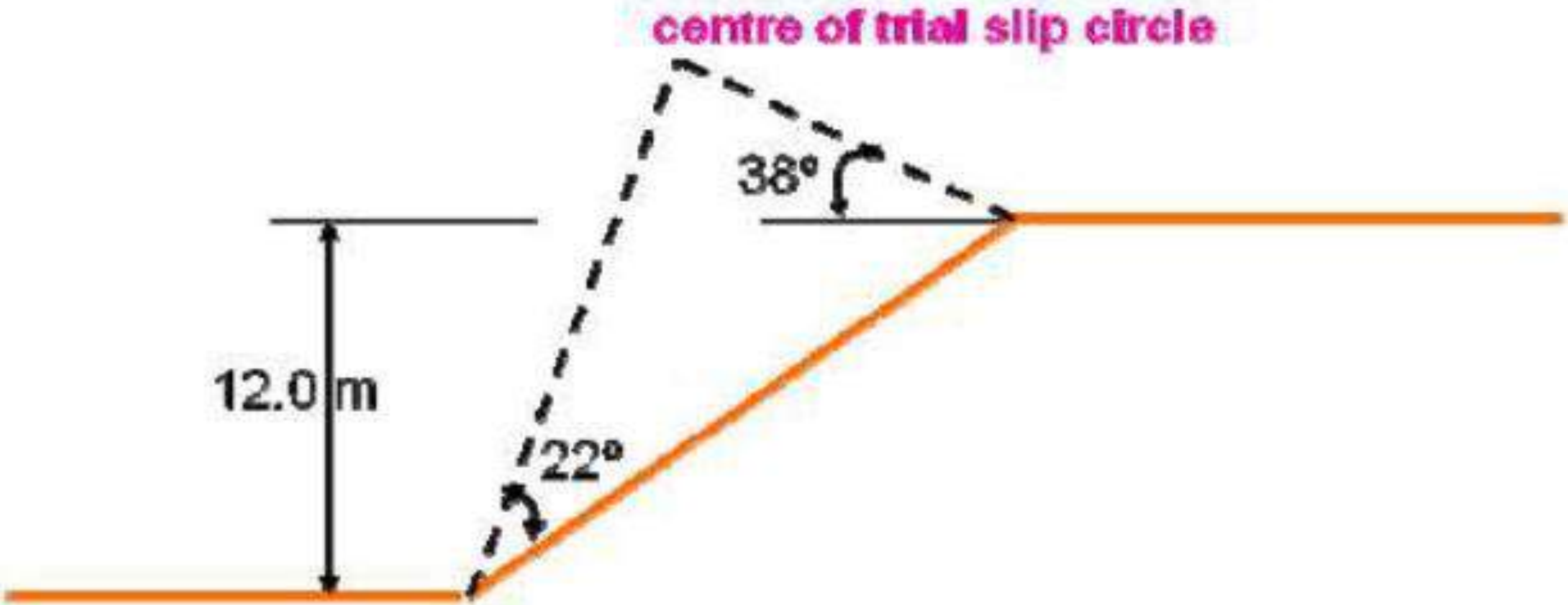
$$= \frac{20 \times 27 + (900 - 216)\tan 18^\circ}{450} = \mathbf{1.70}$$

(*b*) Factor of safety with respect to cohesion

$$F_c = \frac{c' r\theta}{\Sigma T}$$

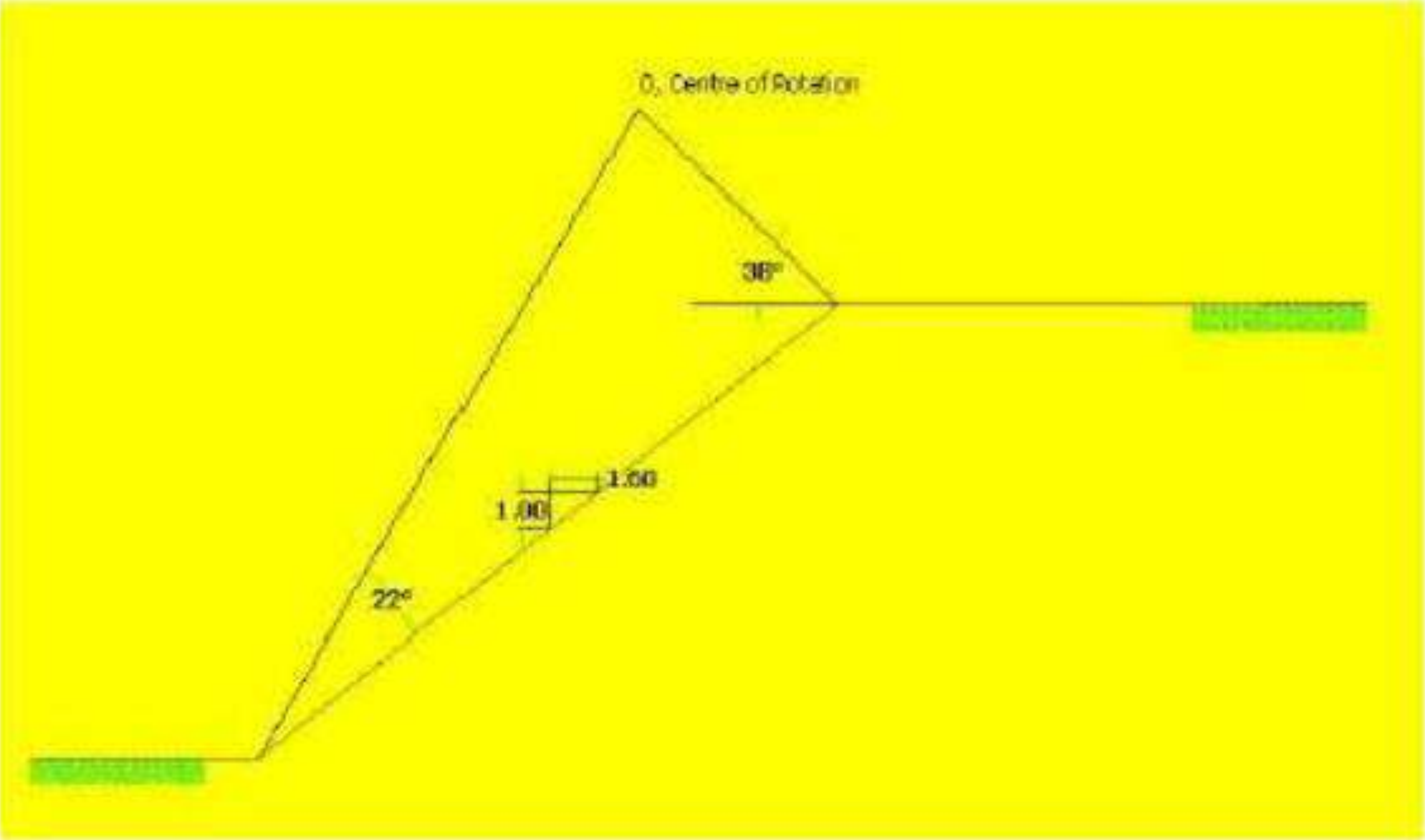
$$= \frac{20 \times 27}{450} = \mathbf{1.20.}$$

An embankment is to be made of a sandy clay, having cohesion of 30 kN/m^2, Angle of internal friction 20° and a unit weight of 18 kN/m^3. The slope and height of embankment are 1.6:1 and 12.0 m respectively. Determine the factor of safety by using the trial circle given in Fig by method of slices



Angle of internal friction $\phi = 20°$

Unit weight $\gamma = 18 kN/m^3$



Step 1: Draw the slope to convenient scale.

Step 2: From the centre of rotation draw the slip surface.

Step 3: Divide the failure plane into 9 slices as shown.

Step 4: Measure the average depth 'Z' and the breadth 'B' of each slice and calculate the areas of each of these slices.

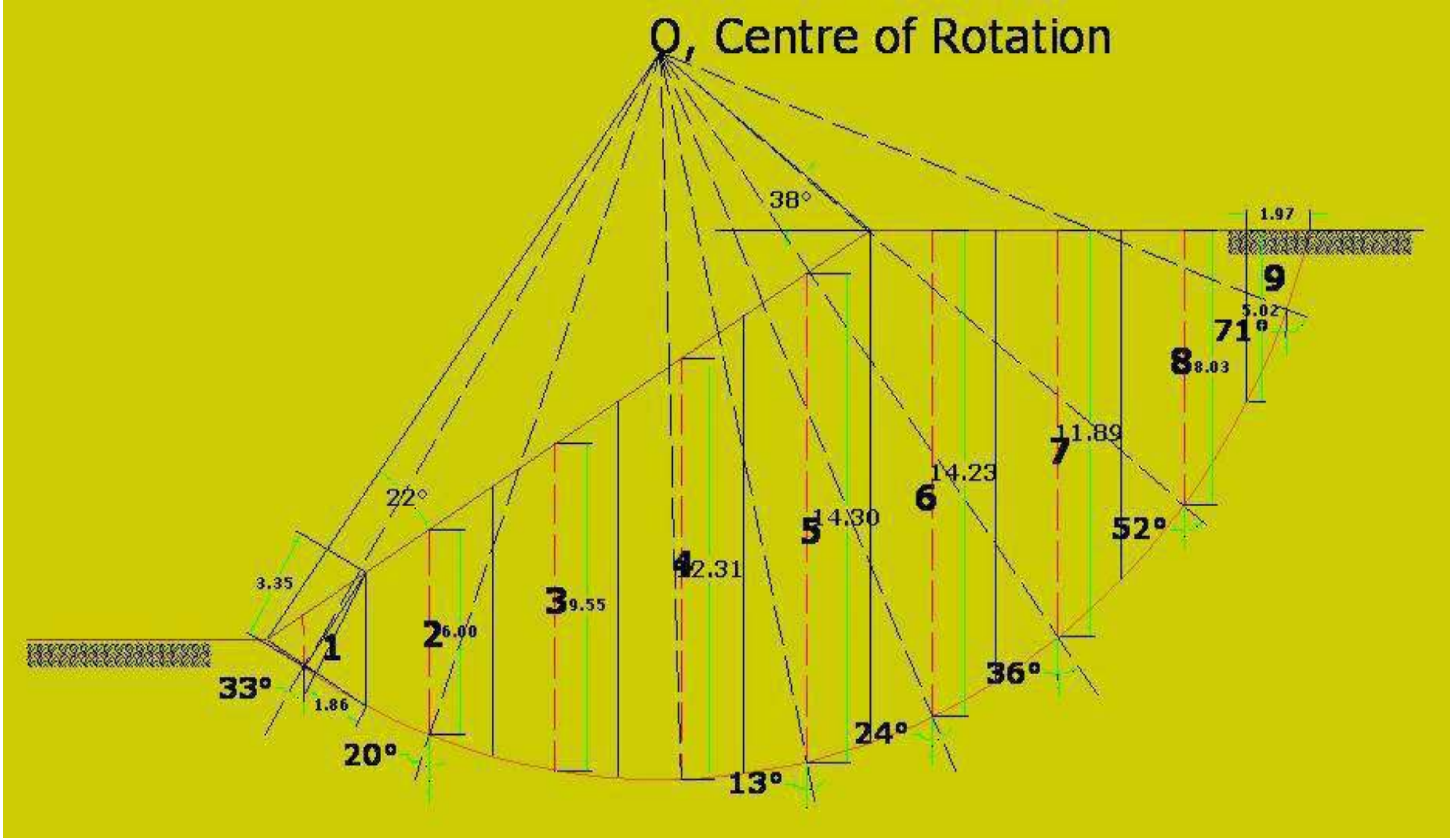
Step 5: Calculate the weight of each slices.

$$W = \gamma A$$

Step 6: Calculate the angle 'δ' for each of the slices.

Step 7: Tabulate all the values as shown in the Table and calculate 'N' & 'T' components.

Step 8: Obtain ΣN and ΣT.



O, Centre of Rotation
38°
1.97
9
5.02
71°
8
8.03
7
11.89
6
14.23
5
14.30
4
12.31
3
9.55
2
6.00
1
22°
3.35
33°
1.86
20°
13°
24°
36°
52°

Slice No.	Z	B	Area (m2)	Weight (kN/m)	δ (deg)	N = W Cos δ	T = W Sin δ
1			6.23	112.16	-33	94.06	-61.08
2	6	4	24.00	432.00	-20	405.94	-147.75
3	9.55	4	38.20	687.60	-9	679.13	-107.56
4	12.31	4	49.24	886.32	0	886.32	0.00
5	14.3	4	57.20	1029.60	13	1003.21	231.60
6	14.23	4	56.92	1024.56	24	935.98	416.72
7	11.89	4	47.56	856.08	36	692.58	503.19
8	8.03	4	32.12	578.16	52	355.95	455.59
9			4.82	86.75	71	28.24	82.02
						ΣN = 5081.41	ΣT = 1372.73

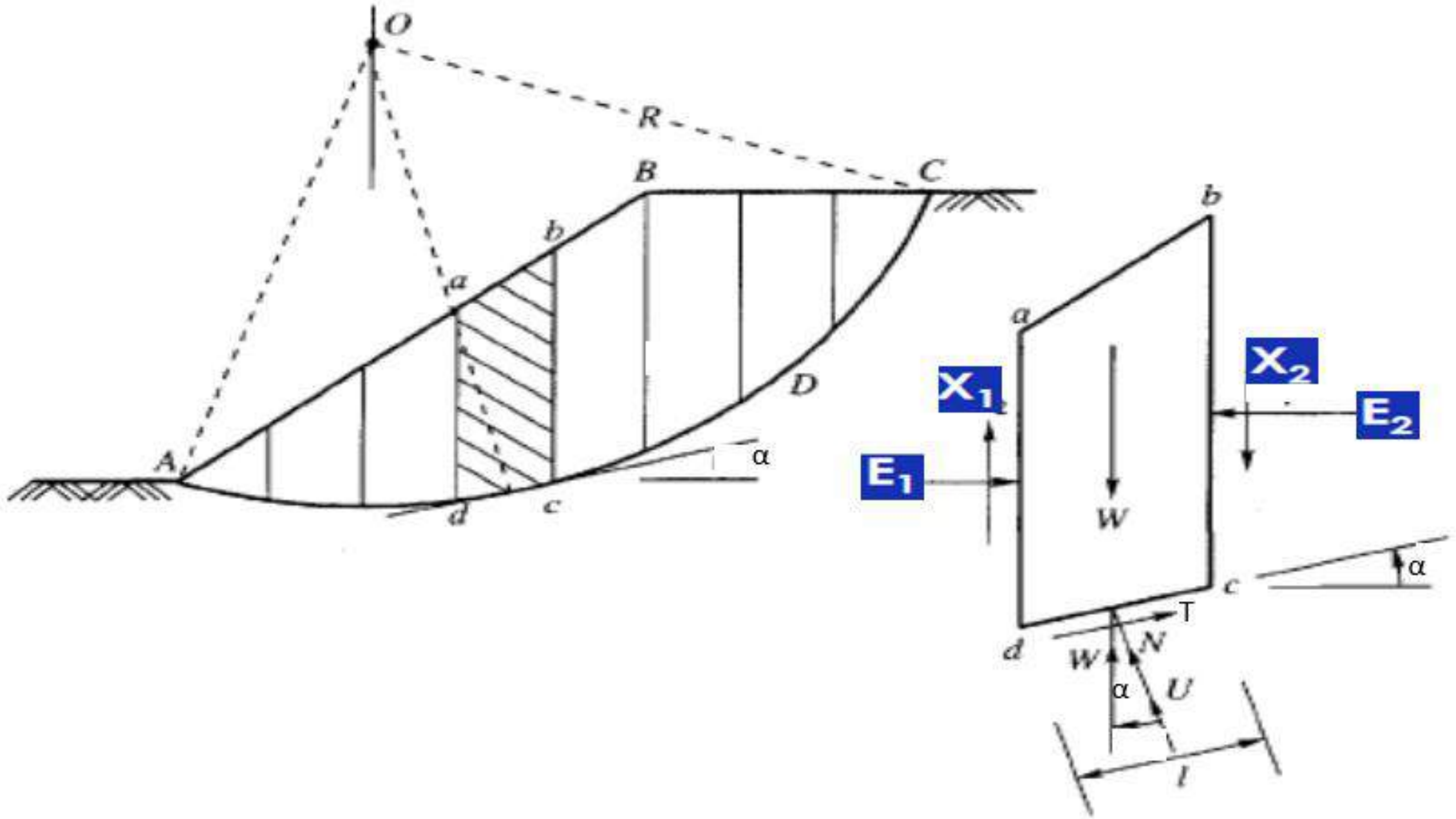
Step 9: Obtain F.S. as

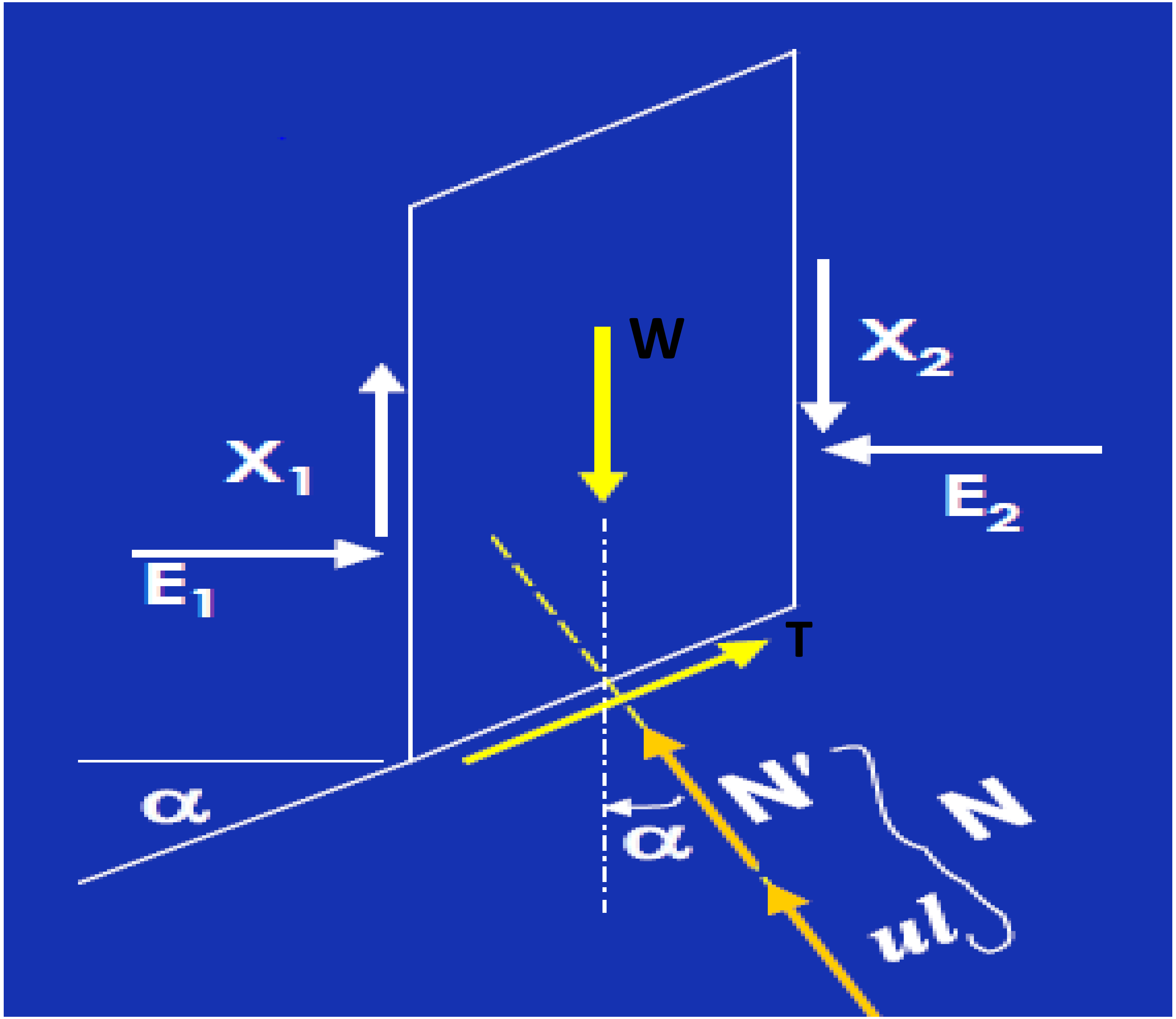
$$\text{Factor of safety, FS} = \frac{(c\,r\,\theta + \sum N \tan\phi)}{\sum T}$$

$$\text{Factor of safety, FS} = \frac{\left(30 \times 21.28 \times 88 \times \dfrac{\pi}{180} + 5081.41 \times \tan 20^{\circ}\right)}{1372.73}$$

$$\text{Factor of safety, FS} = 2.06$$

BISHOP'S SIMPLIFIED METHOD OF SLICES





W
X_2
X_1
E_2
E_1
T
α
α
N'
N
ul

W = weight of the slice

N = total normal force on the failure surface *cd*

U = pore water pressure = ul on the failure surface *cd*

T = shear resistance acting on the base of the slice

X_1, X_2 = normal forces on the vertical faces *bc* and *ad*

E_1, E_2 = shear forces on the vertical faces *bc* and *ad*

α = the inclination of the failure surface *cd* to the horizontal

Bishop simplified Method (BSM)

In this solution it is assumed that the resultant forces on the sides of the slices are horizontal. i.e $X_1 - X_2 = 0$

For equilibrium the shear force on the base of any slice is:

$$T = \frac{1}{FS}\left(c'l + N'\tan\phi'\right)$$

Resolving forces in the vertical direction:

$$W = N'\cos\alpha + ul\cos\alpha + \frac{c'l}{FS}\sin\alpha + \frac{N'}{FS}\tan\phi'\sin\alpha$$

After some rearrangement and using l = b secα:

$$FS = \frac{1}{\sum W\sin\alpha}\sum\left([c'b + (W - ub)\tan\phi']\frac{\sec\alpha}{1 + (\tan\alpha\tan\phi'/FS)}\right)$$

- Bishop's simplified method (BSM) considers the inter slice normal forces but neglects the inter slice shear forces. It further satisfies vertical force equilibrium to determine the effective base normal force(N').

In summary, BSM

- satisfies moment equilibrium for FOS,
- satisfies vertical force equilibrium for N,
- considers interslice normal force,
- more common in practice, and
- applies mostly for circular shear surfaces.

TAYLOR STABILITY NUMBER METHOD

- If the slope angle β , height of embankment H, the effective unit weight of material ɣ, angle of internal friction φ and unit cohesion c are known, the factor of safety may be determined.

- Taylor (1937) conceived the idea of analyzing the stability of a large number of slopes through a wide range of slope angles and angles of internal friction, and then representing the results by an abstract number which he called the "stability number". This number is designated as N

TAYLOR STABILITY NUMBER AND CHART

- **Stability Number** is defined as $S_n = c / (F_c\gamma H) = c_m / (\gamma H)$
- Mobilized cohesion $c_m = c / F_c$
- Reciprocal of **Stability Number- Stability factor**
- Stability number- dimensionless quantity
- For analysis of simple of simple sections and of homogeneous soils
- Charts prepared indicating Stability Number and slope angle β for different values of Φ

TAYLOR STABILITY NUMBER AND CHART

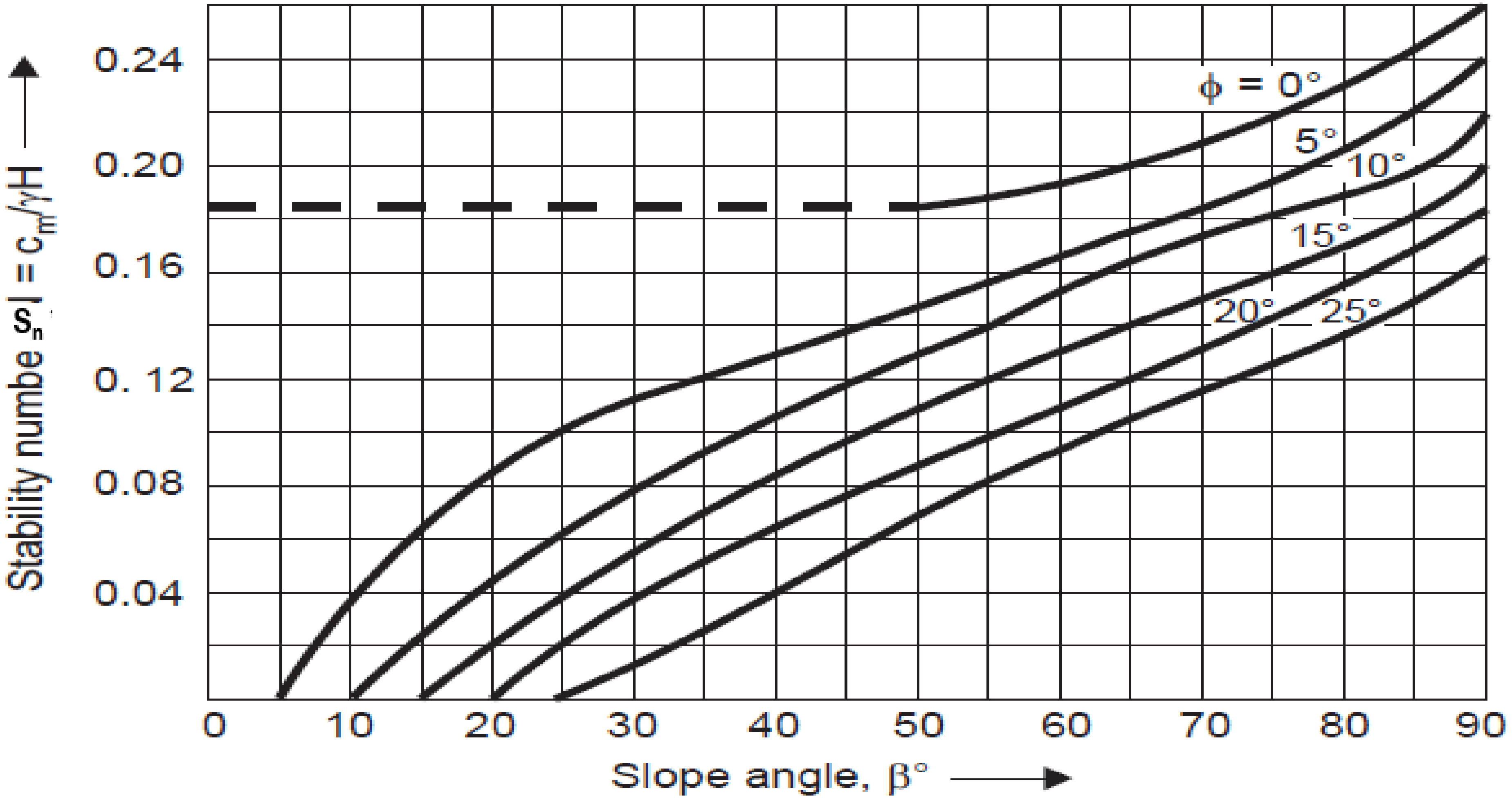


Fig. 9.25 Taylor's charts for slope stability (After Taylor, 1948)

$c - \phi$ soil with the angle of slope less than 53°, $\phi = 0$ and a slope angle greater than 53°

TAYLOR STABILITY NUMBER AND CHART

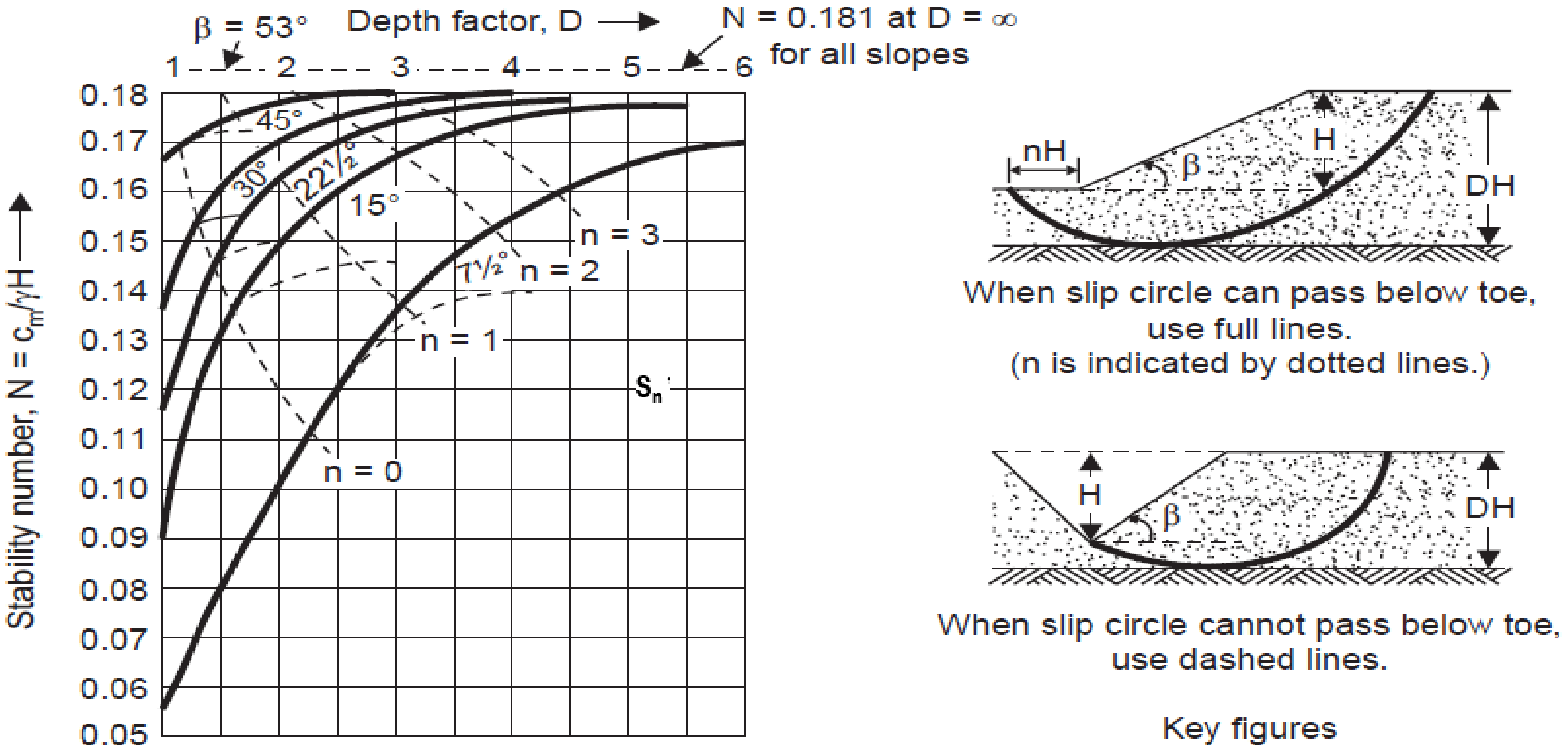


Fig. 9.26 Taylor's chart for slopes with depth limitation

slope angle being less than 53° | ϕ = 0 and a layer of rock or stiff material at a depth *DH* below the top of the embankment

- For cohesive soils, the stability number is related to parameter D
- D = Depth of hard stratum below the top of slope / Height of slope

TAYLOR STABILITY NUMBER AND CHART

- Stability number can be used to determine the factor of safety

  $F_c = c / c_m = c / (\mathbf{S_n} * \gamma * H)$

- Stability charts can be used to determine the steepest slope for a given factor of safety

Example 9.4: An embankment is inclined at an angle of 35° and its height is 15 m. The angle of shearing resistance is 15° and the cohesion intercept is 200 kN/m². The unit weight of soil is 18.0 kN/m³. If Taylor's stability number is 0.06, find the factor of safety with respect to cohesion. (S.V.U.—B.Tech. (Part-time)—Apr., 1982)

$$\beta = 35^\circ$$
$$H = 15 \text{ m}$$
$$\phi = 15^\circ$$
$$c = 200 \text{ kN/m}^2$$
$$\gamma = 18.0 \text{ kN/m}^3$$

Taylor's stability Number $S_n = 0.06$

Since $$S_n = \frac{c_m}{\gamma H}$$

∴ $$0.06 = \frac{c_m}{18 \times 15}$$

∴ Mobilised cohesion,

$$c_m = 0.06 \times 18 \times 15 \text{ kN/m}^2$$
$$= 16.2 \text{ kN/m}^2$$

Cohesive strength $c = 200$ kN/m²

∴ Factor of safety with respect to cohesion:

$$F_c = \frac{c}{c_m} = \frac{200}{16.2} \approx \mathbf{12.3.}$$

Example 9.6: A cutting is to be made in clay for which the cohesion is 35 kN/m² and $\phi = 0°$. The density of the soil is 20 kN/m³. Find the maximum depth for a cutting of side slope $1\frac{1}{2}$ to 1 if the factor of safety is to be 1.5. Take the stability number for a $1\frac{1}{2}$ to 1 slope and $\phi = 0°$ as 0.17.

(S.V.U.—B.E., (N.R.)—Apr., 1966)

$$c = 35 \text{ kN/m}^2 \qquad \phi = 0°$$

$$\gamma = 20 \text{ kN/m}^3 \qquad S_n = 0.17$$

$$F_c = 1.5$$

$$\therefore \quad c_m = c/F_c = 35/1.5 = \frac{70}{3} \text{ kN/m}^2$$

$$\text{But} \quad S_n = \frac{c_m}{\gamma H}$$

$$\therefore \quad 0.17 = \frac{c_m \times 100}{20 \times H} = \frac{70 \times 100}{3 \times 20 \times H}$$

$$\therefore \quad H = \frac{70 \times 100}{60 \times 0.17} \text{ cm}$$

$$= \mathbf{6.86 \text{ m.}}$$

An embankment is to be constructed with $c = 20$ kN/m^2, $\phi = 20°$, $\gamma = 18$ kN/m^3, FS =1.25 and height is 10 m. Estimate side slope required. Taylor's Stability Numbers are as follows for various slope angles.

Slope angle	90	75	60	45	30	20	10
S_n	0.182	0.134	0.097	0.062	0.025	0.005	0

Also find the factor of safety, if the slope is 1V: 2H given $\phi = 20°$

Part-A

$$\text{Factor of safety, FS} = \frac{C}{S_n \times \gamma_w \times H}$$

$$\textbf{Stability number, } S_n = \frac{C}{FS \times \gamma \times H}$$

$$S_n = \frac{20}{1.25 \times 18 \times 10}$$

Stability number, $S_n = 0.088$

Therefore Slope angle $\beta = 56.14°$

Part-B

$$\text{Slope angle } \beta = \tan^{-1}\frac{1}{2}$$

$$\beta = 26.5^\circ$$

For $\beta = 26.5^\circ$, Taylor's Stability Number $S_n = 0.018$

$$\text{Factor of safety, } FS = \frac{C}{S_n \times \gamma \times H}$$

$$= \frac{20}{0.018 \times 18 \times 10}$$

$$FS = 6.17$$

Example 9.7: At cut 9 m deep is to be made in a clay with a unit weight of 18 kN/m³ and a cohesion of 27 kN/m². A hard stratum exists at a depth of 18 m below the ground surface. Determine from Taylor's charts if a 30° slope is safe. If a factor of safety of 1.50 is desired, what is a safe angle of slope ?

Depth factor $D = 18/9 = 2$

From Taylor's charts,

for $D = 2; \beta = 30°$

$S_n = 0.172$

$$0.172 = \frac{c_m}{18 \times 9}$$

$c_m = 0.172 \times 18 \times 9 = 27.86 \text{ kN/m}^2$

$c = 27 \text{ kN/m}^2$

$F_c = c/c_m = 27/27.86 = 0.97$

The proposed slope is therefore not safe.

For $F_c = 1.50$

$c_m = c/F_c = 27/1.5 = 18 \text{ kN/m}^2$

$S_n = 18/18 \times 9 = 1/9 = 0.11$

For $D = 2.0$, and $S_n = 0.11$

from Taylor's charts,

we have $\beta = 8°$

$\therefore$ Safe angle of slope is 8°.

Critical height of slope Hc

- It is the maximum height a slope can have assuming activation of full cohesion

- Height after applying a certain factor of Safety

$$\boldsymbol{H_c = F_c \times H}$$

Question-

What is the maximum unsupported height of a vertical-cut in pure clay???

$4c/\gamma$ nearly

Submerged and sudden drawdown condition of slope

If the slope is submerged, the effective unit weight γ' instead of γ is to be used.

$$\gamma' = \gamma_{sat} - \gamma_w$$

For the case of sudden drawdown, the saturated unit weight γ_{sat} is to be used for γ; in addition, a reduced value of ϕ, ϕ_w, should be used, where:

$$\phi_w = (\gamma'/\gamma_{sat}) \times \phi$$

Example 9.8: A canal is to be excavated through a soil with $c = 15\ \text{kN/m}^2$, $\phi = 20°$, $e = 0.9$ and $G = 2.67$. The side slope is 1 in 1. The depth of the canal is 6 m. Determine the factor of safety with respect to cohesion when the canal runs full. What will be the factor of safety if the canal is rapidly emptied ?

$$\gamma_{sat} = \left(\frac{G+e}{1+e}\right).\gamma_w = \left(\frac{2.67 \times 0.90}{1+0.90}\right) \times 9.81\ \text{kN/m}^3$$

$$= \frac{3.57}{1.90} \times 9.81\ \text{kN/m}^3 = 18.43\ \text{kN/m}^3$$

$$\gamma' = \gamma_{sat} - \gamma_w = 8.62\ \text{kN/m}^3$$

$$\beta = 45°,\ \phi = 20°.$$

(*a*) Submerged condition:

From Taylor's charts, for these values of β and ϕ, the stability number S_n is found to 0.06.

$$\therefore \qquad 0.06 = \frac{c_m}{\gamma'.H} = \frac{c_m}{8.62 \times 6}$$

$$c_m = 8.62 \times 6 \times 0.06\ \text{kN/m}^2 = 3.10\ \text{kN/m}^2.$$

Factor of safety with respect to cohesion, $F_c = c/c_m = 15/3.10 =$ **4.48.**

(b) Rapid drawdown condition:

$$\phi_w = (\gamma'/\gamma_{sat}) \times \phi = (8.62/18.43) \times 20° = 9.35°$$

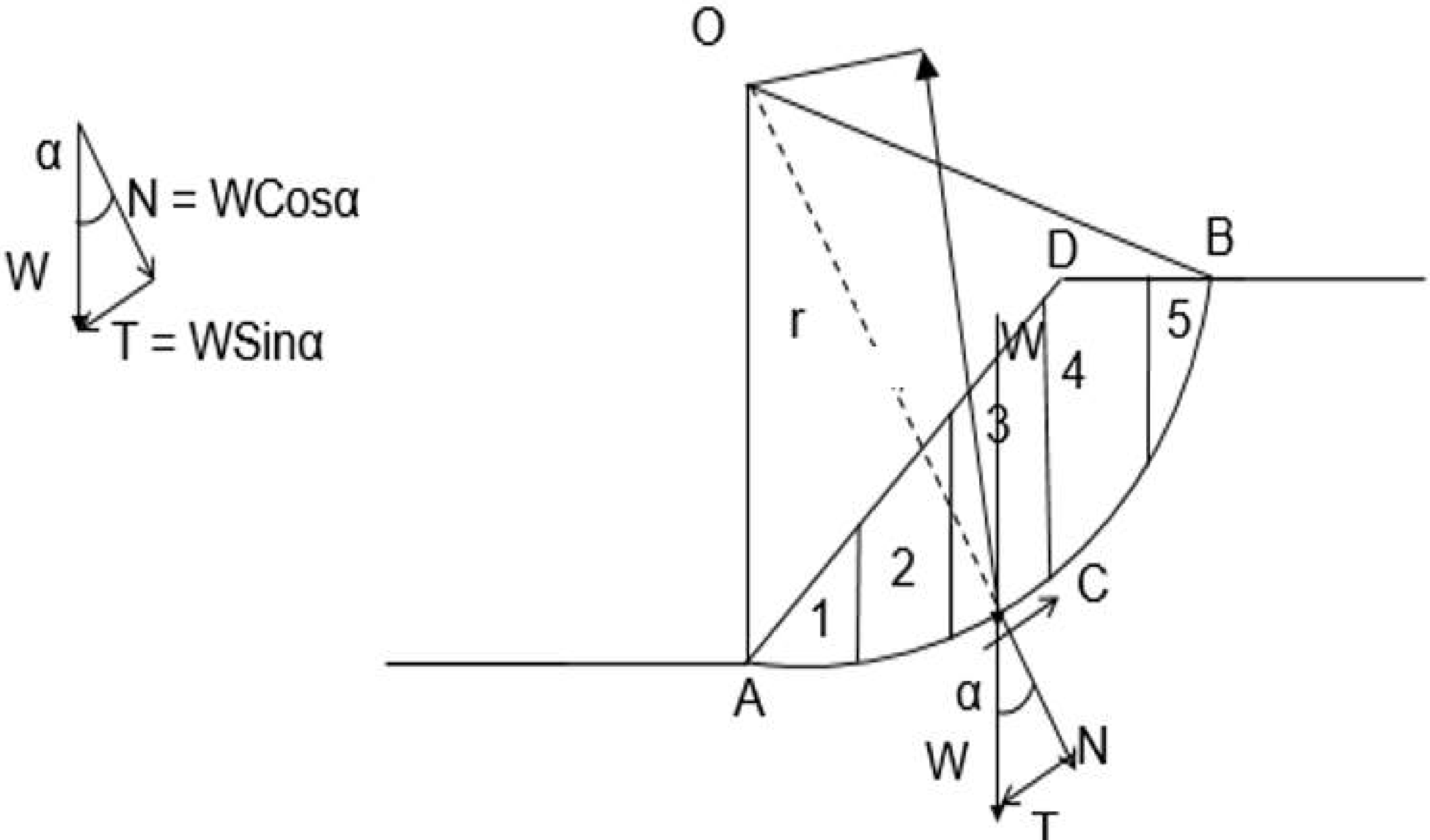
For $\beta = 45°$ and $\phi = 9.35°$, Taylor's stability number from charts is found to be 0.114.

$$\therefore \qquad 0.114 = \frac{c_m}{\gamma_{sat} H} = \frac{c_m}{18.43 \times 6}$$

$$c_m = 0.114 \times 18.43 \times 6 \text{ kN/m}^2 = 12.60 \text{ kN/m}^2$$

Factor of safety with respect to cohesion $F_c = c/c_m = \dfrac{15.0}{12.6} \approx 1.2$

An embankment is 5.4 m high with side slopes of 1.5 H : 1V. The soil has C= 20kPa, φ=50 and γ=15kN/m^3. If the Fellenius angles are $\alpha=26^0$ and $\beta=35^0$, determine the factor of safety of the slope using Swedish method of slices.



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Geotechnical Engineering –II

Assignment No. 2

(Last date of submission- 25/01/2019)

1. How a slope is analyzed using Swedish circle method and Bishop's method of slices? Derive an expression for the factor of safety for both?

2. An embankment is 5.4m high with side slopes of 1.5 H : 1V. The soil has C= 20kPa, φ=50 and γ=15kN/m^3. If the Fellenius angles are α=26^0 and β=35^0, determine the factor of safety of the slope using Swedish method of slices.

3. Explain the Taylor's stability Number method to Analyze finite slopes. Determine the factor of safety with respect to cohesion for a submerged embankment 10 m high and having a slope of 40^0. The properties of the soil are c = 40kN/m^2, φ = 10^0 and γ_{sat}=18 kN/m^3. Given stability numbers for different slope angles are as follows. Also find the critical height of slope.

Slope Angle	15	30	45	60	75	90
S_n	0.070	0.075	0.108	0.138	0.173	0.218

4. Explain different types of slope failures with sketches

5. An embankment of 10m high is inclined at an angle of 36°, to the horizontal. A stability analysis by the method of slices gives following forces per meter.
Sum of Shearing forces=450kN
Sum of Normal forces=900kN
Sum of Neutral forces=216kN
The length of the failure arc is 27 m. Laboratory tests on the soil indicate the effective values c′ and φ′ as 20 kN/m^2 and 18° respectively. Determine the factor of safety with respect to a) Shear strength and b) Cohesion

6. A 5m deep canal has side slopes of 1:1. The properties of soil are C = 30kN/m^2, φ = 20°, e = 0.7 and G =2.7. If Taylor's stability number is 0.11, determine the factor of safety with respect to cohesion when the canal runs full. Also find the same in case of sudden drawdown, if Taylor's stability number for this condition is 0.125.

7. Distinguish between finite and infinite slopes. Write the equation of factor of safety of an infinite slope in a) Cohesionless soil b) Cohesive and Frictional soil (C-ϕ) soil.

