# Arba Minch Water Technology Institute
# Faculty of Water Supply and Environmental Engineering

Course title; **Foundation Engineering**

Course code; ***Ceng-3172***

Target groups; *3rd year Water Supply and Env'tal Eng'g students*

Academic year; 2019/20 Semester; II

Instructor; Gemechis A.

# CHAPTER ONE

# Introduction to Foundation Engineering

## Contents of the chapter

- General Introduction to foundation
- Foundation types
- Selection of appropriate type of foundation
- Geotechnical investigation and methods
- Foundation on Expansive soil

# INTRODUCTION TO FOUNDATIONS

- ❖ Any structure consists of two parts which are the super structure and the substructure which is found below the ground level.

**WHAT IS FOUNDATION?**

- ❖ Foundation is the lowest part of a structure which is used to transfer load of the superstructure to the ground on which it is resting.

# What are the requirements?

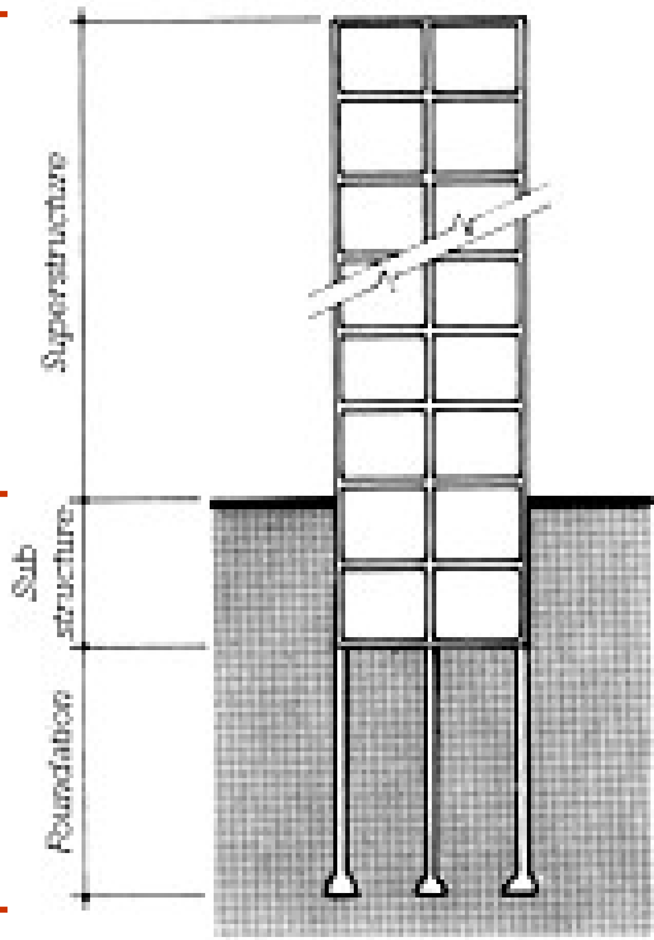
- ❖ The structural load should be transferred throughout the rock/soil without overstressing of soil.
- ❖ Overstress results in either excessive settlement or shear failure, both of which can damage the structure.
- ❖ Foundation engineering applies the knowledge of:
  - ✓ Soil mechanics (physical properties of soils)
  - ✓ Rock mechanics
  - ✓ Geology and
  - ✓ Structural engineering to the design and construction of foundations for buildings and other structures.

# Major building parts

Superstructure

Substructure

Foundation

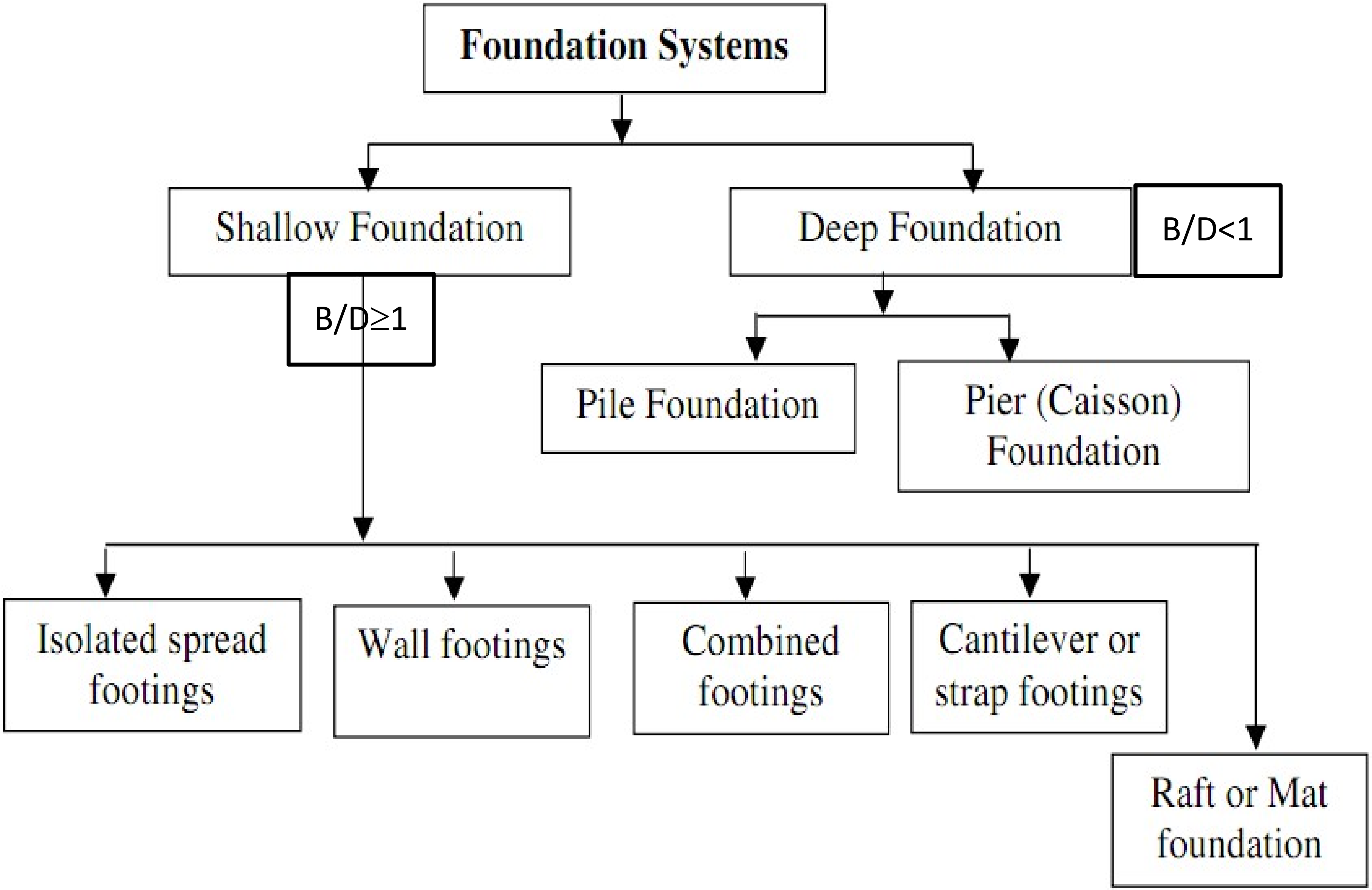


# Objectives of Foundation

❖ A foundation is provided for the following purposes:

- ✓ To distribute the total load coming from the structure on a larger area.
- ✓ To support the structures.
- ✓ To give enough stability to the structures against various disturbing forces, such as wind and rain.
- ✓ To prepare a level surface for concreting and masonry work etc.

# TYPES OF FOUNDATION



# 1. Shallow Foundation

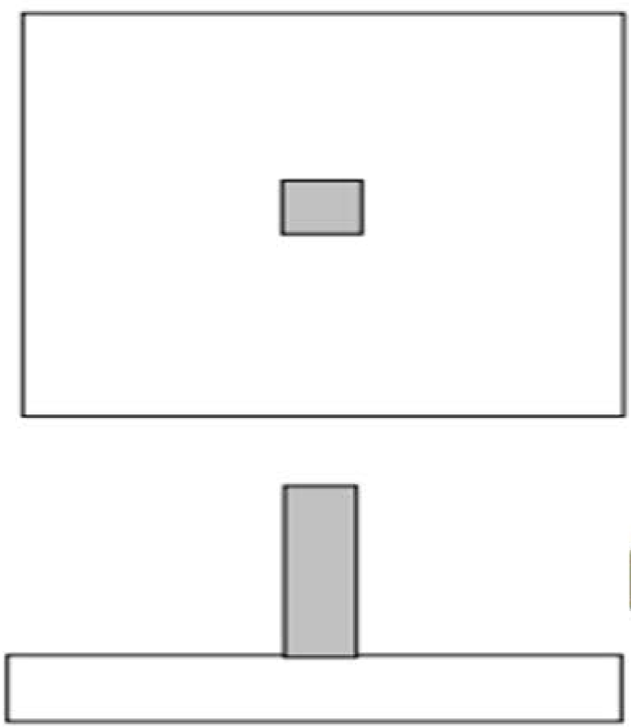
- A shallow foundation is a type of building foundation that transfers building loads to the **earth very near to the surface**, rather than to range of depths as does a deep foundation.
- These types of foundations are so called, because they are placed at a shallow depth (relative to their dimensions) beneath the soil surface.

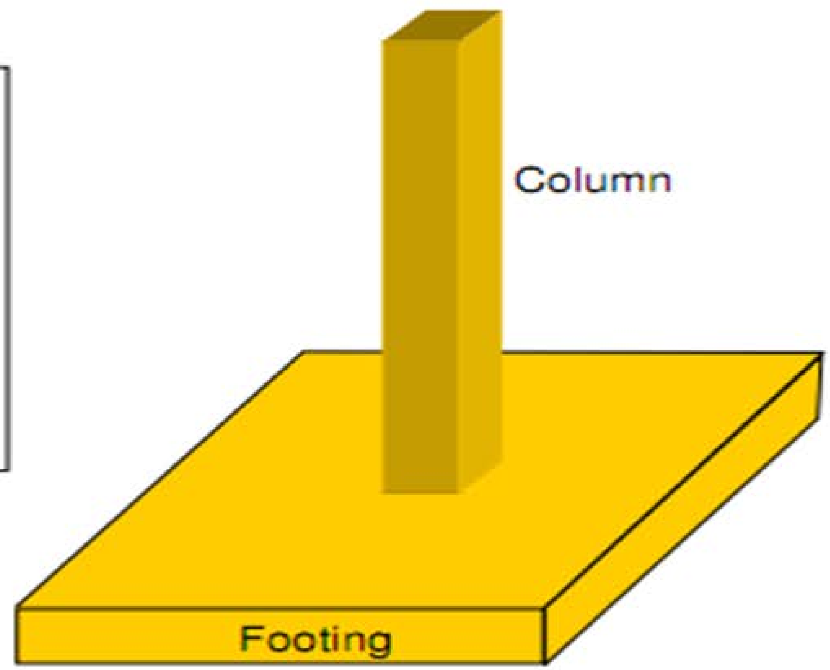
Cont…..

- A shallow foundation system are generally used when:
  - The soil close the ground surface has sufficient bearing capacity
  - Underlying weaker strata do not result in undue settlement.
  - The shallow foundations are commonly used most economical foundation systems.
- The shallow foundation also contains **Footings** and **Columns**

# Footings

- Are structural elements, which transfer loads from columns, walls or lateral loads from earth retaining structures to the soil.





# Cont…..

- Footings must be designed to:
  - Prevent excessive settlement
  - Minimize differential settlement, and
  - Provide adequate safety against overturning and sliding.

# Advantages of Shallow Foundation

- Cost (affordable)
- Construction Procedure (simple)
- Material (mostly concrete)
- Labor (doesn’t need expertise) etc.

# Types of shallow foundation

- Shallow is also called **open** or **spread** foundation
- Spread footings may be built in **different shapes & sizes** to accommodate individual needs. It can be:
  - Square Spread Footings
  - Rectangular Spread Footings
  - Circular Spread Footings

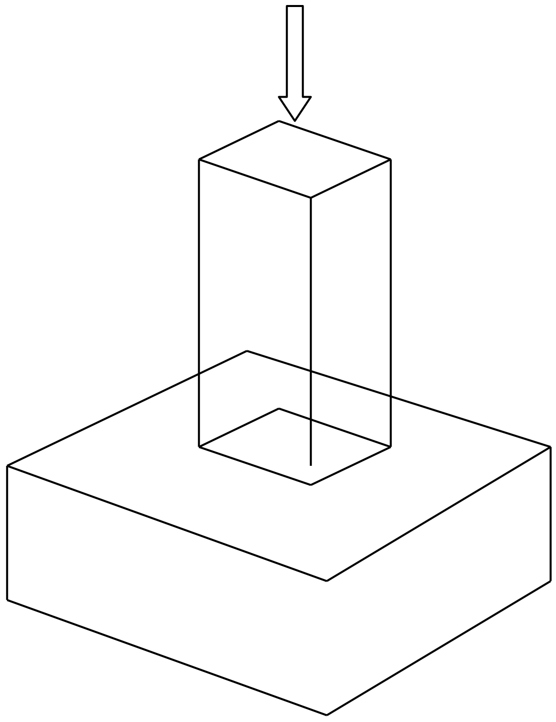
# A) Isolated spread footing (L/B<10):

## Stepped Isolated spread footing

- A type of footing w/c is provided to support an individual columns.
- This can be square footings, Rectangular Spread footings or circular type.

# Square Isolated footings

- Support a single centrally located column
- Square (B x B)
- Usually one column

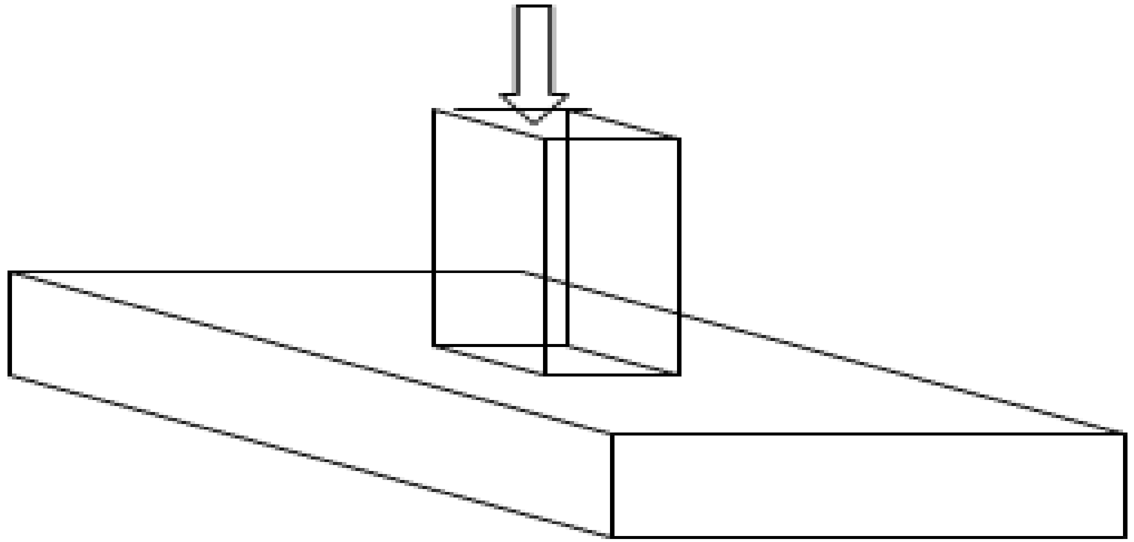


# Cont…

- Sometimes, it is stepped or sloped to spread the load over a large area.

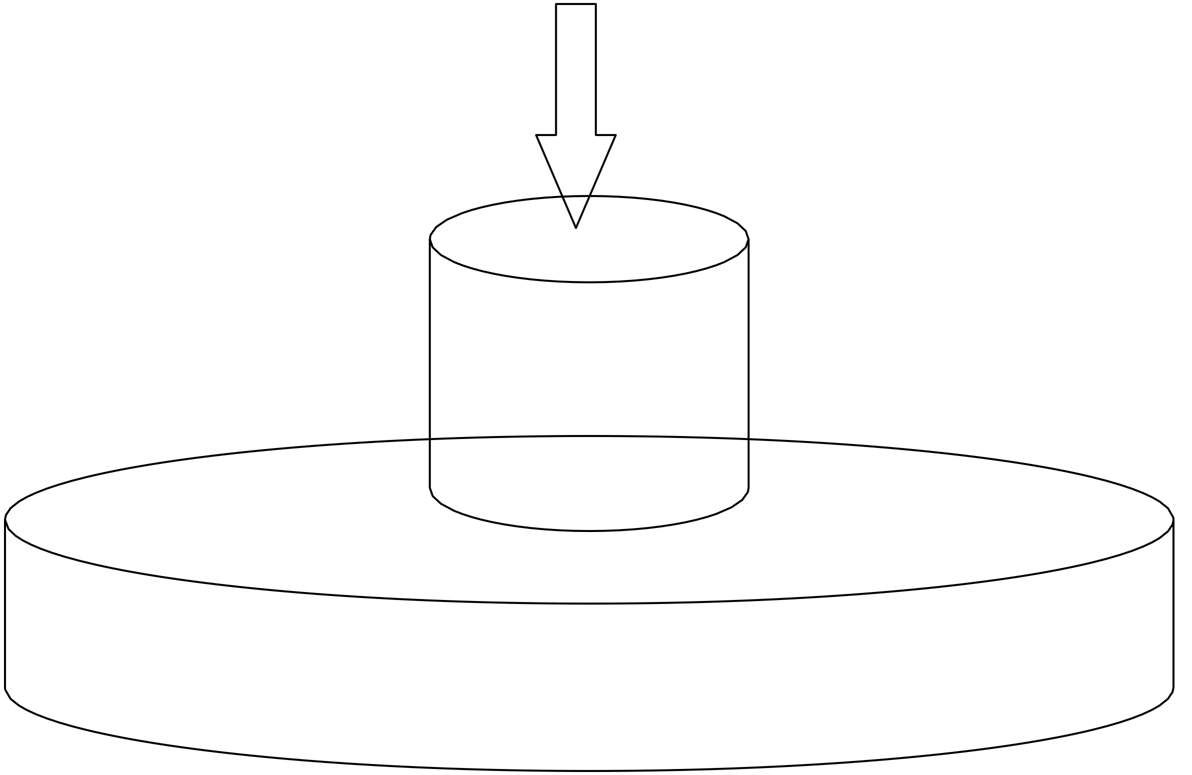
# Rectangular Isolated Footings

- Rectangular (B x L)
- Useful when obstructions prevent construction of a square footing with a sufficiently large base area and
- When large moment loads are present



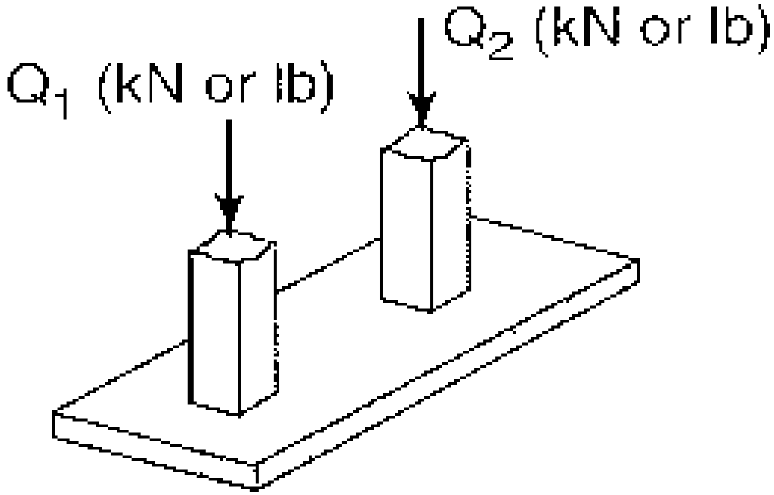
# Circular Isolated Spread Footings

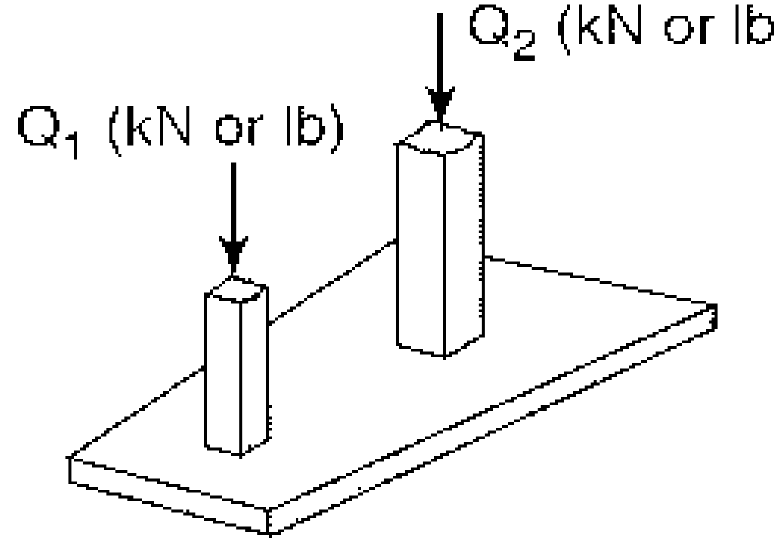
- Are round in plan view
- Most frequently used for power transmission lines and flag poles
  .