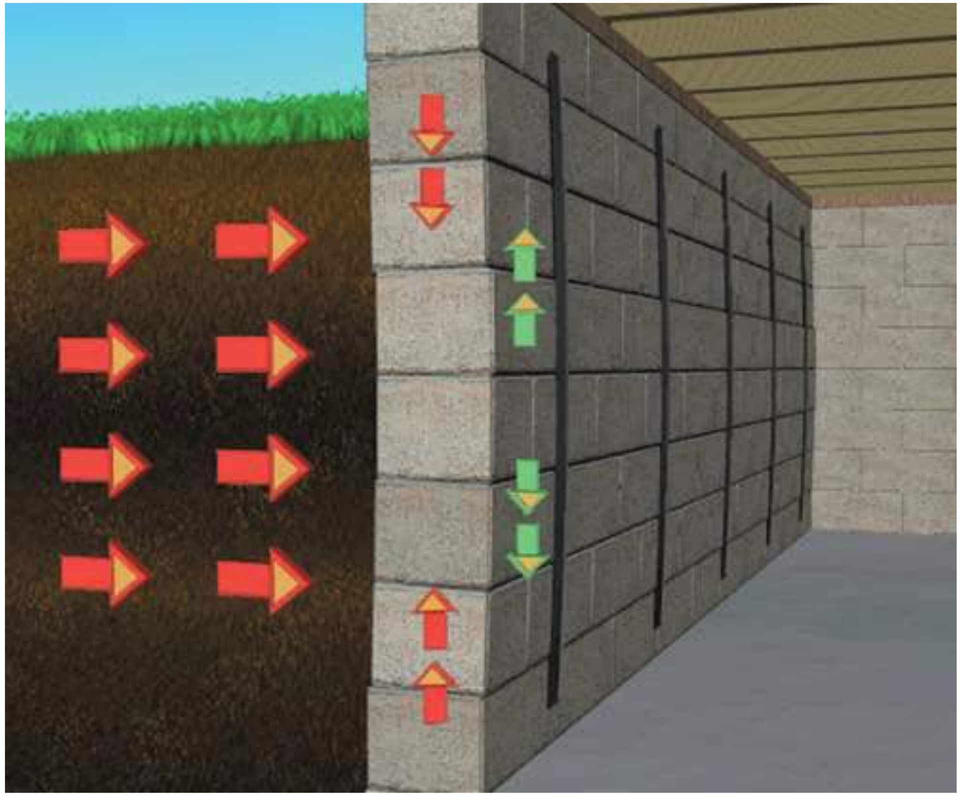
GEOTECHNICAL ENGINEERING- II

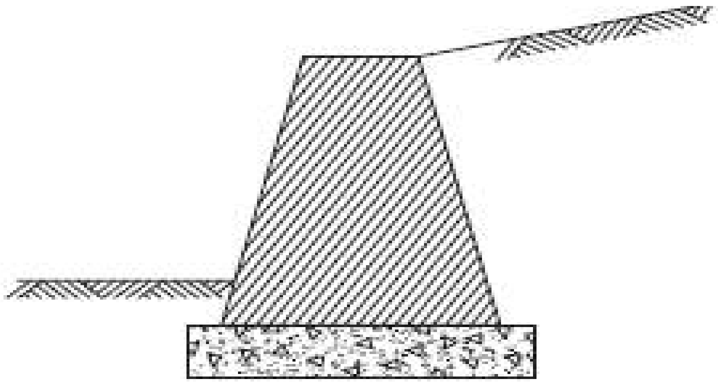
MODULE- III

EARTH PRESSURE

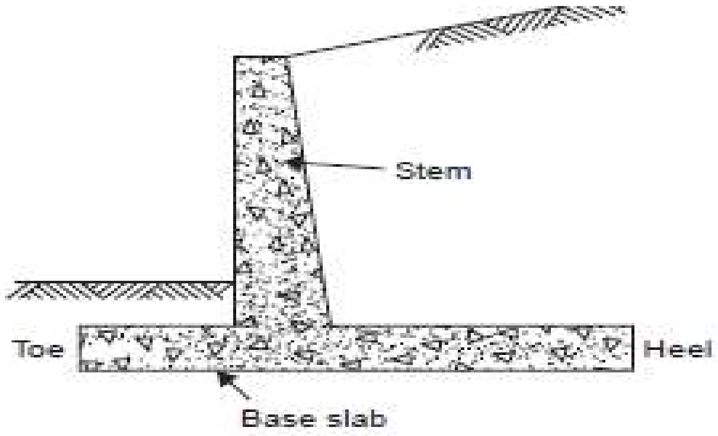
Lateral earth pressure

Lateral earth pressure is the pressure that soil exerts in the horizontal direction

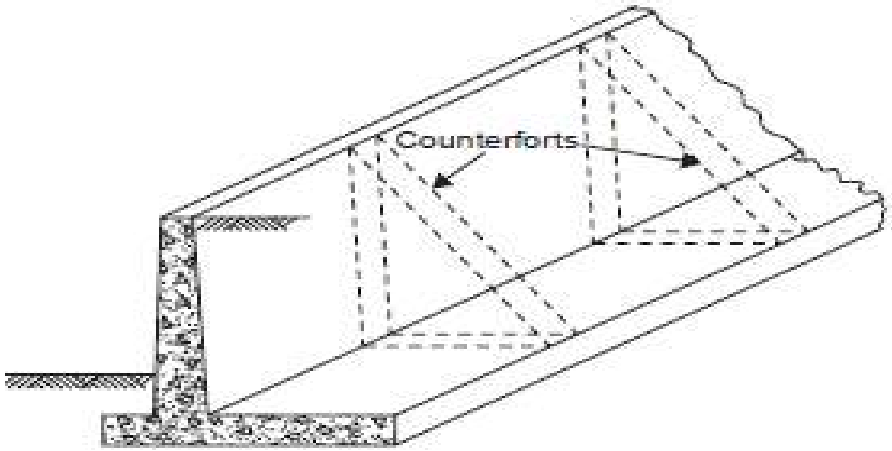




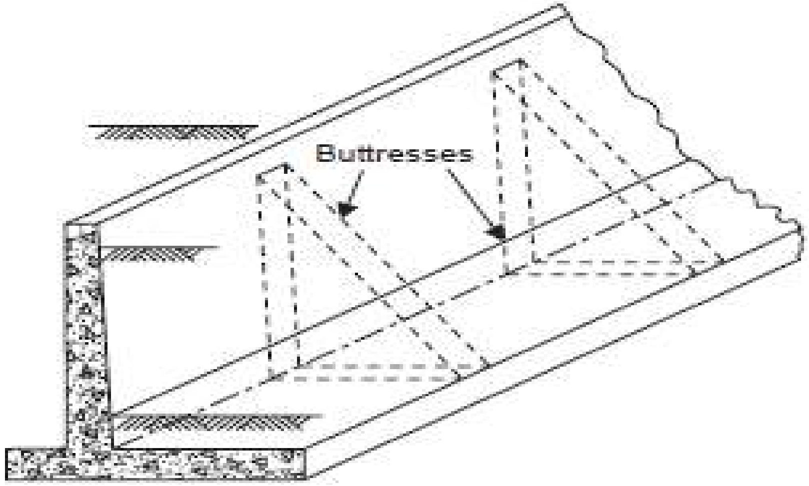
(a) Gravity retaining wall



(b) Cantilever retaining wall



(c) Counterfort retaining wall



(d) Buttress retaining wall

Why We Study Lateral Earth Pressure?

- The lateral earth pressure is important because it is considered in the design of geotechnical engineering structures such as retaining walls, basements, tunnels, deep foundation sand braced excavations.

- Earth retaining structures are common in a manmade environment.

Lateral earth pressure is a function of:

- Type and amount of wall movement- Wall flexibility
- Shear strength parameter of soil
- Unit weight of soil
- Drainage conditions of the soil

- Soil mass is stable- slope of the surface of the soil mass is flatter than the safe slope.

- Where slope is limited- soil has to be retained at a steeper slope.

- Soil has to be supported by retaining structure.

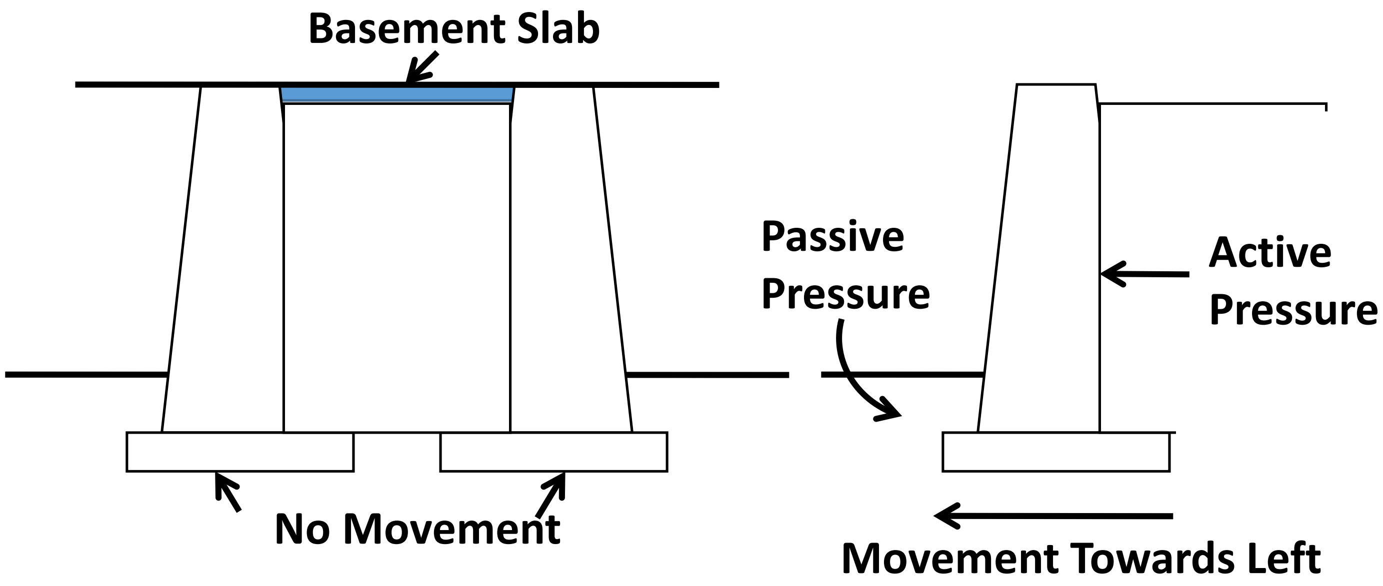
- Soils at different levels are supported by retaining structures on either sides.

- Soil at the higher level will slide and ultimately fail in the absence of retaining structure.

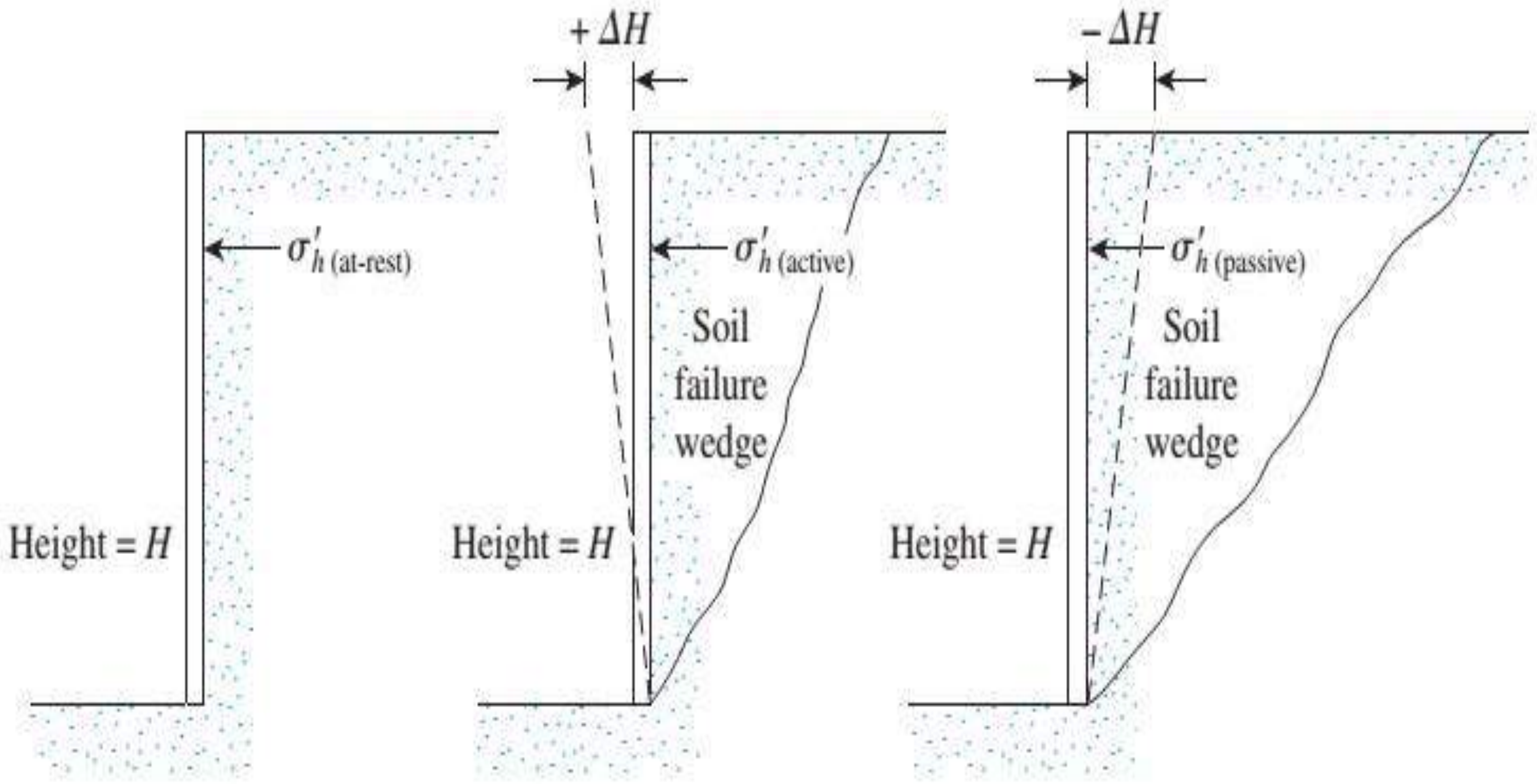
- Determination of magnitude and line of action of force critical for design of earth retaining structures.

Lateral earth pressures

AT REST-PRESSURE	ACTIVE EARTH PRESSURE	PASSIVE EARTH PRESSURE
Not subjected to any lateral yielding or movements	Occurs when soil tends to stretch horizontally	Occurs when soil tends to compress horizontally
Firmly fixed at its top	Not fixed at top	Not fixed at top
Not allowed to move laterally or rotate freely	Allowed to rotate freely or move laterally	Allowed to rotate freely or move laterally
In elastic equilibrium	In plastic equilibrium	In plastic equilibrium
1. Retaining walls with basement slab at top 2. Bridge abutment	1. Retaining wall	1. Retaining wall



At-Rest Pressure



. Illustration of nature of lateral earth pressure on a retaining wall at rest, active, and passive states.

Rankine's Analysis

Assumptions of Rankine's Analysis

- Soil is semi infinite
- Soil is isotropic & homogenous
- Applicable only for dry, cohesionless soil.
- Ground surface is plane which may be horizontal or inclined.
- The back of the wall is smooth, vertical or there is no shearing resistance b/w wall & soil. The stress relationship on any element near the wall is equal to away from the wall.

Coefficient of Earth pressure $K' = \frac{\sigma_H}{\sigma_V}$

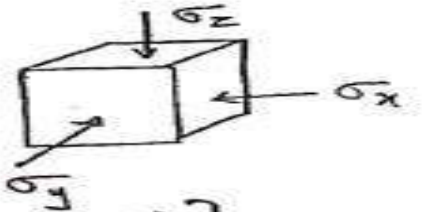
where σ_H = horizontal stress $(K\sigma_v')$,

σ_v' = effective stress

'At rest' condition is an equilibrium condition.

Consider an element in soil under stresses σ_x σ_y σ_z as shown. Soil being a semi-infinite medium

$\sigma_x = \sigma_y$.



ε_0 — strain in rest condition

ε_0 in X direction $= \frac{1}{E}\left[\sigma_x - \mu(\sigma_y + \sigma_z)\right]$

Equating this to zero,

$$\frac{1}{E}\left[\sigma_x - \mu(\sigma_y + \sigma_z)\right] = 0$$

$$(1-\mu)\sigma_x - \mu\sigma_z = 0$$

$$\sigma_x = \frac{\mu\sigma_z}{1-\mu}$$

σ_z is the vertical stress $\therefore \sigma_z = \sigma_v$

$$\therefore \sigma_x = \frac{\mu \sigma_v'}{1-\mu}$$

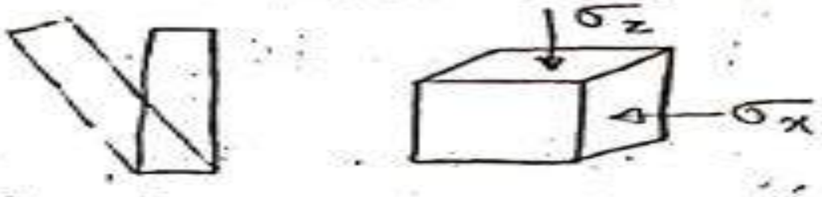
Also $\sigma_x = k_0 \sigma_v'$

$$\therefore k_0 = \frac{\mu}{\ }$$

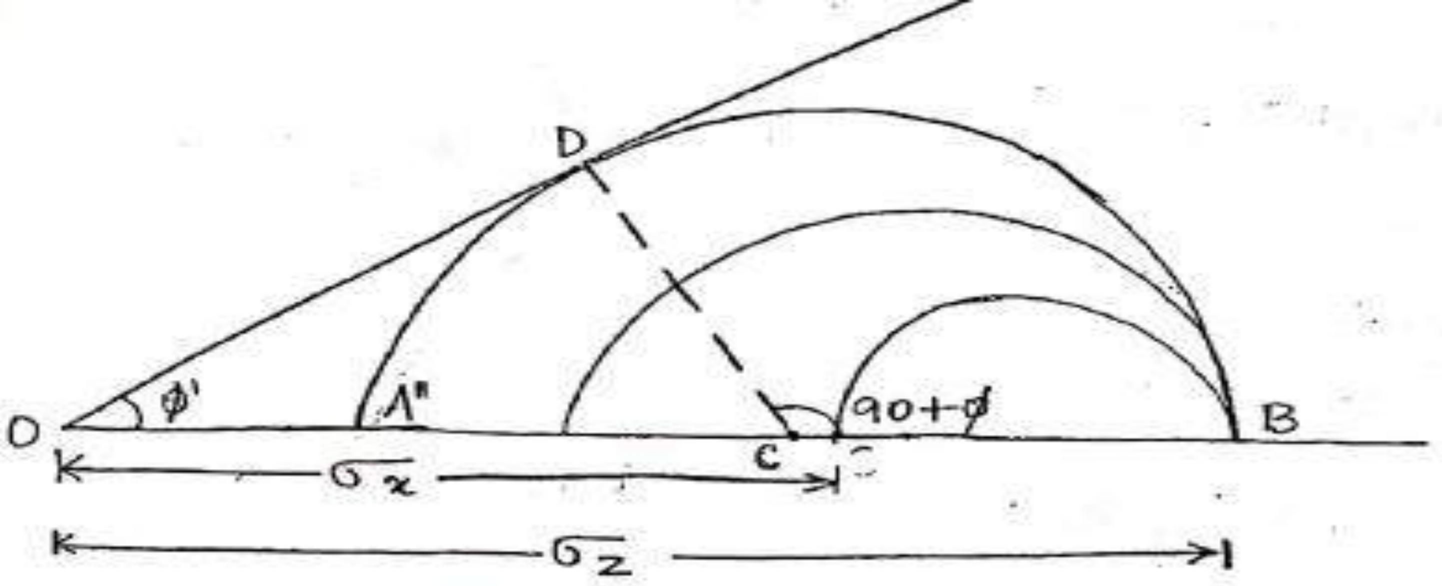
k_0 is also given by

$$k_0 = 1 - \sin\phi$$

When retaining structure moves away from the soil mass only the vertical stress σ_z remains unchanged. σ_x & σ_y will change.



If we draw the Mohr's circle for this case, as the retaining wall shifts away from the soil, $\sigma_x \downarrow$. This will happen till the Mohr's circle touches the failure plane.



Coefficient of Earth pressure at active condition

$$k_a = \sigma_x / \sigma_z = \frac{OC - A''C}{OC + CB}$$

But $DC = A''C = CB$

$$\therefore k_a = \frac{OC - DC}{OC + DC}$$

$$\sin\phi = \frac{DC}{OC}$$

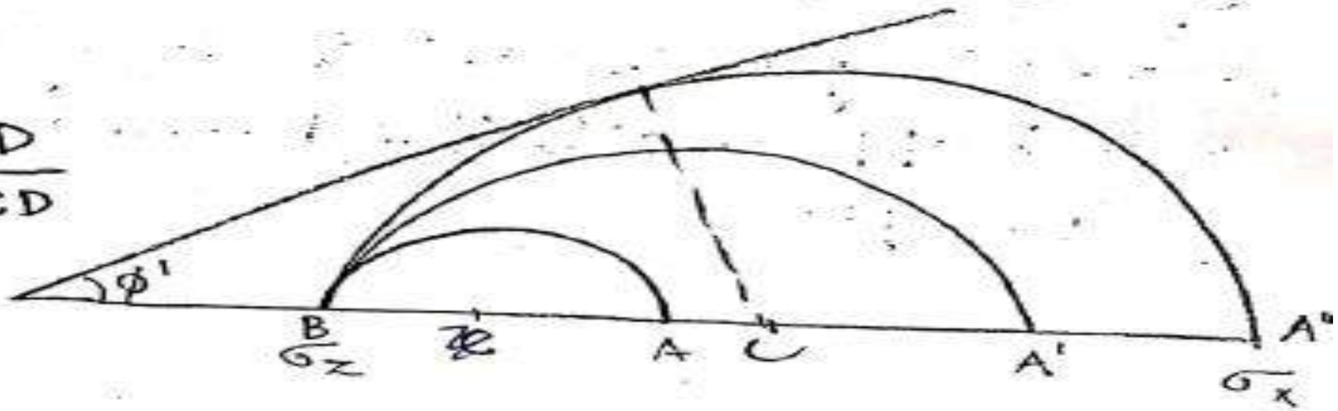
$$\therefore DC = OC \sin\phi$$

$$\therefore \boxed{k_a = \frac{1-\sin\phi'}{1+\sin\phi'}}$$

In passive condition σ_z remains const. & σ_x will be increasing.

Coefficient of earth pressure at passive condition

$$k_p = \frac{\sigma_x}{\sigma_v}$$



$$k_p = \frac{OC+CA''}{OC-CB} = \frac{OC+CD}{OC-CD}$$

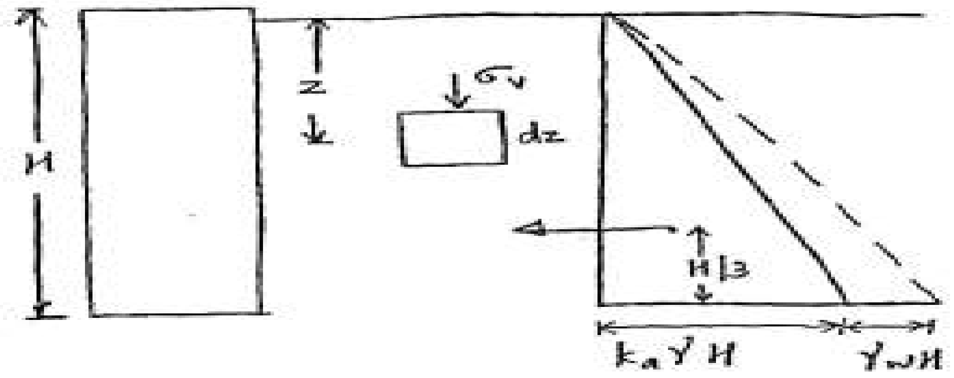
$$= \frac{OC+OC\sin\phi'}{OC-OC\sin\phi'}$$

$$k_p = \frac{1+\sin\phi}{1-\sin\phi}$$

When $\phi = 30^\circ$, $k_o = 0.5$ $k_a = 0.33$ $k_p = 3$.

Total Pressure on Retaining wall

Consider an element at a depth z of size dz.



Horizontal stress,

$$\sigma_x = k_a \sigma_v$$

∴ Horizontal stress,

$$\sigma_x = k_a \gamma z \, dz$$

Total horizontal force

$$P = \int_0^H k_a \gamma z \, dz$$

$$= k_a \gamma \left[\frac{z^2}{2} \right]_0^H$$

$$= \frac{k_a \gamma H^2}{2}$$

It is equal to the area of triangular pressure distribution.

A wall 5.4 m high retains sand in a loose state with a void ratio of 0.63 and angle of friction $\phi = 27°$. If the same soil compacted to a void ratio of 0.36 and angle of friction 45°, compare the ratio of active & passive earth pressure for the two cases. $G = 2.64$

$$\gamma_{d_1} = \frac{G\gamma_w}{1+e} = \frac{[illegible]}{1+0.63} = 15.89\ kN/m^3$$

$$\frac{G\gamma_w}{1+e} = \frac{2.64 \times 9.8}{1+0.63}$$

$$k_{a_1} = \frac{1-\sin\phi_1}{1+\sin\phi_1} = 0.375$$

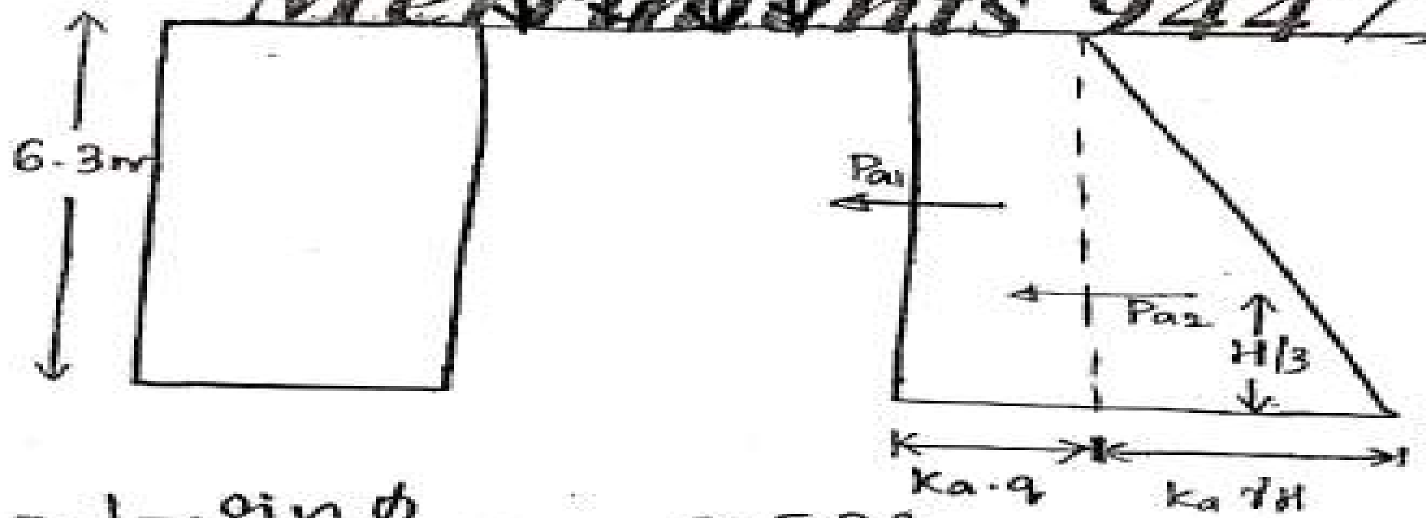
$$k_{p_1} = \frac{1+\sin\phi_1}{1-\sin\phi_1} = 2.663$$

Case(i) $P_a = k_{a_1}[15.89 \times 5.4]$

$P_p = k_{p_1}[15.89 \times 5.4]$

Case(ii) $\gamma_{d_2} = \frac{2.64 \times 9.81}{1+0.36}$

A vertical wall with smooth surface is 6.3 m high & retain soil with a bulk unit wgt of 18 kN/mm and $\phi = 18°$. The top of the soil is in level with the top of the wall and is horizontal. If the soil surface carries a VDL of 4.5 kN/m², determine the total active thrust on the wall per m of the wall & its point of application.



$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.528$$

$$P_{a1} = H \cdot K_a \cdot q = 15.025 \text{ kN/m}^2$$

$$P_{a2} = \frac{1}{2} \cdot k_a \gamma H \cdot H = 189.32 \text{ kN/m}^2$$

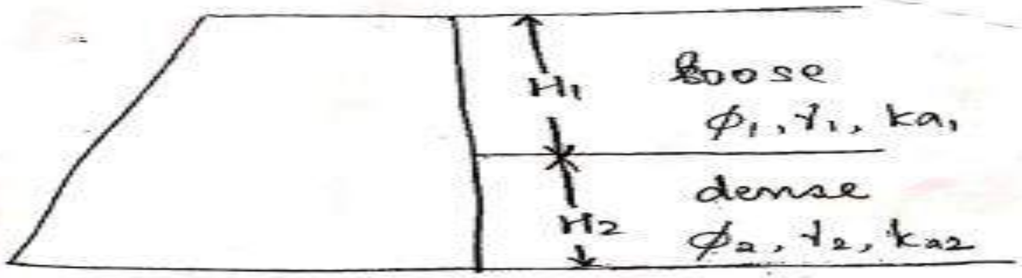
$$P_a = P_{a1} + P_{a2} = 204.35 \text{ kN/m}^2$$

Let line of application be x from base.

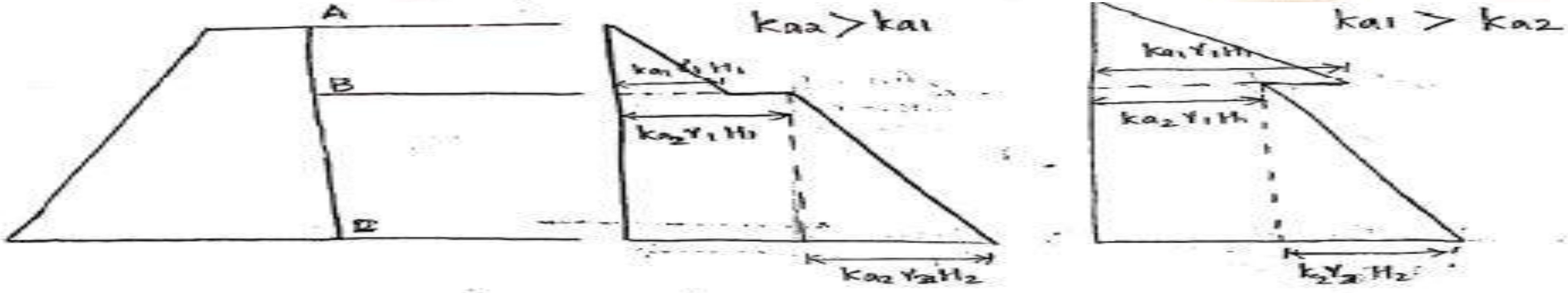
$$P_a \times x = P_{a1} \times H/2 + P_{a2} \times H/3$$

$$204.35 \times x = 15.025 \times \frac{6.3}{2} + 189.32 \times \frac{6.3}{3}$$

Lateral pressure on layered soils



Lateral pressure depends on k_a, which in turn depends on ϕ.



At any pt b/w B & C at a depth z from level B

$\sigma_v = \gamma_1 H_1 + \gamma_2 z$ [z varies from 0 to H_2]

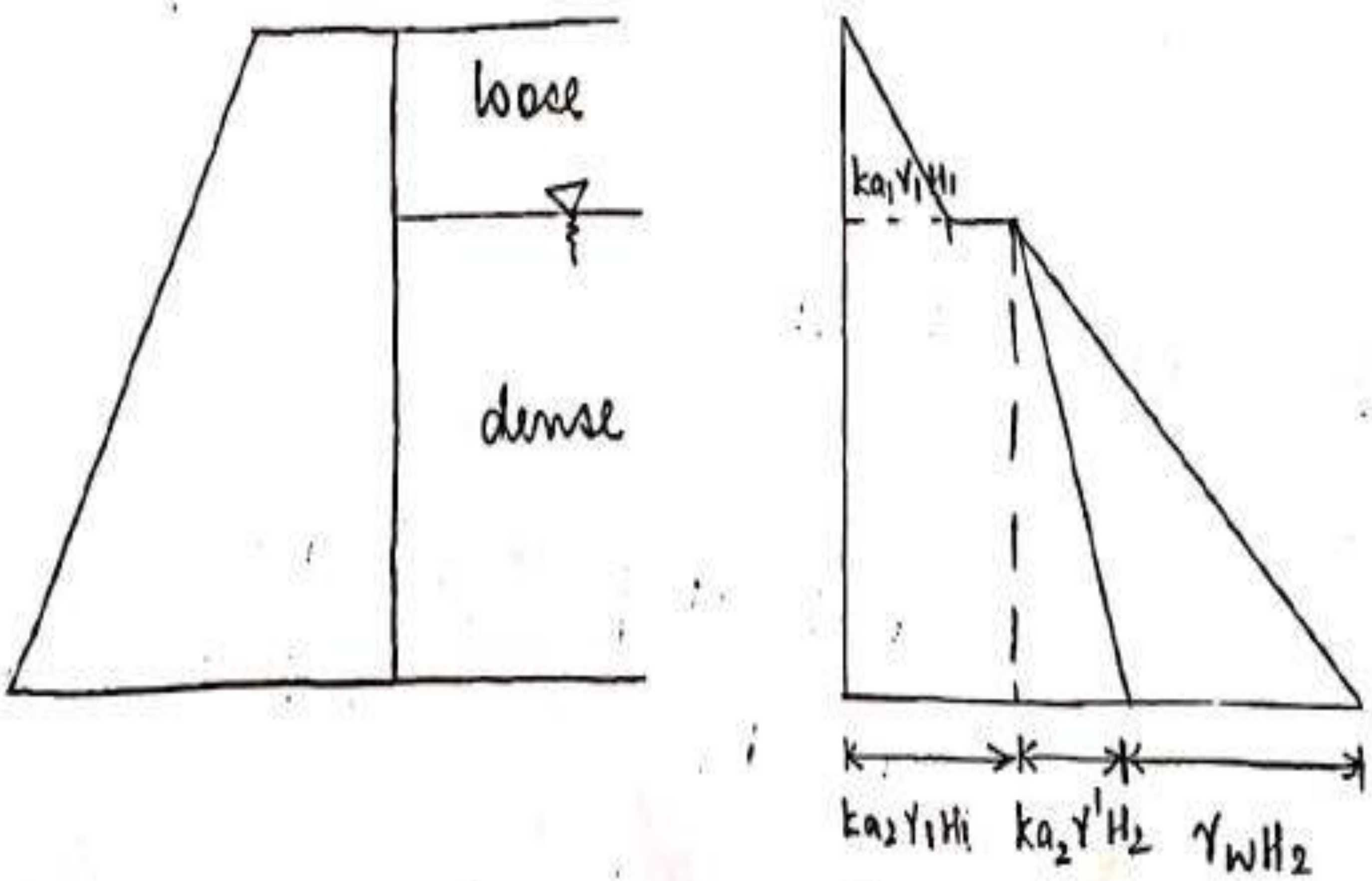
$\sigma_H = \sigma_v \cdot k_{a2}$

$= (\gamma_1 H_1 + \gamma_2 z)\, k_{a2}$

$= k_{a2}\gamma_1 H + k_{a2}\gamma_2 z$

∴ At level B, just above B $\sigma_H = k_{a1}\gamma_1 H_1$

just below B $\sigma_H = k_{a2}\gamma_1 H_1$



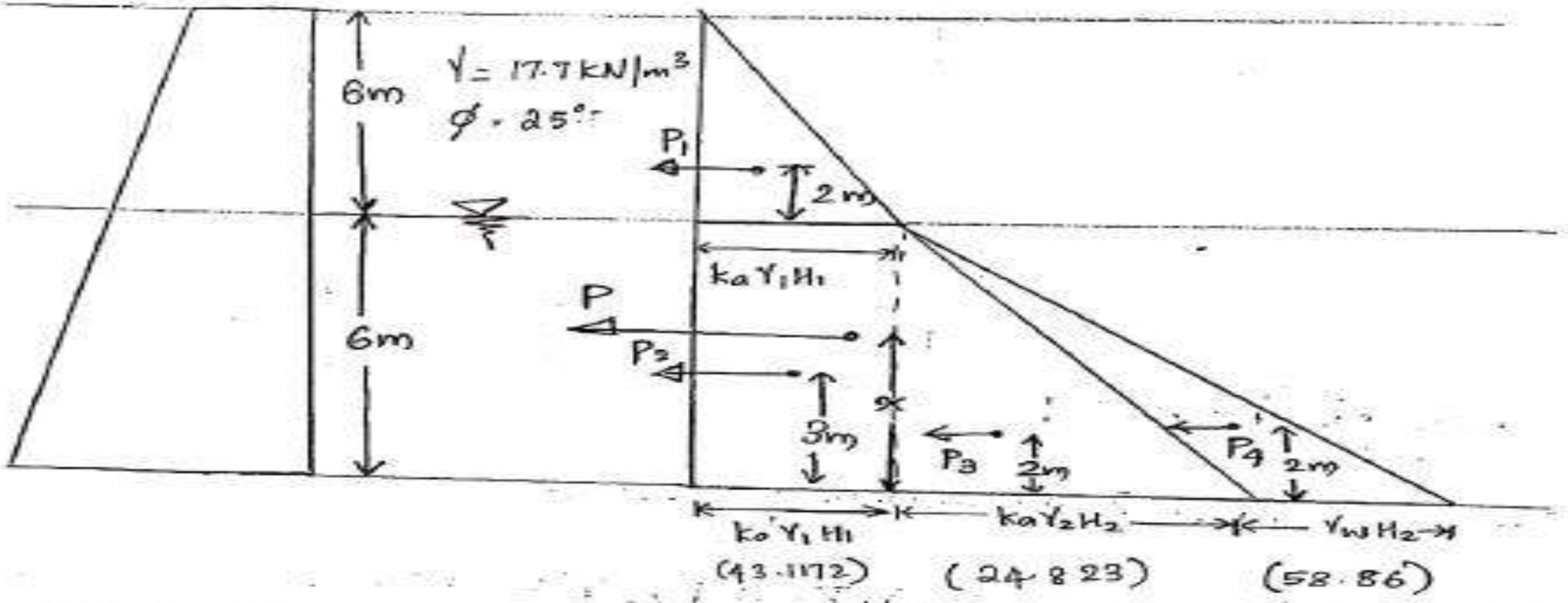
$\gamma' = \gamma_{sub}$.

As ϕ value $\uparrow$, $k_a \downarrow$

A gravity retaining wall retains 12 m of backfill as $\gamma = 17.7 \, kN/m^3$ & $\phi = 25°$, with a uniform horizontal surface. Assume the wall interface to be vertical. Determine the magnitude & pt of application of the total active pressure. If water table is at a ht of 6m, how far does the magnitude & pt of appln of active pressure change? $\gamma_{sat} = 20 \, kN/m^3$

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.406$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = 20 - 9.81 = 10.19 \, kN/m^3$$



At level B

$$\sigma_v = \gamma \times 6 = 17.7 \times 6 = 106.2 \text{ kN/m}^2$$

$$\sigma_H = 106.2 \times k_a = 43.1172 \text{ kN/m}^2$$

At level C

$$\sigma_H = k_a \gamma_1 H_1 + k_a \gamma_2 H_2 + \gamma_w H_2 = 17.7 \times 6 \times .406 + .406 \times 10.19 \times 6 + 9.81 \times 6$$

$$= \underline{126.8} \text{ kN/m}^2$$

To find the pt at which the net force acts.

$$P_1 = \frac{1}{2} \times 6 \times 43.1172 = 129.35 \text{ kN}$$

$$P_2 = 43.1172 \times 6 = 258.703 \text{ kN}$$

$$P_3 = \frac{1}{2} \times 24.823 \times 6 = 74.469 \text{ kN}$$

$$P_4 = \frac{1}{2} \times 6 \times (126.8 - 67.94) = 176.58 \text{ kN/m}^2$$

$$P = 639.102 \text{ kN/m}^2$$

$$639.102 \times x = 129.35 \times 8 + 258.703 \times 3 + 74.469 \times$$
$$+ 176.58 \times 2$$

$$\underline{x = 3.619 \text{ m}}$$

Rankine's Analysis for Cohesive Soil

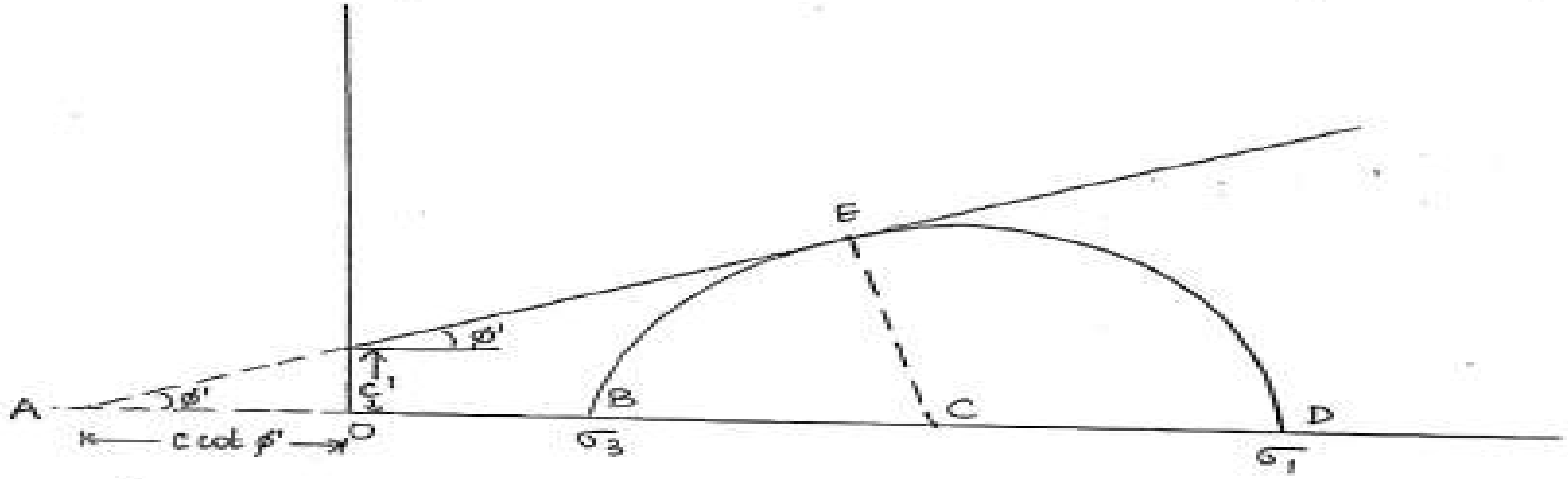
Active Earth pressure :-

σ_1 = major principal stress

σ_3 = minor "

c' = effective cohesion

ϕ' = effective ∠ of friction.



$$\sin \phi' = \frac{EC}{AC} = \frac{EC}{AO + OC} = \frac{(\sigma_1 - \sigma_3)/2}{c' \cot \phi' + (\sigma_1 + \sigma_3)/2}$$

$$\sin\phi' \times c'.\cot\phi' + \sin\phi' \frac{(\sigma_1+\sigma_3)}{2} = \frac{\sigma_1-\sigma_3}{2}$$

$$c'.\cos\phi' + \frac{\sigma_1}{2}\sin\phi' + \frac{\sigma_3}{2}\left(\sin\phi' + 1\right) = \frac{\sigma_1}{2}$$

$$\frac{\sigma_3}{2} = \frac{\frac{\sigma_1}{2}(1-\sin\phi) - c'\cos\phi'}{1+\sin\phi}$$

$$\frac{\sigma_3}{2} = \frac{\sigma_1}{2}\,\frac{1-\sin\phi}{1+\sin\phi} - \frac{c'.\cos\phi'}{1+\sin\phi}$$

We have $k_a = \dfrac{1-\sin\phi}{1+\sin\phi}$

$$\sqrt{k_a} = \sqrt{\frac{1-\sin\phi}{1+\sin\phi}} = \sqrt{\frac{(1-\sin\phi)(1+\sin\phi)}{(1+\sin\phi)^2}} = \sqrt{\frac{1-\sin^2\theta}{(1+\sin\phi)^2}}$$

$$\therefore \sqrt{k_a} = \frac{\cos\phi'}{1+\sin\phi}$$

$$\therefore \boxed{\sigma_3 - k_a \cdot \sigma_1 - 2c'\sqrt{k_a}}$$

Also, $k_a = \dfrac{1-\sin\phi}{1+\sin\phi} = \tan^2(45-\phi/2)$

$$\therefore \boxed{\sigma_3 = \sigma_1 \tan^2(45-\phi/2) - 2c' \tan(45-\phi/2)}$$

σ_1 = vertical stress = $\gamma.z$.

$$\boxed{\sigma_3 = \gamma z k_a - 2c'\sqrt{k_a}}$$ active earth pressure for $c-\phi$ soil

When $z=0$, $\sigma_3 = -2c'\sqrt{k_a}$

ie, the soil is tensile in nature. Cracks develop b/w retaining wall & soil.

To find the depth at which earth pressure = 0.