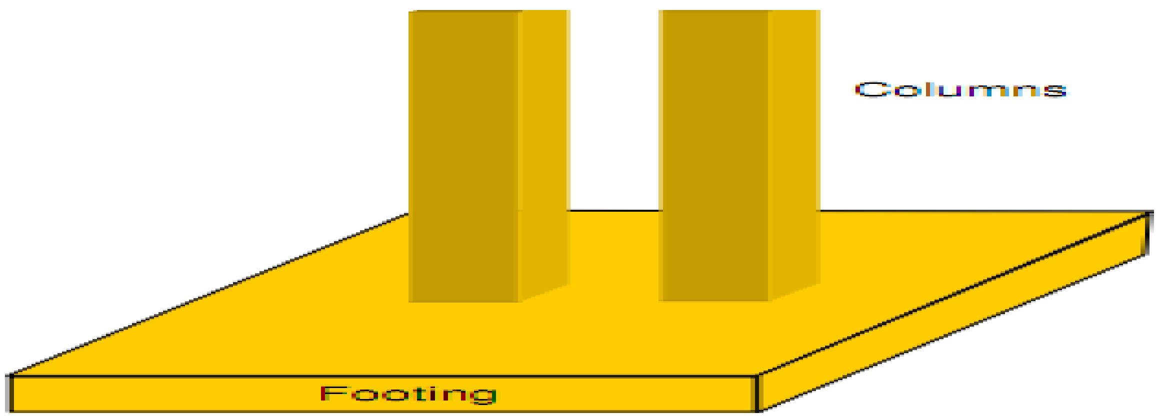


# B. Combined Footings

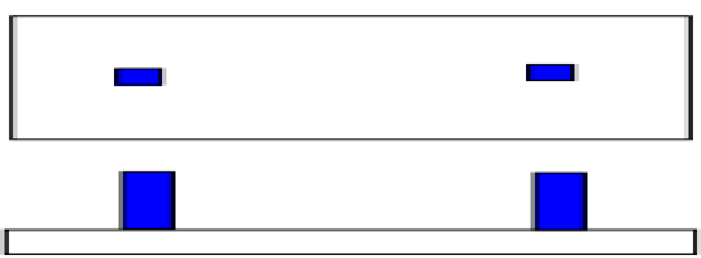
- ❖ A type of footing used to Support more than one column
- ❖ Useful when columns are located too close each other
- ❖ Provides necessary Moment to prevent failure.
- ❖ May be trapezoidal or rectangular
  - ✓ Rectangular combined footing are used when the two columns carry equal loads
  - ✓ where as Trapezoidal combined footings are used when the two columns have different loads.

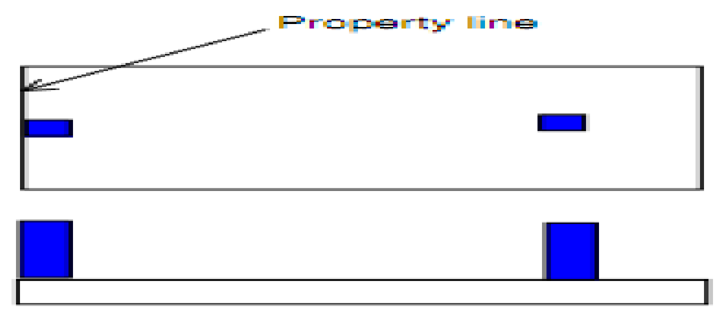



Combined Footing

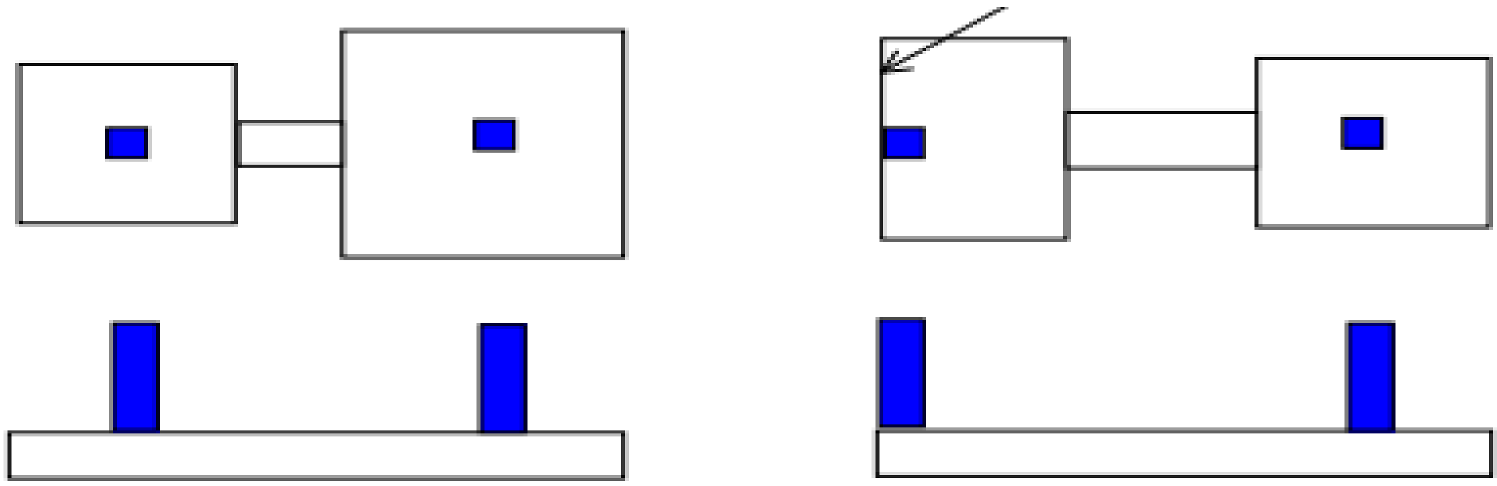




# C) Strap or Cantilever Footing

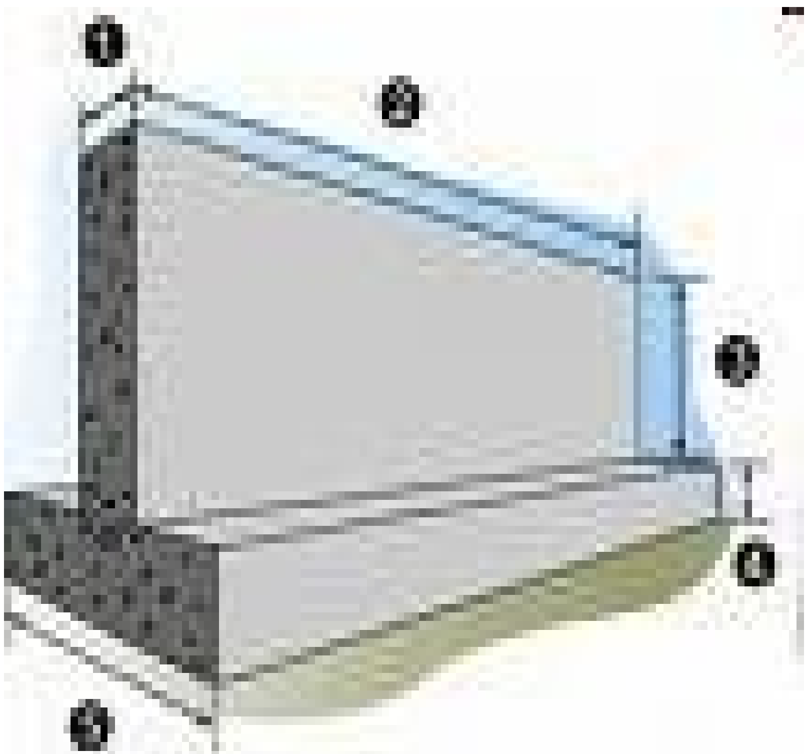
- These are similar to combined footings, except that the **footings under columns are built independently, and are joined by strap beam.**

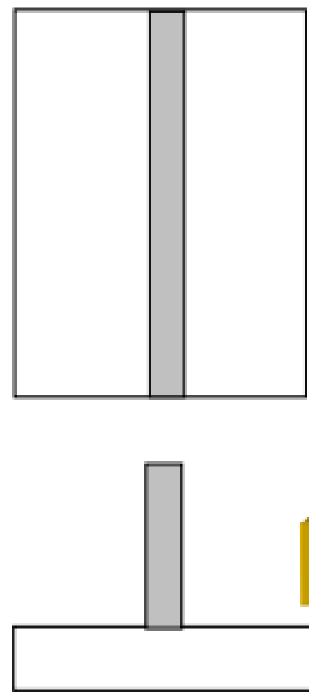
- The strap connects the footings such that they **behave as one unit.**

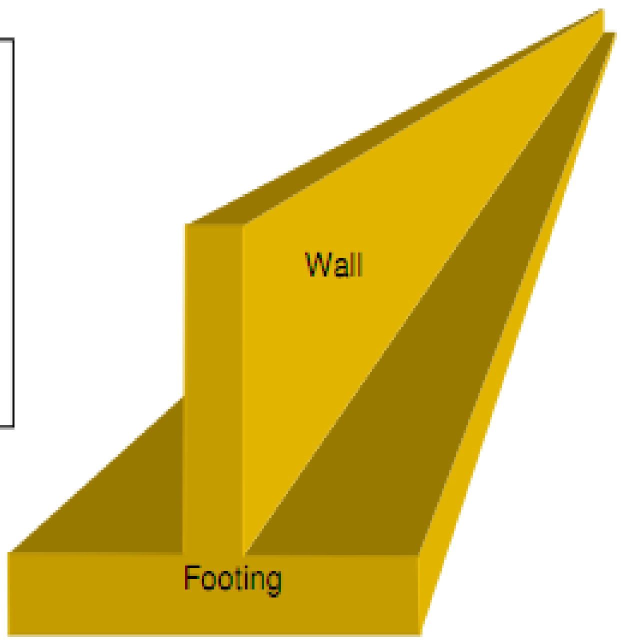


# D. Wall footings

- Are also called **Continuous spread footings** or **Strip foundation(L/B≥10).**
- Used to support bearing/resisting walls.







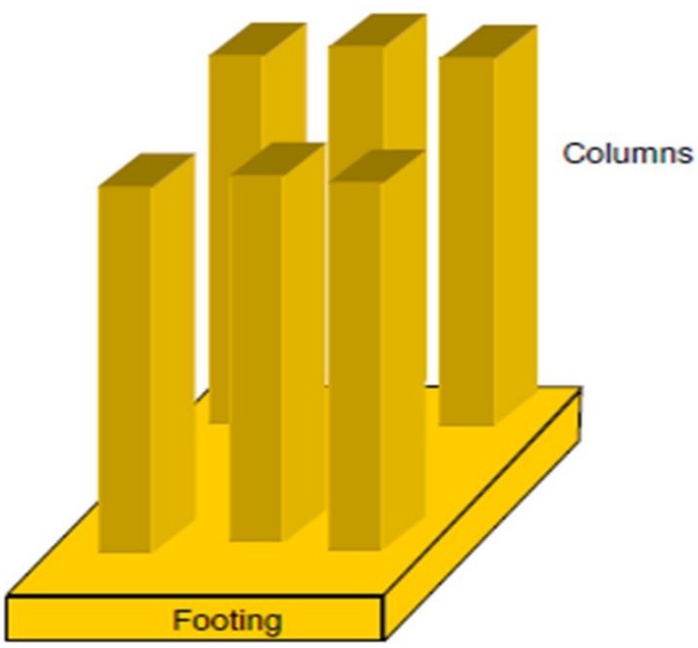
# E. MAT(RAFT) FOUNDATION

- ❖ Mat or raft foundation is a large concrete slab supporting several columns in two or more rows.
- ❖ A Mat or Raft **cover the entire area beneath a structure** and support all the walls and columns
- ❖ Mat/Raft foundation will be used when:
  - ✓ The allowable soil pressure is low
  - ✓ The building loads are heavy
  - ✓ Use of spread footing covers more than the one half the area.
  - ❖ **Thus mat foundations are often used for supporting structures that are sensitive to differential settlement.**

Cont…

# Advantages of Raft Foundation

- Spread the load in a larger area-Increase bearing pressure
- Provides more structural rigidity-Reduce settlement
- Heavier-More resistant to uplift
- Distributes loads more evenly



# Deep Foundations

- Deep foundations are used to transfer loads to a stronger layer, which may be located at a significant depth below the ground surface.
- Deep foundations are of the following types.
    1. Pile foundations- more commonly used.
    2. Pier foundation
    3. Caisson or well foundation

# PILE FOUNDATIONS

- ❖ A pile is basically a long cylinder of a strong material such as concrete that is pushed into the ground to act as a steady support for structures built on top of it

- ❖ Pile foundations are preferable under the following situation:
  - ➢ When the load of the super structure is heavy and its distribution is uneven.
  - ➢ The top soil has poor bearing capacity
  - ➢ The sub soil water level is high so that pumping of water from the open trenches for the shallow foundations is **difficult and uneconomical**.

# Cont…

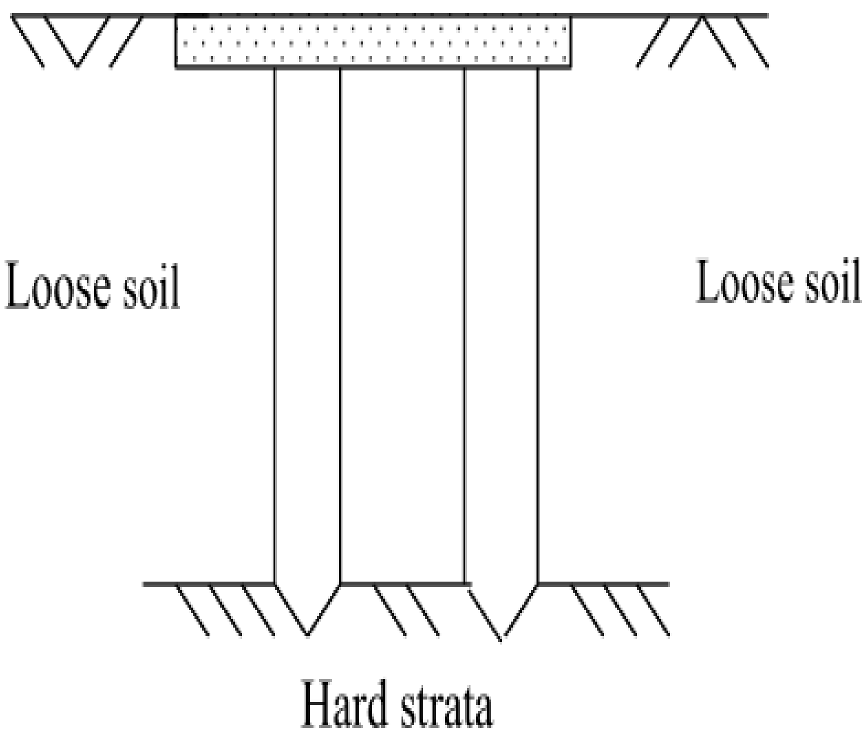
- When there is large fluctuations in sub soil water level
- When the structure is situated on the sea shore or river bed
- Canal or deep drainage lines exist near the foundations

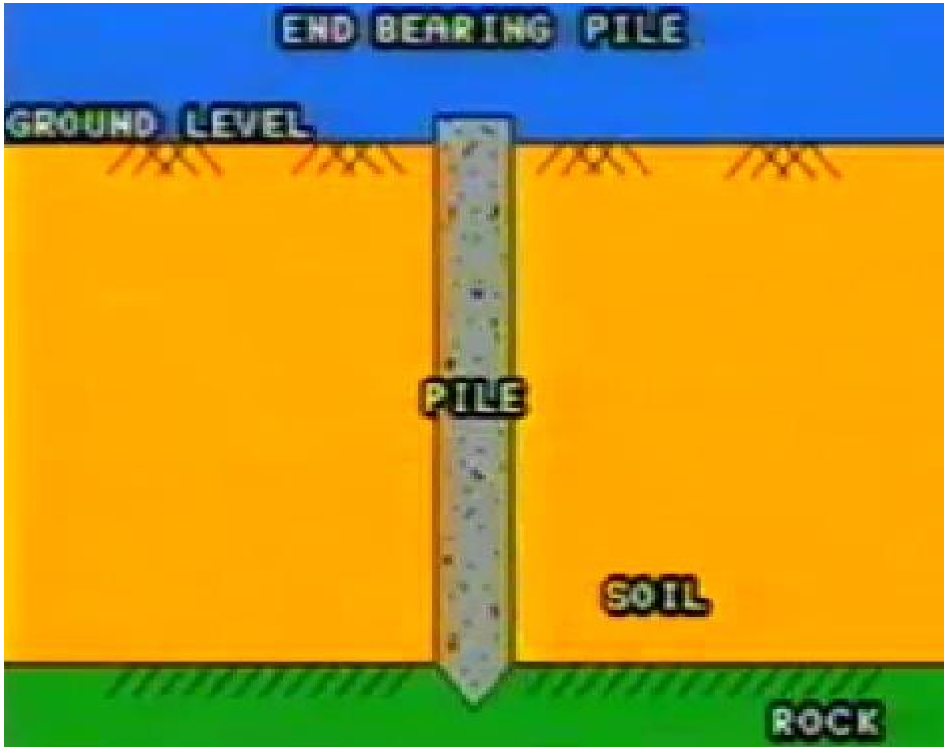
- Piles used for building foundation may be of four types; based on the function they serve.
  - End bearing pile
  - Friction pile
  - combined and
  - Compaction piles

# I)End bearing piles

Used to transfer load through water or soft soil to a suitable bearing stratum.

- ❖ Such piles are used to carry heavy loads safely to hard strata. Multi-storey
- ❖ Buildings are regularly founded on end bearing piles, so that the settlements are minimized.        So, End bearing piles **Qu=Qp**