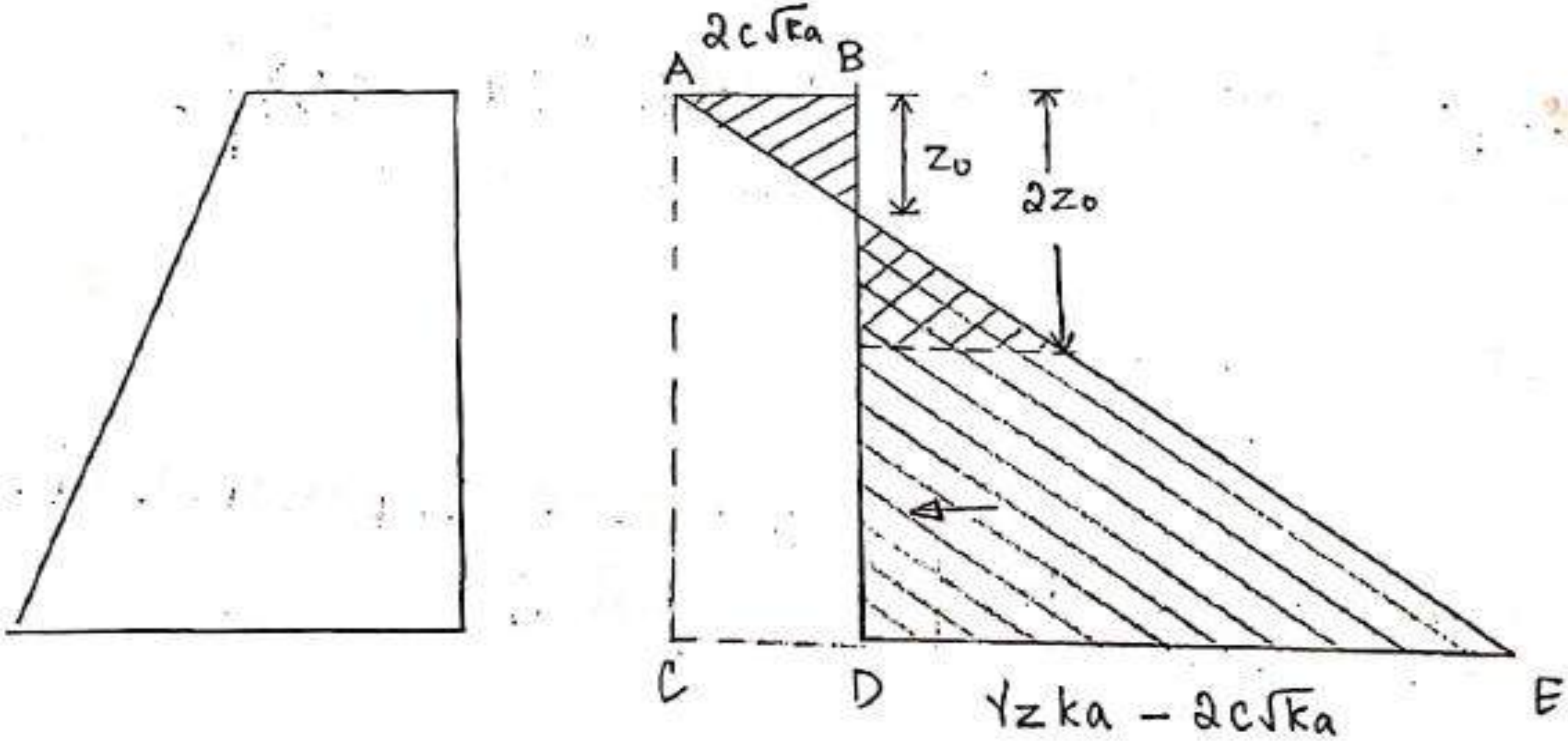
Let 'z_0' be the reqd depth.

$$\gamma . z_0 k_a - 2c' \sqrt{k_a} = 0$$

$$z_0 = \frac{2c' \sqrt{k_a}}{\gamma . k_a}$$

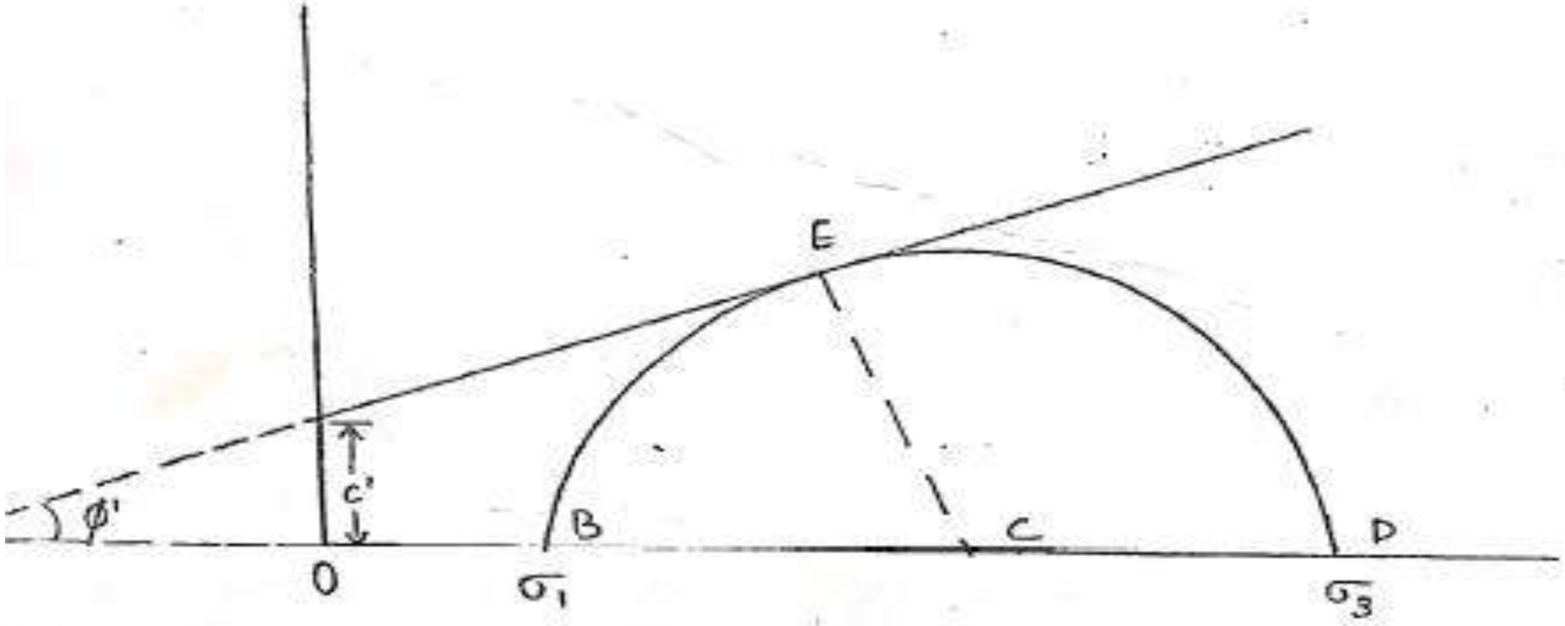
$$\boxed{z_0 = \frac{2c'}{\gamma \sqrt{k_a}}}$$

ie, crack formation occurs upto a depth of $\frac{2c'}{\gamma \sqrt{k_a}}$.



The soil can stand without retaining structure upto a depth of $2z_0$. At that depth the tension gets cancels out.

Passive Earth pressure



$$\sigma_3' = \sigma_1 k_p + 2c'\sqrt{k_p}$$

$$k_p = \frac{1+\sin\phi}{1-\sin\phi}$$

$$\sin\phi' = \frac{EC}{AC} = \frac{EC}{AO+OC} = \frac{(\sigma_3-\sigma_1)/2}{c'\cot\phi' + (\sigma_3+\sigma_1)/2}$$

$$c'\cos\phi' + \left(\frac{\sigma_3+\sigma_1}{2}\right)\sin\phi' = \frac{\sigma_3-\sigma_1}{2}.$$

$$\frac{\sigma_1}{2}\left[1-\sin\phi\right] = \frac{\sigma_3}{2}\left[1+\sin\phi\right]$$

$$\frac{\sigma_3}{2} = \frac{c'\cos\phi'}{1-\sin\phi} + \frac{1+\sin\phi}{1-\sin\phi}\cdot\frac{\sigma_1}{2}$$

$$k_p = \frac{1+\sin\phi}{1-\sin\phi}$$

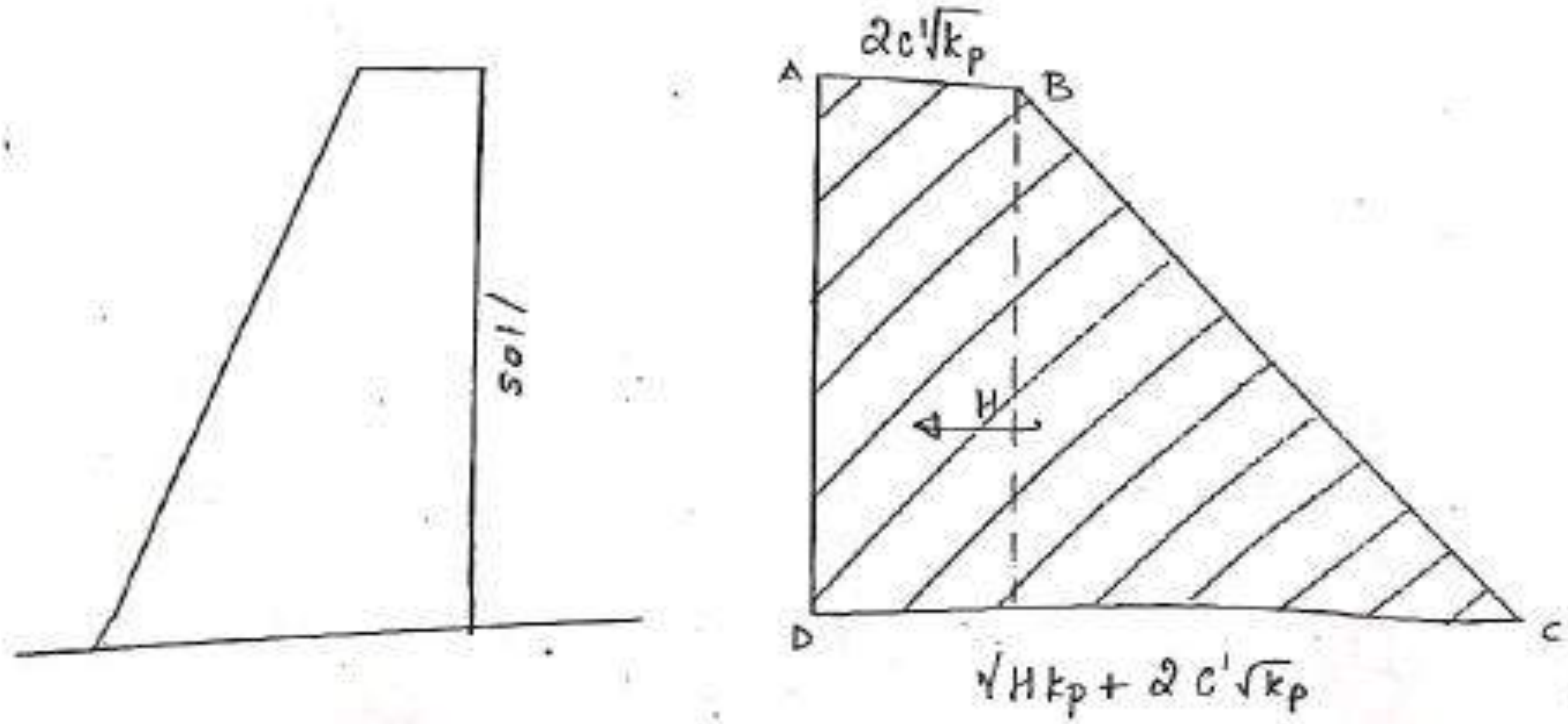
$$\sqrt{k_p} = \sqrt{\frac{(1+\sin\phi)(1-\sin\phi)}{(1-\sin\phi)^2}} = \frac{\cos\phi}{1-\sin\phi}$$

$$\therefore \sigma_3 = k_p\cdot\sigma_1 + \sqrt{k_p}\,2c'$$

$$\sigma = \gamma z$$

$$\boxed{\sigma_3 = \gamma z k_p + 2c'\sqrt{k_p}}$$

When $z = 0$, $\sigma_3 = 2c'\sqrt{k_p}$



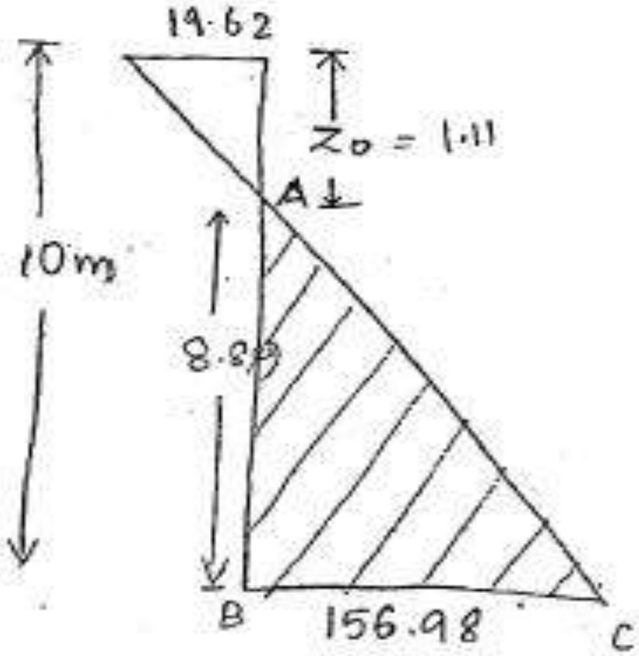
Q. A wall with a smooth vertical backfill (10 m high) supports a purely cohesive soil with $c = 9.81\ kN/m^2$ and $\gamma = 17.66\ kN/m^3$. Determine the Rankine's active pressure against the wall, Position of zero pressure & dist. of centre of active pressure above the base.

$\phi = 0$ (purely cohesive)

$$k_a = \frac{1 - \sin 0}{1 + \sin 0} = 1$$

$c = 9.81\ kN/m^2$

$\gamma = 17.66\ kN/m^3$



$$\sigma_3 = \gamma z k_a - 2c' \sqrt{k_a}$$

$$= 17.66 \times 10 \times 1 - 2 \times 9.81 \times \sqrt{1}$$

$$= 156.98$$

$$z_0 = \frac{2c'}{\gamma \sqrt{k_a}}$$

$$= \frac{2 \times 9.81}{17.66 \times \sqrt{1}}$$

$$= \underline{\underline{1.11\,m}}.$$

$$\frac{z_0}{10 - z_0} = \frac{19.62}{156.98}$$

$$156.98\, z_0 = 19.62 \times 10 - 19.62\, z_0$$

$$\underline{\underline{z_0 = 1.11\,m}}.$$

Total pressure = area of ABC

$$= \frac{1}{2} \times 8.89 \times 156.98 = \underline{\underline{697.776\ kN}}$$

Q. In the above Q, if $\phi = 20°$, calculate passive earth pressure

$$k_p = \frac{1+\sin 20}{1-\sin 20} = 2.0396$$

$$\sigma_3 \text{ at base} = \gamma z k_p + 2c' \sqrt{k_p}$$

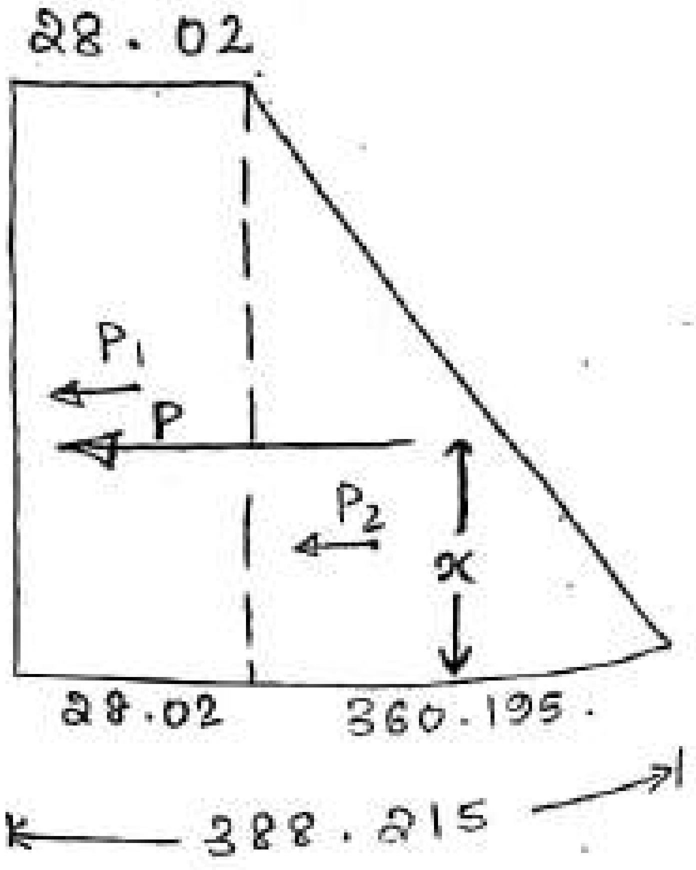
$$= 17.66 \times 10 \times 2.0396 + 2 \times 9.81 * \sqrt{2.0396}$$

$$= 388.215 \text{ kN/m}^2.$$

Total pressure

$$= \frac{1}{2}(28.02 + 388.215) \times 10$$

$$= \underline{\underline{2081.175 \text{ kN}}}$$



$$P_1 \times 5 + P_2 \times \frac{10}{3} = 2081.175 \times x$$

$$28.02 \times 10 + \frac{1}{2} \times 360.195 \times 10 \times \frac{10}{3}$$

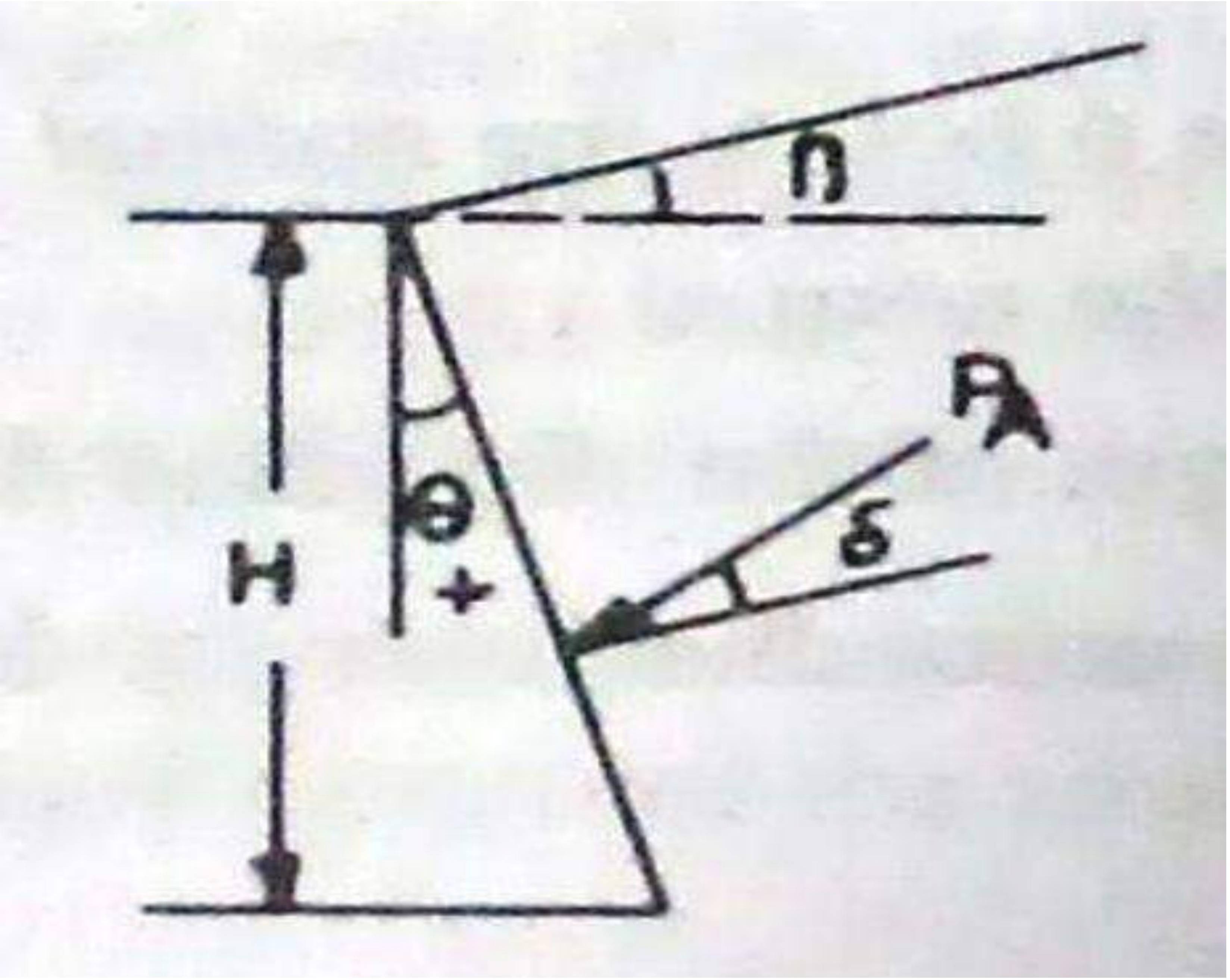
$$= 2081.175 \times x$$

$$x = \underline{\underline{3.019 \text{ m}}}$$

Coulomb’s Theory of Earth Pressure

Assumptions

- The backfill is a dry, cohesion less, homogeneous, isotropic soil.
- The backfill surface is planar and can be inclined.
- The back of the wall can be inclined to the vertical.
- The failure surface is a plane surface which passes through the heel of the wall.
- The position and the line of action of the earth pressure are known.



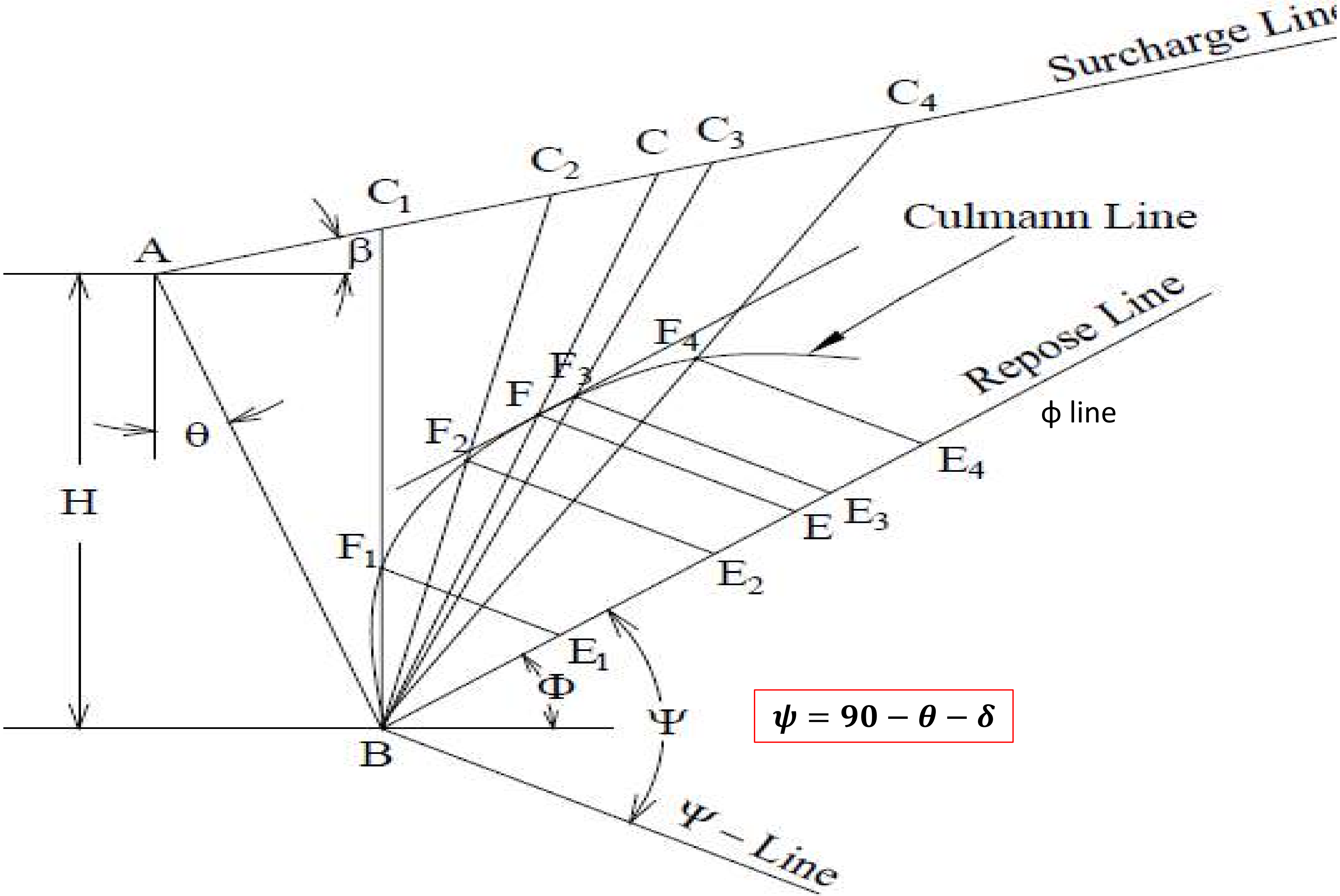
Θ = Batter Angle

δ = Angle of Wall friction

β = Surcharge angle

δ = This is an angle of friction between the wall and backfill soil and is usually called ‘wall friction’.

Culmann's Graphical Method for Active Earth Pressure of Cohesionless Soil



$$\psi = 90 - \theta - \delta$$

The steps involved in the Culmann's method are as follows:

1. Given height H and batter angle θ , the back AB of the wall is constructed.
2. Through A, the surcharge line (β-line) is drawn inclined at angle β to the horizontal.
3. Through B, the repose line (∅-line) is drawn inclined at an angle ∅ to the horizontal.
4. Again through B, the ψ-line is drawn inclined at an angle ψ to the ∅-line ($\psi = 90° - \theta - \delta$).
5. Trial slip planes BC_1, BC_2…… are drawn. The weights of the wedges ABC_1, ABC_2…… are calculated and plotted to scale as BE_1, BE_2,…… on the ∅-line.
6. Through E_1, E_2,…. lines are drawn parallel to ψ-line, intersecting BC_1, BC_2,……. at F_1, F_2..... respectively.
7. A smooth curveis drawn through points B, F_1, F_2…… This curve is called Culmann line.

8. A line is drawn parallel to Ø-line and tangential to Culmann line. Let it touch Culmann line at F. BF is joined and produced to intersect the β-line at C. Then BC is the critical slip plane.

9. Through F, line FE is drawn parallel to ψ-line, intersecting Ø-line at E.

10. The weight W of the wedge ABC is calculated. The resultant active earth pressure P_a is given by

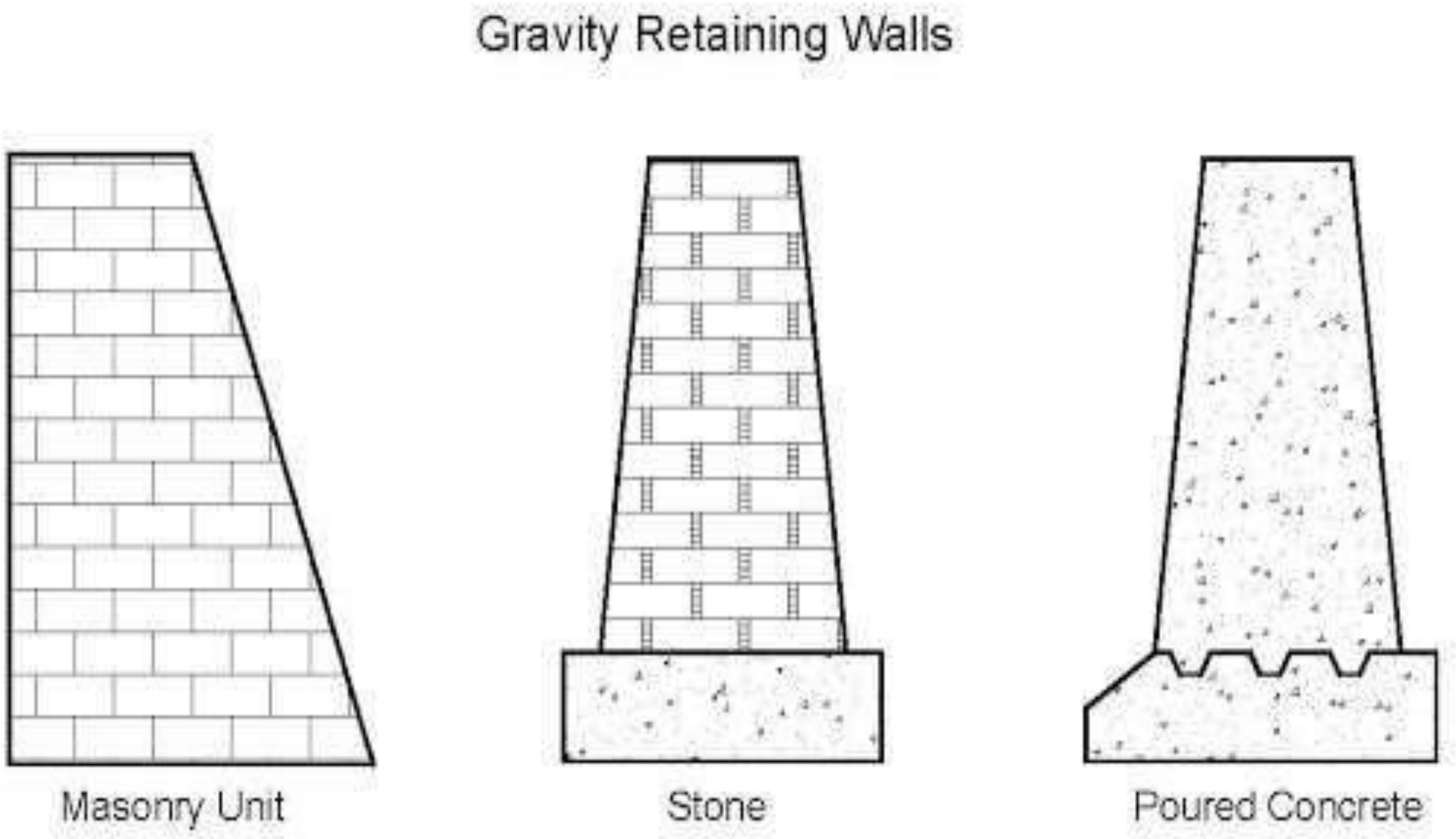
$$\frac{P_a}{W} = \frac{FE}{BE}$$

$$\therefore\ P_a = W.\left(\frac{FE}{BE}\right)$$

TYPES OF RETAINING WALLS

- Gravity retaining walls
- Cantilever retaining walls
- Sheet pile retaining walls
- Counter-fort / Buttressed retaining wall

Gravity retaining walls

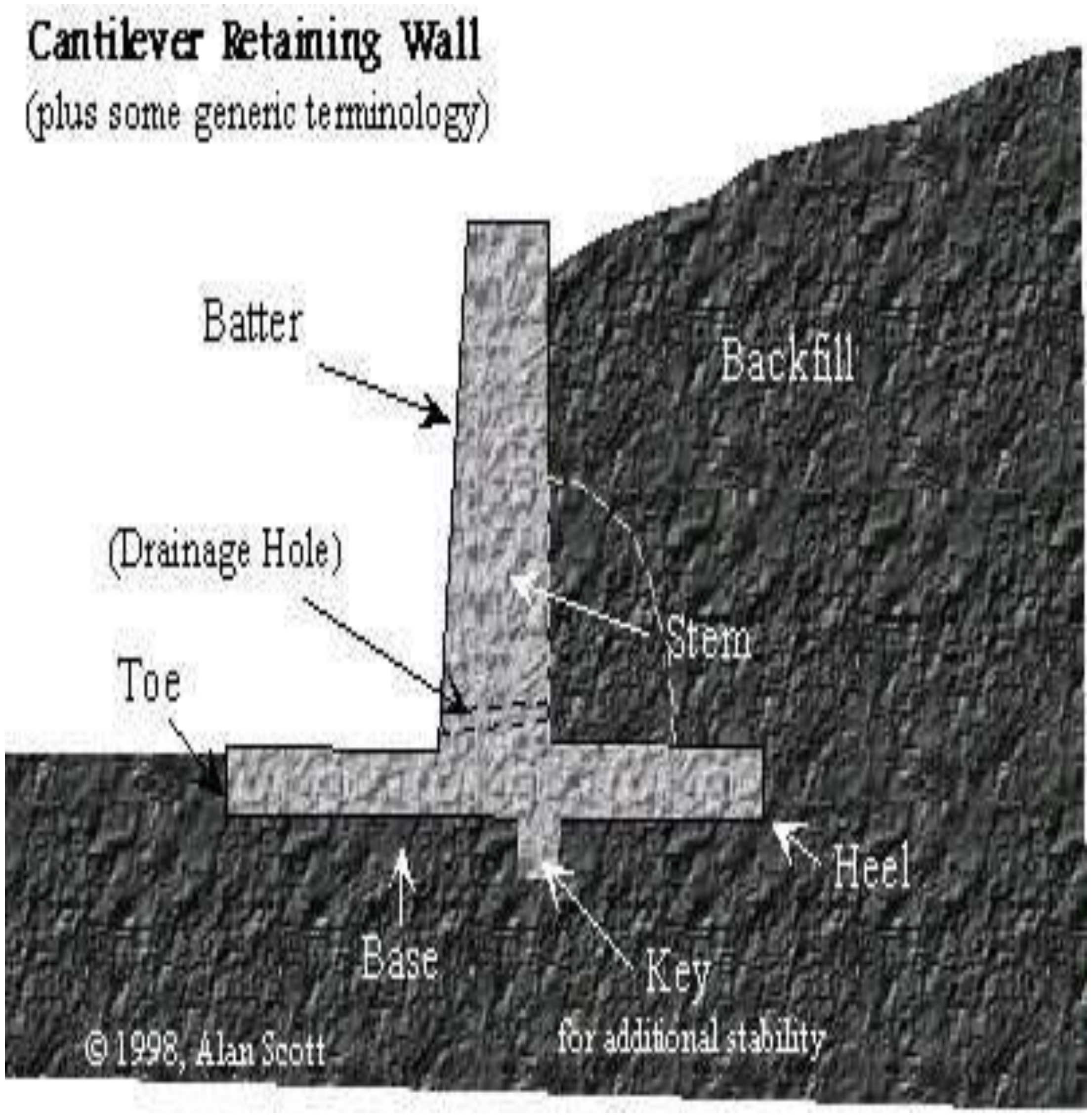


- It is that type of retaining wall that relies on their huge weight to retain the material behind it and achieve stability against failures.

- Gravity Retaining wall can be constructed from concrete, stone or even brick masonry. Gravity retaining walls are much thicker in section.

- Geometry of these walls also help them to maintain the stability.

Cantilever retaining walls



- A cantilever retaining wall is one that consists of a wall which is connected to foundation.
- They are the most common type used as retaining walls. Cantilever wall rest on a slab foundation.
- This slab foundation is also loaded by back-fill and thus the weight of the backfill and surcharge also stabilizes the wall against overturning and sliding.

Sheet pile retaining walls

- Sheet pile retaining walls are usually used in soft soils and tight spaces.

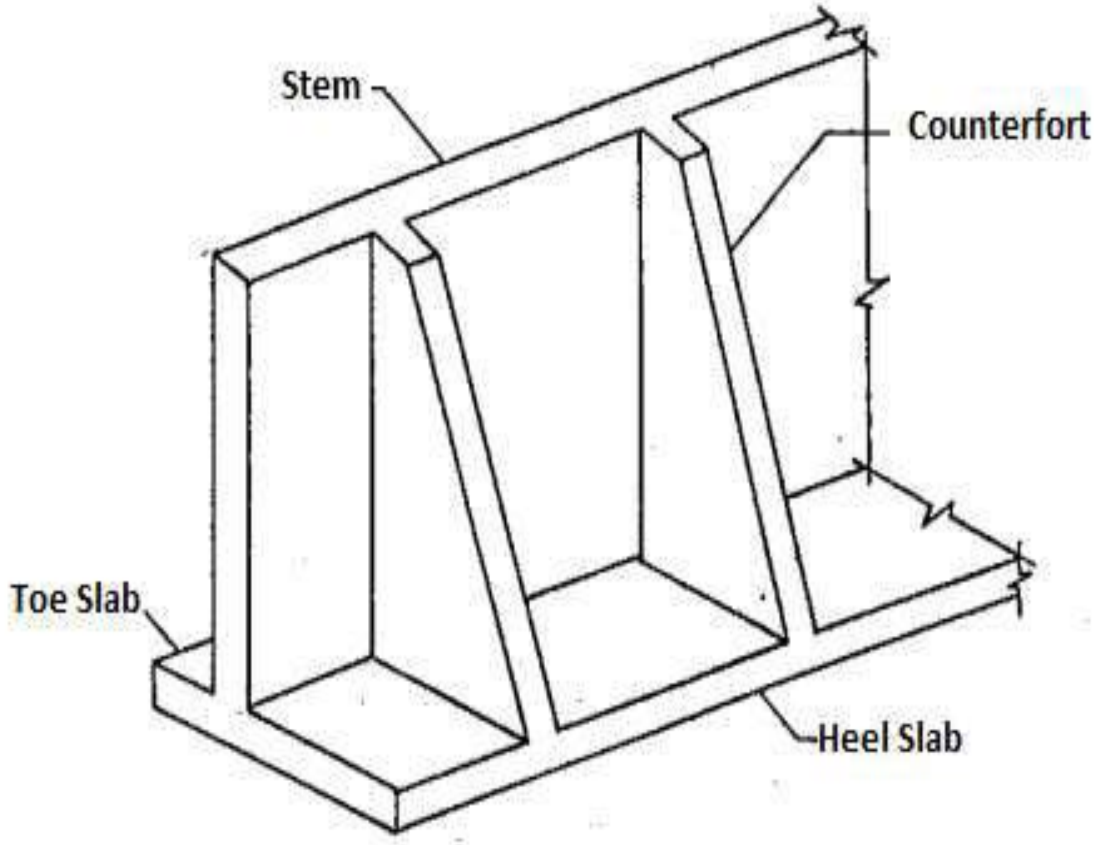
- Sheet pile walls are made out of steel, vinyl or wood planks which are driven into the ground.

- They are mainly used as temporary structures

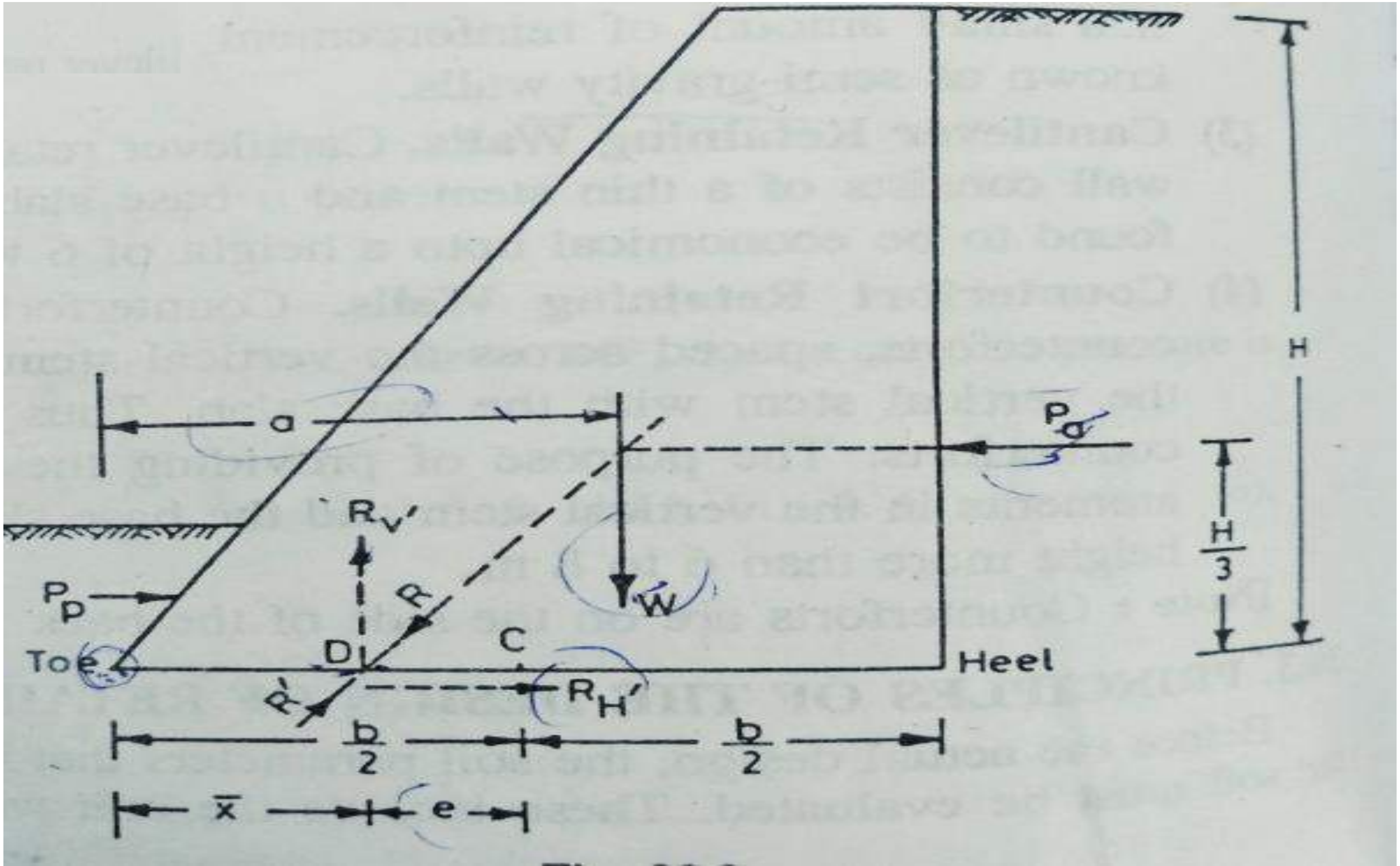
- Taller sheet pile walls will need a tieback anchor for stability

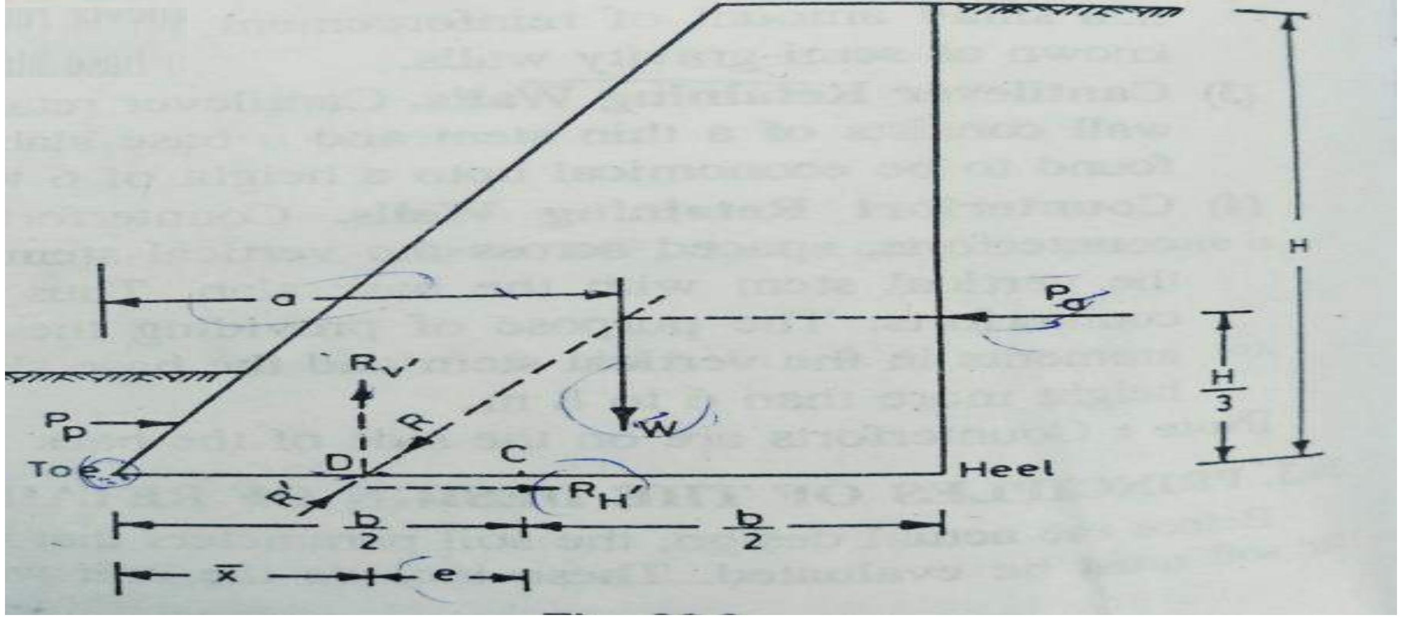
Counter-fort / Buttressed retaining wall

- Counterfort retaining wall consists of a stem, toe slab and heel slab as in case of cantilever retaining wall.
- It also consists of counterforts are regular interval which divides the stem.