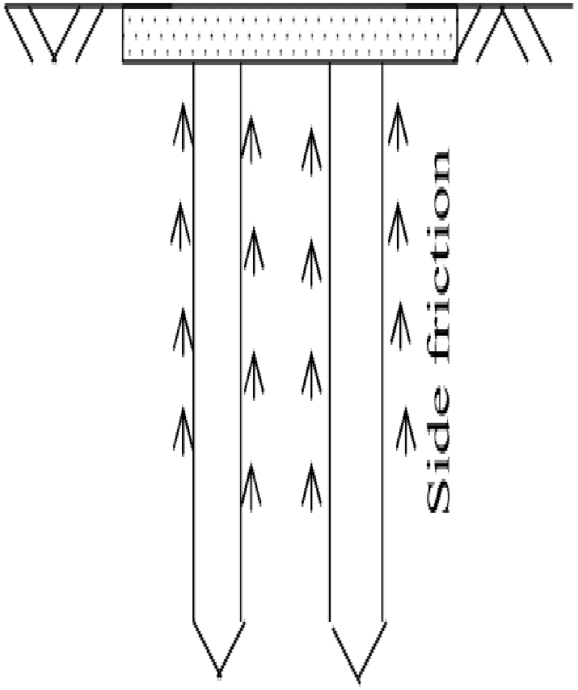


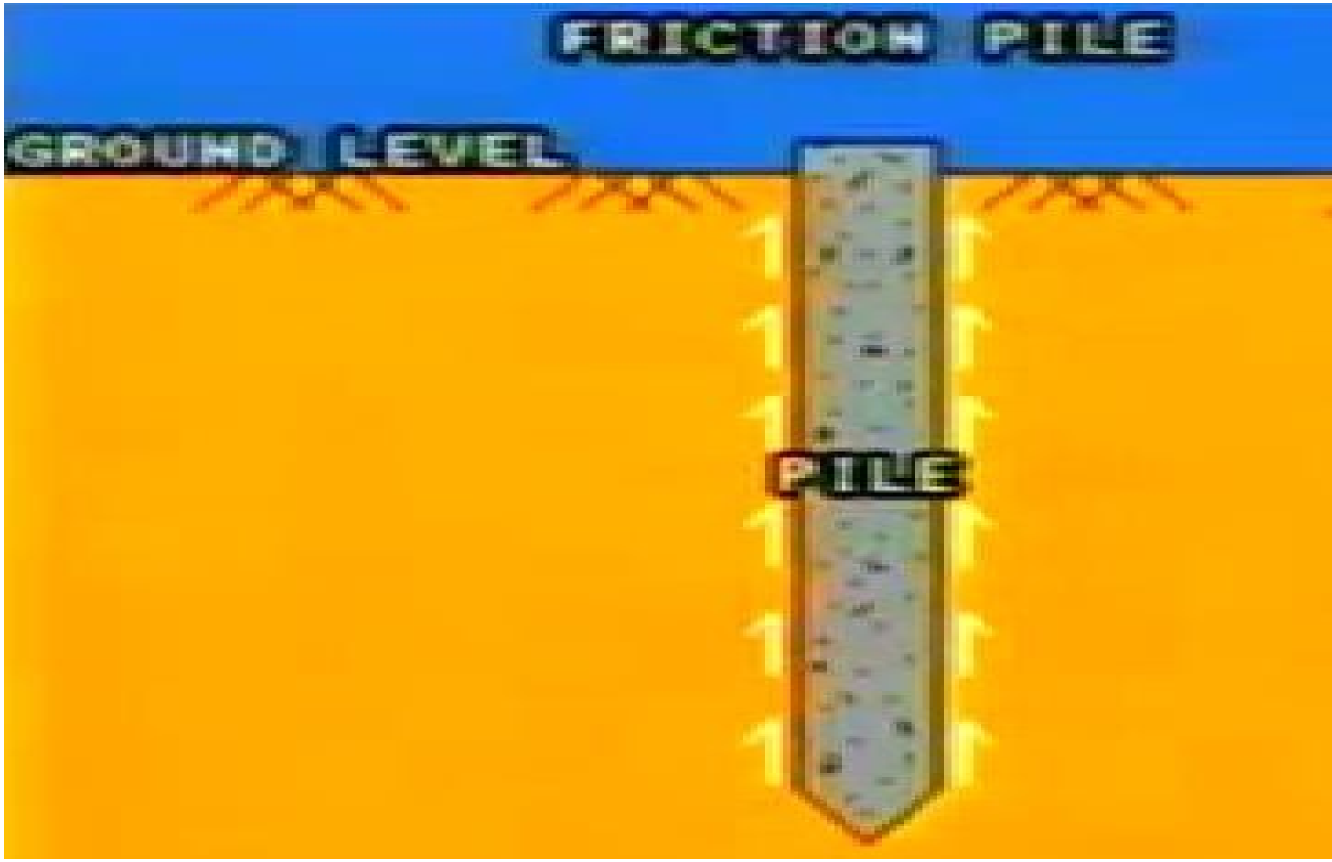



# II) Friction Piles

- Used to transfer load to a depth of a friction – load- carrying material by means of skin friction along the length of the pile

- Generally used in granular soil where the depth of hard stratum is very great. Qu=Qs

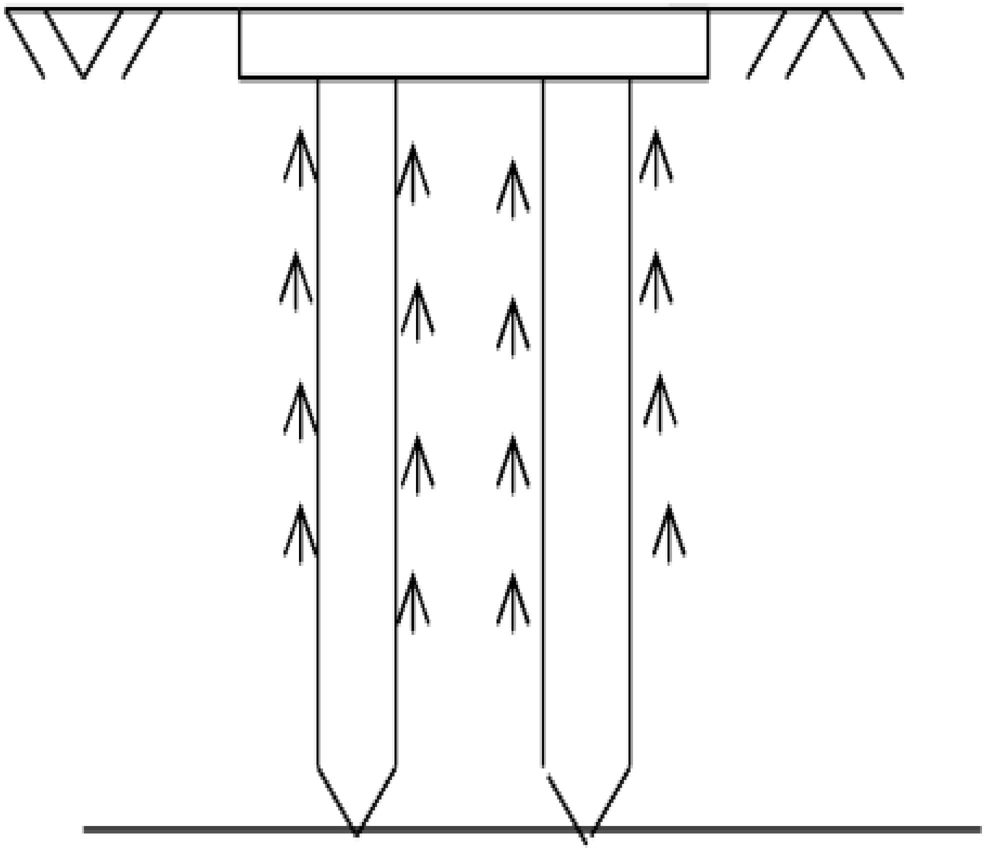


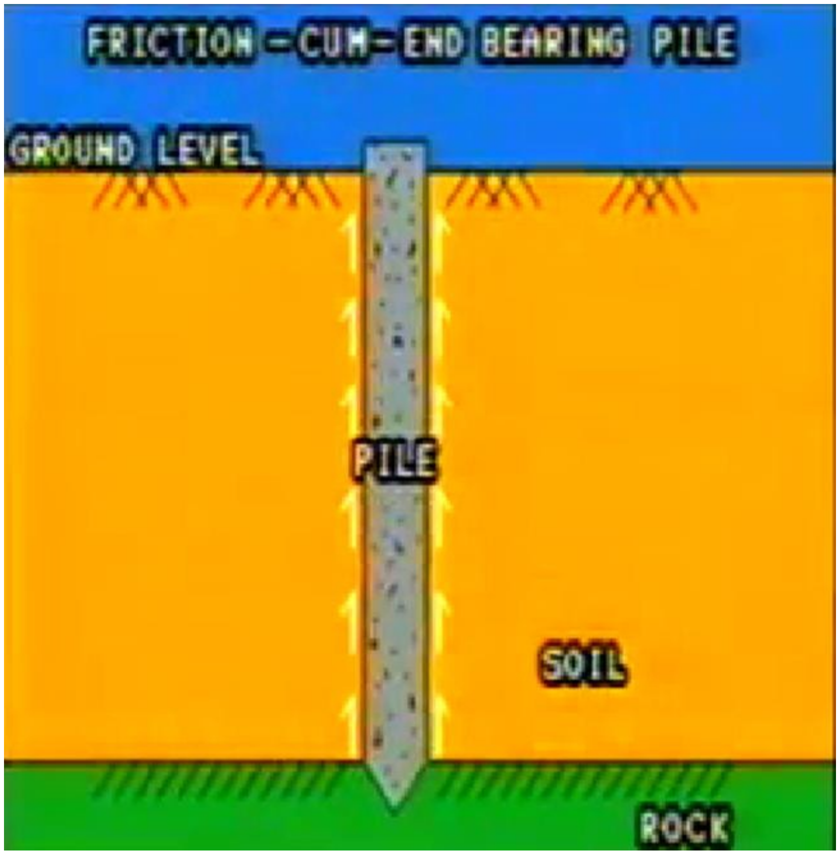



# III. COMBINED PILES

- Some times the super imposed load is transferred both through side friction as well as end bearing.

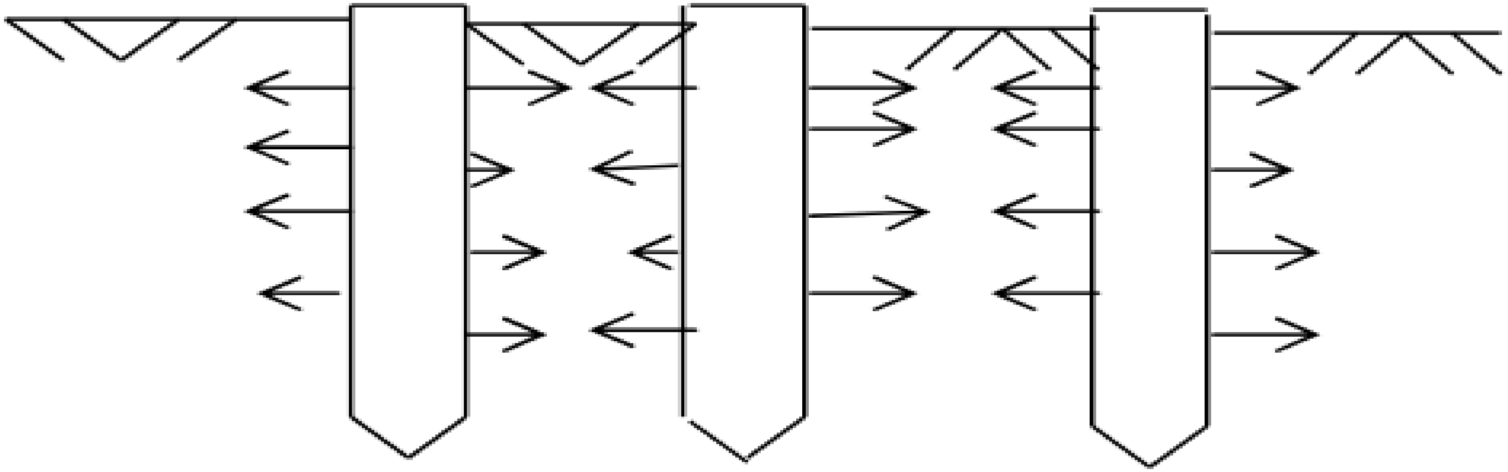
$$Qu=Qp+Qs$$





# IV. Compaction Piles

- Used to compact loose granular soils, thus increasing their bearing capacity.
- The compaction piles themselves do not carry a load. Hence it may be of weaker material (eg timber, bamboo, etc)
- The pile tube, driven to compact the soil, is gradually taken out and sand is filled in its place thus forming a 'sand pile'



# Selection of Appropriate type of Foundation

- ❖ Selection criteria for foundation for buildings depend on two factors, i.e.