Stability of retaining walls





$$R_V' = W, \quad \text{and} \quad R_H' = P_a$$

The third equation of equilibrium, namely the moment equation, is used to determine the eccentricity e of the force R_v' relative to the centre C of the base of the wall. Obviously, by taking moments about the toe,

$$R_v' \times \bar{x} = W \times a - P_a(H/3)$$

or

$$\bar{x} = \frac{W \times a - P_a \times H/3}{R_V'} \quad \ldots(20.1)$$

where $\bar{x}$ is the distance of the point D from the toe.

Thus, eccentricity,

$$e = b/2 - \bar{x} \quad \ldots(20.2)$$

where b = width of the base.

For a safe design, the following requirements must be satisfied.

(1) **No Sliding**

The wall must be safe against sliding. In other words,

$$\mu\ R_V > R_H$$

where R_V and R_H are vertical and horizontal components of R, respectively. The factor of safety against sliding is given by

$$F_s = \frac{\mu R_V}{R_H} \quad ...(20.3)$$

where μ = coefficient of friction between the base of the wall and the soil (= tan δ).

A minimum factor of safety of 1.5 against sliding is generally recommended.

(2) **No Overturning**

The wall must be safe against overturning about toe. The factor of safety against overturning is given by

$$F_0 = \frac{\Sigma M_R}{\Sigma M_O} \quad ...(20.4)$$

where ΣM_R = sum of resisting moment about toe,

and ΣM_O = sum of overturning moment about toe.

In Fig. 20.2,

$$F_0 = \frac{W \times a}{P_a \times H/3} \quad ...(20.5)$$

The factor of safety against overturning is usually kept between 1.5 to 2.0.

(3) **No bearing capacity failure**

The pressure caused by R_V at the toe of the wall must not exceed the allowable bearing capacity of the soil.

The pressure distribution at the base is assumed to be linear. The maximum pressure is given by

$$p_{max} = \frac{R_V}{b}\,(1 + 6e/b) \quad ...(20.6)$$

The factor of safety against bearing failure is given by

$$F_b = \frac{q_{na}}{p_{max}} \quad ...(20.7)$$

where q_{na} = allowable bearing pressure.

A factor of safety of 3 is usually specified, provided the settlement is also within the allowable limit.

Geotechnical Engineering –II

Assignment No. 3

(Last date of submission- 25/03/2019)

1. Differentiate active, passive and earth pressure at rest.

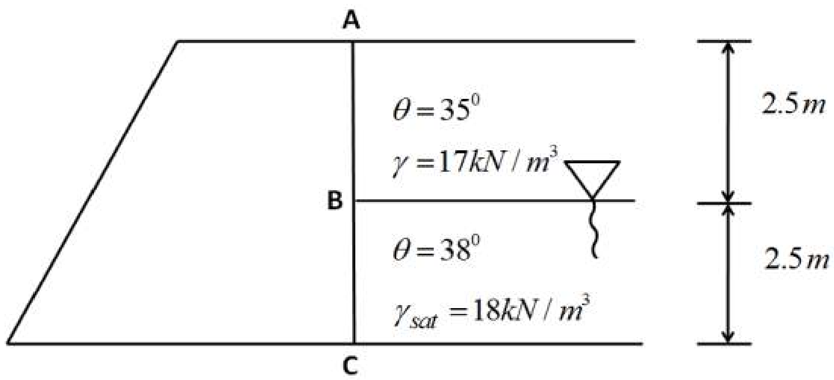
2. An unsupported excavation is made in a clay layer. The properties of clay are c =23 kN/m^2, γ =19 kN/m^3 and Φ =15^0.
   Determine.
   i. Depth of tension crack.
   ii. Draw active earth pressure diagram.
   iii. Determine the total thrust
   Assume the depth of clay layer as 6m.

3. A retaining wall 6m high, with a smooth vertical back is pushed against a soil mass having C= 36 kN/m^2 and Φ =15° and γ =18 kN/m^3. What is the total Rankine passive pressure, if the horizontal soil surface carries a uniform load of 35 kN/m^2? What is the point of application of the resultant thrust?

4. Describe the construction procedure for Culmann's graphical method.

5. A smooth vertical wall 6 m high retains a soil with c = 2.5 kN/m^2, φ = 28°, and γ = 20 kN/m^3. Show a) Rankine passive pressure distribution, b) Rankine Active earth pressure distribution and also determine the magnitude and point of application

6. Determine the active pressure on the retaining wall shown in Figure. Take γ_w=10 kN/m^3.



7. Discuss the design principles of retaining walls.

GEOTECHNICAL ENGINEERING-II

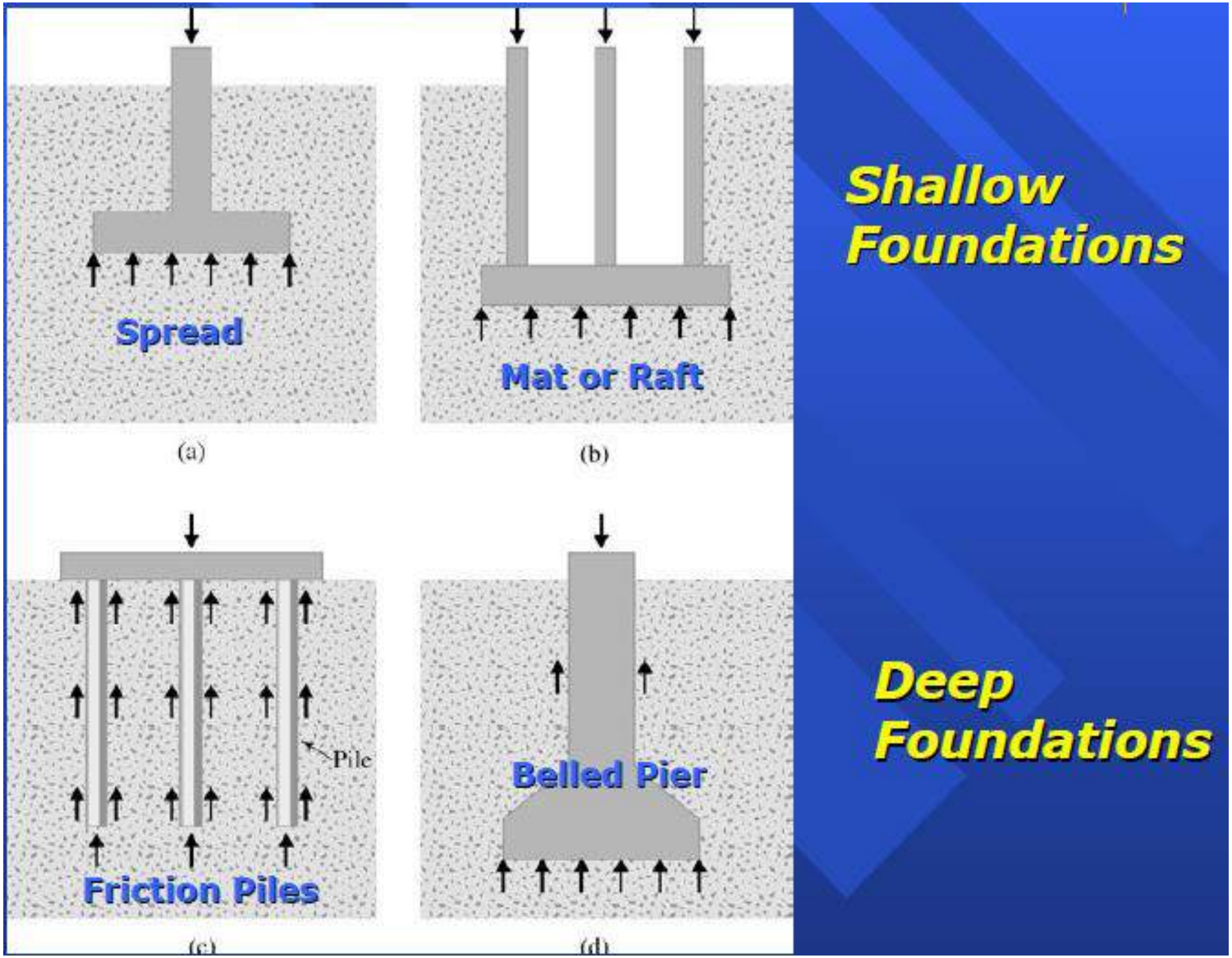
MODULE-IV

BEARING CAPACITY

INTRODUCTION

- The foundation should be designed such that
  - The soil below does not fail in shear, i.e the load applied to soil should be such that the induced stresses in soil is lesser than its capacity
  - Settlement is within the safe limits
  - No differential settlement should occur

Types of Foundations



Bearing Capacity of soil

- Bearing capacity is the ability of soil to safely carry the pressure/load placed on the soil from any engineered structure without undergoing shear failure and excessive large settlements.

CRITERIA FOR THE DETERMINATION OF BEARING CAPACITY

1. LOCATION AND DEPTH:

A **foundation** must be **properly located** and founded at **such a depth** that its **performance does not affected by factors** such as **lateral expulsion** of soil from beneath the foundation, **seasonal volume changes**, presence of **adjoining structures** etc.

2. ***Shear failure*** *of the foundation or* ***bearing capacity failure,*** *as it is sometimes called,* shall not occur. (This is **associated with plastic flow of the soil** material underneath the foundation, and lateral expulsion of the soil from underneath the footing of the foundation); and,

3. *The* ***probable settlements****, differential as well as total, of the foundation must be* limited to safe, tolerable or acceptable magnitudes. In other words, the anticipated settlement under the applied pressure on the foundation should not be detrimental to the stability of the structure.

Last two criteria are known as the shear strength criterion, and settlement criterion, respectively. The **design value** of the safe bearing capacity, obviously, would be the **smaller of the two values,** obtained from these two criteria.

BASIC DEFINITIONS

- **Gross pressure intensity (q)**

  Total pressure at the base of the footing due to weight of superstructure, self weight of the footing and the weight of the earthfill/soil

- **Net Pressure Intensity(q_n)**

  Difference in intensities of the gross pressure after the construction of the structure and the original overburden pressure

$$q_n = q - \gamma D$$

- **Ultimate Bearing Capacity (q_u) :**

The ultimate bearing capacity is the minimum gross pressure intensity at the base of the foundation at which soil fails in shear

- **Net ultimate Bearing Capacity (q_{nu}) :**

It is the minimum net pressure intensity at the base of foundation that cause shear failure of the soil

Thus, $q_{nu} = q_u - \gamma D$ (overburden pressure)

- **Net Safe Bearing Capacity (q_{ns}) :**

  Net ultimate bearing capacity divided by a Factor of safety

  Thus, $q_{ns} = q_{nu} / F$

  F - Factor of safety usually taken as 2.00 -3.00

- **Safe Bearing Capacity (q_s) :**

  It is the maximum pressure which the soil can carry safely without risk of shear failure

  It is equal to net safe bearing capacity plus overburden

  $$q_s = q_{nu} / F + \gamma D$$

- **Net Safe Settlement Pressure (q_{np}) :**

  It is the net pressure which the soil can carry without exceeding allowable settlement

- **Net Allowable Bearing Pressure (q_{na}):**

  It is the net bearing pressure which can be used for design of foundation
  Thus,

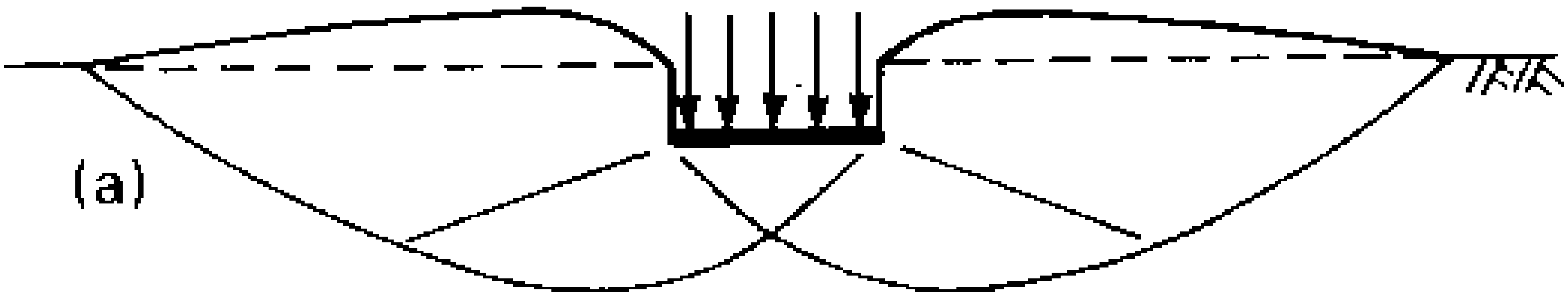
$$q_{na} = q_{ns} \quad ; \text{if } q_{np} > q_{ns}$$
$$q_{na} = q_{np} \quad ; \text{if } q_{ns} > q_{np}$$

It is also known as Allowable Soil Pressure

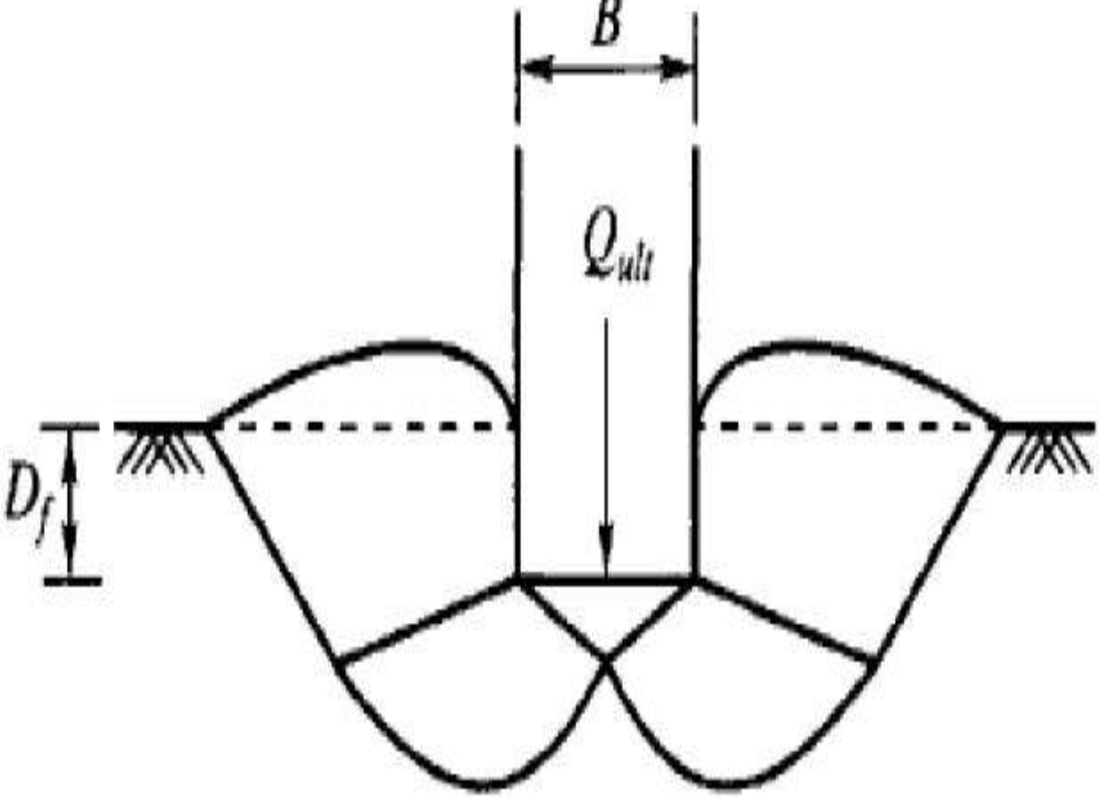
TYPES OF BEARING CAPACITY FAILURES

- Distinct failure patterns are developed depending on failure mechanism
- Vesic (1973) classified shear failure of soil under a foundation base into three categories depending on the type of soil & location of foundation
  - General Shear failure
  - Local Shear failure
  - Punching Shear failure

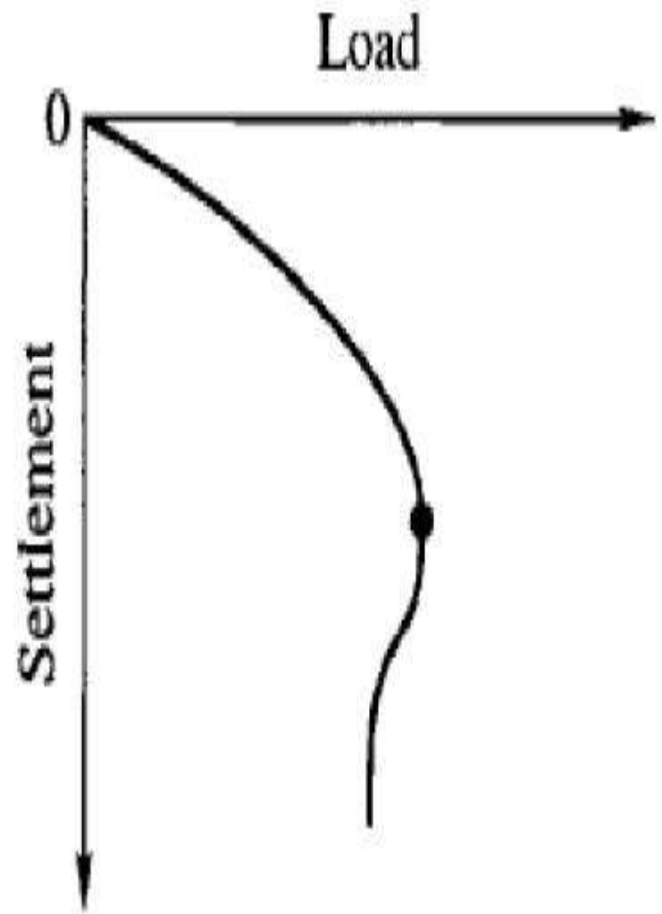
BEARING CAPACITY FAILURES- GENERAL SHEAR FAILURE



- In low compressibility (dense or stiff) soils
- Heaving on both sides of foundation
- Final slip (movement of soil) on one side only causing structure to tilt



(a) General shear failure

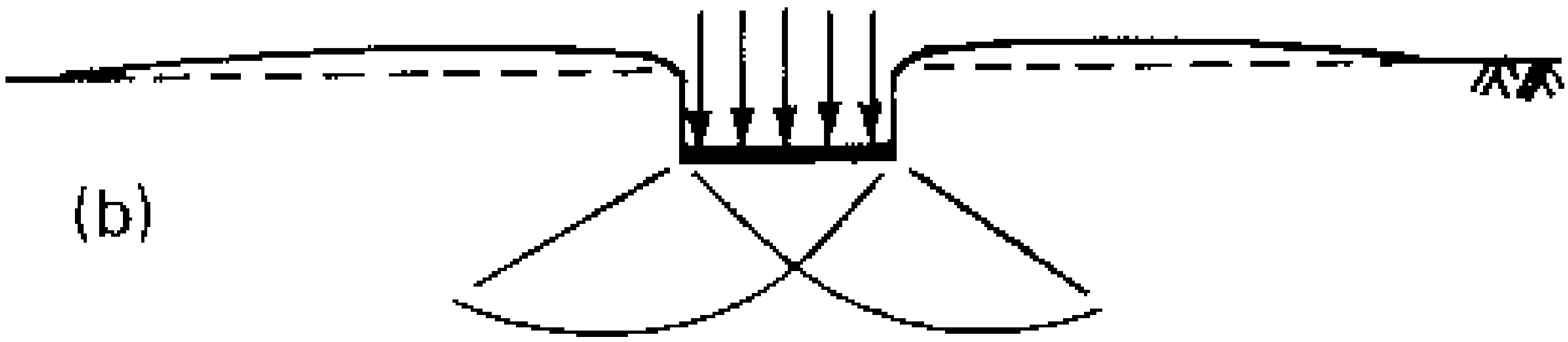


Load vs. Settlement behaviour

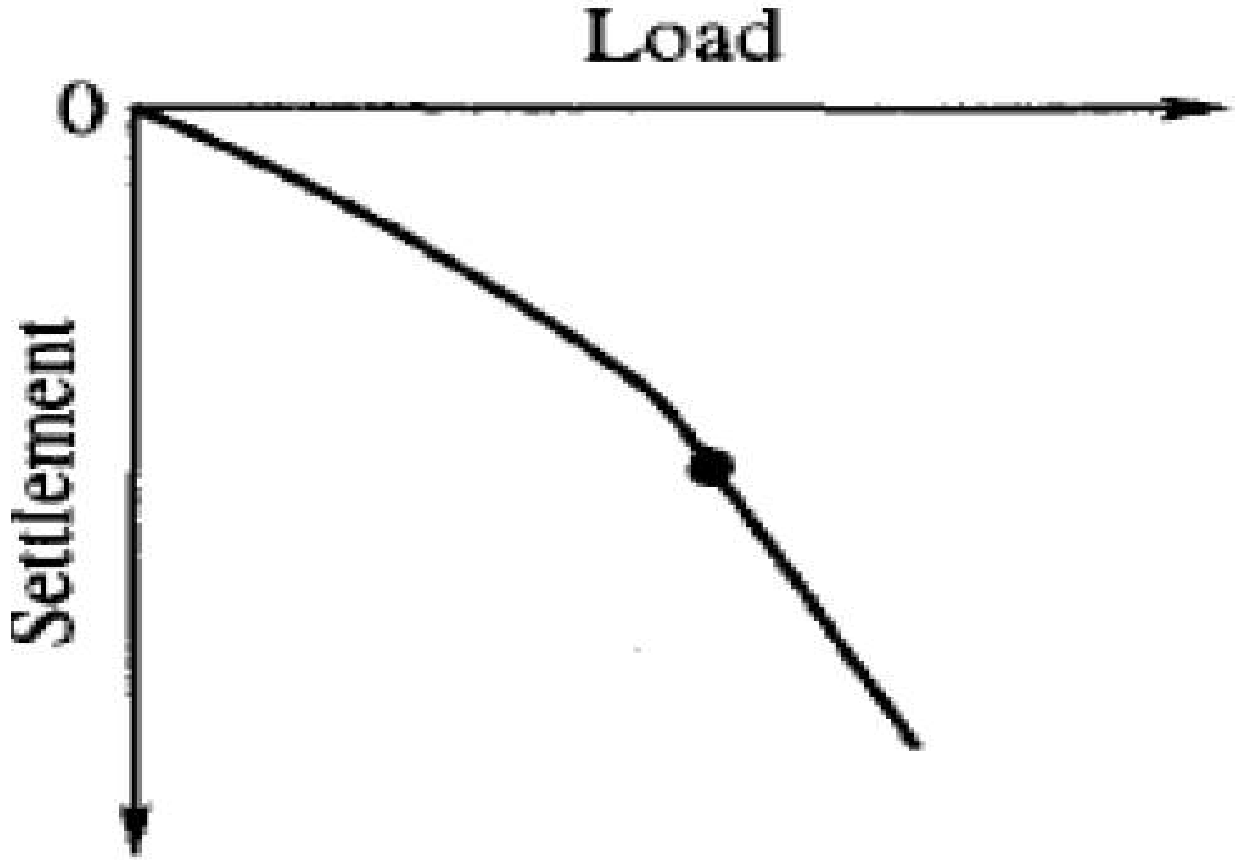
BEARING CAPACITY FAILURES- GENERAL SHEAR FAILURE

- It has well defined failure surface reaching to ground surface
- There is considerable bulging of sheared mass of soil adjacent to footing
- Failure is accompanied by tilting of footing
- Failure is sudden
- Ultimate bearing capacity is well defined

BEARING CAPACITY FAILURES- LOCAL SHEAR FAILURE



- In highly compressible soils
- Only slight heaving on sides
- Significant compression of soil under footing but no tilting

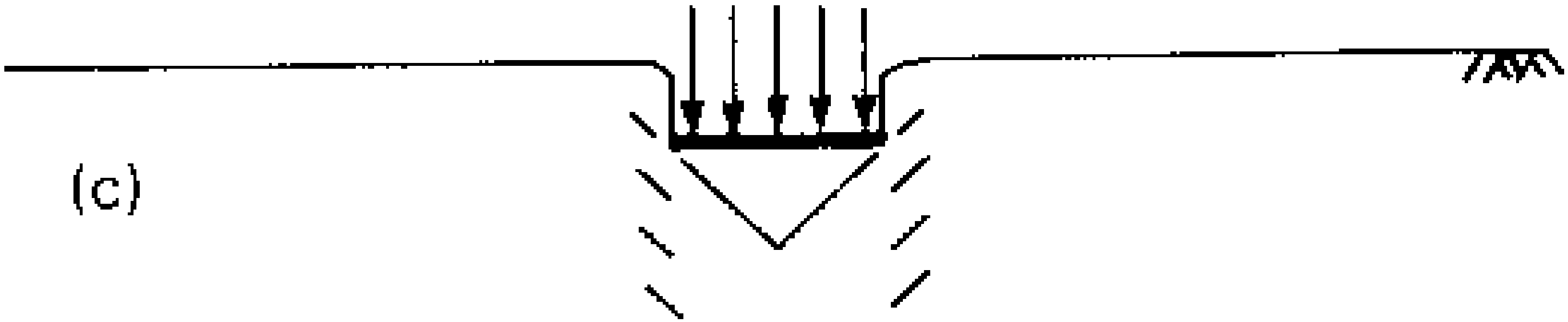


Load vs. Settlement behaviour

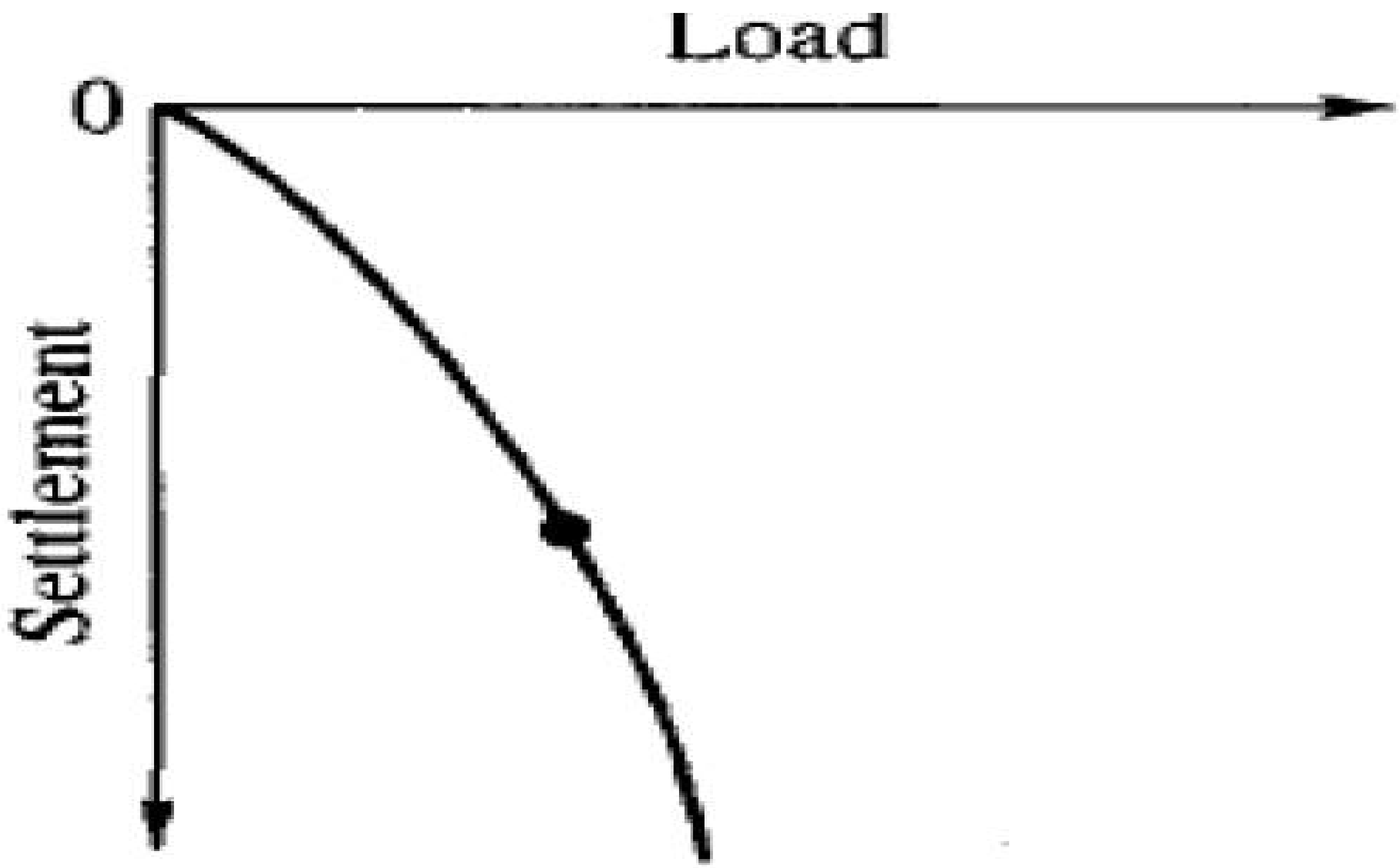
BEARING CAPACITY FAILURES- LOCAL SHEAR FAILURE

- In soils of high compressibility and in sands having relative density between 35 and 70 percent
- Failure pattern is clearly defined only immediately below the footing
- Failure surface do not reach ground surface
- Only slight bulging of soil around the footing
- Failure is not sudden and there is no tilting of the footing
- Failure is defined by large settlements
- Ultimate bearing capacity not well defined

BEARING CAPACITY FAILURES- PUNCHING SHEAR FAILURE



- In loose, uncompacted soils
- Vertical shearing around edges of footing
- High compression of soil under footing, hence large settlements
- No heaving, no tilting



Load vs. Settlement behaviour

BEARING CAPACITY FAILURES- PUNCHING SHEAR FAILURE

- No failure pattern observed
- No bulging of soil around the footing
- No tilting of footing
- Failure is characterised in terms of very large settlements
- Ultimate bearing capacity not well defined

TERZAGHI METHOD (1943).

ASSUMPTIONS:

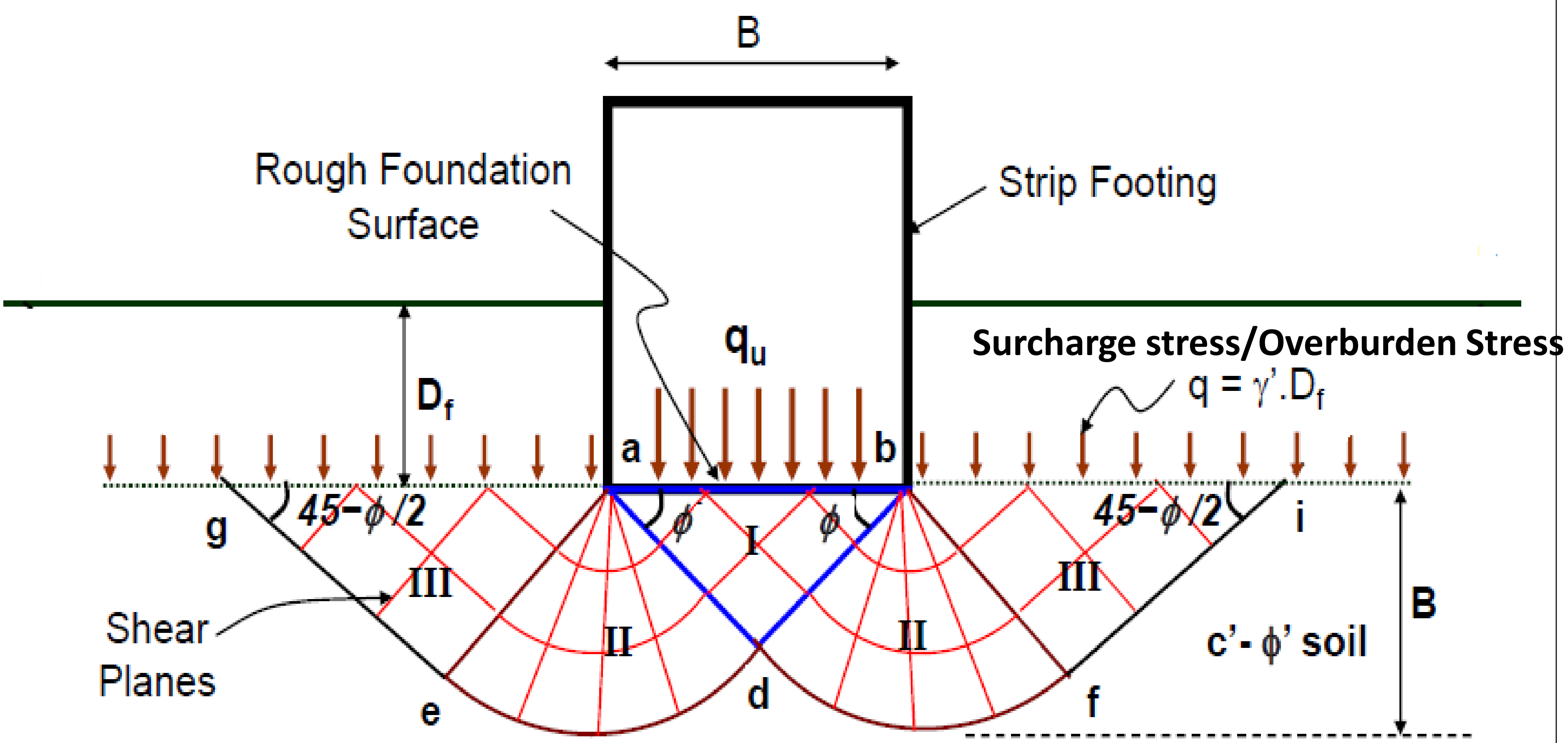
The soil is a semi-infinite, homogeneous, isotropic, rigid–plastic material.

The embedment depth is not greater than the width of the footing ($D_f \leq B$).

General shear failure occurs & The base of the footing is rough.

The soil above the footing base can be replaced by a surcharge stress. This, in effect, means that the shearing resistance of the soil located above the base is neglected.

TERZAGHI'S BEARING CAPACITY THEORY



Terzaghi's Bearing capacity equation for determining ultimate bearing capacity of strip footing (Only)

$$q_{ult} = cN_c + \gamma D_f N_q + 0.5\,\gamma\,B\,N_\gamma$$

$N_\gamma, N_q,$ *and* N_c *are Bearing capacity factors and depends on angle of shearing resistance (ø)*

C= Cohesion B= Width of foundation Df= Depth of foundation	ɣ = Unit weight of soil

Bearing-capacity factors for the Terzaghi equations

ϕ, deg	N_c	N_q	N_γ
0	5.7*	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
34	52.6	36.5	36.0
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
48	258.3	287.9	780.1
50	347.5	415.1	1153.2

Example 14.5: A continuous footing of width 2.5 m rests 1.5 m below the ground surface in clay. The unconfined compressive strength of the clay is 150 kN/m². Calculate the ultimate bearing capacity of the footing. Assume unit weight of soil is 16 kN/m³.

(S.V.U.—B.E., (R.R.)—May, 1969)

Continuous footing $b = 2.5$ m $D_f = 1.5$ m

Pure clay.

$\phi = 0°$ $\quad q_u = 150 \text{ kN/m}^2$ $\quad \gamma = 16 \text{ kN/m}^3$

$$c = \frac{q_u}{2} = 75 \text{ kN/m}^2$$

For $\phi = 0°$, Terzaghi's factors are: $N_\gamma = 0$, $N_q = 1$, and $N_c = 5.7$.

$$q_{ult} = cN_c + \frac{1}{2}\gamma b N_\gamma + \gamma D_f N_q = cN_c + \gamma D_f N_q, \text{ in this case.}$$

$$\therefore \quad q_{ult} = 5.7 \times 75 + 16 \times 1.5 \times 1 = 451.5 \text{ kN/m}^2 \approx \mathbf{450\ kN/m^2}.$$

Example 14.6: Compute the safe bearing capacity of a continuous footing 1.8 m wide, and located at a depth of 1.2 m below ground level in a soil with unit weight $\gamma = 20$ kN/m^3, $c = 20$ kN/m^2, and $\phi = 20°$. Assume a factor of safety of 2.5. Terzaghi's bearing capacity factors for $\phi = 20°$ are $N_c = 17.7$, $N_q = 7.4$, and $N_\gamma = 5.0$, what is the permissible load per metre run of the footing ?

$b = 1.8$ m continuous footing $D_f = 1.2$ m

$\gamma = 20$ kN/m^3 $c = 20$ kN/m^2

$\phi = 20°$ $N_c = 17.7$

$N_q = 7.4$ $N_\gamma = 5.0$ $\eta = 2.5$

$$q_{\text{ult}} = cN_c + \frac{1}{2}\gamma b N_\gamma + \gamma D_f N_q$$

$$= 20 \times 17.7 + \frac{1}{2} \times 20 \times 1.8 \times 5.0 + 20 \times 1.2 \times 7.4$$

$$= 621.6 \text{ kN/m}^2$$

$$q_{\text{net ult}} = q_{\text{ult}} - \gamma D_f = 621.6 - 20 \times 1.2 = 597.6 \text{ kN/m}^2$$

$$q_{\text{net safe}} = \frac{q_{\text{net ult}}}{\eta} = \frac{597.6}{2.5} = 239 \text{ kN/m}^2$$

$$q_{\text{safe}} = q_{\text{net safe}} + \gamma D_f = 239 + 20 \times 1.2 = \mathbf{263 \text{ kN/m}^2}$$

Permissible load per metre run of the wall = 263 × 1.8 kN = **473.5 kN**.

Terzaghi's Bearing capacity equation for determining ultimate bearing capacity of other Foundations (Square and Round)

General form of Terzaghi's Bearing Capacity Theory

$$q_{ult} = c' N_c s_c + \gamma' D_f N_q + 0.5\gamma\ B N_\gamma s_\gamma$$

s_c and $s_\gamma \rightarrow$ shape factors

For:	strip	round	square
$s_c =$	1.0	1.3	1.3
$s_\gamma =$	1.0	0.6	0.8

Example 14.9: A square footing, 1.8 m × 1.8 m, is placed over loose sand of density 16 kN/m^3 and at a depth of 0.8 m. The angle of shearing resistance is 30°. N_c = 30.14, N_q = 18.4, and N_γ = 15.1. Determine the total load that can be carried by the footing.

(S.V.U.—Four-year B.Tech.,—Apr., 1983)

Square footing $b = 1.8$ m

$$\gamma = 16 \text{ kN/m}^3, \quad c = 0, \phi = 30°, \quad D_f = 0.8 \text{ m}$$

$$N_c = 30.14, N_q = 18.4, \quad N_\gamma = 15.1$$

$$q_{ult} = 1.3\, cN_c + 0.4\gamma\, b\, N_\gamma + \gamma D_f N_q = 0.4\, \gamma b\, N_\gamma + \gamma D_f N_q, \text{ in this case}$$

$$\therefore \quad q_{ult} = 0.4 \times 16 \times 1.8 \times 15.1 + 16 \times 0.8 \times 18.4 = 174 + 236 = 410 \text{ kN/m}^2$$

The ultimate load that can be carried by the footing

$$= q_{ult} \times \text{Area} = 410 \times 1.8 \times 1.8 \text{ kN} = \mathbf{1328.4 \text{ kN}}.$$

Example 14.7: What is the ultimate bearing capacity of a square footing resting on the surface of a saturated clay of unconfined compressive strength of 100 kN/m^2.

(S.V.U.—Four-year B. Tech.—Apr., 1983)

Square footing.

Saturated clay,

$$\phi = 0^\circ \qquad D_f = 0.$$

Terzaghi's factors for $\phi = 0^\circ$ are : $N_c = 5.7$, $N_q = 1$, and $N_\gamma = 0$.

$$q_u = 100 \text{ kN/m}^2$$

$$\therefore \qquad c = \frac{1}{2} q_u = 50 \text{ kN/m}^2$$

$$q_{\text{ult}} = 1.3\, cN_c = 1.3 \times 50 \times 5.7 = 370 \text{ kN/m}^2$$

$$\therefore \qquad q_{\text{ult}} = \mathbf{370\ kN/m^2}.$$

Example 14.14: A circular footing is resting on a stiff saturated clay with $q_u = 250$ kN/m^2. The depth of foundation is 2 m. Determine the diameter of the footing if the column load is 600 kN. Assume a factor of safety as 2.5. The bulk unit weight of soil is 20 kN/m^3.

(S.V.U.—Four-year B. Tech.—Dec., 1982)

Circular footing: $\phi = 0°$, $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$

$$q_u = 250 \text{ kN/m}^2 \qquad c = \frac{1}{2} q_u = 125 \text{ kN/m}^2 \qquad D_f = 2 \text{ m}$$

Column load = 600 kN $\quad \eta = 2.5 \quad \gamma = 20$ kN/m^3

$$q_{\text{ult}} = 1.3\, c\, N_c + 0.3\, \gamma\, D\, N_\gamma + \gamma\, D_f N_q = 1.3\, c\, N_c + \gamma\, D_f N_q, \text{ in this case.}$$

$$\therefore \quad q_{\text{ult}} = 1.3 \times 125 \times 5.7 + 20 \times 2 \times 1 = 966 \text{ kN/m}^2$$

$$q_{\text{net ult}} = q_{\text{ult}} - \gamma D_f = 966 - 20 \times 2 = 926 \text{ kN/m}^2$$

$$q_{\text{safe}} = \frac{q_{\text{net ult}}}{\eta} + \gamma D_f = \frac{926}{2.5} + 20 \times 2 = 410 \text{ kN/m}^2$$

Safe load on the column

$$= q_{\text{safe}} \times \text{Area} = 600 \text{ kN}$$

$$\therefore \quad 600 = \frac{410 \times \pi d^2}{4}$$

$$\therefore \quad d = \sqrt{\frac{4 \times 600}{410}} \text{ m} = 1.365 \text{ m}$$

A diameter of **1.5 m** may be adopted in this case.

Presence of the Water Table

In granular soils, the presence of water in the soil can substantially reduce the bearing capacity.

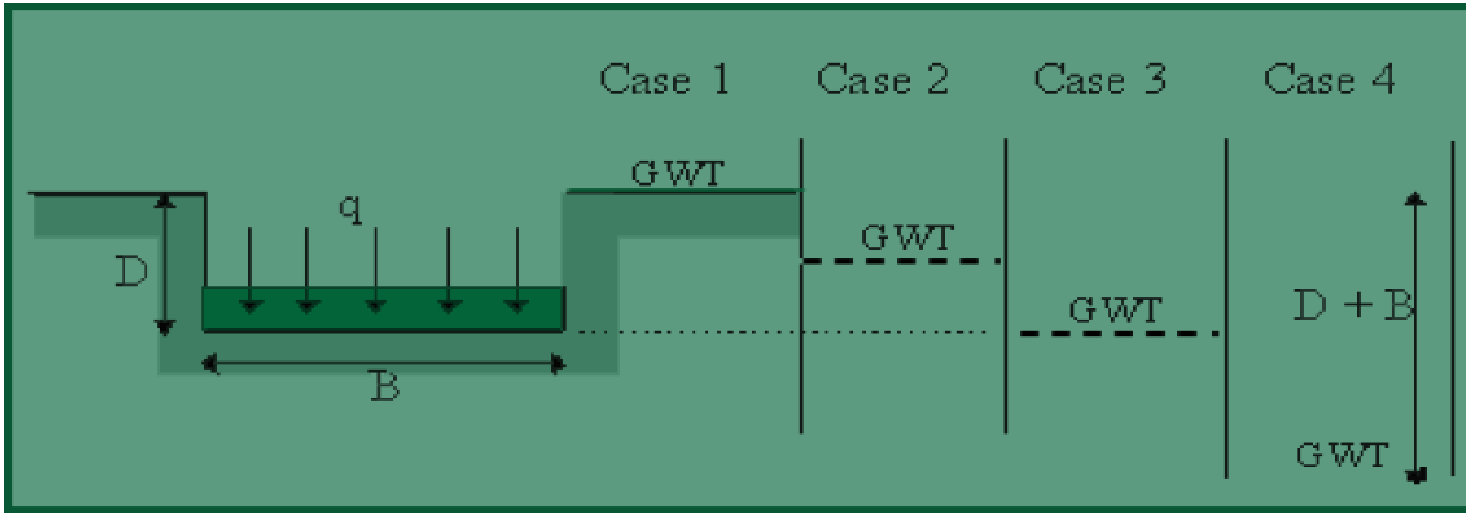
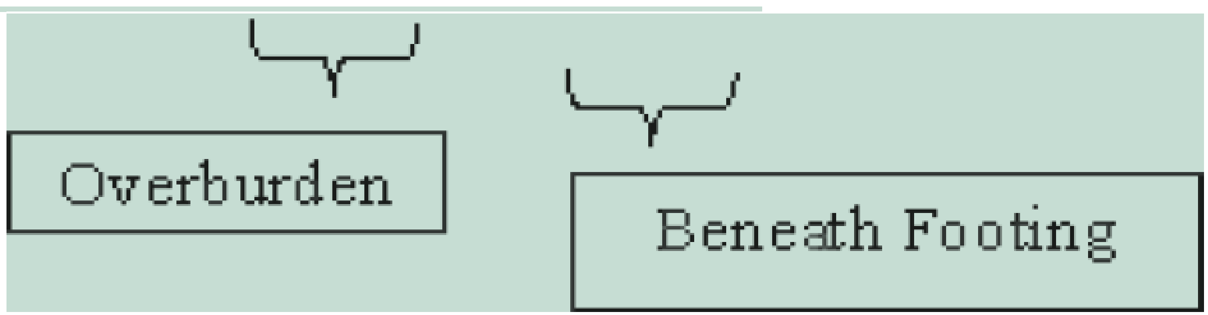


Fig 4.17 footing with various levels of water table

Case 1 : use γ' for the $\gamma D N_q$ and $\frac{1}{2} B \gamma N \gamma$ terms



Case 2 : for the $\gamma D N_q = \sigma' N_q$ term calculate the effective stress at the depth of the footing

$\sigma' = \sigma - u = \gamma D - \gamma_w h_w$, and

for the $\frac{1}{2} B \gamma N$ use γ'.

Case 3 : use γ for the $\gamma D N_q$ term, and

use γ' for the $\frac{1}{2} B \gamma N \gamma$ term.

Case 4 : use γ for the $\gamma D N_q$ and $\frac{1}{2} B \gamma N \gamma$ terms.

γ' is the submerged or effective unit weight ($\gamma_{sat} - \gamma_w$)

Example 14.11: A foundation, 2.0 m square is installed 1.2 m below the surface of a uniform sandy gravel having a density of 19.2 kN/m^3, above the water table and a submerged density of 10.1 kN/m^3. The strength parameters with respect to effective stress are $c' = 0$ and $\phi' = 30°$. Find the gross ultimate bearing capacity for the following conditions:

(*i*) Water table is well below the base of the foundation (*i.e.*, the whole of the rupture zone is above the water table);

(*ii*) Water table rises to the level of the base of the foundation; and

(*iii*) the water table rises to ground level.

(For $\phi = 30°$, Terzaghi gives $N_q = 22$ and $N_\gamma = 20$)

(S.V.U.—B. Tech., (Part-time)—Sept., 1982)

Square $b = 2$ m $D_f = 1.2$ m $c' = 0$ $\phi' = 30°$

$\gamma = 19.2$ kN/m^3 $\gamma' = 10.1$ kN/m^3 $N_q = 22$ $N_\gamma = 20$

$$q_{ult} = 1.3\, c\, N_c + 0.4\, \gamma\, b\, N_\gamma + \gamma D_f N_q = 0.4\, \gamma\, b\, N_\gamma + \gamma D_f N_q, \text{ in this case.}$$

or

$$q_{ult} = 0.4 \times 19.2 \times 2 \times 20 + 19.2 \times 1.2 \times 22 = \mathbf{814\ kN/m^2}$$

(*ii*) Water table rises to the level of the base of the foundation:

$$q_{ult} = 0.4\, \gamma'\, b\, N_\gamma + \gamma\, D_f N_q$$

$$= 0.4 \times 10.1 \times 2 \times 20 + 19.2 \times 1.2 \times 22 = \mathbf{668\ kN/m^2}$$

(*iii*) Water table rises to the ground level:

$$q_{ult} = 0.4\, \gamma'\, bN_\gamma + \gamma' D_f N_q$$

$$= 0.4 \times 10.1 \times 2 \times 20 + 10.1 \times 1.2 \times 22 = \mathbf{428\ kN/m^2}$$

Thus, as the water table rises, there is about 20% to 50% decrease in the ultimate bearing capacity.

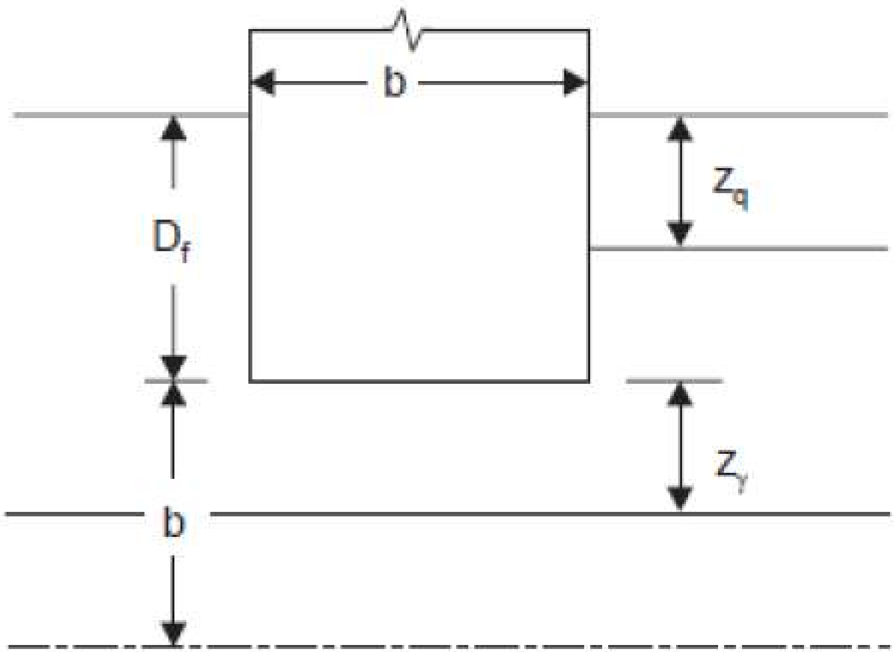
Alternate Approximate method

$$q_{ult} = {}^{*}c'N_c + \gamma D_f N_q R_q + \frac{1}{2}{}^{**}\gamma b\, N_\gamma \cdot R_\gamma$$

*appropriate multiplying factor should be used for isolated footings.
**Appropriate shape factor.

$$R_q = 0.5\left(1 + \frac{z_q}{D_f}\right)$$

$$R_\gamma = 0.5\left(1 + \frac{z_\gamma}{b}\right)$$



$$z_q = 0 \ldots R_q = 0.5 \qquad z_\gamma = 0 \ldots R_\gamma = 0.5$$

$$z_q = D_f \ldots R_q = 1.0 \qquad z_\gamma = b \ldots R_\gamma = 1.0$$

Note.

- For **$z_q > D_f$** *(the water table is below the base of the footing)*, ***R_q is limited to 1.0.***
- For **$0 \le z_q \le D_f$** *(the water table is above the base of the footing)*, ***R_γ is limited to 0.5.***
- *for* ***$z_q > (D_f + b)$*** or ***$z_\gamma > b$***, ***R_q*** as well as ***R_γ*** are ***limited to 1.0.***
- For ***$z_q = 0$***, ***R_q*** as well as ***R_γ*** are ***limited to 0.5***.

Example 14.10: Compute the safe bearing capacity of a square footing 1.5 m × 1.5 m, located at a depth of 1 m below the ground level in a soil of average density 20 kN/m³. $\phi = 20°$, $N_c = 17.7$, $N_q = 7.4$, and $N_\gamma = 5.0$. Assume a suitable factor of safety and that the water table is very deep. Also compute the reduction in safe bearing capacity of the footing if the water table rises to the ground level. (S.V.U.—B.Tech., (Part-time)—Sept., 1983)

$b = 1.5$ m Square footing $D_f = 1$ m

$\gamma = 20$ kN/m³ $\quad \phi = 20° \quad N_c = 17.7, \quad N_q = 7.4$, and $N_\gamma = 5.0$

$c = 0$ and $\eta = 3$

$q_{\text{ult}} = 1.3\, c\, N_c + 0.4\, \gamma\, b\, N_\gamma + \gamma D_f N_q = 0.4\, \gamma\, b\, N_\gamma + \gamma\, D_f N_q$, in this case.

$= 0.4 \times 20 \times 1.5 \times 5.0 + 20 \times 1 \times 7.4 = 60 + 148 = 208 \text{ kN/m}^2$

$q_{\text{net ult}} = q_{\text{ult}} - \gamma\, D_f = 208 - 20 \times 1 = 188 \text{ kN/m}^2$

$$q_{\text{safe}} = \frac{q_{\text{net ult}}}{\eta} + \gamma\, D_f = \frac{188}{3} + 20 \times 1 = 83 \text{ kN/m}^2$$

If the water table rises to the ground level,

$R_\gamma = 0.5 = R_q$

$\therefore \quad q_{\text{ult}} = 0.4\, \gamma^b N_\gamma \,.\, R_\gamma + \gamma D_f N_q \,.\, R_q$

$= 0.4 \times 20 \times 1.5 \times 5.0 \times 0.5 + 20 \times 1 \times 7.4 \times 0.5 = 30 + 74 = 104 \text{ kN/m}^2$

$q_{\text{net ult}} = q_{\text{ult}} - \gamma D_f = 104 - 10 \times 1 = 94 \text{ kN/m}^2$

$$q_{\text{safe}} = \frac{q_{\text{net ult}}}{\eta} + \gamma D_f = \frac{94}{3} + 10 \times 1 = 41 \text{ kN/m}^2$$

Percentage reduction in safe bearing capacity

$$= \frac{42}{83} \times 100 = 50.$$

Extension of Terzaghi's Equation for Local shear failure of soils

For 'local shear failure', as given by Terzaghi :

$$c' = (2/3)c$$

$$\tan \varphi' = (2/3) \tan \varphi$$

SKEMPTON METHOD (1951)

He found that the factor ***Nc*** *is a function* of the depth of foundation and also of its shape.

(For Cohesive soils only)

The net ultimate bearing capacity is given by:

$$q_{net\ ult} = c \cdot N_c$$

Strip footings:

$$N_c = 5\,(1 + 0.2\,D_f\,/b)$$

Square or Circular footings:

$$N_c = 6(1 + 0.2D_f\,/b)$$

IS 6403:1981 METHOD

$$q_{nu} = cN_c s_c d_c i_c + q(N_q - 1) s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma W'$$

$q = \gamma * D_f$

If the water table is at or below a depth of $D_f + B$, measured from the ground surface, w'=1. If the water table rises to the base of the footing or above, w'=0.5. If the water table lies in between then the value is obtained by linear interpolation.

$$d_c = 1 + 0.2(D_f / B)\tan(45 + \phi/2)$$

$$d_q = d_\gamma = 1 \; for \; \phi' < 10^0$$

$$d_\gamma = d_q = 1 + 0.1(D_f / B)\tan(45^0 + \phi/2) \; for \; \phi > 10^0$$

shape factors	s_c	s_q	s_γ
Continuous (Width b)	1.0	1.0	1.0
Rectangular ($b \times L$)	$1 + 0.2\dfrac{b}{L}$	$1 + 0.2\dfrac{b}{L}$	$1 - 0.4\dfrac{b}{L}$
Square (Size b)	1.3	1.2	0.8
Circular (Diameter b)	1.3	1.2	0.6

inclination factors:

$$i_c = i_q = \left(1 - \frac{\theta}{90^0}\right)^2$$

$$i_\gamma = \left(1 - \frac{\theta}{\phi}\right)^2$$

Inclination

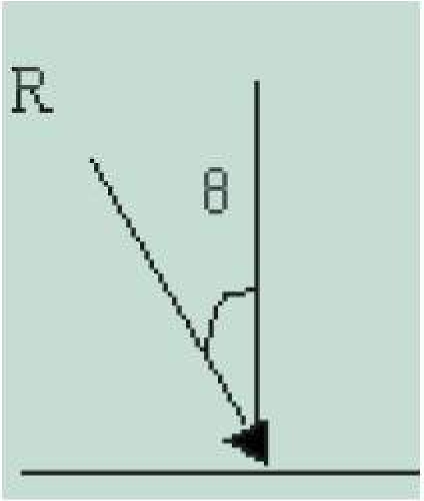
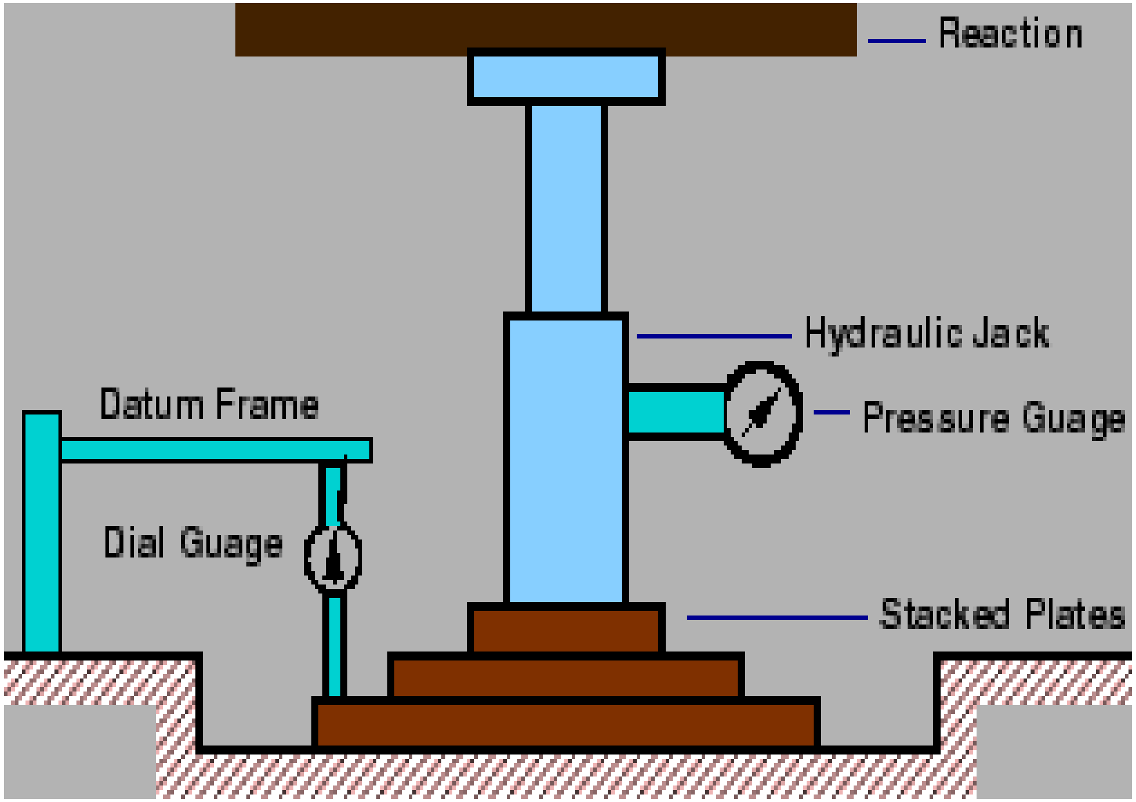
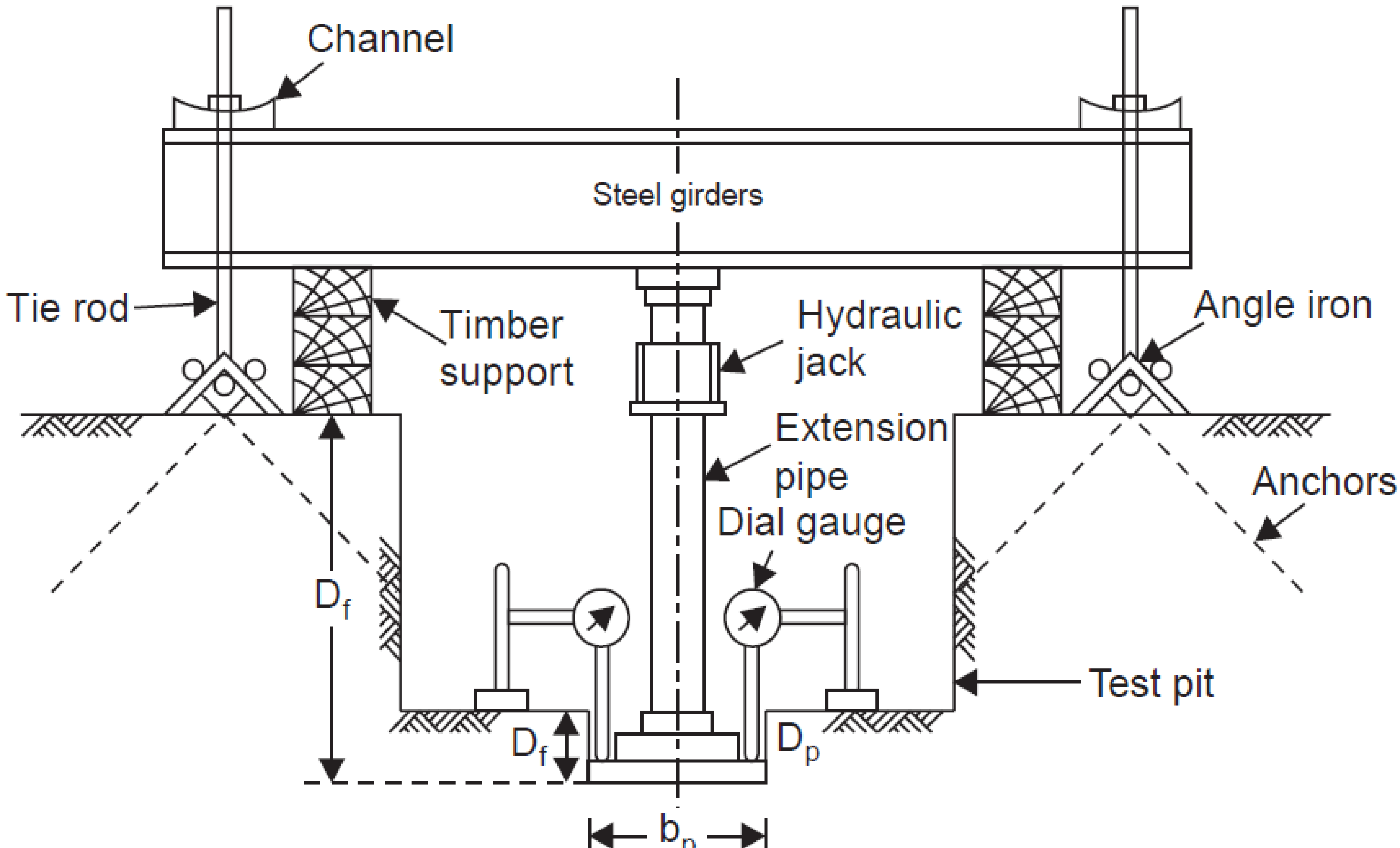
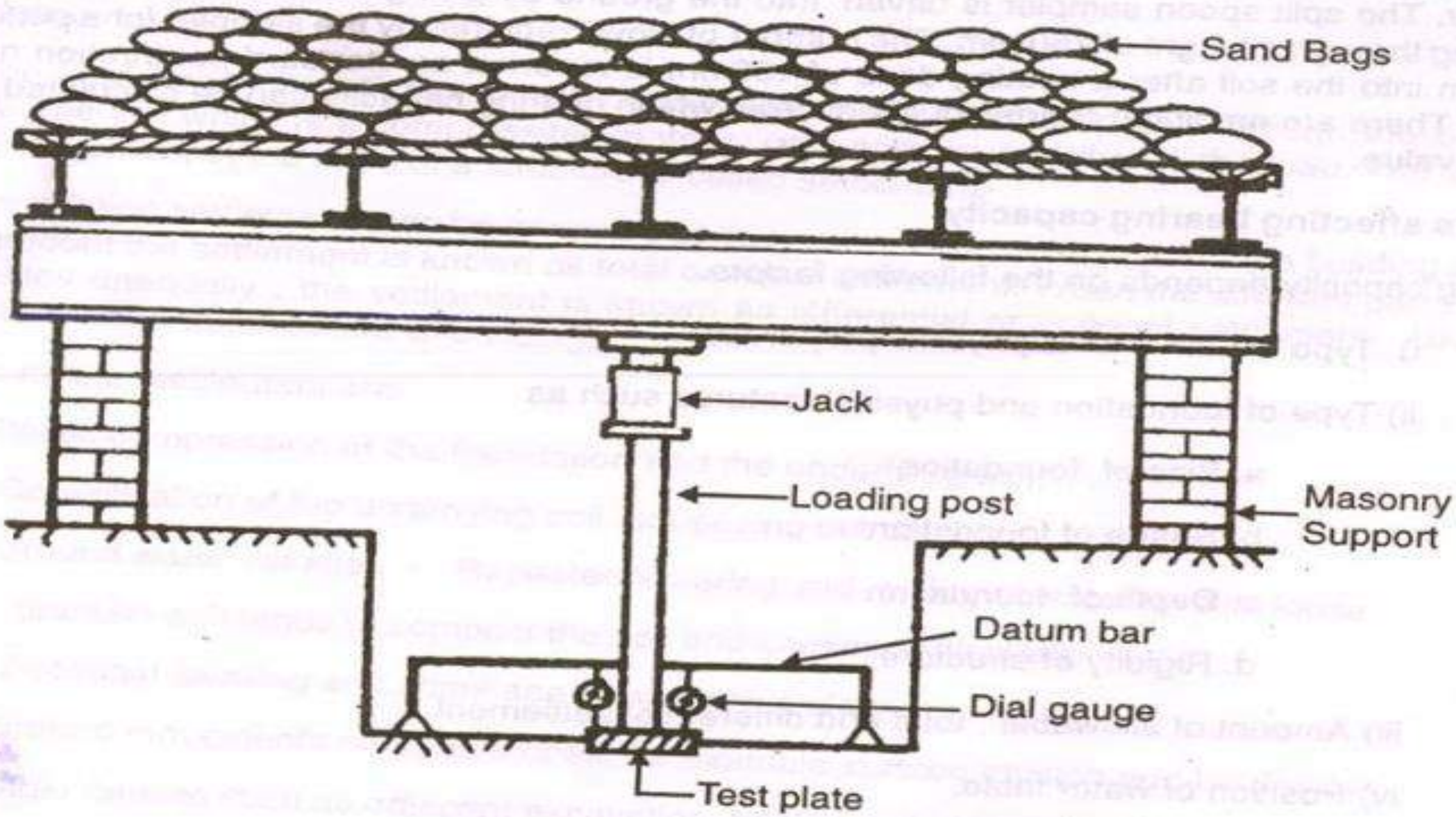


PLATE LOAD TEST





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


Sand Bags
Jack
Loading post
Masonry
Support
Datum bar
Dial gauge
Test plate

- It is a field test used to determine the ultimate bearing capacity of soil
- A pit is dug up to the foundation level
- A square plate of 300mm x 300mm & 25 mm is placed at the centre of the pit
- A dial gauge is connected to the test plate
- Now weights in the form of sand bags are placed on the platforms in equal increments.
- The test is continued till the failure occurs or the plates settled by 25 mm whichever occurs earlier
- The load settlement curve is then recorded.

- The test pit should be at least five times as wide as the test plate and the bottom of the test plate should correspond to the proposed foundation level.
- At the centre of the pit, a small square hole is made the size being that of the test plate and the depth being such that,

$$\frac{D_p}{b_p} = \frac{D_f}{b}$$

(*i*) After excavating the pit of required size and levelling the base, the test plate is seated over the ground.

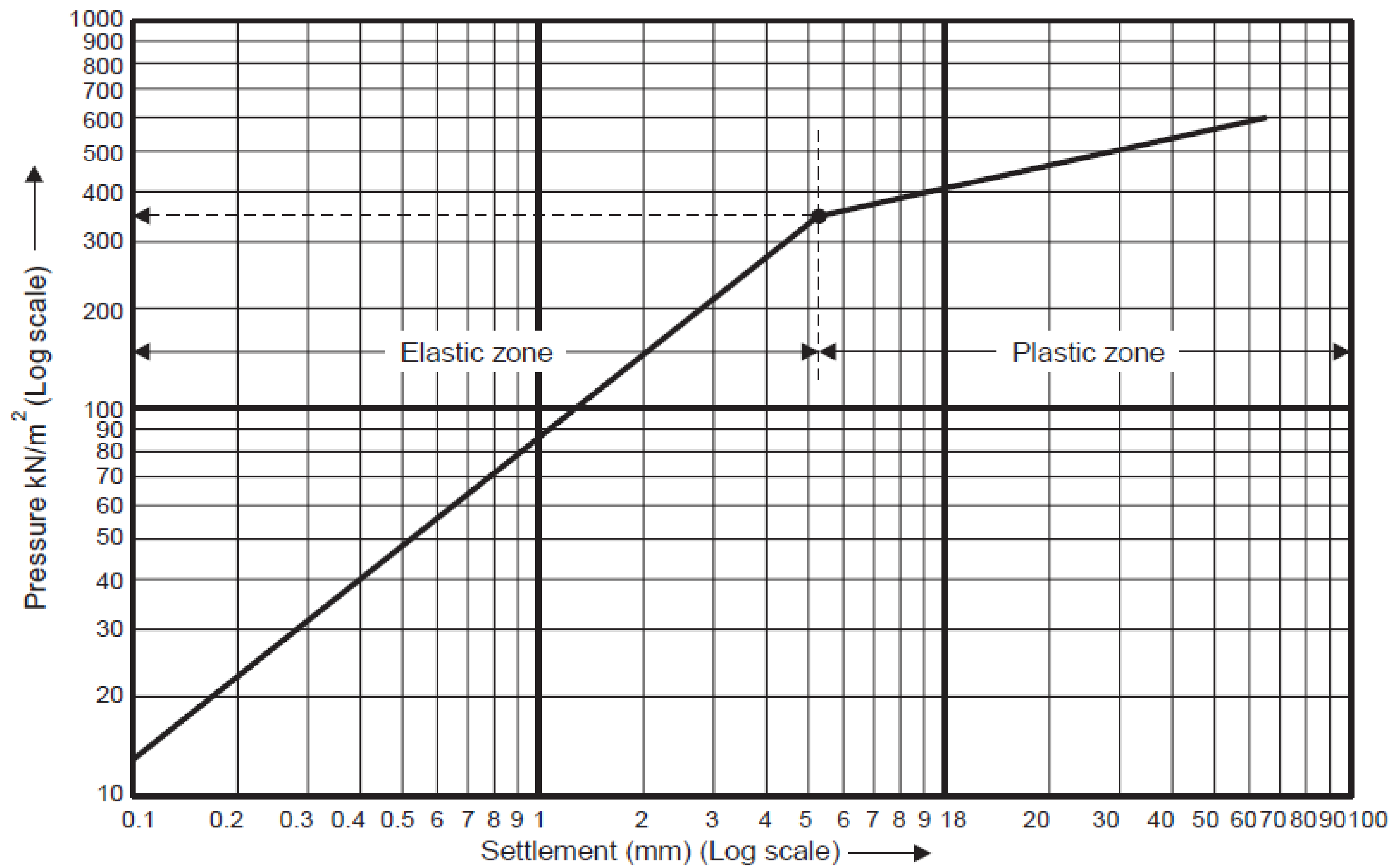
(*ii*) A seating pressure of 7.0 kN/m2 (70 g/cm2) is applied and released before actual loading is commenced.

(*iii*) The first increment of load, say about one-tenth of the anticipated ultimate load, is applied. Settlements are recorded with the aid of the dial gauges after 1 min., 4 min., 10 min., 20 min., 40 min., and 60 min., and later on at hourly intervals until the rate of settlement is less than 0.02 mm/hour, or at least for 24 hours.

(*iv*) The test is continued until a load of about 1.5 times the anticipated ultimate load is applied. According to another school of thought, a settlement at which failure occurs

or at least 2.5 cm should be reached.

(*v*) From the results of the test, a plot should be made between pressure and settlement, which is usually referred to as the ''load-settlement curve'',. The bearing capacity is determined from this plot

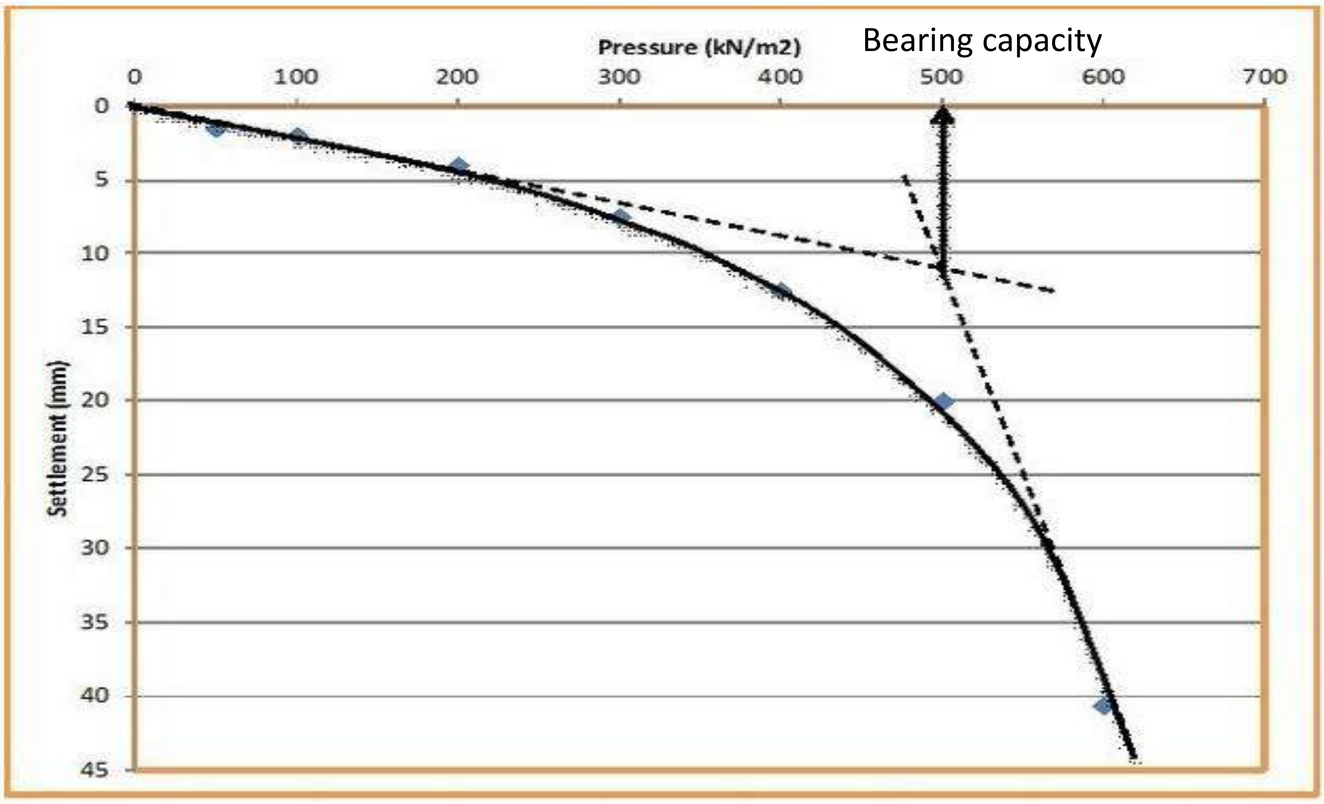


1000
900
800
700
600
500
400
300
200
100
90
80
70
60
50
40
30
20
10
Pressure kN/m2 (Log scale)
Elastic zone
Plastic zone
0.1 0.2 0.3 0.4 0.5 6 7 8 9 1 2 3 4 5 6 7 8 9 18 20 30 40 50 60 70 80 90 100
Settlement (mm) (Log scale)

- The plot between pressure and settlement usually consists of two straight lines as shown in Figure. The point corresponding to the break gives the failure point and the pressure corresponding to it is taken as the bearing capacity.

- IS: 1888–1971 also recommends this method for use with plate load tests.

ALTERNATE METHOD FOR DETERMINATION OF BEARING CAPACITY



Load settlement curve

Settlement of original foundation (S)

Sandy soils

$$\frac{S_f}{S_p} = \left[\frac{b_f(b_p + 0.3)}{b_p(b_f + 0.3)}\right]^2$$

Clayey soils

$$\frac{S_f}{S_p} = \frac{b_f}{b_p}$$

where S_f = settlement of the proposed foundation (mm),
S_p = settlement of the test plate (mm), (same units)
b_f = size of the proposed foundation (m), and
b_p = size of the test plate (m).

Ultimate bearing capacity (qu) for foundation

$$q_{u(f)} = q_{u(p)} \; for\ Clay$$

$$q_{u(f)} = q_{u(p)} \frac{b_f}{b_p} \; for\ sady\ layer$$

where

$q_{u(f)}$ = Ultimate Bearing Capacity for the Proposed Foundation

$q_{u(p)}$ = ultimate Bearing capacity of the Test Plate

Settlement in Soil

- **Immediate Settlement**
- **Consolidation Settlement**

Immediate Settlement in Cohesive Soils

The immediate settlement of a **flexible foundation,** according to Terzaghi (1943), is given by:

$$S_i = q \cdot B \left(\frac{1-\nu^2}{E_s} \right) \cdot I_t$$

Si = immediate settlement at a corner of a rectangular flexible foundation of size L × B,

B = Width of the foundation,

q = Uniform pressure on the foundation,

Es = Modulus of elasticity of the soil beneath the foundation,

ν = Poisson's ratio of the soil, and

I_t = Influence Value, which is dependent on L/B

L/B	*1*	*2*	*3*	*4*	*5*
Influence value I_t	0.56	0.76	0.88	0.96	1.00

Consolidation settlement

$$S_c = \frac{C_C \times H}{1 + e_0} \times \log_{10}\left(\frac{\sigma_0{}' + \Delta\sigma'}{\sigma_0{}'}\right)$$

Example 7.4: A layer of soft clay is 6 m thick and lies under a newly constructed building. The weight of sand overlying the clayey layer produces a pressure of 260 kN/m^2 and the new construction increases the pressure by 100 kN/m^2. If the compression index is 0.5, compute the settlement. Water content is 40% and specific gravity of grains is 2.65.

(S.V.U.—B.E., (R.R.)—Dec., 1976)

Initial pressure, $\bar{\sigma}_0 = 260$ kN/m^2

Increment of pressure, $\Delta\bar{\sigma} = 100$ kN/m^2

Thickness of clay layer, $H = 6$ m $= 600$ cm.

Compression index, $C_c = 0.5$

Water content, $w = 40\%$

Specific gravity of grains, $G = 2.65$

Void ratio, $e = wG$, (since the soil is saturated) $= 0.40 \times 2.65 = 1.06$

This is taken as the initial void ratio, e_0.

Consolidation settlement,

$$S = \frac{H.C_c.}{(1+e_0)} \log_{10}\left(\frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}\right)$$

$$= \frac{600 \times 0.5}{(1+1.06)} \log_{10}\left(\frac{260+100}{260}\right) \text{ cm}$$

$$= \frac{300}{2.06} \log_{10}\left(\frac{360}{260}\right) \text{ cm}$$

$= 21.3$ cm.

Allowable/Permissible Settlements

(IS: 1904-1961) recommends a permissible total settlement of 65 mm for isolated foundations on clay, 40 mm for isolated foundations on sand, 65 to 100 mm for rafts on clay and 40 to 65 mm for rafts on sand.

The permissible differential settlement is 40 mm for foundations on clay and 25 mm for foundations on sand. The angular distortion in the case of large framed structures must not exceed 1/500 normally and 1/1000 if all kinds of minor damage also are to be prevented.

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ABHISHEK SHARMA

MODULE V
DEEP FOUNDATIONS

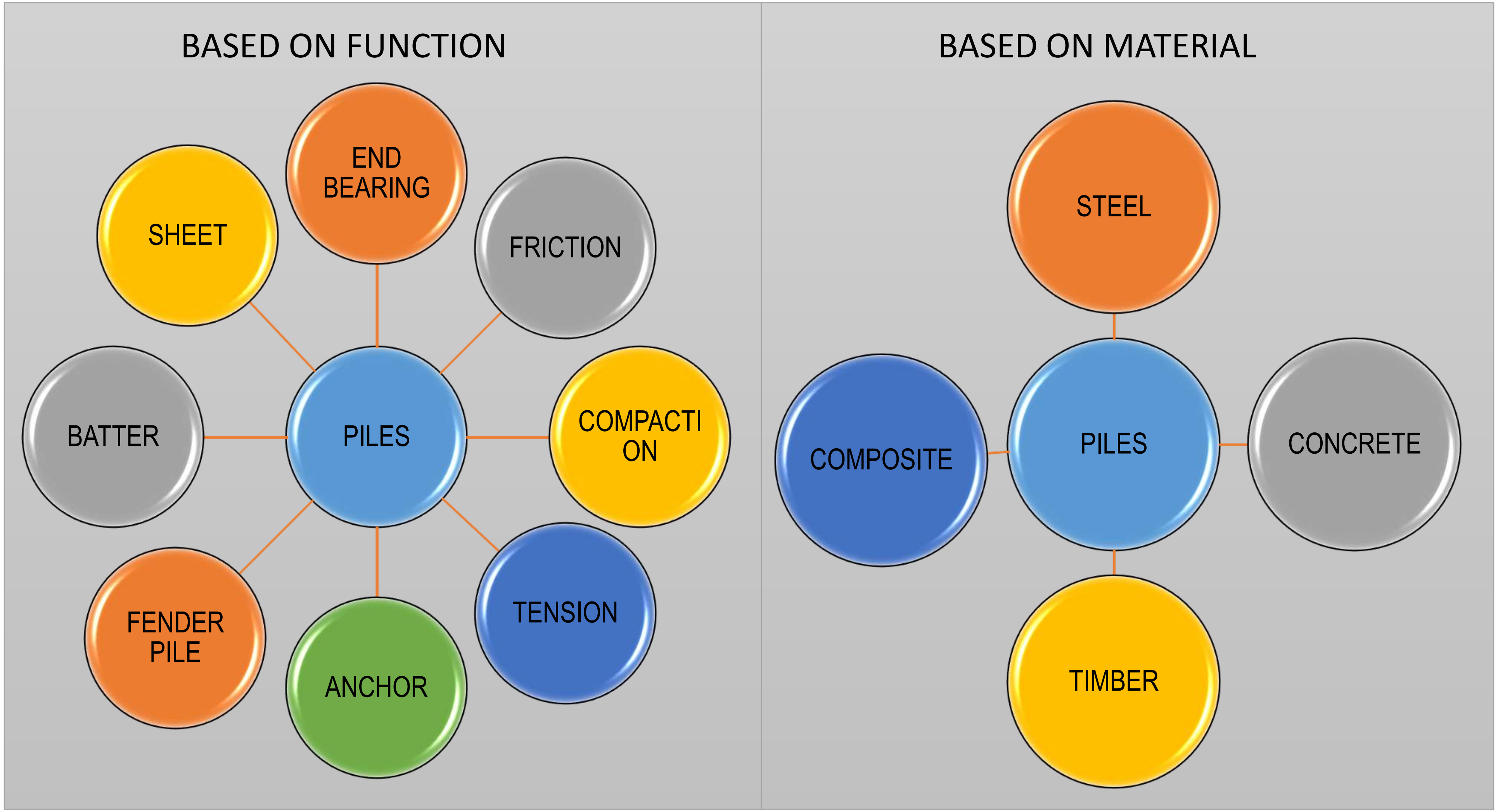
What is Pile Foundations?

- Pile Foundation means a construction for the foundation of a abutment or pier which is supported on piles.
- Pile is as like a column that is driven into the foundation soil or constructed inside the foundation soil.

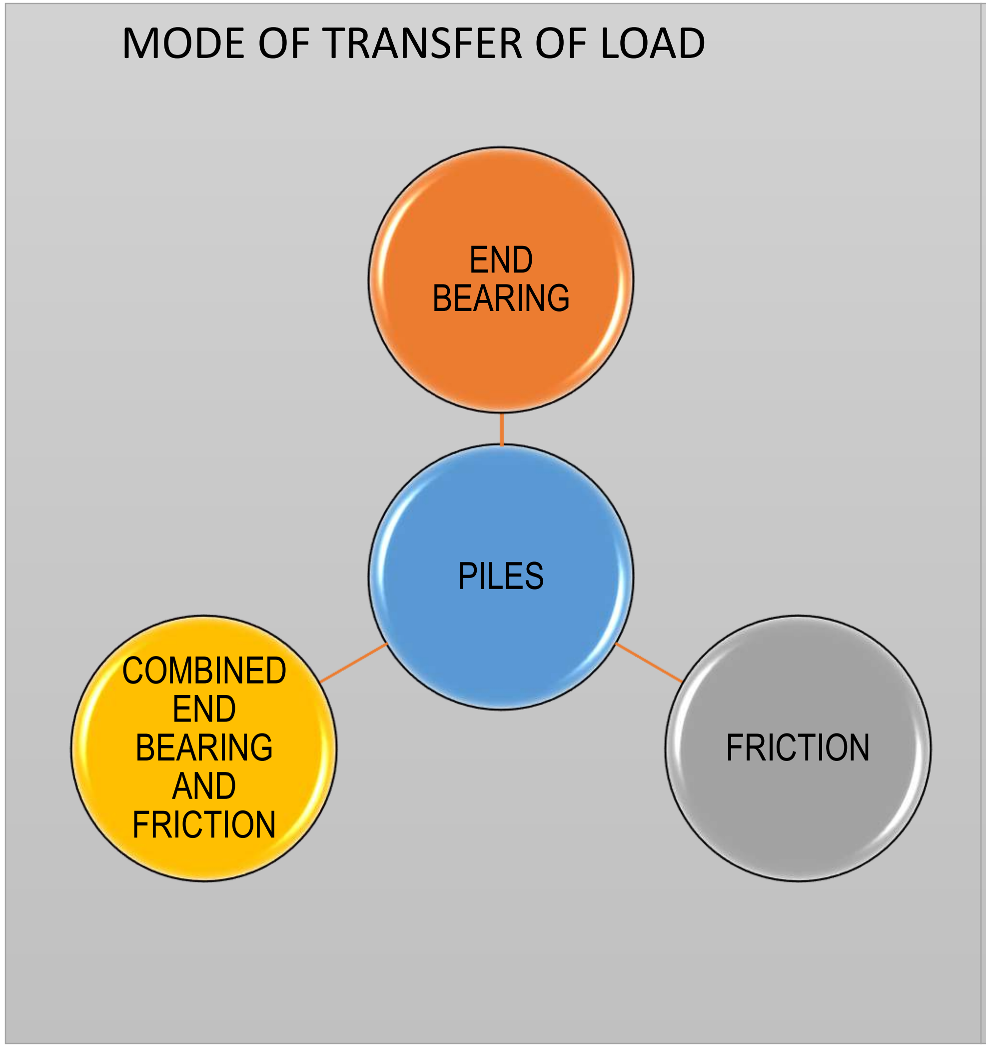
PILE FOUNDATIONS- USES

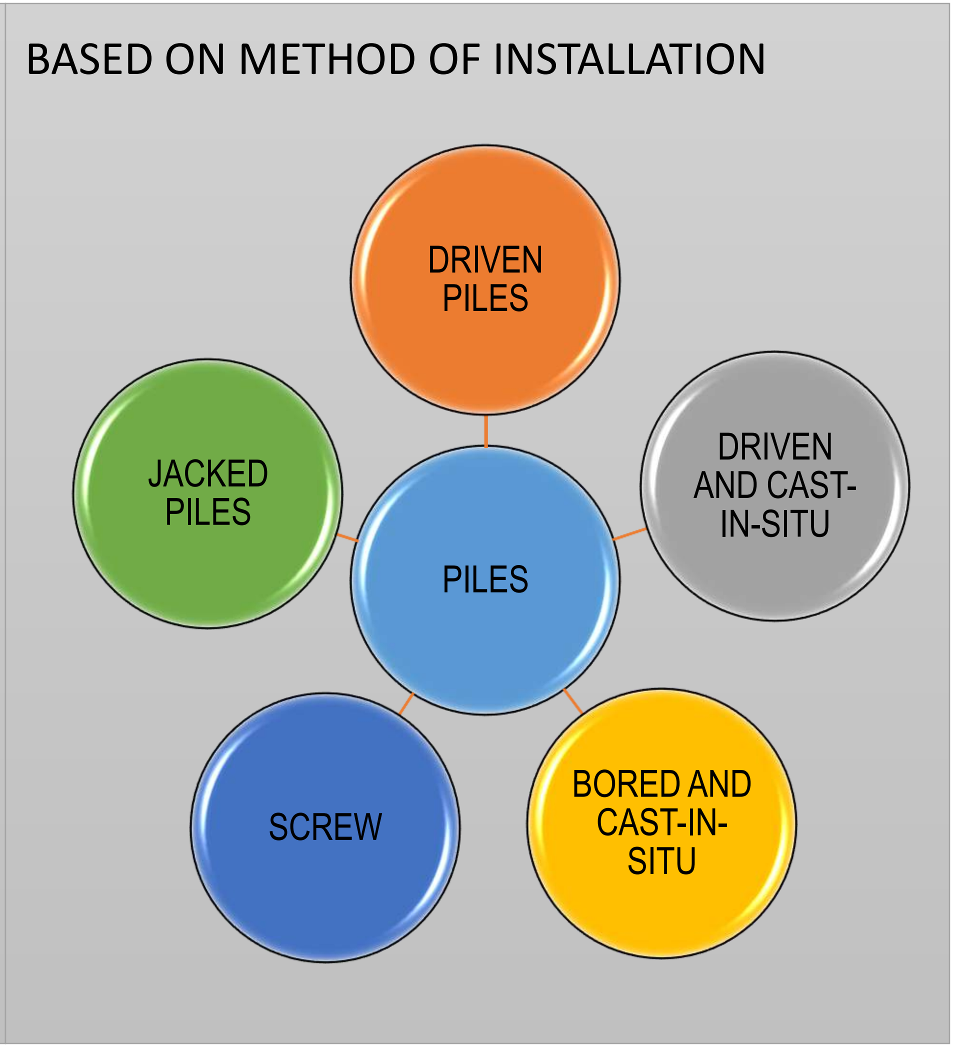
- Highly compressible or weak strata directly below the ground surface
- Foundations for irregular structures- irregular relative to the plan and load distribution
- Transmission of load through deep waters to a hard stratum
- Structures with risk of soil being washed out- shallow foundations almost impossible
- In expansive soils- subject to swelling or shrink
- In collapsible soils

PILE FOUNDATIONS



PILE FOUNDATIONS





End-bearing piles

Used to transfer load through the pile tip to a suitable bearing stratum, passing soft soil or water.