- ➢ Factors related to ground (soil) conditions and

- ➢ Factors related to loads from the structure.

# Criteria for selecting suitable foundation based on soil condition

- If the soil close to the surface is capable of supporting structural loads, **shallow foundations can be provided.**
- If the ground close to surface is not capable of supporting structural loads, hard strata is searched for, and in some cases, it may be very deep, like in case of multi-storey buildings, where loads are very high. So, **deep foundations are suitable for such cases.**
- If the ground is not leveled, and has gradient then **step foundation may be preferred.**

# Selection of foundation based on Loads from structure

- In case of low rise building with large span, the extent of loading is relatively modest, so in this case **shallow foundation is preferred**.
- While high-rise building with short span has high loads. Therefore, deep foundation is required in such cases.
- Deep foundation is provided because ground/soil at greater depth are highly compacted(denser).
- In case of framed structure multi-storey building, where loads are concentrated at the point of application, the use of pads and piles are common.

# Geotechnical investigation and methods

# Soil Investigation

- The proper design of civil engineering structures requires adequate information of **surface and subsurface** conditions at the site of the structures.
- A geotechnical investigation is required to obtain **information about the soil conditions below the surface.**
- The process of **determining the layers of natural soil deposits** that will underlie a proposed structure and their physical properties is generally referred to as **site investigation.**
- In general, Site investigation deals with **determining the suitability of the site for the proposed construction.**

cont….

- F**ield and laboratory investigations** required to obtain the necessary data for the soils for proper design and successful construction of any structure at the site are collectively called **soil exploration.**

- Site investigation is done by two methods:

## Surface exploration

- like geologic mapping, surface sample collection, collection of reports, maps, etc.

**Subsurface exploration**: These are two types

**Geotechnical**: Collection of soil/rock samples below the ground surface by different means like coring, making pits, trenching, etc. and in situ and laboratory tests of the samples repossessed.

**Geophysical**: These investigations involve geophysical methods

# 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 ground water table (GWT).
- Obtaining soil&rock samples from the various strata.
- The determination of the engineering properties of the soil & rock strata that affect the performance of the structure.
- Determination of the in-situ properties by performing field tests.

# Procedures of Subsurface Explorations

❖ **Reconnaissance/Inspection Survey**

- Visual inspection to the site to obtain information about:
  - ✓ General topography of the site, possible existence of drainage ditches.
  - ✓ Soil stratification from deep cuts, such as those made for construction of other structures.
  - ✓ Type of vegetation, which may indicate the type of soil.
  - ✓ Type of construction nearby and existence of any cracks in walls or other problems.
- Generally the nature of stratification and physical properties of the soil nearby can also be obtained from any available soil exploration report for existing structures.

# Desk study or collection of primary information

- In this step try to get the following:
  - ✓ Information regarding the type of structure to be built and its general use.

**For example Regarding building:**

- ✓ Appropriate column loads.
- ✓ Spacing of columns.
- ✓ Code requirements etc.

# Sub –Surface Investigation

- Sub soil exploration should enable the engineer to draw the soil profile indicating the sequence of the strata and properties.
- Sub surface investigation like
  - ✓Sinking bore holes
  - ✓Collecting soil samples
  - ✓Laboratory testing and Field testing
- The choice of appropriate testing method is affected by:
  - ✓ Economy
  - ✓ Type of structure
  - ✓ Type of foundation, if predetermined
  - ✓ Type of soil

# Soil data required from subsurface explorations

- **Soil profile-**layer thickness and soil identification
- **Index properties-** water content, Atterberg limits (Liquid Limit, Plastic Limit, Shrinkage Limit) etc.
- Used as supplementary for other
- **Strength & compressibility characteristics , consolidation, cohesiveness, E,v etc.**
- Others (e.g., water table depth)

# SOIL SAMPLING

- Soil sampling is the process of taking samples from the filed or site to determine soil types and in-place characteristics of soil.
- Geotechnical engineers collect soil samples to study about the properties of the strata below the ground surface.

- Samples obtained for testing should be representative of the ground from which they are taken.

- Samples obtained for engineering testing and analysis, in general, are of two main categories:

✓ Disturbed samples

✓ Undisturbed samples

# I) Disturbed Soil Samples

- Disturbed samples are those obtained using equipment that destroy the macro structure of the soil but do not alter its mineralogical composition.
- Disturbed soil samples do not retain the in-situ properties of the soil during the collection process.

- Disturbed samples allow an engineer to determine the geotechnical properties :
  - Moisture content
  - Atterberg limits /liquid limits & plastic limits/
  - Grain size analysis/sieve
  - Compaction tests
  - Specific gravity
  - CBR tests
  - Density index e.t.c

- Disturbed samples are taken from cuttings produced by the drilling process.


# II)Undisturbed Soil Samples

- Undisturbed soil samples retain the structural integrity of the in-situ soil and have a high recovery rate within the sampler.

- It should be noted that the term “undisturbed” soil sample refers to the relative degree of disturbance to the soil’s in-situ properties.

- B/c Collecting a perfectly undisturbed sample is difficult and the samplers may contain a small portion of undisturbed soil at the top and bottom of the sample length.

# Cont….

- Used to determine the Geotechnical properties like Shear strength, Permeability, Compressibility, Fracture patterns, stratification, in situ density, consolidation and other engineering characteristics of soils.
- N.B: Undisturbed samples are generally taken by cutting blocks of soil or rock, or by pushing or driving thin wall tubes into the ground.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20
40
60
80
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# Some of the Laboratory Tests

- Specific Gravity
- Sieve analysis
- Density
- Atterbergs limit
- Compaction
- CBR
- Direct Shear Test
- Tri-axial Test
- Permeability tests
- Unconfined Compression Test
- Consolidation, Vane Shear Test

# The Methods available for Soil Exploration

- ✓ Direct methods ......................Test pits and trenches.
- ✓ Semi-direct methods ..............Borings
- ✓ Indirect methods .....................Soundings or penetration tests and geophysical methods.

# Direct Methods -Test Pit

- Test pits or trenches are open type or accessible exploratory methods.
- Soils can be inspected in their natural condition.
- The necessary soils samples may be obtained by sampling techniques and used for finding strength and engineering properties by laboratory tests.

**Cont.…**

- Test pits are considered suitable only for small depths, up to 3m the cost increases with depth.
- For greater depths, lateral supports or bracing of the excavations will be necessary.
- Test pits are usually made only for supplementing other methods or minor structures

# Semi Direct Methods - Boring

- Making or drilling bore holes into the ground with a view to obtaining soil or rock samples from specified or known depths is called ***'boring'***

- Boring is required:
  - To obtain representative soil and rock samples for laboratory tests.
  - To identify the ground water conditions.

But, we can't observe the natural condition as that of direct method.

## Cont…..

- The most common methods of advancing boreholes are:
  - Auger boring
  - Wash boring
  - Rotary drilling
  - Percussion drilling

# 1. **Auger boring**

- Soil auger is a device that is used for advancing/ digging a bore hole into the ground.
- Augers may be hand operated(**<5m**) or power/machine driven **up to 70m**.
- Hand auger is the simplest method of boring used for small projects in **soft cohesive soils**.
- The power required to rotate the auger depends on the **type and size of Auger** and the **type of soil.**

- The soil samples collected in this manner are **disturbed samples** and can be used for **classification test**.
- **Auger boring** may not be possible in very soft clay or coarse sand because the hole tends to collapse when auger is removed.
- **Auger boring** is convenient in case of partially saturated sand, silts, medium to stiff cohesive soils.

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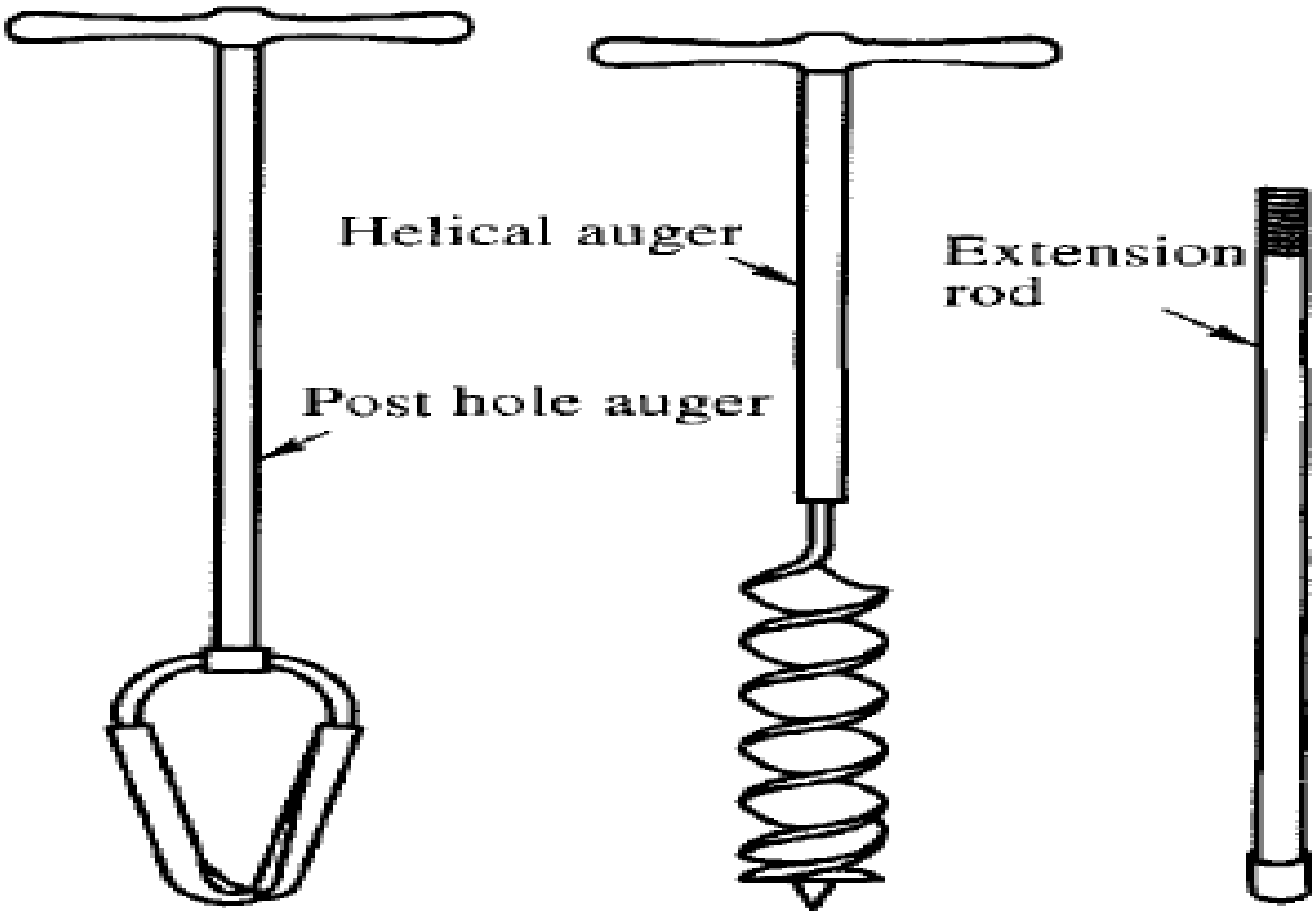



Hand augers

# 2. Wash Boring

- Wash boring is commonly used for exploration below ground water table **for which the auger method is unsuitable.**
- This method may be used in all kinds of soils except those mixed with gravel and boulders.
- Water jet under pressure is forced through the rod and the bit in to the hole.
- This loosens the soil at the lower end and forces the soil water suspension upward along the surface between the rod and the side of the hole.

- This suspension is led to a settling tank where the soil particles settle while the water overflows into a sump.
- The water collected in the sump is used for circulation again.

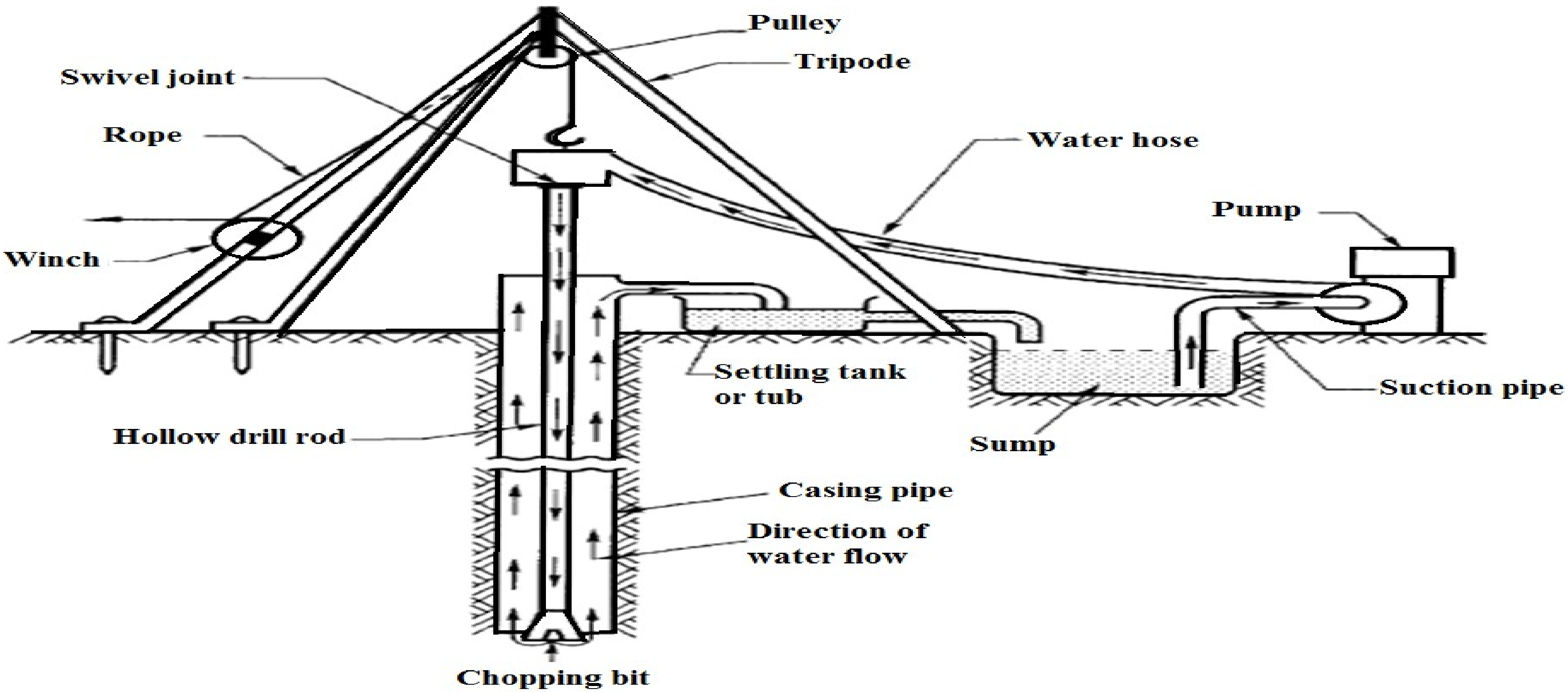



Fig. Wash boring

Cont….

- The soil particles collected represented a very disturbed sample and is not very useful for the evaluation of the engineering properties.
- Wash borings are primarily used for advancing bore holes; whenever a soil sample is required, the cutting bit is to be replaced by a sampler.
- The change of the rate of progress and change of color of wash water indicate changes in soil strata.

# 3. Rotary drilling

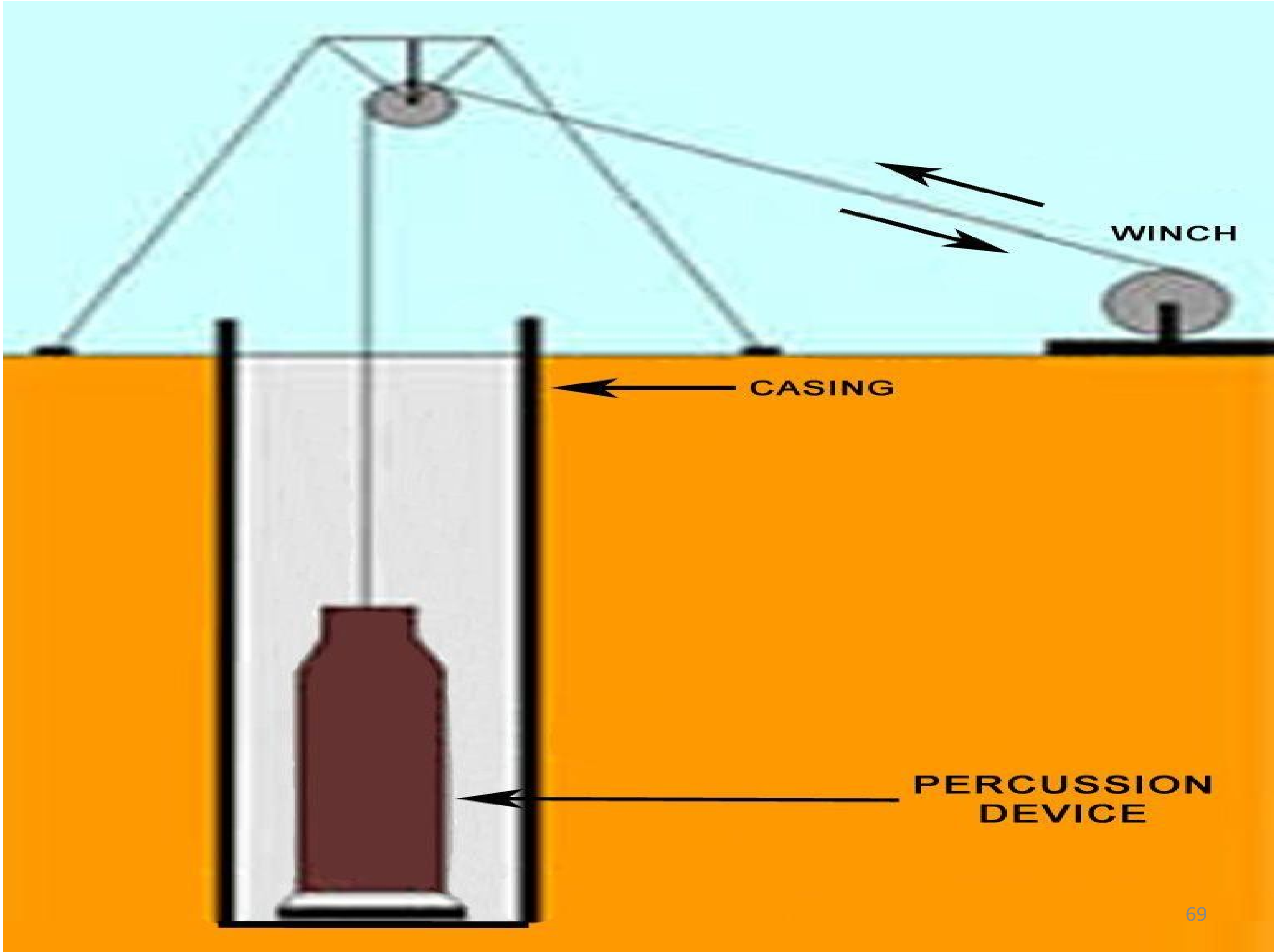
- This method is Primarily intended for **investigation in rock**, but also used in soils.
- There are two forms of rotary drilling, open-hole drilling and core drilling.
- Open- hole drilling, which is generally used in soils and weak rock, just for advancing the hole
- The drilling rods can then be removed to allow tube samples to be taken or in-situ tests to be carried out.