Friction piles

Used to transfer loads to a depth in a frictional material by means of skin friction along the surface area of the pile.

Tension or uplift piles

Used to anchor structures subjected to uplift due to hydrostatic pressure or to overturning moment due to horizontal forces.

Compaction piles

Used to compact loose granular soils in order to increase the bearing capacity. Since they are not required to carry any load, the material may not be required to be strong; in fact, sand may be used to form the pile. The pile tube, driven to compact the soil, is gradually taken out and sand is filled in its place thus forming a 'sand pile'.

Anchor piles

Used to provide anchorage against horizontal pull from sheetpiling or water.

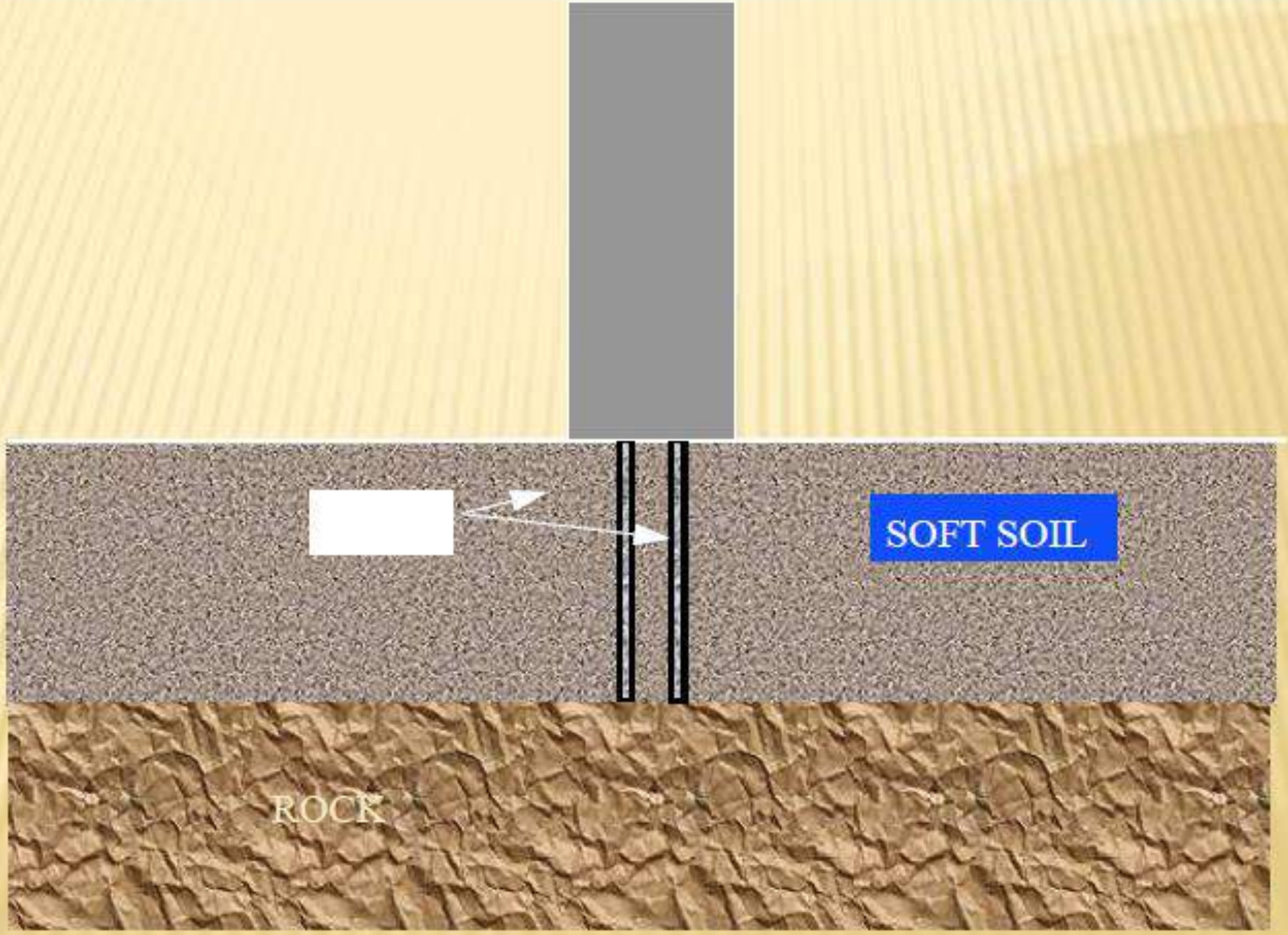
Fender piles

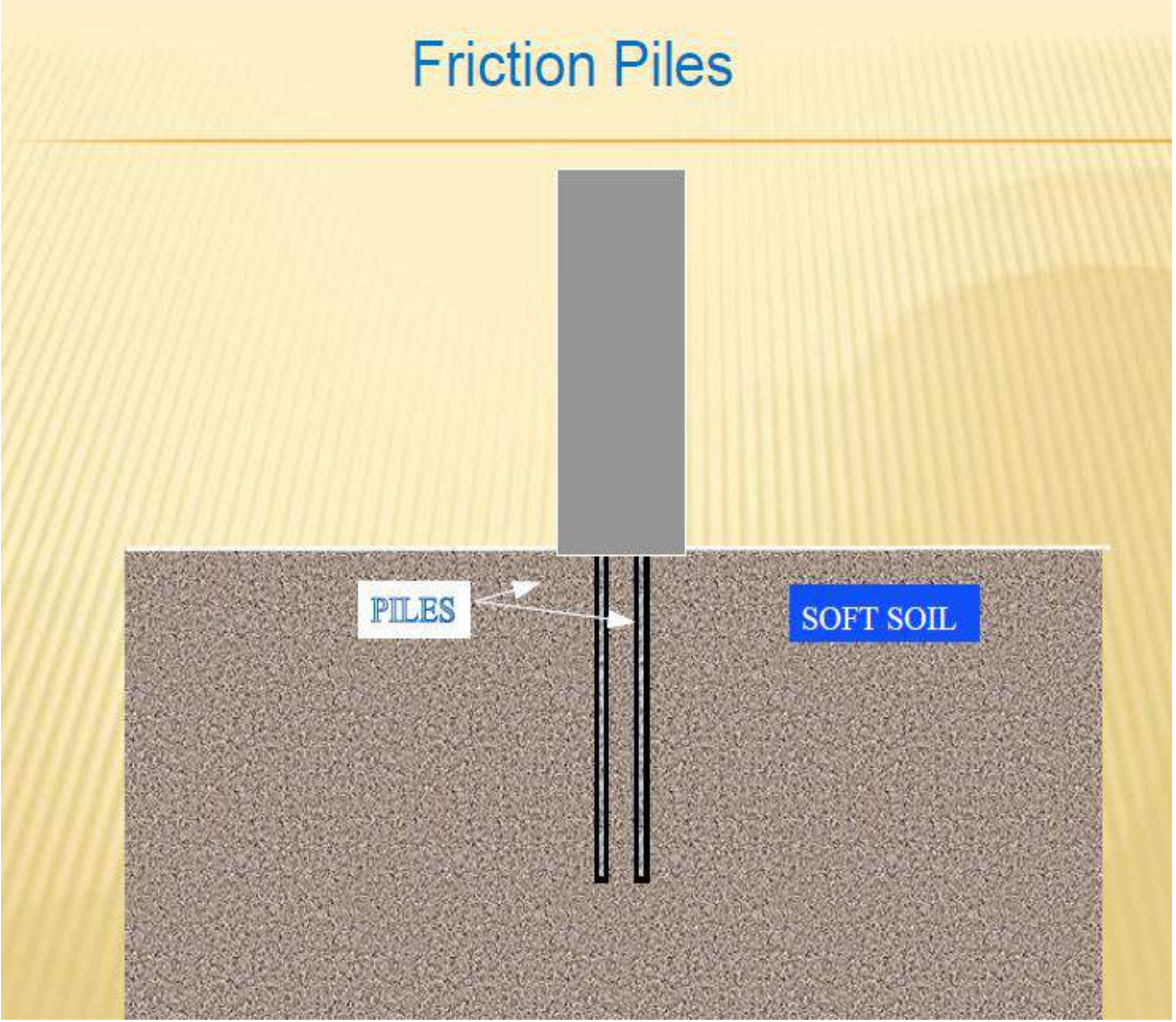
Used to protect water-front structures against impact from ships or other floating objects.

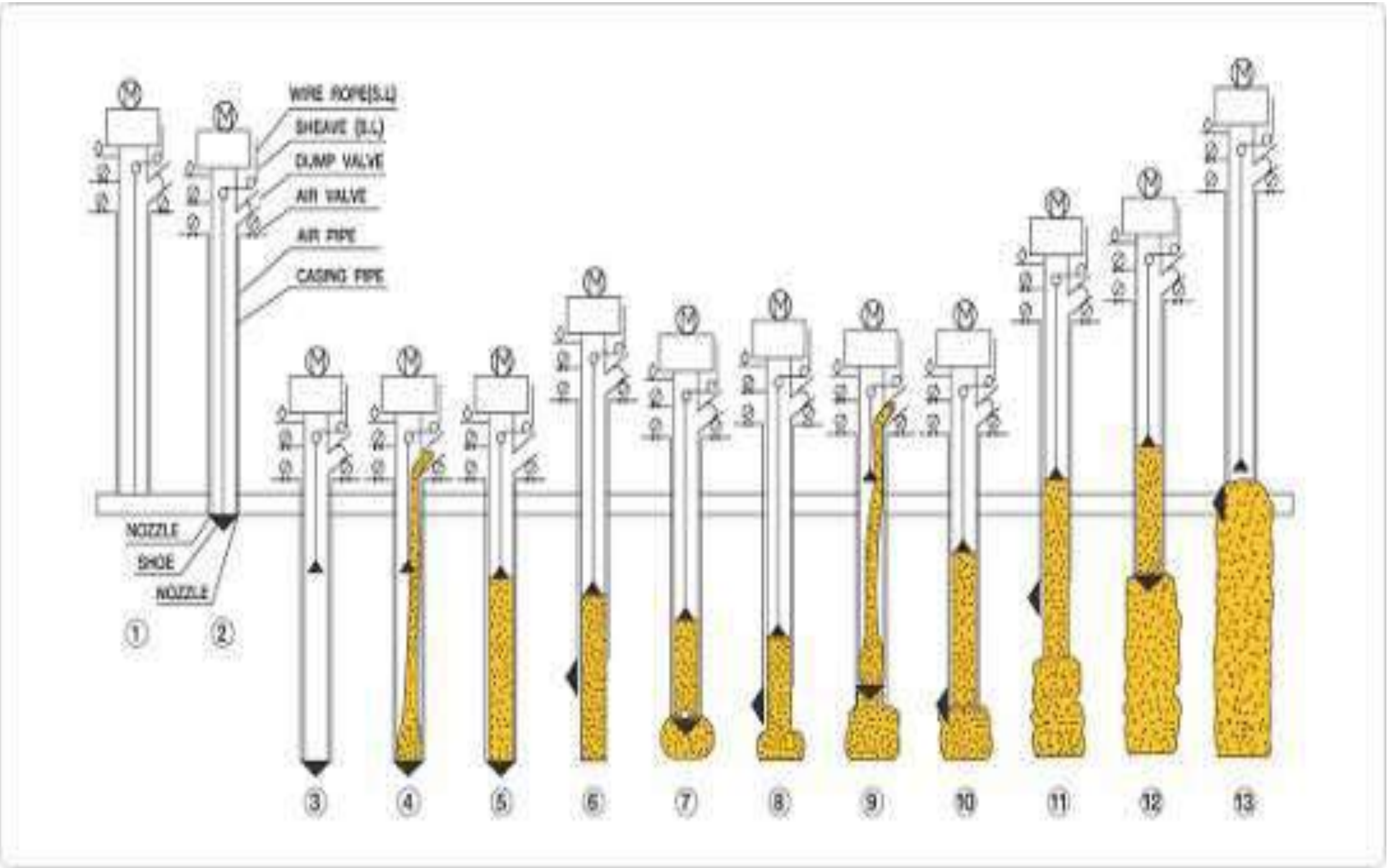
Batter piles

Used to resist horizontal and inclined forces, especially in water front structures.

End Bearing Piles




Friction Piles
PILES
SOFT SOIL



Compaction Piles

Fender Piles


STEEL SHEET PILING

Batter Piles

16.2.2 Classification Based on Material and Composition

Piles may be classified as follows based on material and composition:

Timber piles

These are made of timber of sound quality. Length may be up to about 8 m; splicing is adopted for greater lengths. Diameter may be from 30 to 40 cm. Timber piles perform well either in fully dry condition or submerged condition. Alternate wet and dry conditions reduce the life of a timber pile; to overcome this, creosoting is adopted. Maximum design load is about 250 kN.

Steel piles

These are usually H-piles (rolled H-shape), pipe piles, or sheet piles (rolled sections of regular shapes). They may carry loads up to 1000 kN or more.

Concrete piles

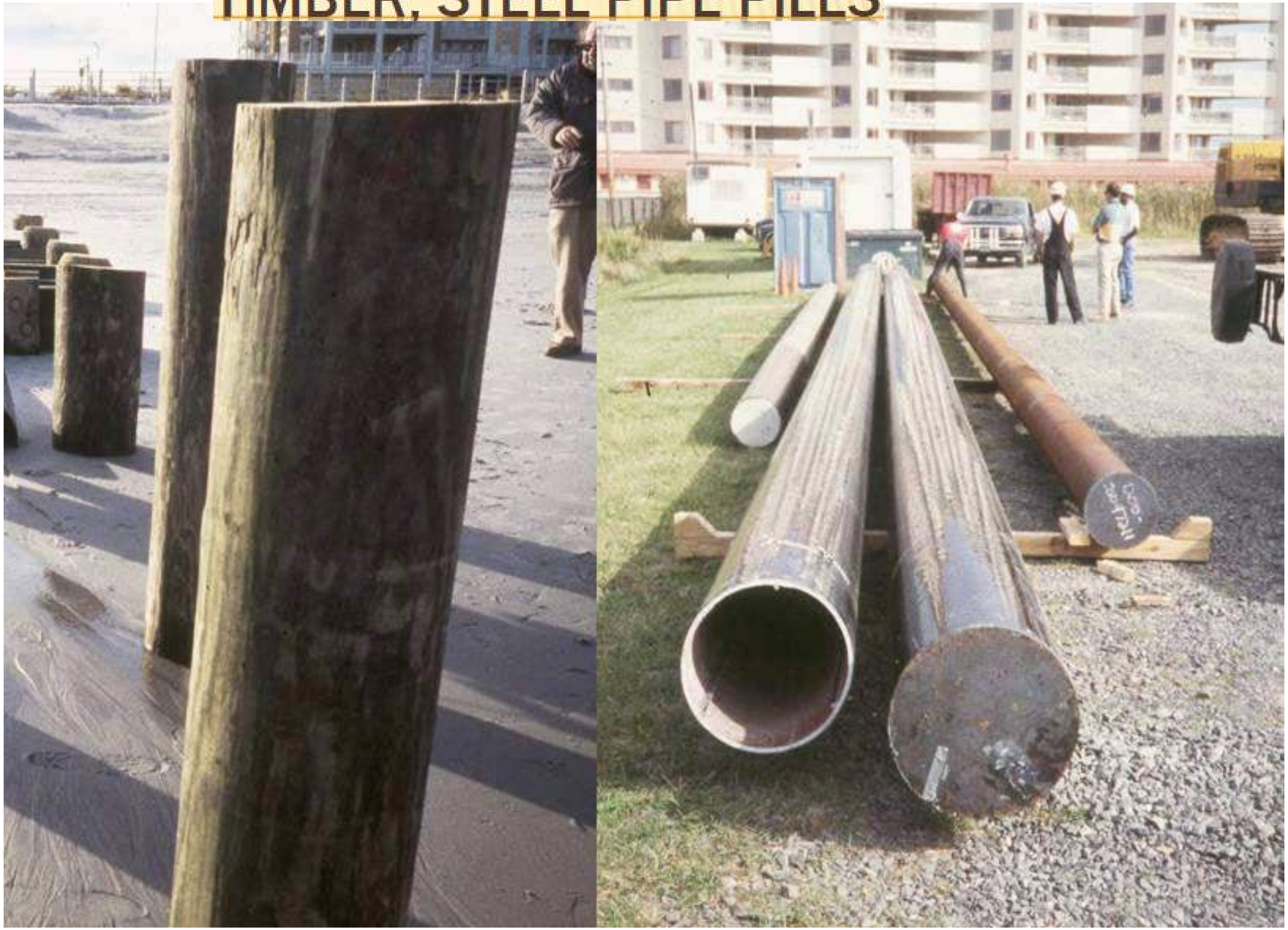
These may be 'precast' or 'cast-in-situ'. Precast piles are reinforced to withstand handling stresses. They require space for casting and storage, more time to cure and heavy equipment for handling and driving.

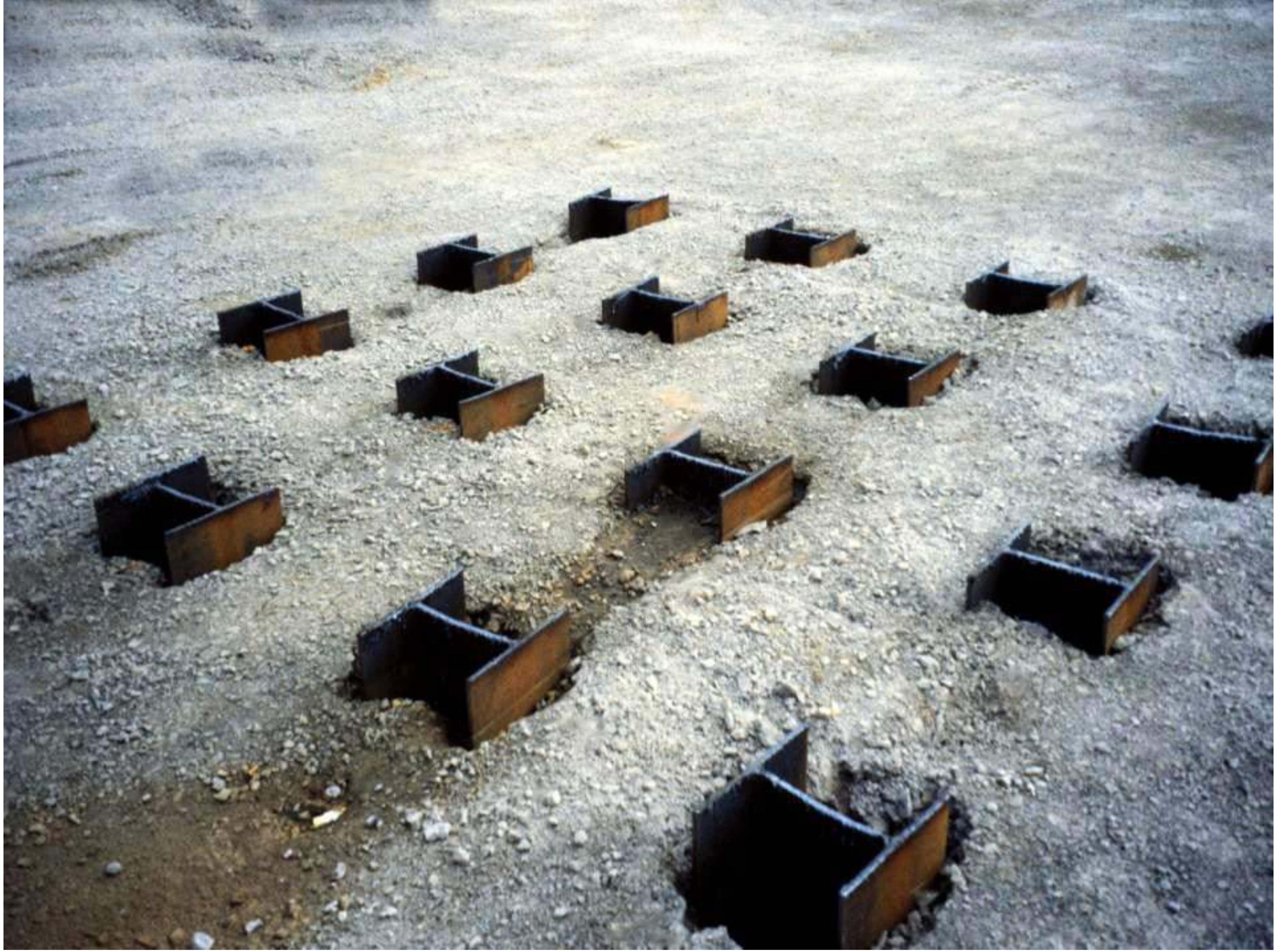
Cast-in-situ piles are installed by pre-excavation, thus eliminating vibration due to driving and handling. The common types are Raymond pile, Mac Arthur pile and Franki pile.

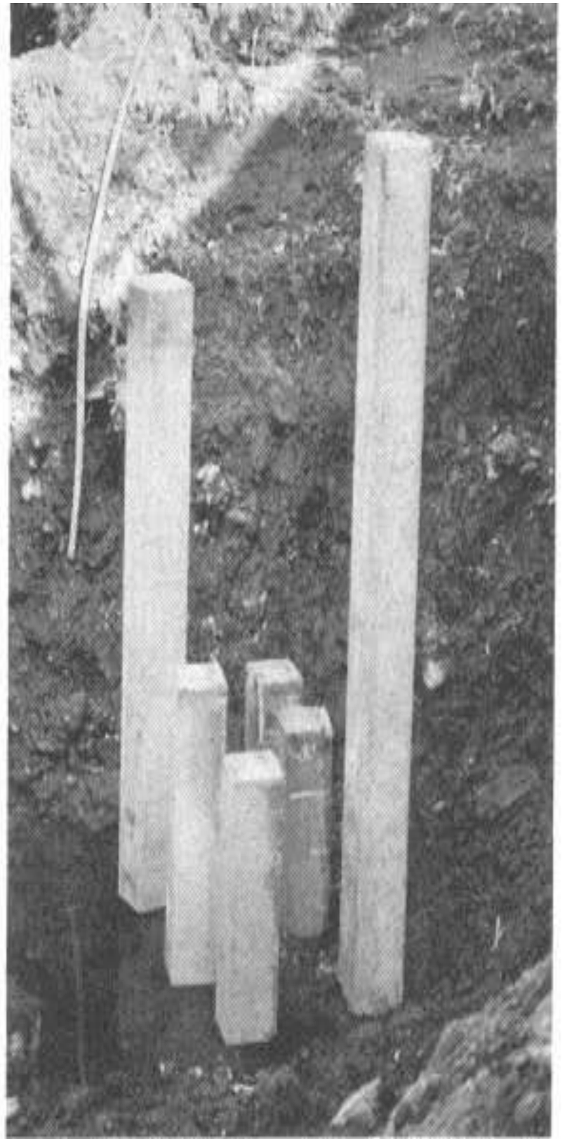
Composite piles

These may be made of either concrete and timber or concrete and steel. These are considered suitable when the upper part of the pile is to project above the water table. Lower portion may be of untreated timber and the upper portion of concrete. Otherwise, the lower portion may be of steel and the upper one of concrete.

TIMBER, STEEL PIPE PILES







16.2.3 Classification Based on Method of Installation

Piles may also be classified as follows based on the method of installation:

Driven piles

Timber, steel, or precast concrete piles may be driven into position either vertically or at an inclination. If inclined they are termed 'batter' or 'raking' piles. Pile hammers and pile-driving equipment are used for driving piles.

Cast-in-situ piles

Only concrete piles can be cast-in-situ. Holes are drilled and these are filled with concrete. These may be straight-bored piles or may be 'under-reamed' with one or more bulbs at intervals. Reinforcements may be used according to the requirements.

Driven and cast-in-situ piles

This is a combination of both types. Casing or shell may be used. The Franki pile falls in this category.

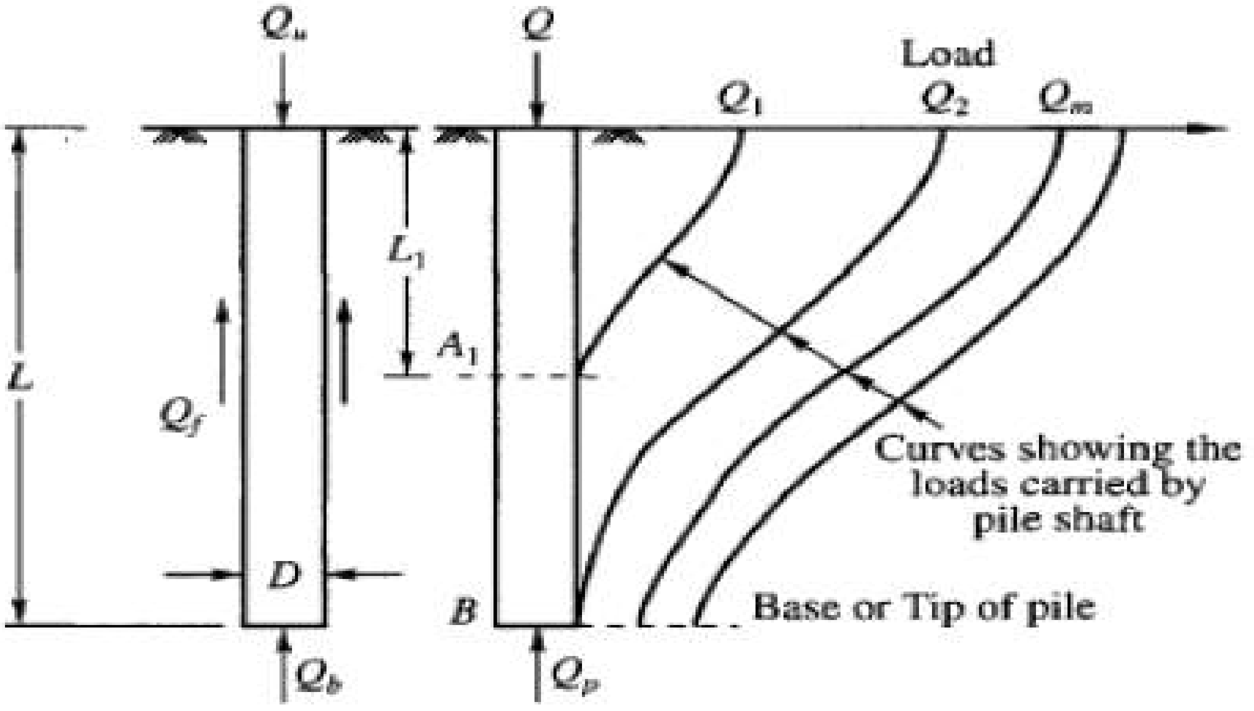

Pile Lifting


Pile Positioning
KELPILE

Load Transfer Mechanism

- Fig. gives a single pile of uniform diameter *d (circular or any other shape)* and length L driven into a homogeneous mass of soil of known physical properties. A static vertical load is

  applied on the top. It is required to determine the ultimate bearing capacity Q_u of the pile.



(a) Single pile (b) Load-transfer curves

LOAD CARRYING CAPACITY OF PILE

- The following is the classification of the methods of determining pile capacity:

(i) Static analysis

(*ii*) Dynamic analysis

(*iii*) Load tests on pile

(*iv*) Penetration tests

Static Analysis

- The ultimate bearing load of a pile is considered to be the sum of the end-bearing resistance and the resistance due to skin friction:

$$Q_{up} = Q_{eb} + Q_{sf}$$

Q_{up} = ultimate bearing load of the pile,
Q_{eb} = end-bearing resistance of the pile, and
Q_{sf} = skin-friction resistance of the pile.

$$Q_{eb} = q_b \cdot A_b$$
$$Q_{sf} = f_s A_s$$

q_b = bearing capacity in point-bearing for the pile,
f_s = unit skin friction for the pile-soil system,
A_b = bearing area of the base of the pile, and
A_s = surface area of the pile in contact with the soil.

Piles in sand

$$q_b = q \cdot N_q$$

q = γ*Depth

$q = \gamma.z$ if $Z < Z_c$, and
$q = \gamma.Z_c$ if $Z > Z_c$

Critical depth can be taken as

10 * B- loose sands

20 * B- dense sands

Z being the embedded length of pile and Z_c the critical depth.

B is the diameter of the pile

The general form for the unit skin friction resistance, f_s, is given by

$$f_s = c_a + \sigma_h \tan \delta$$

c_a = adhesion

σ_h = average lateral pressure of soil against the pile surface;

δ = angle of wall friction

For piles in sands:

$f_s = \sigma_h \tan \delta$, c_a being zero.

σ_h = K * σ_v [K = coefficient of earth pressure]

Average vertical stress/ surcharge is considered for analysis

$$fs = K * \frac{\sigma v}{2} * \tan\delta$$

Piles in Clay

$$q_b = cN_c$$

N_c = bearing capacity factor for deep foundation

9 - commonly used for piles

For piles in clays:

$$f_s = c_a, \text{ since } \tan\delta \text{ is zero.}$$

The adhesion c_a may be expressed as

$$c_a = \alpha \cdot c$$

where α is called the 'adhesion factor',

Ultimate load (Q_u) = $\mathbf{c * N_c * A_b + \alpha * c * A_s}$

Example 16.3: A pile is driven in a uniform clay of large depth. The clay has an unconfined compression strength of 90 kN/m². The pile is 30 cm diameter and 6 m long. Determine the safe frictional resistance of the pile, assuming a factor of safety of 3. Assume the adhesion factor $\alpha = 0.7$.

Cohesion of clay $= \frac{1}{2} \times 90 = 45 \text{ kN/m}^2$

Frictional resistance $= \alpha \, . \, cA_s$

$= 0.7 \times 45 \times \pi \times 0.3 \times 6 \text{ kN} = 178.13 \text{ kN}$

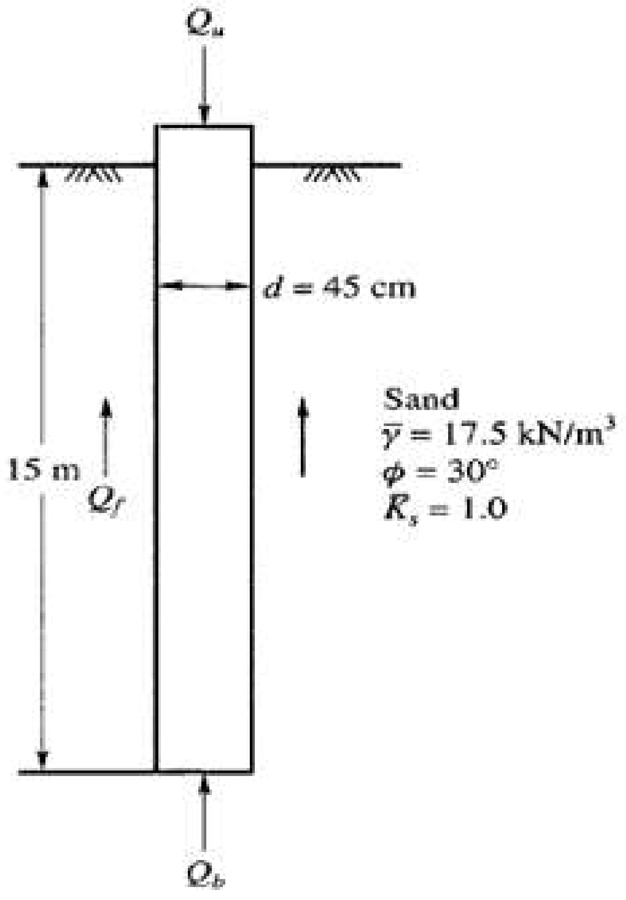
$\therefore$ Safe frictional resistance $= \frac{178.13}{3} \approx$ **59.3 kN.**

A concrete pile of 45 cm diameter was driven into sand of loose to medium density to a depth of 15 m. The following properties are known: **Neglect critical depth factor**

(a) Average unit weight of soil along the length of the pile, $\bar{\gamma} = 17.5$ kN/m^3, average $\phi = 30°$,

(b) average $\overline{K}_s = 1.0$ and $\delta = 0.75\phi$. Calculate (a) the ultimate bearing capacity of the pile,

N_q is equal to 16.5.



$$Q_u = Q_b + Q_f = q_0' A_b N_q + \overline{q}_o' A_s \overline{K}_s \tan \delta$$

where $q_0' = \overline{\gamma} L = 17.5 \times 15 = 262.5 \text{ kN/m}^2$

$$\overline{q}_o' = \frac{1}{2} \overline{\gamma} L = \frac{262.5}{2} = 131.25 \text{ kN/m}^2$$

$$A_b = \frac{3.14}{4} \times 0.45^2 = 0.159 \text{ m}^2$$

$$A_s = 3.14 \times 0.45 \times 15 = 21.195 \text{ m}^2$$

$$\delta = 0.75\phi = 0.75 \times 30 = 22.5°$$

$$\tan \delta = 0.4142$$

Substituting the known values, we have

$$Q_u = Q_b + Q_f = 262.5 \times 0.159 \times 16.5 + 131.25 \times 21.195 \times 1.0 \times 0.4142$$

$$= 689 + 1152 = 1841 \text{ kN}$$

DYNAMIC FORMULAE

- **Engineering News Record Formulae**

$$Q_u = \frac{(W * h * \eta_h)}{(S + C)}$$

- S = penetration of pile per hammer blow; obtained from the average for the last few blows of the hammer
- C = constant
  - 2.54 cm- drop hammer
  - 0.254 cm- steam hammer
- $Q_u = \frac{(En * \eta_h)}{(S + C)}$ E_n = energy of hammer in kN-cm

- **Engineering News Record Formulae**

- Efficiency of drop hammer- 0.7 to 0.9
  - Single acting - 0.75
  - Double acting- 0.85
  - Diesel hammer- 0.80 to 0.90

- Factor of safety- 6 (recommended)

- Formula not dependable

Modified Engineering News Record Formulae

$$Q_u = \frac{Wh\eta_h}{S + C}\left(\frac{W + e^2P}{W + P}\right)$$

- P = weight of pile; e = coefficient of restitution; η_h= hammer efficiency
- Hammer efficiency dependent on
  - Pile driving equipment
  - Driving procedure
  - Ground conditions

Hammer Type	Drop	Single Acting	Double Acting	Diesel
Efficiency	0.75 – 1	0.75 - 0.85	0.85	0.85 - 1

Coefficient of Restitution (e)

Type of Pile	**Coefficient of Restitution**
Broomed timber pile	0.0
Good timber pile	0.25
Driving cap with timber dolly on steel pile	0.3
Driving cap with plastic dolly on steel pile	0.5
Helmet with composite plastic dolly and packing on R.C. C. Pile	0.4

- **Hiley's Formulae**
- Takes into account the various losses

$$Q_f = \frac{\eta_h W H \eta_b}{S + \frac{C}{2}}$$

- Q_f = ultimate load on pile
- W = weight of hammer, in kg
- H = height of drop of hammer, in cm
- S = penetration or set, in cm per blow
- C = total elastic compression = $C_1+C_2+C_3$

DYNAMIC FORMULAE

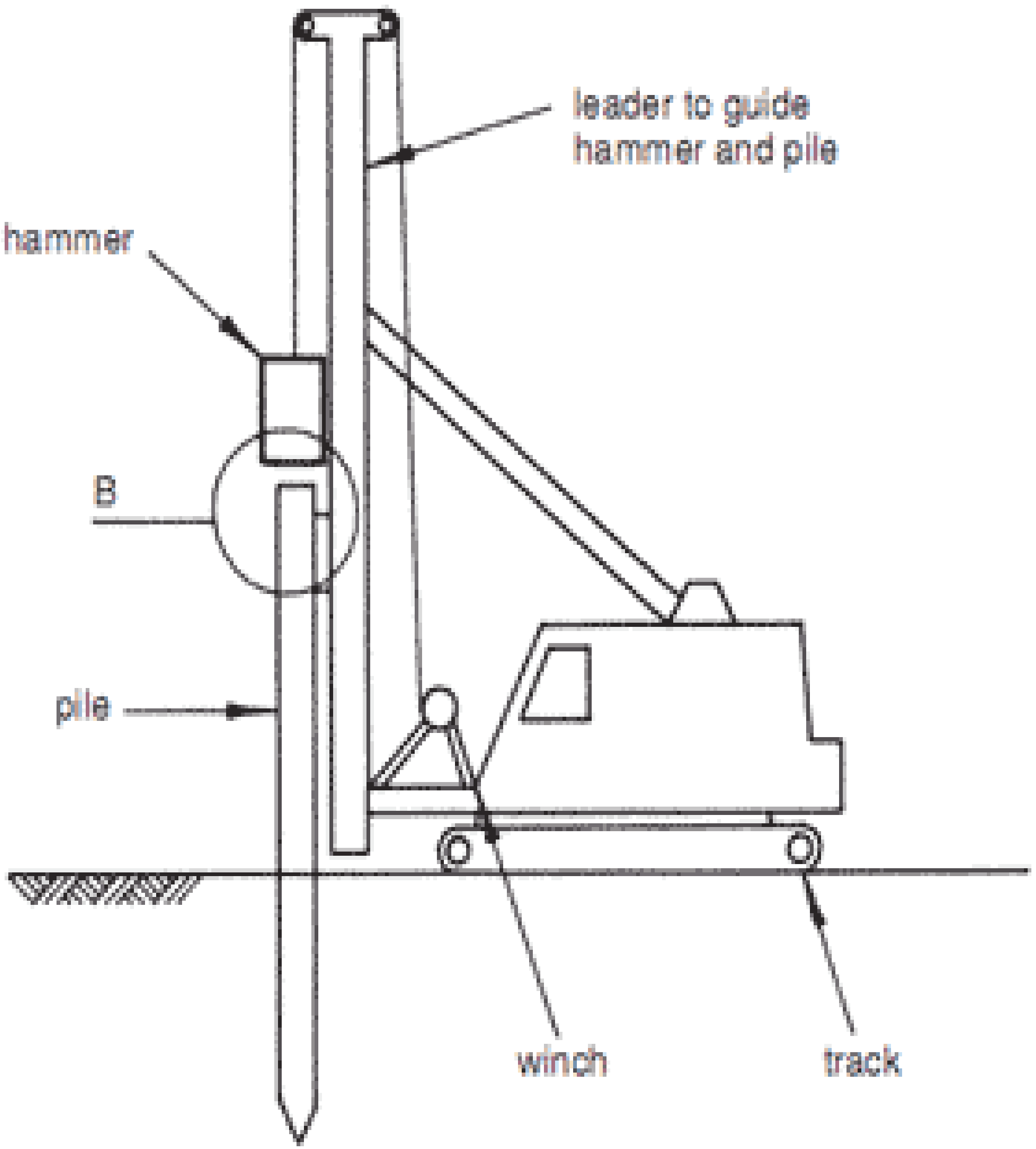
- C_1, C_2, C_3 = temporary elastic compression of dolly and packing, pile and soil respectively
- η_h = efficiency of hammer
- η_b = efficiency of hammer blow

$$= \frac{W + e^2P}{W + P} \quad (if\ W > eP)$$

$$= \frac{W + e^2P}{W + P} - \left\{\frac{W - eP}{W + P}\right\}^2 \quad (W < eP)$$

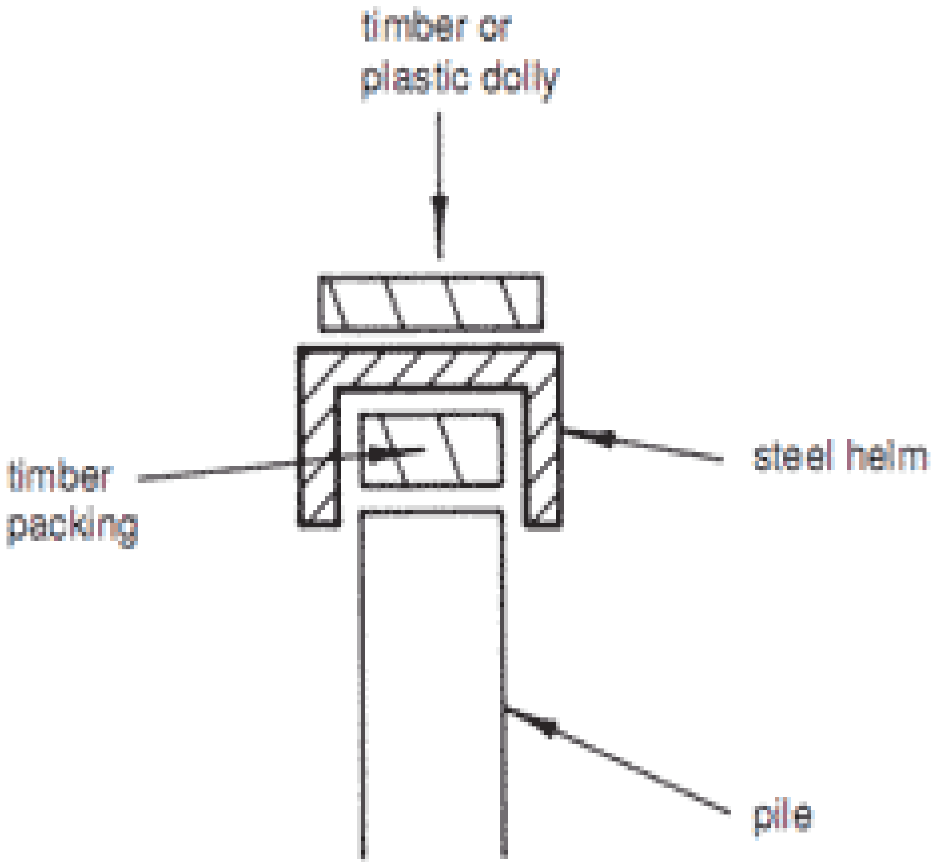
P = weight of pile

e = coefficient of restitution



piling frame

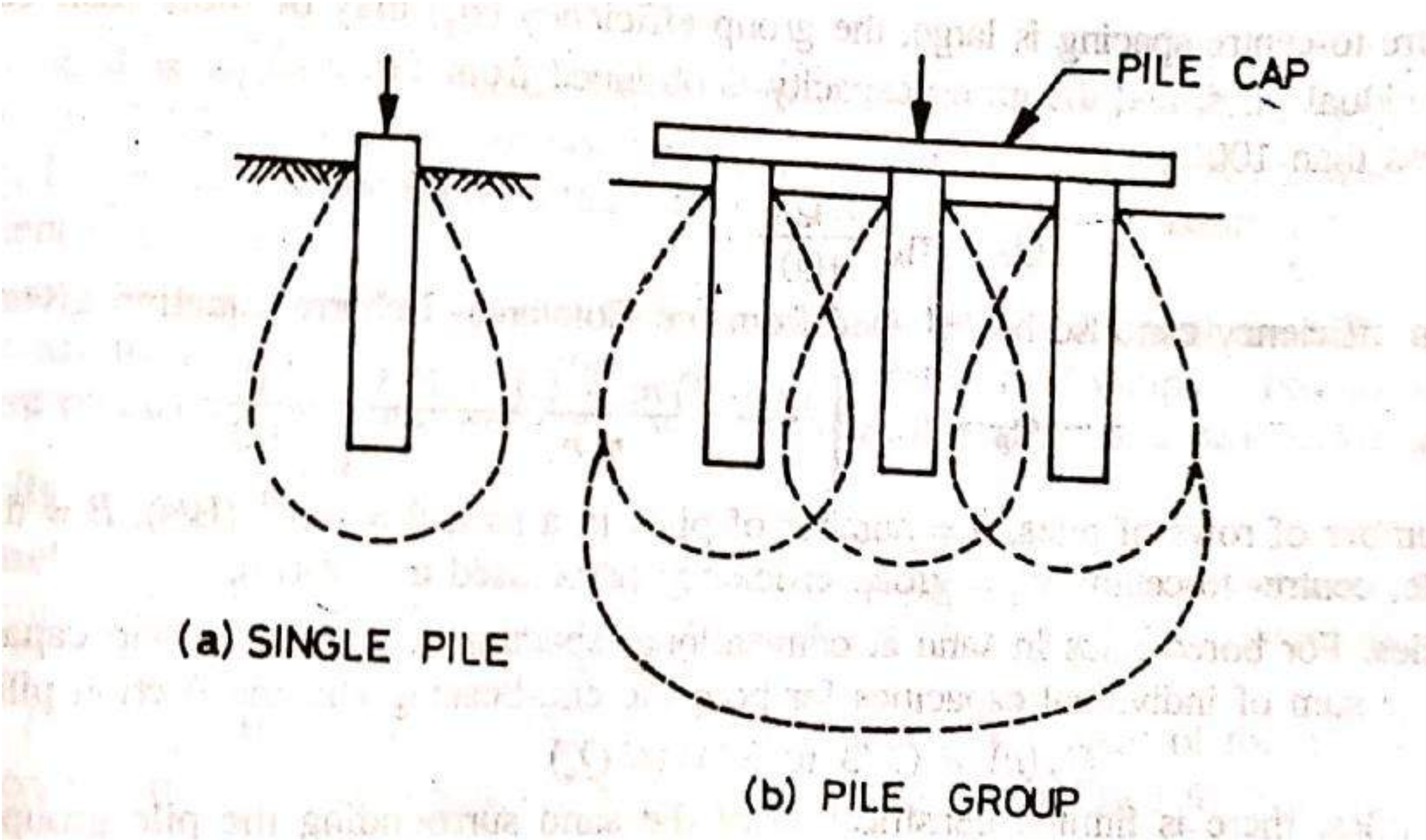
(a)



detail at B

(b)

Group Capacity of piles



25.18. GROUP ACTION OF PILES

A pile is not used singularly beneath a column or a wall, because it is extremely difficult to drive the pile absolutely vertical and to place the foundation exactly over its centre line. If eccentric loading results, the connection between the pile and the column may break or the pile may fail structurally because of bending stresses. In actual practice, structural loads are supported by several piles acting as a group. For columns, a minimum of three piles in a triangular pattern are used. For walls, piles are installed in a staggered arrangement on both sides of its centre line. The loads are usually transferred to the pile group through a reinforced concrete slab, structurally tied to the pile tops such that the piles act as one unit. The slab is known as a *pile cap.* The load acts on the pile cap which distributes the load to the piles (Fig. 25.15).

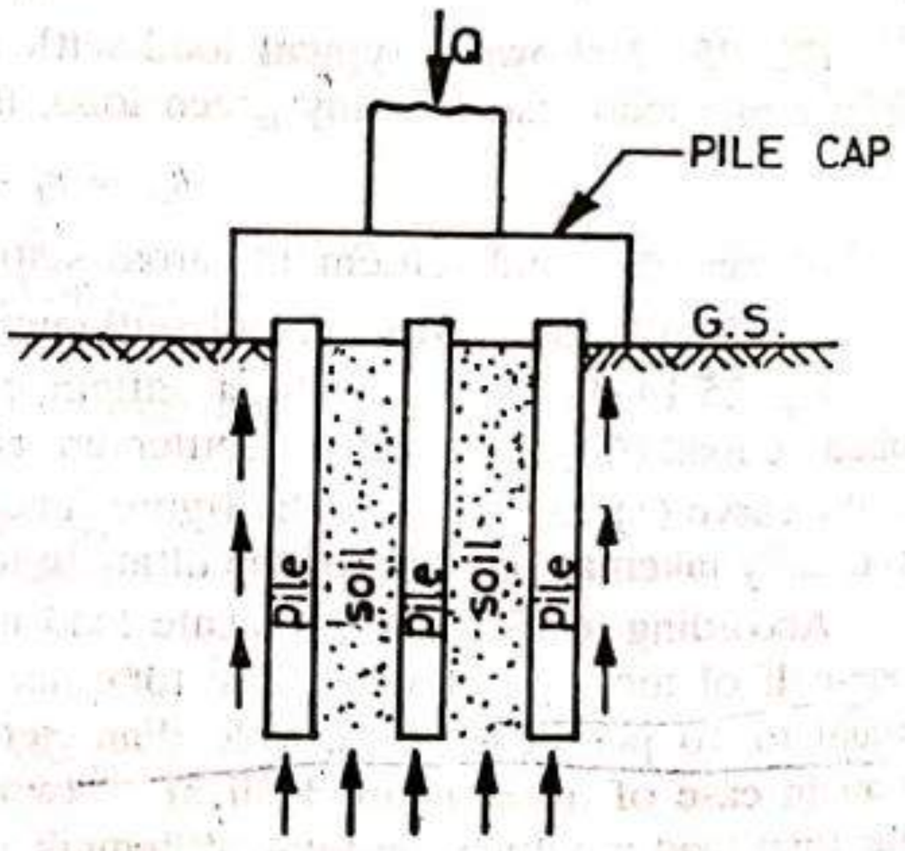


Fig. 25.15.

The load carrying capacity of a pile group is not necessarily equal to the sum of the capacity of the individual piles. Estimation of the load-carrying capacity of a pile-group is a complicated problem. When the piles are spaced a sufficient distance apart, the group capacity may approach the sum of the individual capacities. On the other hand, if the piles are closely spaced, the stresses transmitted by the piles to the soil may overlap, and this may reduce the load-carrying capacity of the piles (Fig. 25.16). For such a case, the capacity is limited by the group action.

The efficiency (η_g) of a group of piles is defined as the ratio of the ultimate load of the group to the sum

The efficiency (η_g) of a group of piles is defined as the ratio of the ultimate load of the group to the sum of individual ultimate loads.

Thus
$$\eta_g = \frac{Q_{g(u)}}{NQ_u} \times 100 \qquad ...(25.38)$$

25.20. PILE GROUPS IN CLAY

As the pile group acts as a block, its ultimate capacity is determined by adding the base resistance an the shaft resistance of the block. The capacity of the block having closely spaced piles ($s \leq 3B$) is ofte limited by the behaviour of the group acting as a block. The group capacity of the block is given by

or $$Q_g(u) = CN_cA_B + \alpha C \times A_{sB}$$

$C = Cohesion, A_B = Area\ of\ the\ block, A_{sB} = Surface\ area of\ block$

α = adhesion factor (= 1.0 for soft clays), c = undrained cohesion.

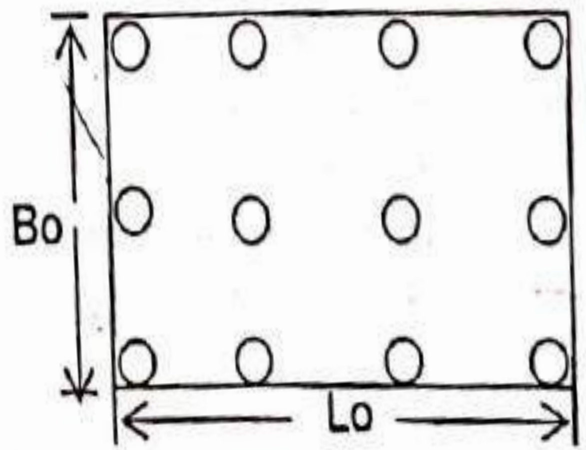
As discussed earlier, the individual pile capacity is given by Eq. 25.15,

Ultimate load (Q_u) = $\mathbf{c * N_c * A_b + \alpha * c * A_s}$

The group capacity considering the piles as individual piles is given by

$$Q_g(u) = NQ_u$$

The lower of the two values, given by is the actual capacity.



A_B = Block area, $L_0 \times B_0$

$B_0 = 2s + d$; $L_0 = 3s + d$

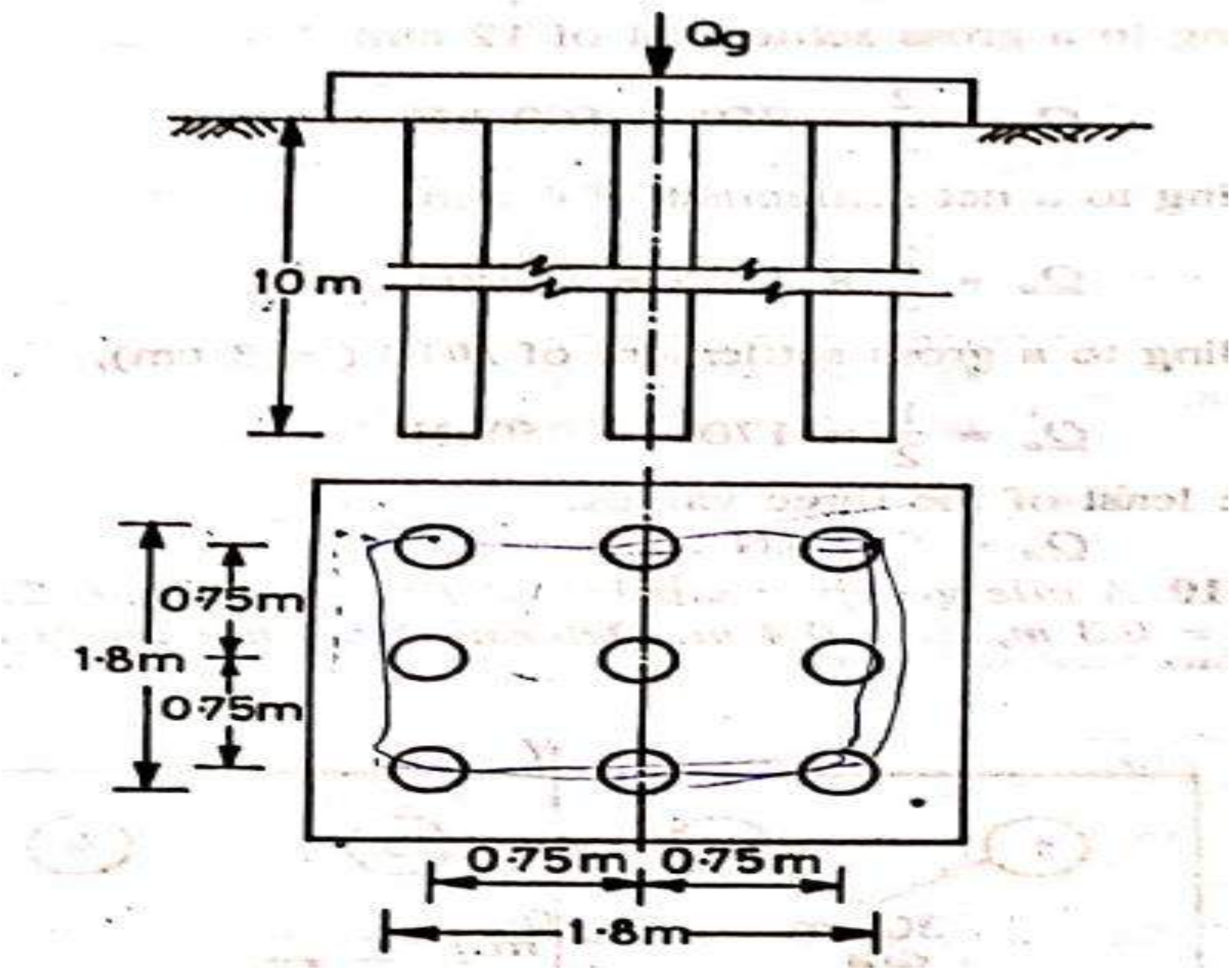
s = c/c spacing of piles d = diameter of piles

Surface area of block
$= 2\,(L_0 + B_0)\,L$

Illustrative Example 25.11. *A pile group consists of 9 friction piles of 30 cm diameter and 10 m length driven in clay (c_u = 100 kN/m^2, γ = 20 kN/m^3), as shown in Fig. E-25.11. Determine the safe load for the group (FS = 3, α = 0.6).*

$$Q_g(u) = CN_cA_B + \alpha C \times A_{sB}$$

C = Cohesion, A_B = Area of the block, A_{sB} = Surface area of block



Qg
10 m
0·75m
1·8m
0·75m
0·75m
0·75m
1·8m

$$= (9 \times 100)(1.8 \times 1.8) + 0.6 \times 100 \times (4 \times 1.8 \times 10)$$

or $$Q_g(u) = 7236 \text{ kN}$$

From Eq. 25.45, Ultimate load (Q_u) = $c * N_c * A_b + \alpha * c * A_s$

$$= (9 \times 100) \times \pi/4 \times (0.3)^2 + 0.6 \times 100 (\pi \times 0.3) \times 10$$

or $$Q_u = 628.8 \text{ kN}$$

From Eq. 25.46, $$Q_g(u) = NQ_u$$

$$= 9 \times 628.8 = 5659.2 \text{ kN}$$

As the ultimate load for individual pile failure is less than the pile group load, the safe load is given by

$$Q_a = \frac{5659.2}{3} = 1886.4 \text{ kN}$$

... and 1 m

Example 16.4: A group of 16 piles of 50 cm diameter is arranged with a centre to centre spacing of 1.0 m. The piles are 9 m long and are embedded in soft clay with cohesion 30 kN/m^2. Bearing resistance may be neglected for the piles—Adhesion factor is 0.6. Determine the ultimate load capacity of the pile group.

$n = 16 \quad d = 50\text{ cm} \quad L = 9\text{ m} \quad s = 1.5\text{ m}$

Width of group, $B = (1 \times 3 + 0.50) = 3.5\text{ m}$

For Individual Piles $\text{Ultimate load } (Q_u) = \mathbf{c * N_c * A_b + \alpha * c * A_s}$

Since bearing resistance is neglected Qu= $0.6 \times 30 \times \pi \times 0.5 \times 9$

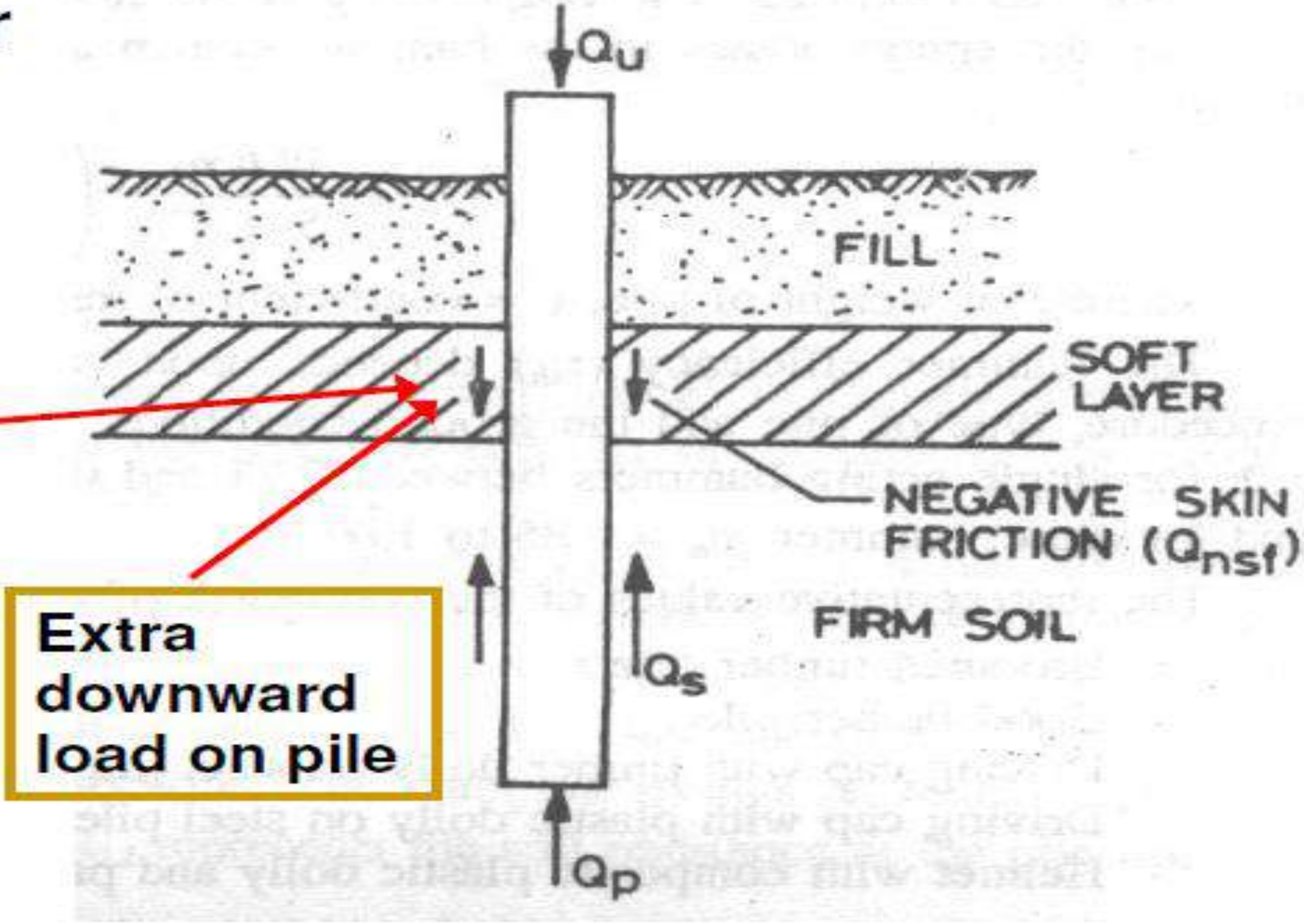
Individual capacity of pile

$$For\ pile\ group \quad Q_g(u) = \boldsymbol{\alpha C * A_{sB} = 0.6 \times 30 \times 4B \times 9}$$

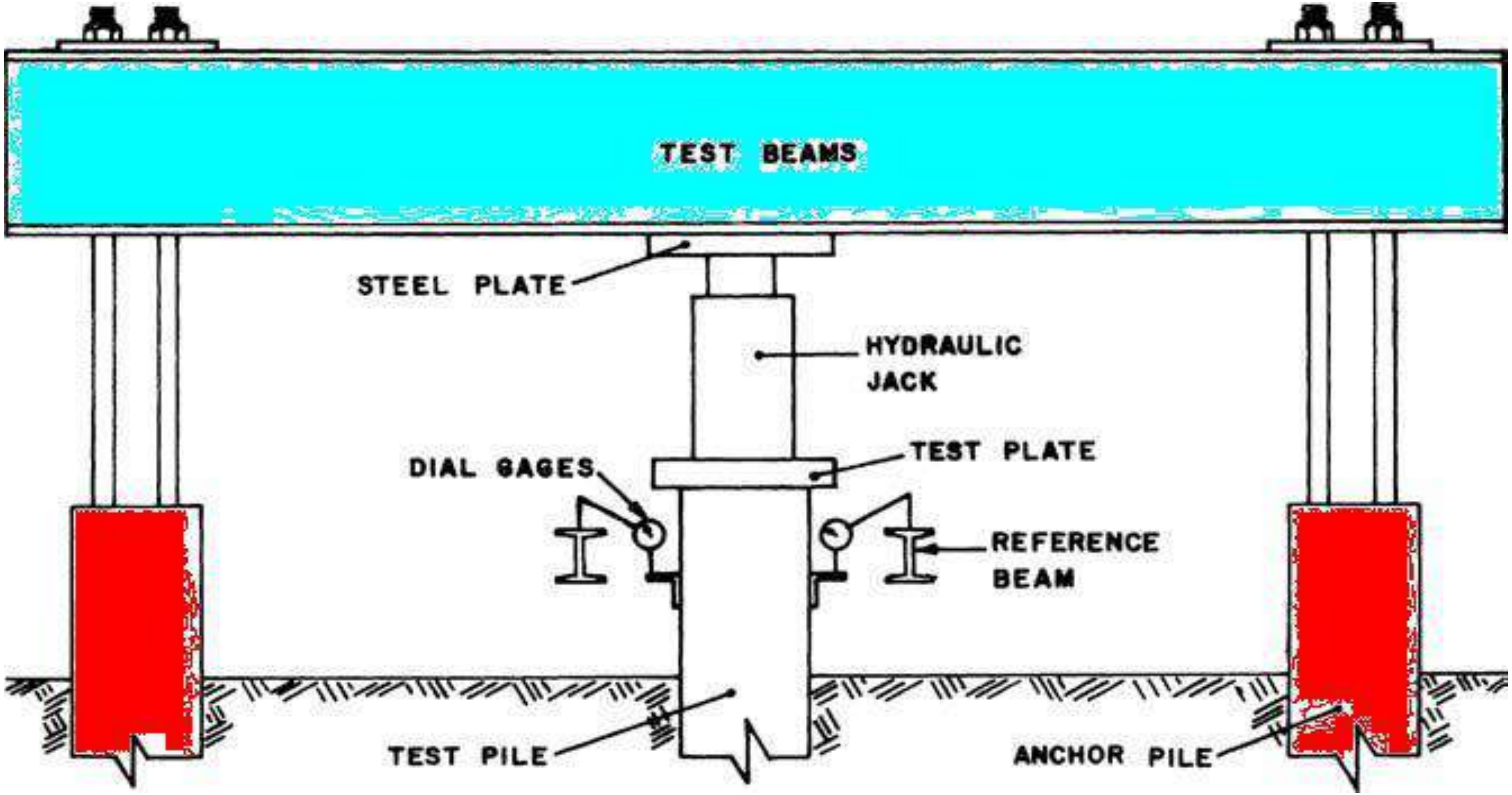
Group capacity of pile

Negative Skin Friction

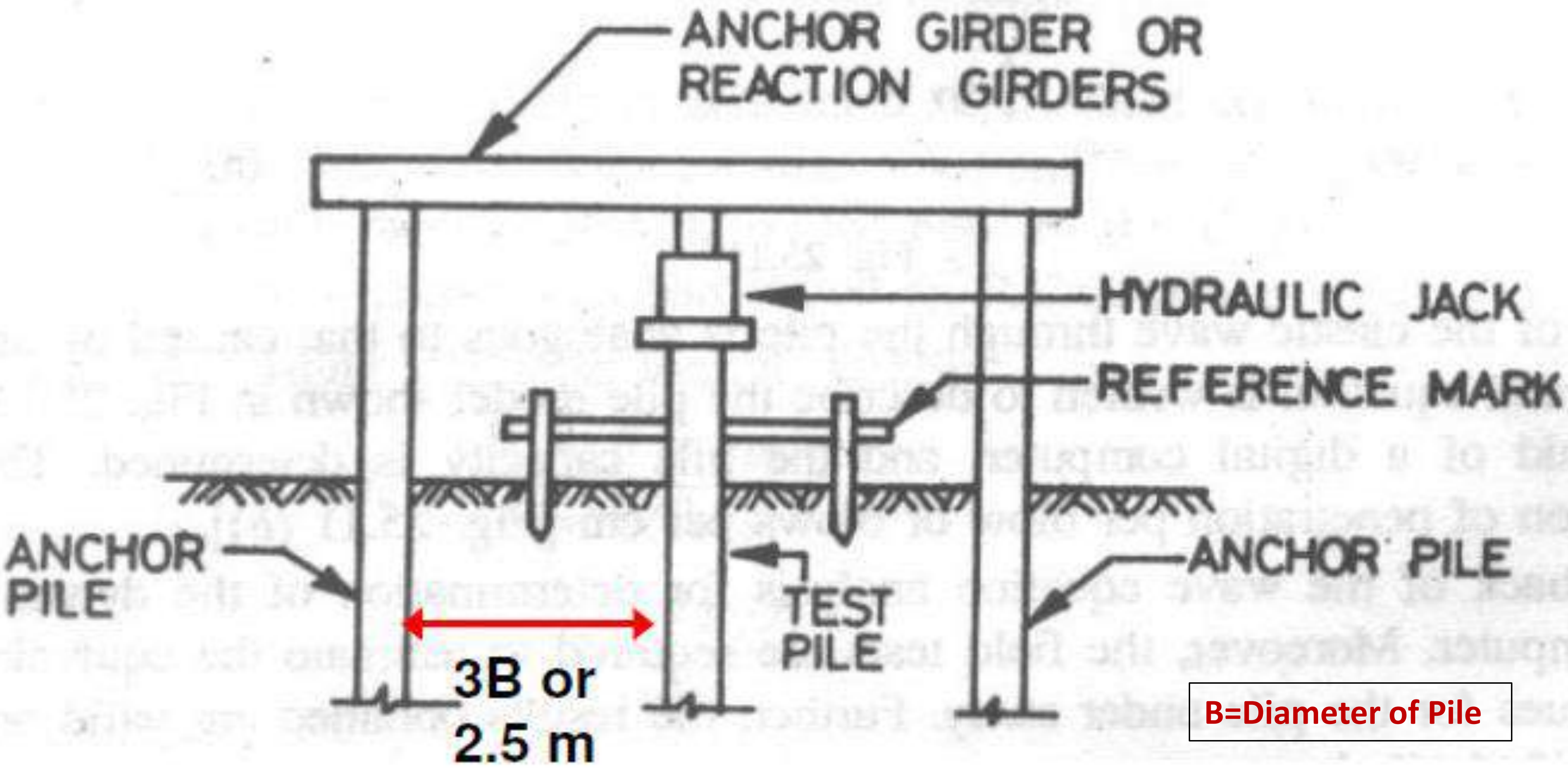
- When soil layer surrounding the portion of the pile shaft settles more than pile
- A downward drag occurs on the pile
- The drag is known as negative skin friction



Pile Load test



Pile Load Test Setup



LOAD TEST BY CONCRETE BLOCKS 1000 MT

LOAD TEST BY SAND BAGS

When pile load test shall be conducted

- 3 days after installation in sandy soils

- One month after installation in silts and clays (soft clays)

How shall be the application of load

- Equal increments of about 20% of the allowable load (Load at which pile will be tested)

- (Test load= Twice the safe load or the load at which the total settlement reaches the specified value

How to record settlement

- At least with three dial gauges

When to increase/change the load

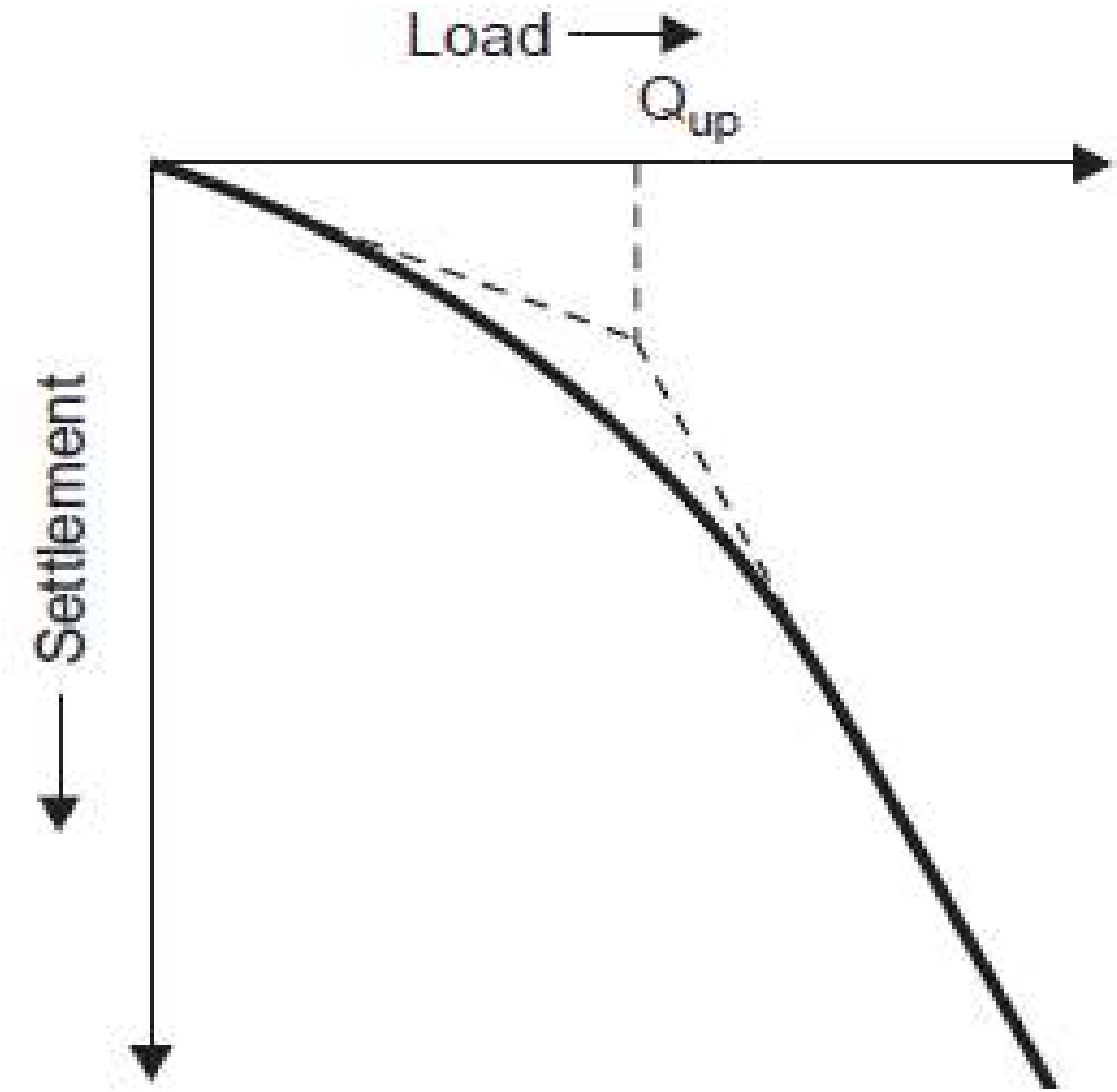
Rate of movement of the pile top is not more than

- 0.1 mm/hour for sandy soils
- 0.02 mm/hour for clayey soils
- Two hours maximum

Time interval to observe settlement

- 0.5 min, 1 min, 2 min, 4 min, 8 min, 15 min, 30 min, 1 hour and 2 hours

- The last stage of loading will be continued for 24 hours or even more depending upon the settlement



Determination of ultimate load from load-settlement curve for a pile

Safe load?? The least of below

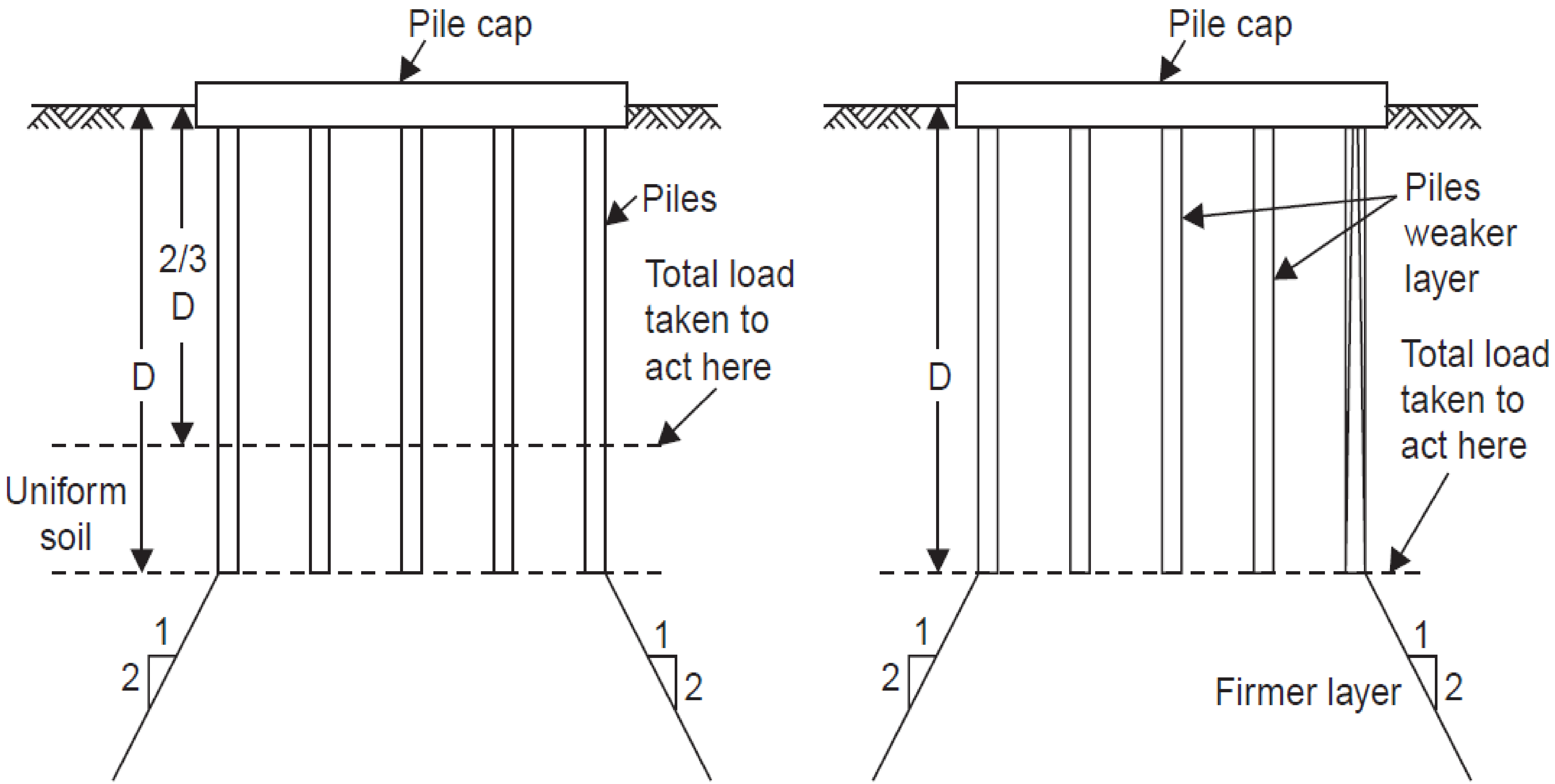
- 2/3 of the final load at which total settlement is 12 mm

- 2/3 of the load corresponding to a net settlement of 6 mm

- 1/2 of load corresponding to a total settlement of (B/10) (7.5% incase of under-reamed pile)

Settlement of pile groups in clay

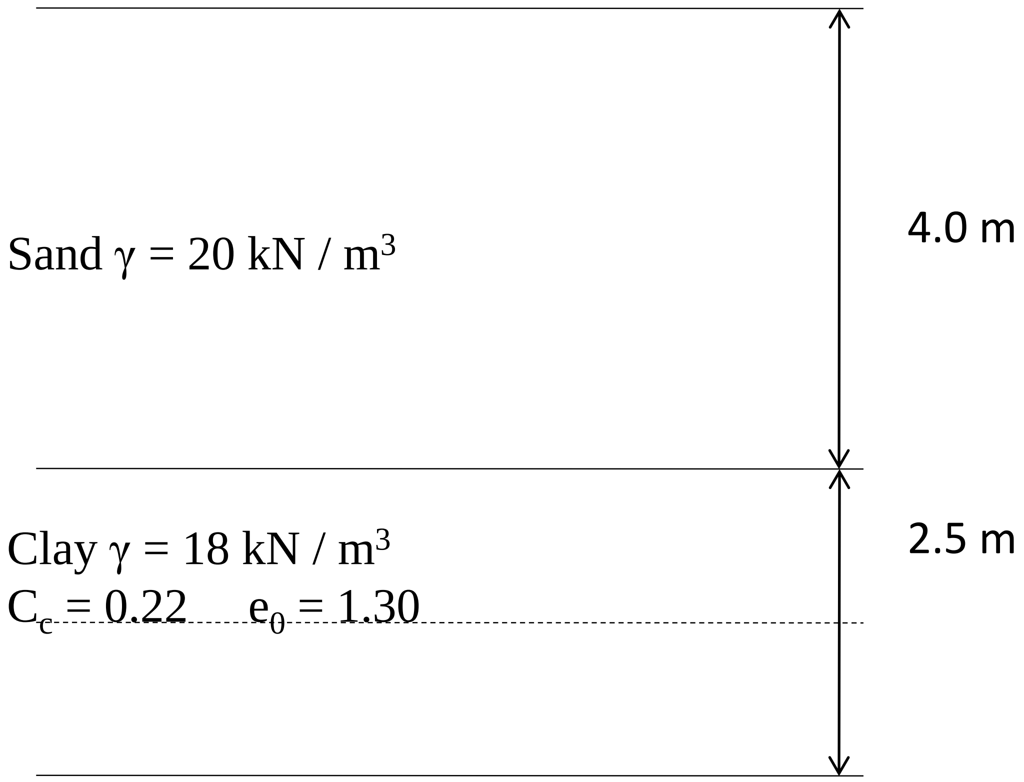
- The equation for consolidation settlement may be used treating the pile group as a block or unit.
- The increase in stress is to be evaluated appropriately under the influence of the load on the pile group.
- When the piles are embedded in a uniform soil (friction and end-bearing piles), the total load is assumed to act at a depth equal to two-thirds the pile length.
- Conventional settlement analysis procedures assuming the Boussinesq or Westergaard stress distribution are then applied to compute the consolidation settlement of the soil beneath the pile tip.



(a) Friction and end-bearing piles in uniform soil

(b) Pile tips in firm soil (end-bearing piles)

- Calculate the final settlement of the clay layer shown below due to an increase in pressure of 30 kN / m^2 at the mid-height of the layer. Take γ_w = 10 kN / m^3.



$\gamma_w = 10$ kN / m^3; $\gamma_{sand} = 20$ kN / m^3; $\gamma_{clay} = 18$ kN / m^3

Height of sand layer = $h_1 = 4$ m

Height of clay layer = $h_2 = 1.25$ m

$C_C = 0.22$; $e_0 = 1.30$

Additional pressure = 30 kN / m^2

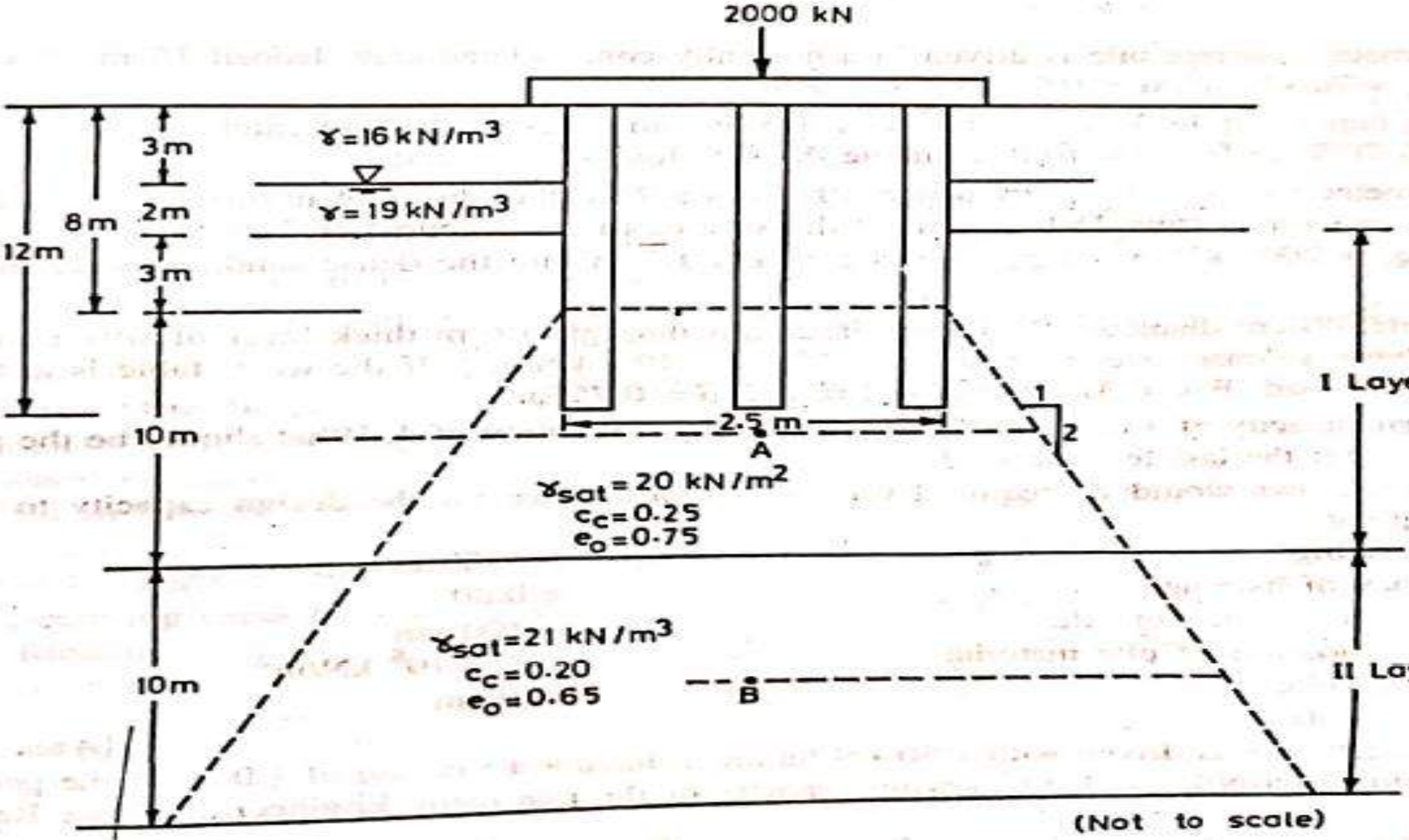
Pressure at the center of the clay layer = 20 * 4 + 18 * 1.25 = 102.5 kN / m^2

Settlement $\Delta H = C_C * H * \log_{10}((\sigma_0' + \Delta\sigma') / \sigma_0') / (1 + e_0)$

$= 0.22 * 2.5 * \log_{10}((102.5 + 30) / 102.5) / (1 + 1.30)$

$= 0.0266$ m = 2.66 cm

Illustrative Example 25.13. *A group of friction piles of 30 cm diameter is subjected to a net load of 2000 kN, as shown in Fig. E-25.13. Estimate the consolidation settlement.*



Solution. σ_0 at point A, middle of I layer

$$= 3 \times 16 + 2 \times (19 - 10) + 8 \times 10.0 = 146 \text{ kN/m}^2$$

σ_0 at point B, middle of II layer $= 3 \times 16 + 2 \times 9.0 + 13 \times 10.0 + 5 \times 11 = 251 \text{ kN/m}^2$

$$\text{Cross-sectional area at } A = \left(2.5 + 2 \times 5 \times \frac{1}{2}\right) = 7.5 \text{ m}^2$$

$$\Delta\sigma = \frac{2000}{7.5 \times 7.5} = 35.56 \text{ kN/m}^2$$

$$\text{Cross-sectional area at } B = \left(2.5 + 15 \times 2 \times \frac{1}{2}\right) = 17.5 \text{ m}^2$$

$$\Delta\sigma = \frac{2000}{17.5 \times 17.5} = 6.53 \text{ kN/m}^2$$

$$\text{Settlement of I layer} = C_c \left(\frac{H}{1 + e_0}\right) \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

$$= 0.25 \times \frac{10}{1 + 0.75} \log \frac{146 + 35.56}{146} = 0.135 \text{ m}$$

$$\text{Settlement of II layer} = 0.20 \times \frac{10}{1 + 0.65} \log \frac{251 + 6.53}{251} = 0.014$$

Total settlement

$$= 0.135 + 0.014 = \mathbf{0.149 \text{ m}}$$

Well Foundations

- Well foundation is the most commonly adopted foundation for major bridges in India.
- Since then many major bridges across wide rivers have been founded on wells.
- Well foundation is preferable to pile foundation when foundation has to resist large lateral forces
- The construction principles of well foundation are similar to the conventional wells sunk for underground water.
- Well foundations have been used in India for centuries.

Advantages of well foundation

(*i*) The effect of scour can be better withstood by a well foundation because of its large cross-sectional area and rigidity.

(*ii*) The depth can be decided as the sinking progresses, since the nature of the strata can be inspected and tested, if necessary, at any desired stage. Thus, it is possible to ensure that it rests upon a suitable bearing stratum of uniform nature and bearing power.

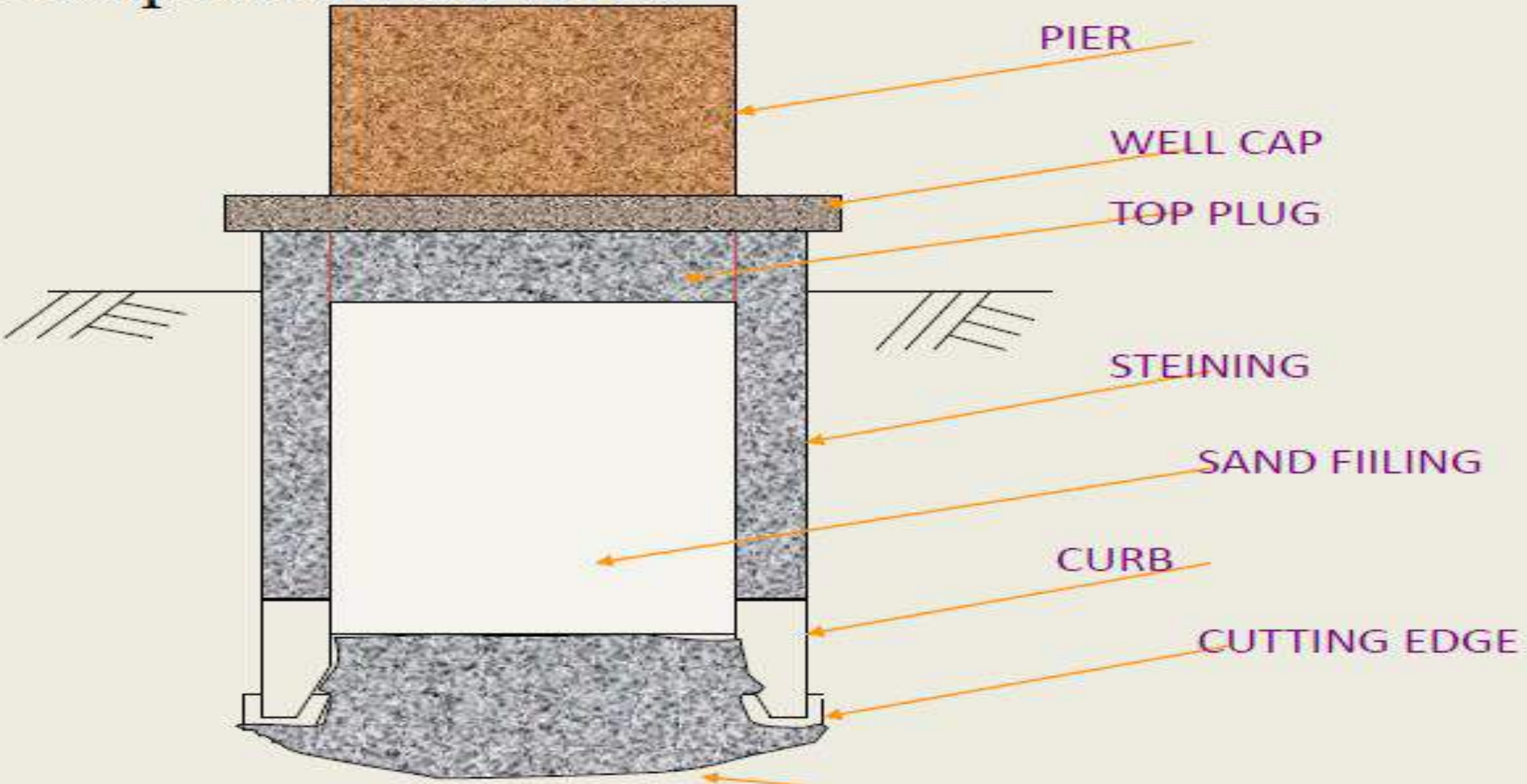
(*iii*) A well foundation can withstand large lateral loads and moments that occur in the case of bridge piers, abutments, tall chimneys, and towers; hence it is preferred to support such structures.

(*iv*) There is no danger of damage to adjacent structures since sinking of a well does not cause any vibrations.

61


lejárat
a sírkamrához
tartóoszlopok
Jamuna

Components of wells



WELL FOUNDATION

(*i*) *Cutting Edge:* The function of the cutting edge is to facilitate easy penetration or sinking into the soil to the desired depth. As it has to cut through the soil, it should be as sharp as possible, and strong enough to resist the high stresses to which it is subjected during the sinking process. Hence it usually consists of an angle iron with or without an additional plate of structural steel. It is similar to the sharp-edged cutting edge of a caisson shown in Fig. 19.2 (*a*).

(*ii*) *Steining:* The steining forms the bulk of the well foundation and may be constructed with brick or stone masonry, or with plain or reinforced concrete occasionally. The thickness of the steining is made uniform throughout its depth. It is considered desirable to provide vertical reinforcements to take care of the tensile stresses which might occur when the well is suspended from top during any stage of sinking.

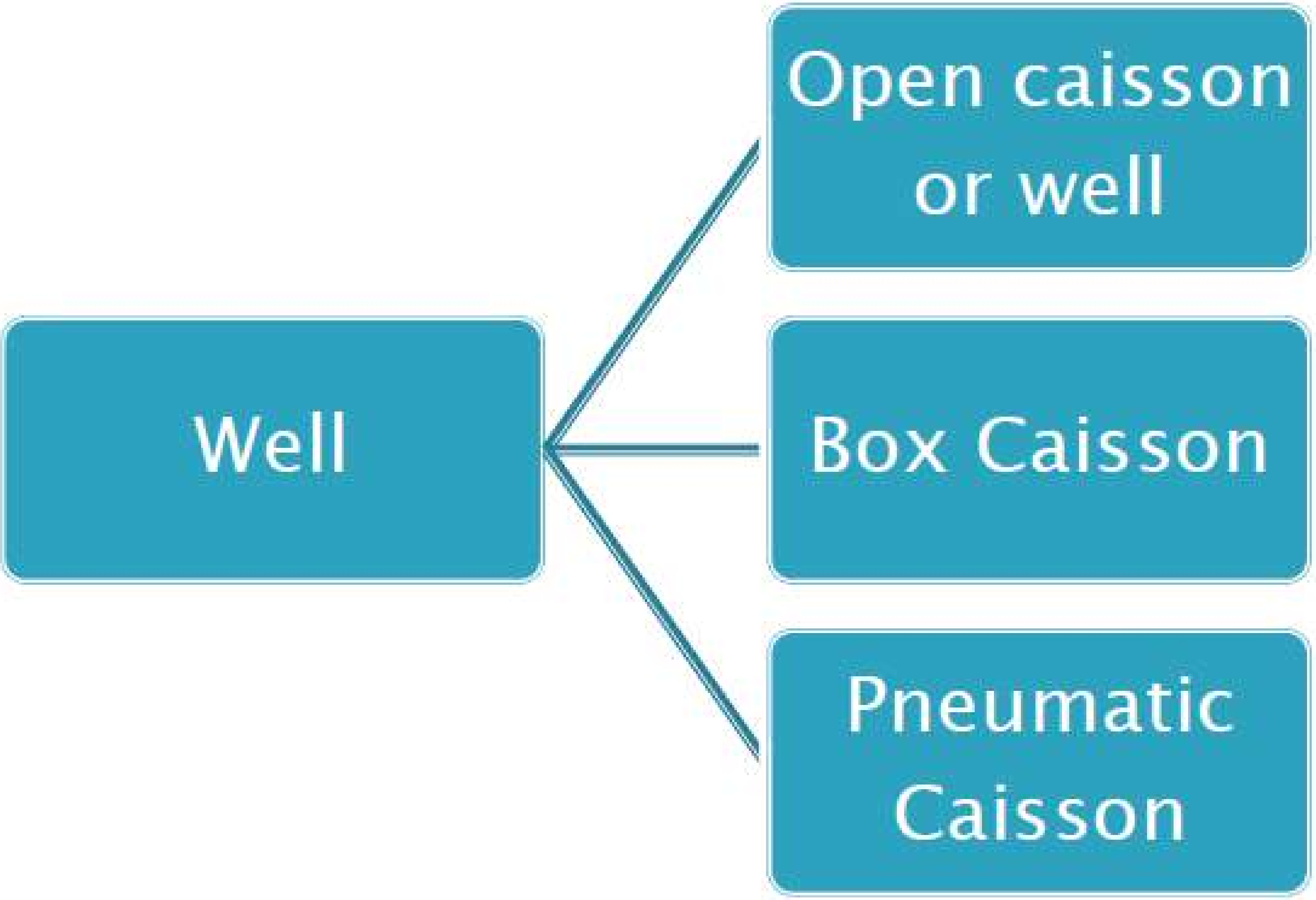
(*iii*) *Curb:* The well curb is a transition member between the sharp cutting edge and the thick steining. It is thus tapering in shape. It is usually made of reinforced concrete as it is subjected to severe stresses during the sinking process.

(*iv*) *Concrete Seal or Bottom Plug:* After the well foundation is sunk to the desired depth so as to rest on a firm stratum, a thick layer of concrete is provided at the bottom inside the well, generally under water. This layer is called the concrete seal or bottom plug, which serves as the base for the well foundation. This is primarily meant to distribute the loads on to a large area of the foundation, and hence may be omitted when the well is made to rest on hard rock.

(*v*) *Top Plug:* After the well foundation is sunk to the desired depth, the inside of the well is filled with sand either partly or fully, and a top layer of concrete is placed. This is known as 'top plug'.

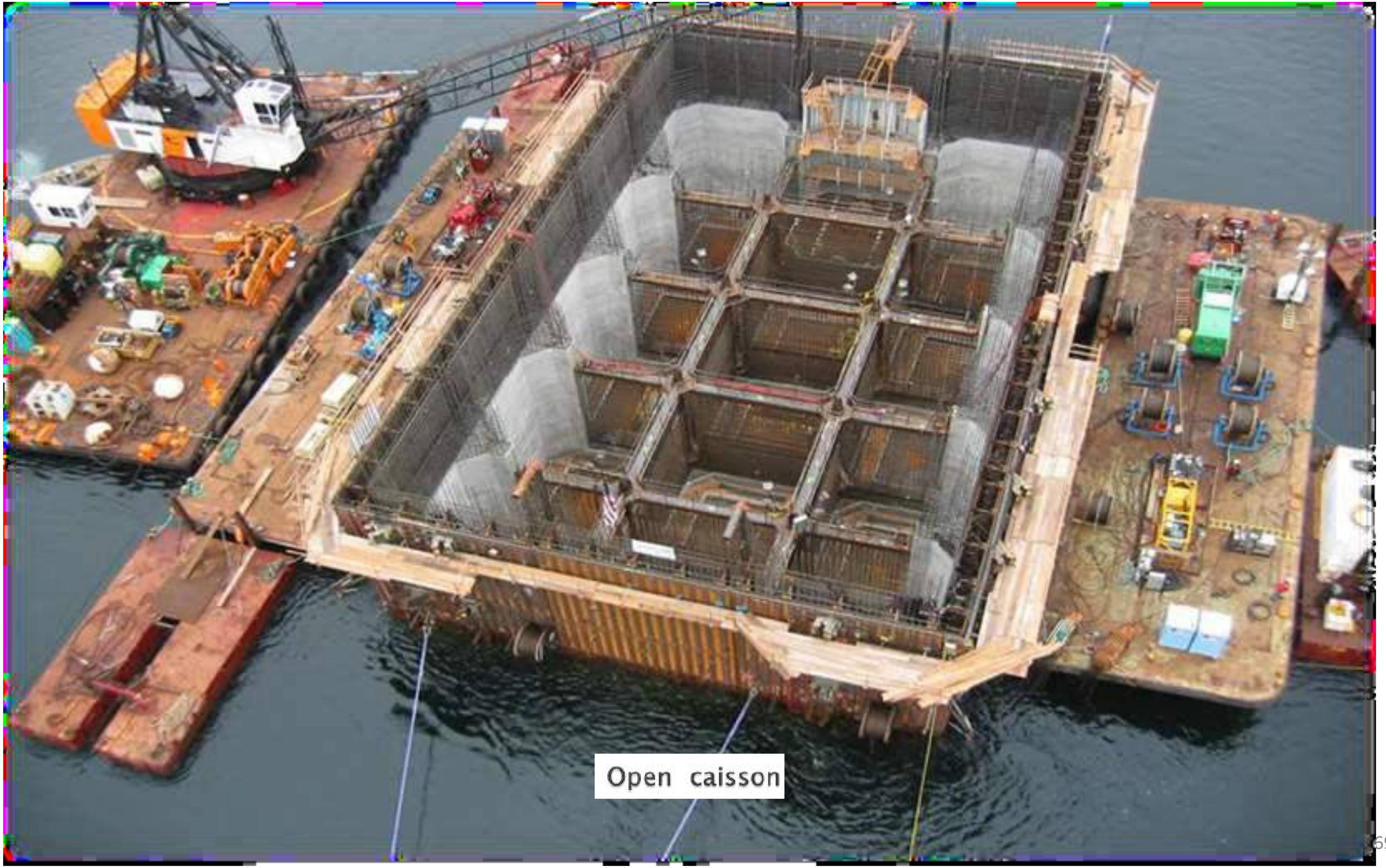
(*vi*) *Well Cap:* The well cap serves as a bearing pad to the superstructure, which may be a pier or an abutment. It distributes the superstructure load onto the well steining uniformly.

Types of Well Foundation



Types of well foundation:-

- **Open caisson or well**: The top and bottom of the caisson is open during construction. It may have any shape in plan.
- **Box caisson**: It is open at the top but closed at the bottom.
- **Pneumatic caisson**: It has a working chamber at the bottom of the caisson which is kept dry by forcing out water under pressure, thus permitting excavation under dry conditions.


Open caisson

2011-11.2_0310+311 First walls in Pier 3 caisson are ready to be poured. photo: John Stamets

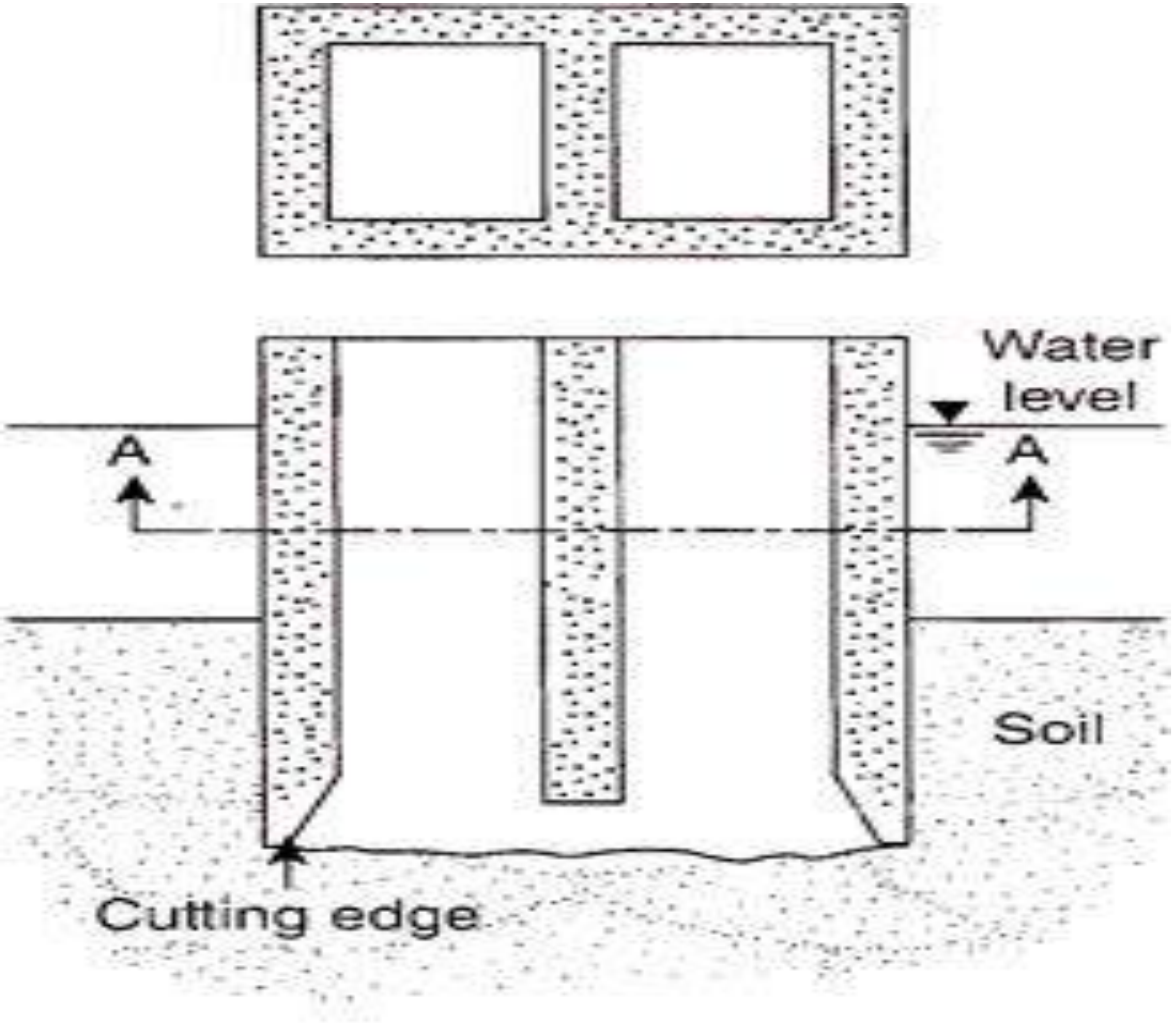


Figure 21.1 Open caisson.

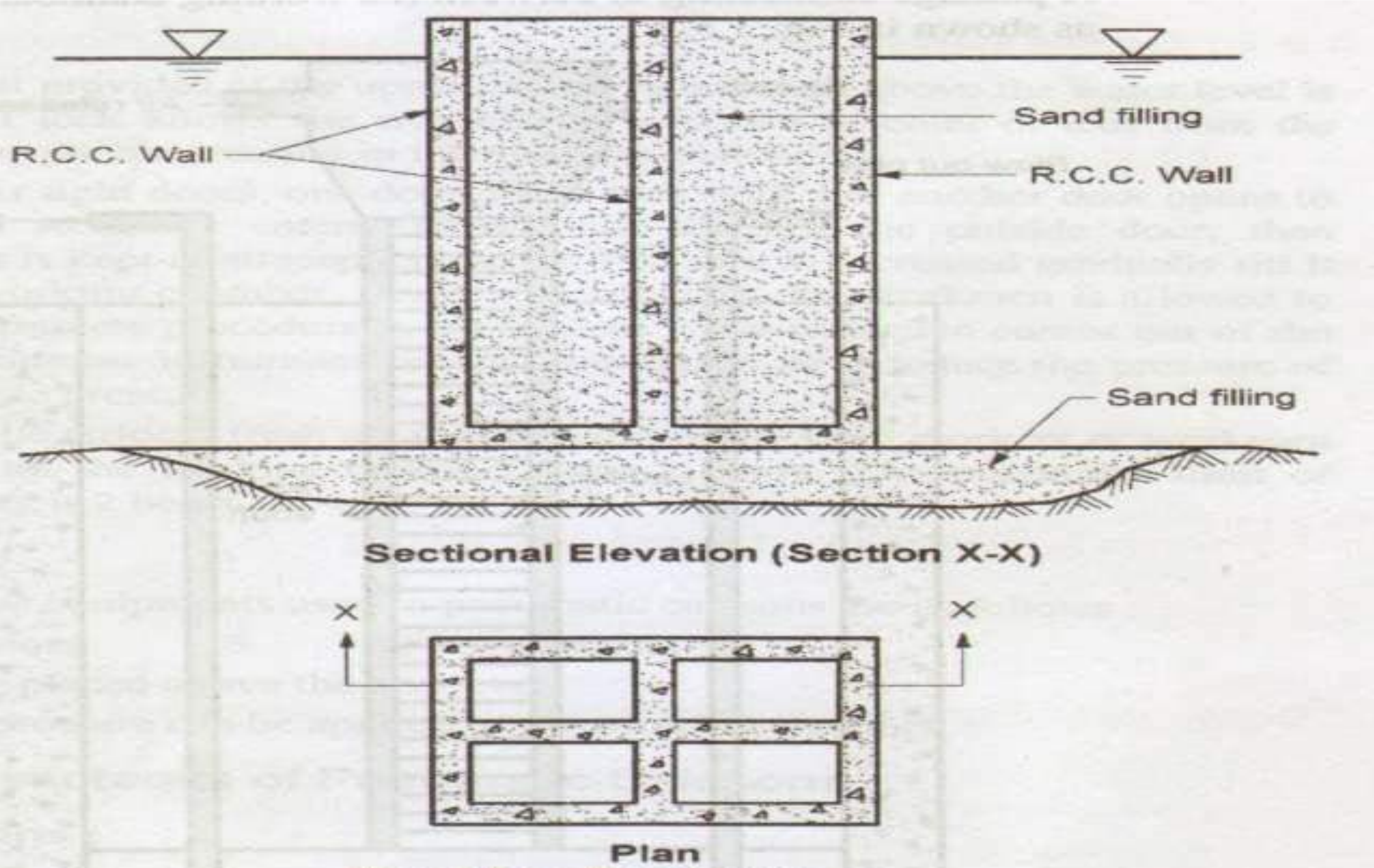


Fig. 3.3.6 : Box caisson

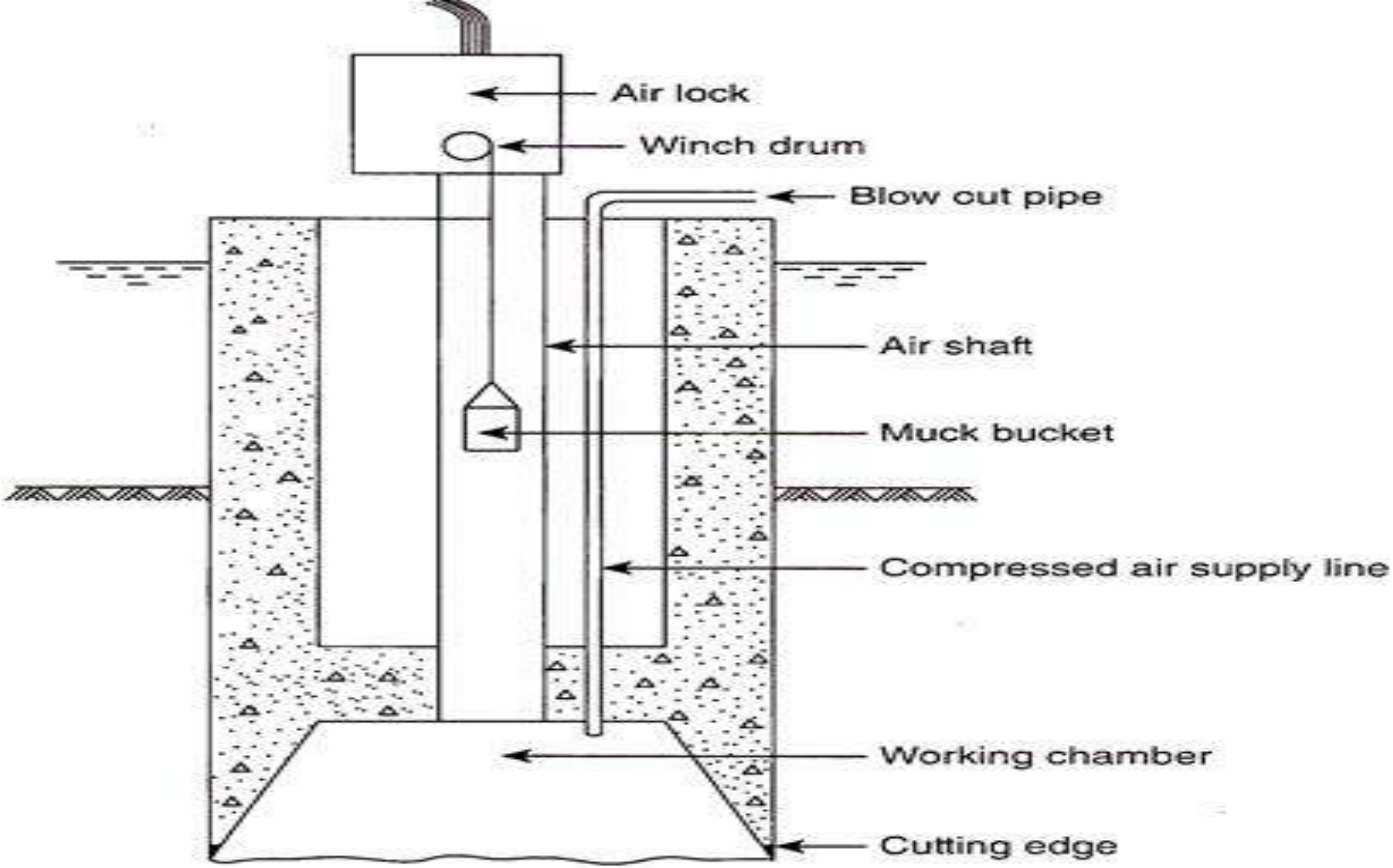


Figure 21.3 Pneumatic caisson.

Types of well shapes

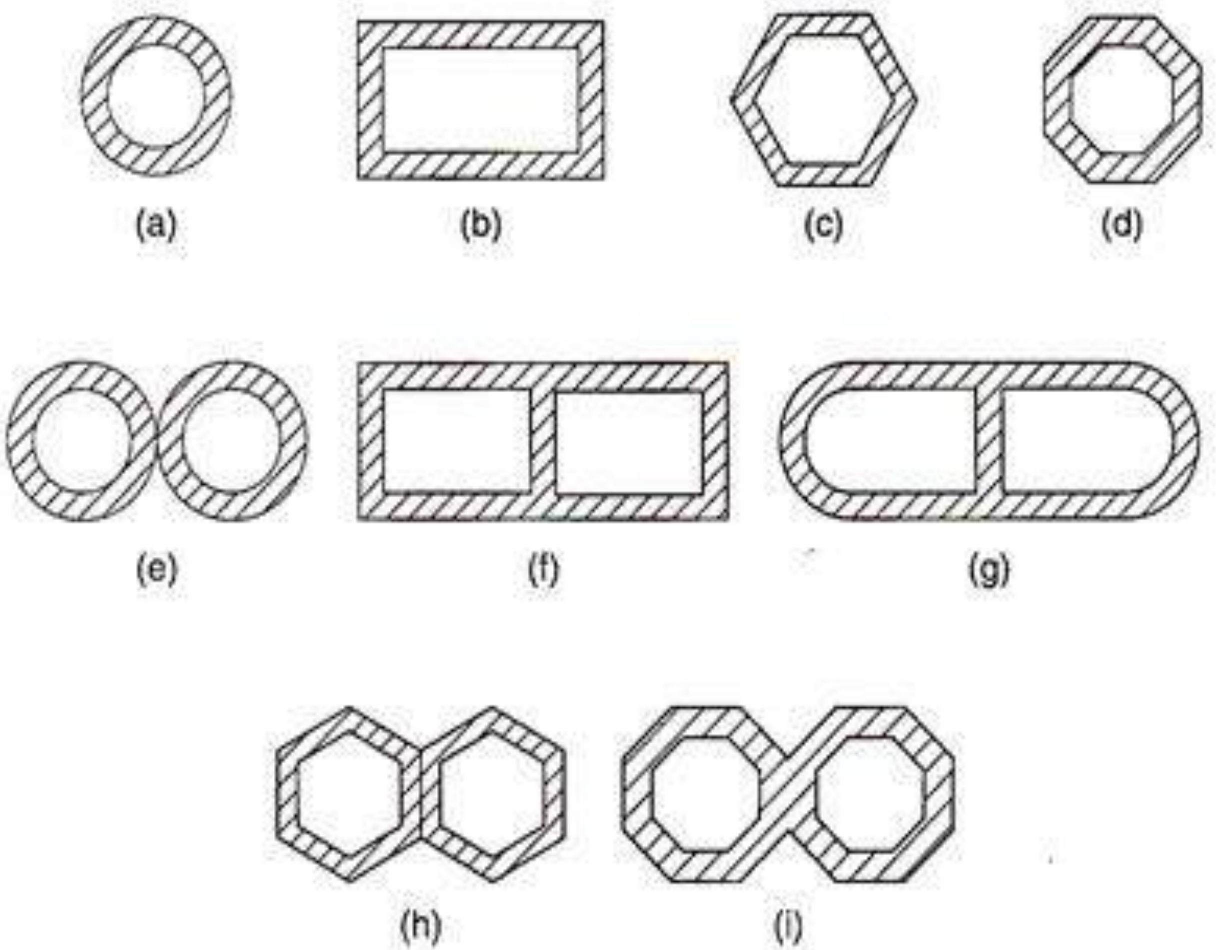


Figure 21.4 Shapes of well foundations: (a) Circular, (b) rectangular, (c) hexagonal, (d) octagonal, (e) twin circular, (f) double rectangular, (g) double D, (h) double hexagonal, and (i) double octagonal.

Procedure for Sinking of Well foundations

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.

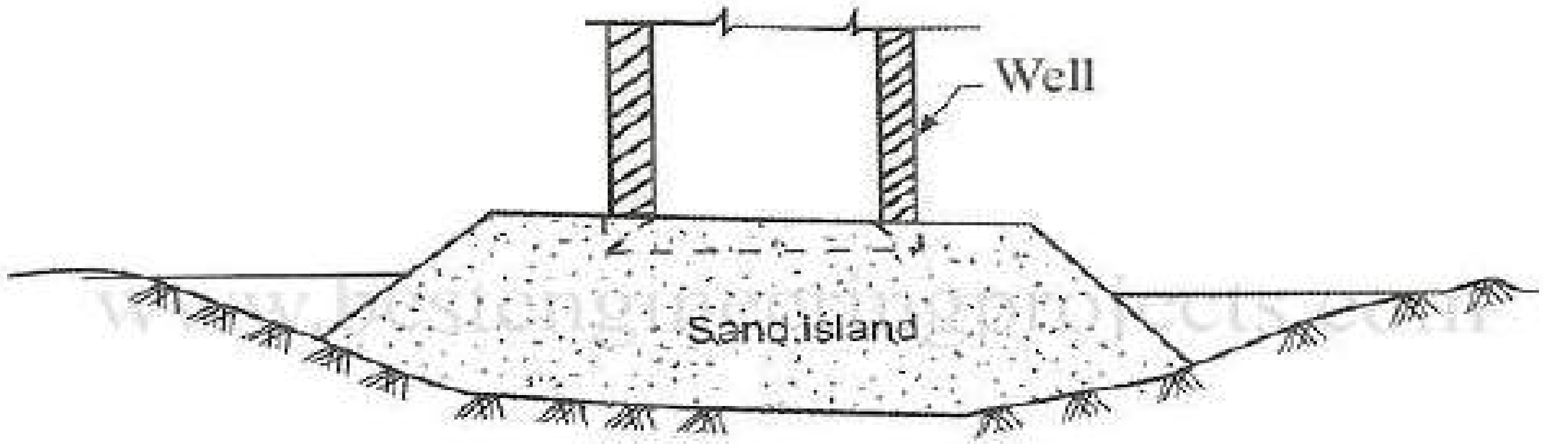


Fig 1 A Typical Sand Island

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 Jhams 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 2.

•Go on adding sections of steining (2 to 5 m in length) up to the required founding strata.

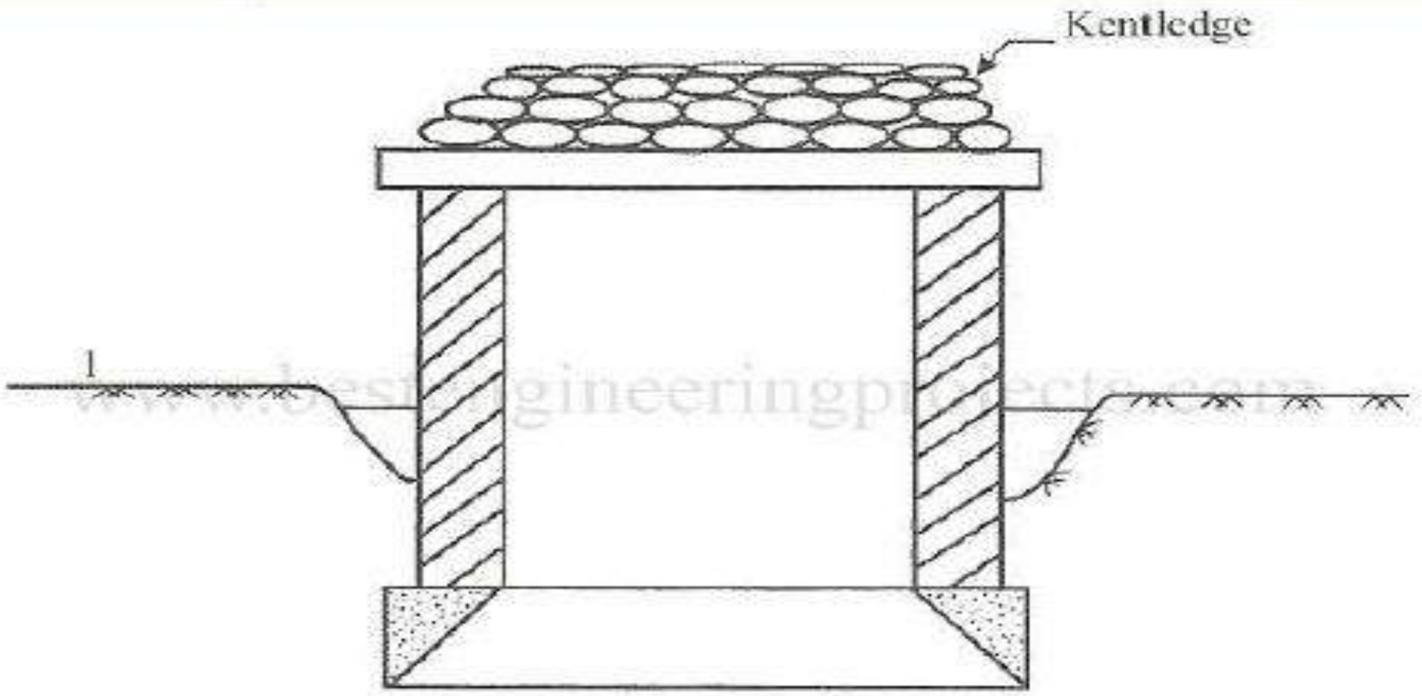


Fig 2 Loading Steining by Kentledge

Tilt and Shift

- 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

Remedial Measures for Rectification of Tilts and Shifts

The following remedial measures may be taken to rectify tilts and shifts:

(1) *Regulation of Excavation:* The higher side is grabbed more be regulating the dredging. In the initial stages this may be all right. Otherwise, the well may be dewatered if possible, and open excavation may be carried out on the higher side [Fig. 19.23 (*a*)].

(2) *Eccentric Loading:* Eccentric placing of the kentledge may be resorted to provide greater sinking effort on the higher side. If necessary a platform with greater projection on the higher side may be constructed and used for this purpose. As the depth of sinking increases, heavier kentledge with greater eccentricity would be required to rectify tilt [Fig. 19.23 (*b*)].

(3) *Water Jetting:* If water jets are applied on the outer face of the well on the higher side, the friction is reduced on that side, and the tilt may get rectified [Fig. 19.23 (*c*)].

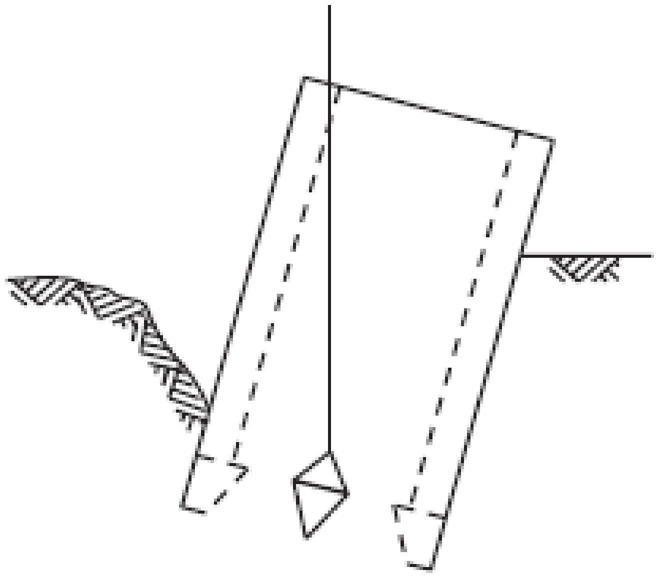
(4) *Excavation under the Cutting Edge:* If hard clay is encountered, open excavation is done under the cutting edge, if dewatering is possible; if not, divers may be employed to loosen the strata.

(5) *Insertion of Wood Sleeper under the Cutting Edge:* Wood sleepers may be inserted temporarily below the cutting edge on the lower side to avoid further tilt.

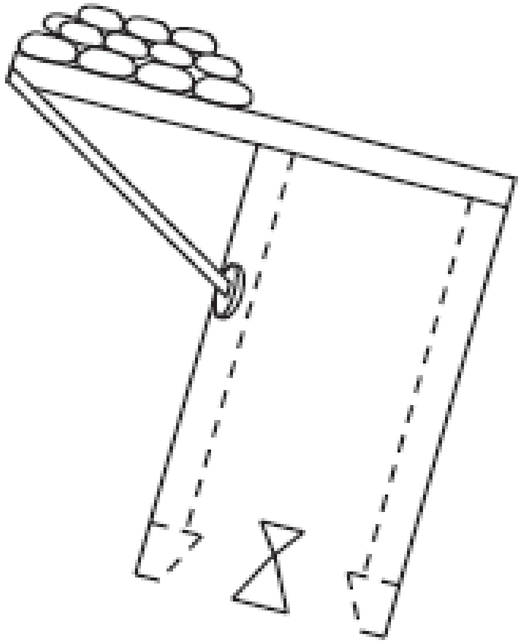
(6) *Pulling the Well:* In the early stages of sinking, pulling the well to the higher side by placing one or more steel ropes round the well, with vertical sleepers packed in between to distribute pressure over larger areas of well steining, is effective [Fig. 19.23 (*d*)].

(7) *Strutting the Well:* The well is strutted on its tilted side with suitable logs of wood to prevent further tilt. The well steining is provided with sleepers to distribute the load from the strut. The other end of the logs rest against a firm base having driven piles [Fig. 19.23 (*e*)].

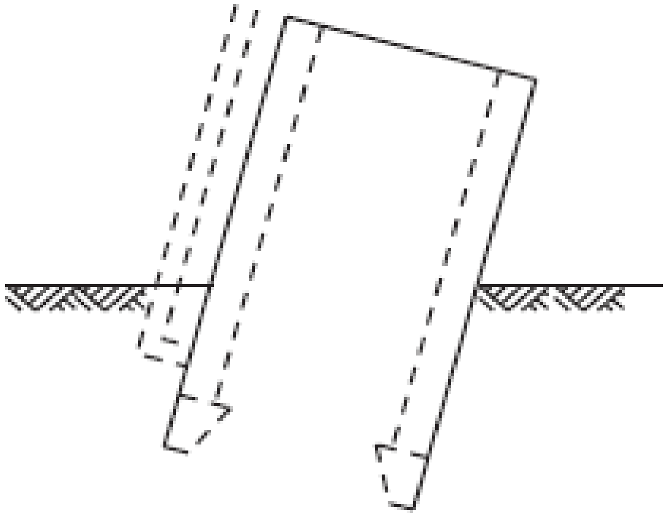
(8) *Pushing the Well with Jacks:* Tilt can be rectified by pushing the well by suitably arranging mechanical or hydraulic jacks.



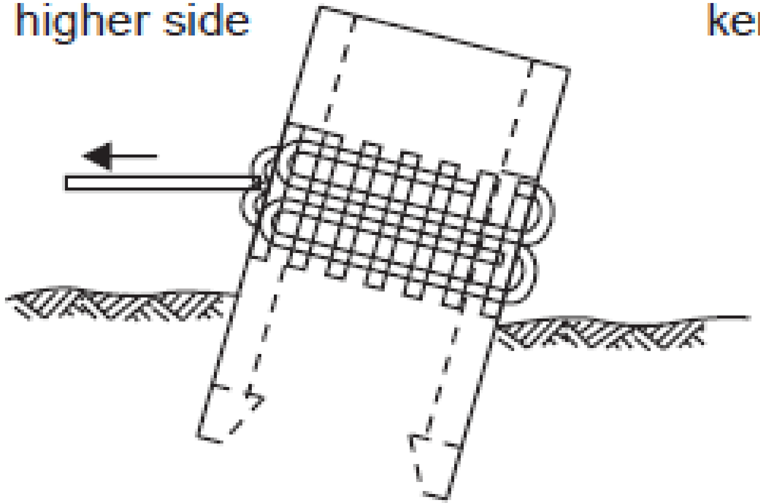
(a) Excavation on higher side



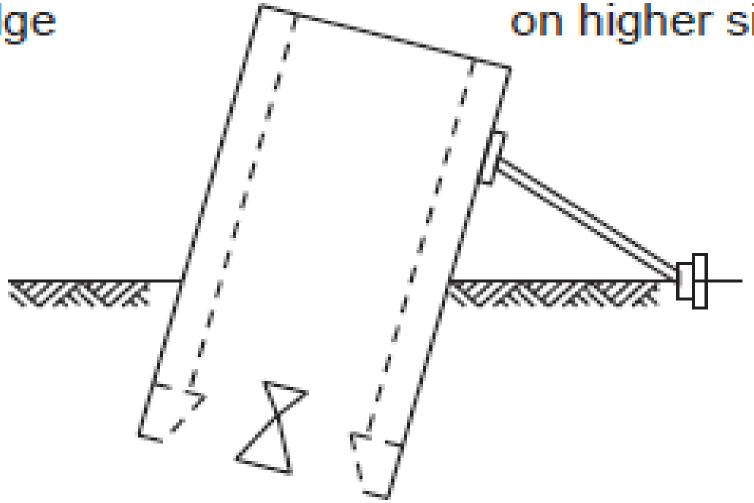
(b) Using eccentric kentledge



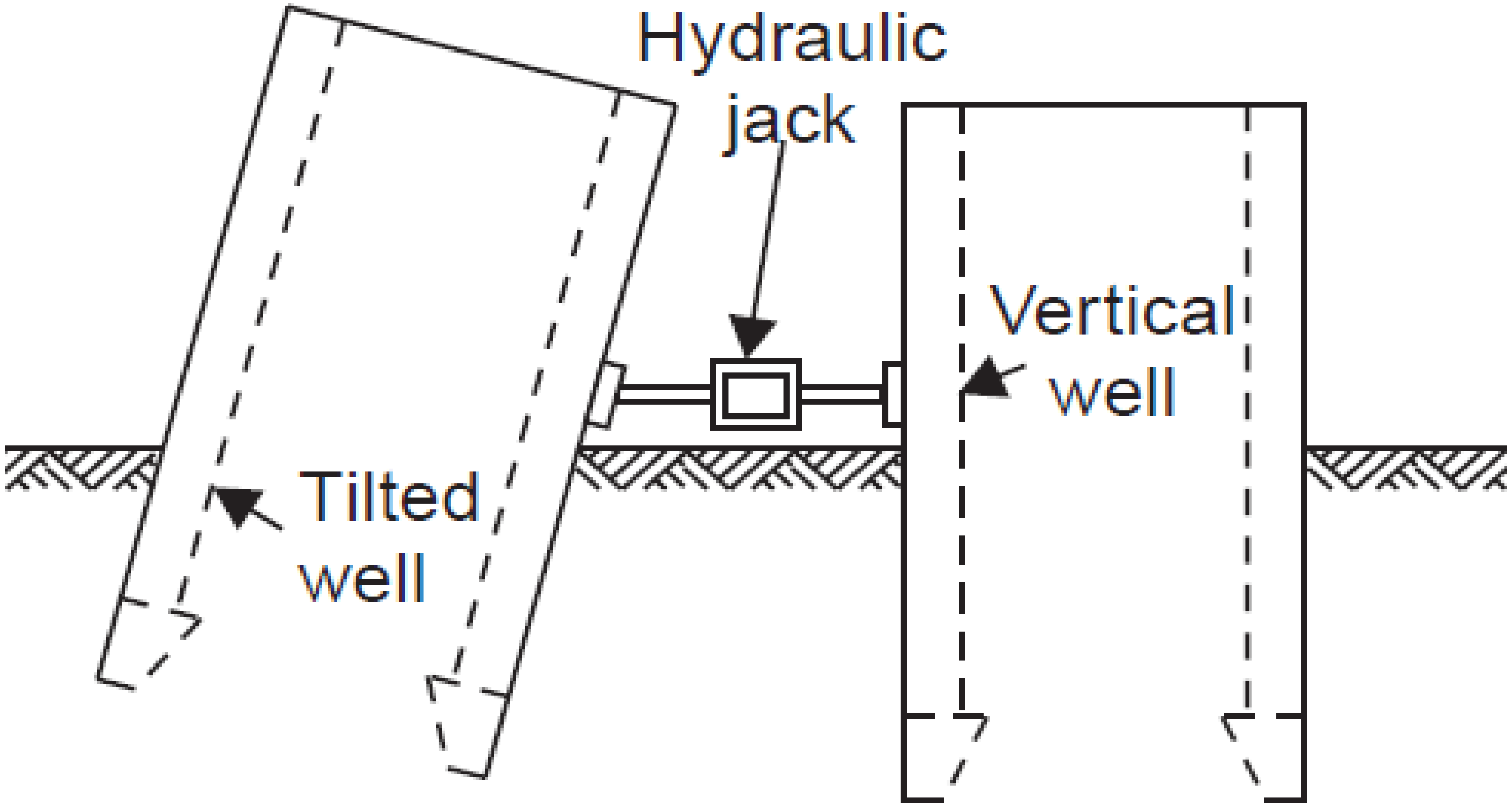
(c) Water or air jetting on higher side



(d) Pulling from higher side



(e) Strutting the well from lower side



(f) Pushing the well with jacks

Design aspects of well foundation

The depth of well foundation is based on the following 2 criteria.

1. There should be adequate embedded length of well, called the grip length below the lowest scour level.
2. The well should be taken deep enough to rest on strata of adequate bearing capacity in relation to the loads transmitted.

normal depth of scour may be calculated by Lacey's formula:

$$d = 0.473(Q/f)^{1/3}$$

where d = normal scour depth, measured below high flood level (m),

Q = design discharge (m^3/s),

and f = Lacey's silt factor.

The silt factor may be calculated from the equation

$$f = 1.76\sqrt{d_m}$$

where d_m = mean size of the particle (mm).

The grip length for wells of railway bridges is taken as 50% of maximum scour depth, generally, while for road bridges 30% of maximum scour depth is considered adequate. base of the well is usually taken to a depth of 2.67 d' below the HFL.

According to IS:3955-1967, the depth should not be less than 1.33 times the maximum scour depth. The depth of the base of the well below the scour level is kept not less than 2 m for piers and abutments with arches, and 1.2 m for piers and abutments supporting other types of structures.

Terzaghi and Peck have suggested the ultimate bearing capacity can be determined from the following expression.

$$Q_u = Q_p + 2\pi R f D_f$$

$$Q_p = \pi R^2 \left(1.2cN_c + \gamma D_f N_q + 0.6\gamma R N_\gamma\right)$$

where N_c , N_q , N_γ= Terzaghi's bearing capacity factors.

R= radius of well

D_f =depth of well

f_s =average skin friction