## Cont..

- In core drilling, which is used in rocks and hard clays, the Diamond or Tungsten carbide bit cuts an annular hole in the material and an intact core enters the barrel, to be removed as a sample.
- The advantage of rotary drilling in soils is that progress is much **faster than with other investigation methods.**

# 4. Percussion drilling

- Percussion Drilling is the process of making boreholes **by striking the soil then removing it**.
- The tools are repeatedly dropped down the borehole while suspended by wire from the power winch.
- Water is circulated to bring the soil cuttings to the ground surface.
- A casing and a pump are required to circulate the water.


WINCH
CASING
PERCUSSION DEVICE

# Number of borings

| Approximate spacing of bore holes | |
|---|---|
| Type of project | Spacing in m |
| Multistory building | 10-30 |
| Industrial plant | 20-60 |
| Highway | 250-500 |
| Residential subdivision | 250-500 |
| Dams and dykes | 40-80 |

# Depth of Boring

| Building width (m) | Number of Stories | | | | |
|---|---|---|---|---|---|
| | 1 | 2 | 4 | 8 | 16 |
| | Boring Depth (m) | | | | |
| 30.5 | 3.4 | 6.1 | 10.1 | 16.2 | 24.1 |
| 61.0 | 3.7 | 6.7 | 12.5 | 20.7 | 32.9 |
| 122.0 | 3.7 | 7.0 | 13.7 | 24.7 | 41.5 |

# Field tests

- We used them when it is difficult to obtain "undisturbed" samples.

## Advantage:

- Testing on natural soil under undisturbed conditions

## Disadvantage:

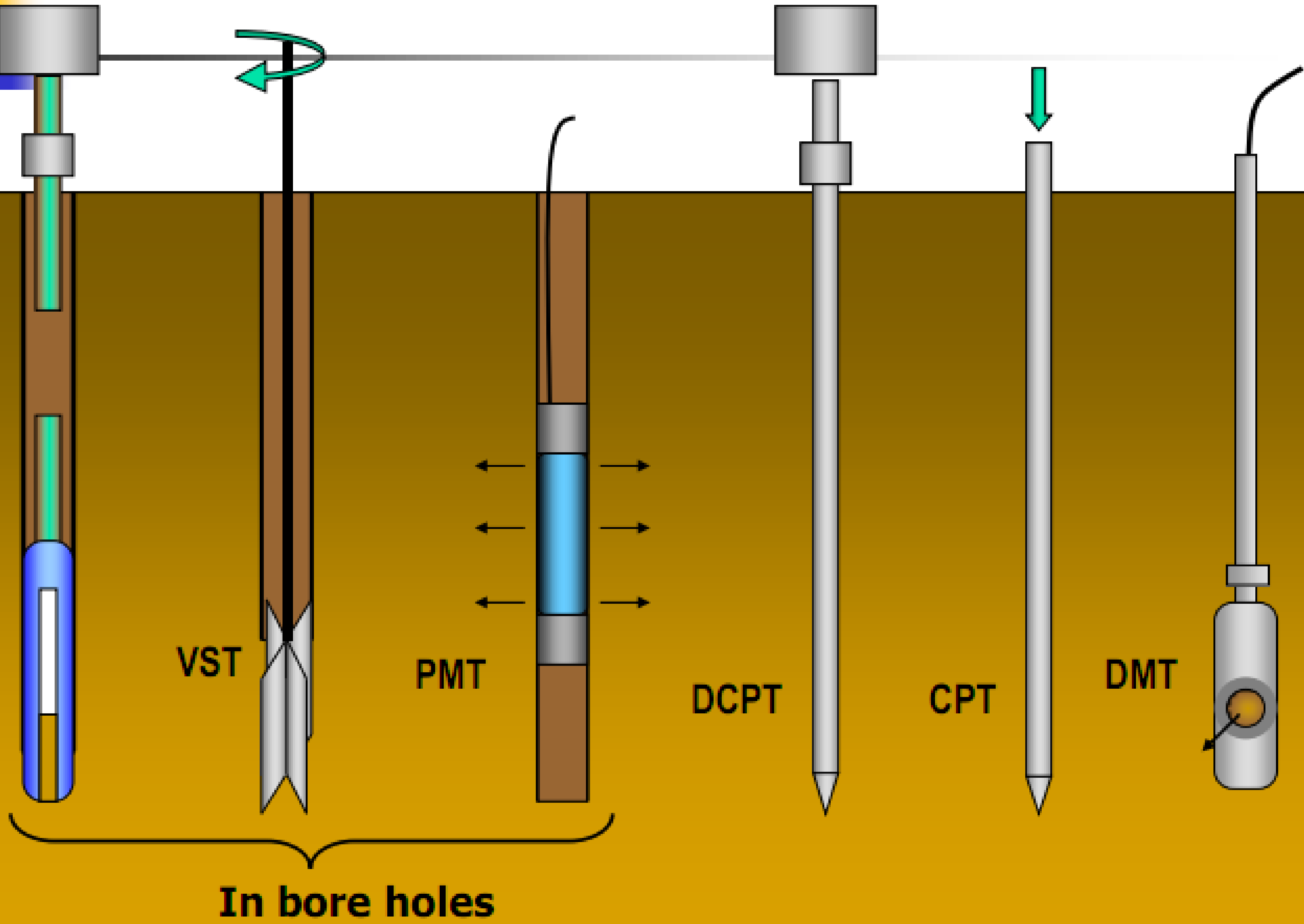
- Testing conditions are not controlled
- Time dependent phenomenon are difficult to control due to large scale
- Measurements/instrumentation is tricky and rather a difficult task

Cont….

# In-situ shear strength tests(Filed tests)

- Standard Penetration Test (SPT)
- Cone Penetration Test (CPT)
- Vane Shear Test (VST)
- Geophysical methods
- Pressure meter Test (PMT)
- Dynamic Cone Penetration Test (DCPT)
- Dilatometer Test (DMT)
- Plate Load Test

# Common In Situ Testing Devices



# 1. STANDARD PENETRATION TEST (SPT)

- The standard penetration test (SPT) is a dynamic test and is a measure of the density of the soil.
- The SPT is carried out in a borehole by lowering the **split spoon sampler** of about 650 mm length, 50 mm external diameter, and 35 mm internal diameter, and driving it using repeated blows by a freely dropped hammer at falling height of 765 mm.

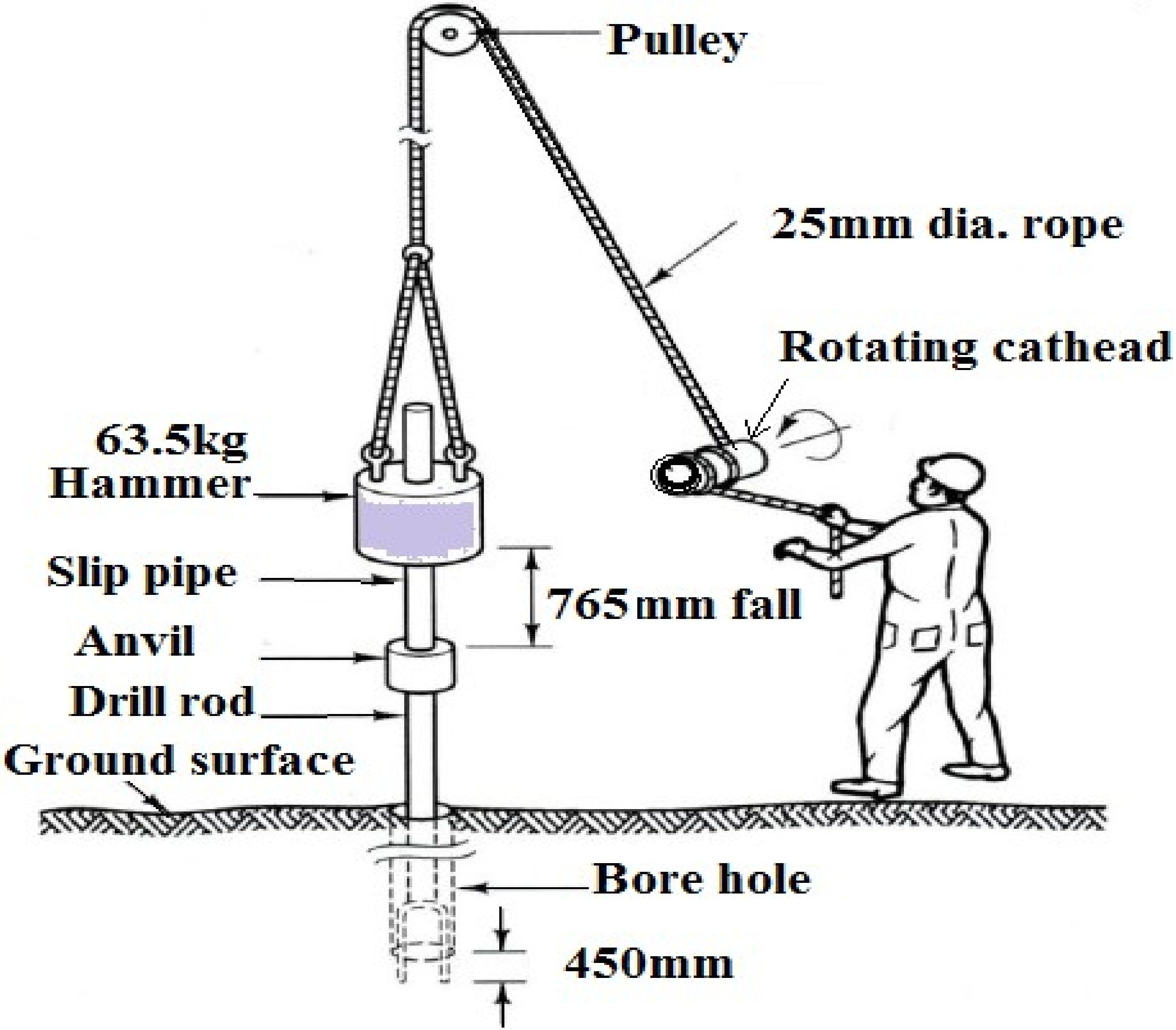
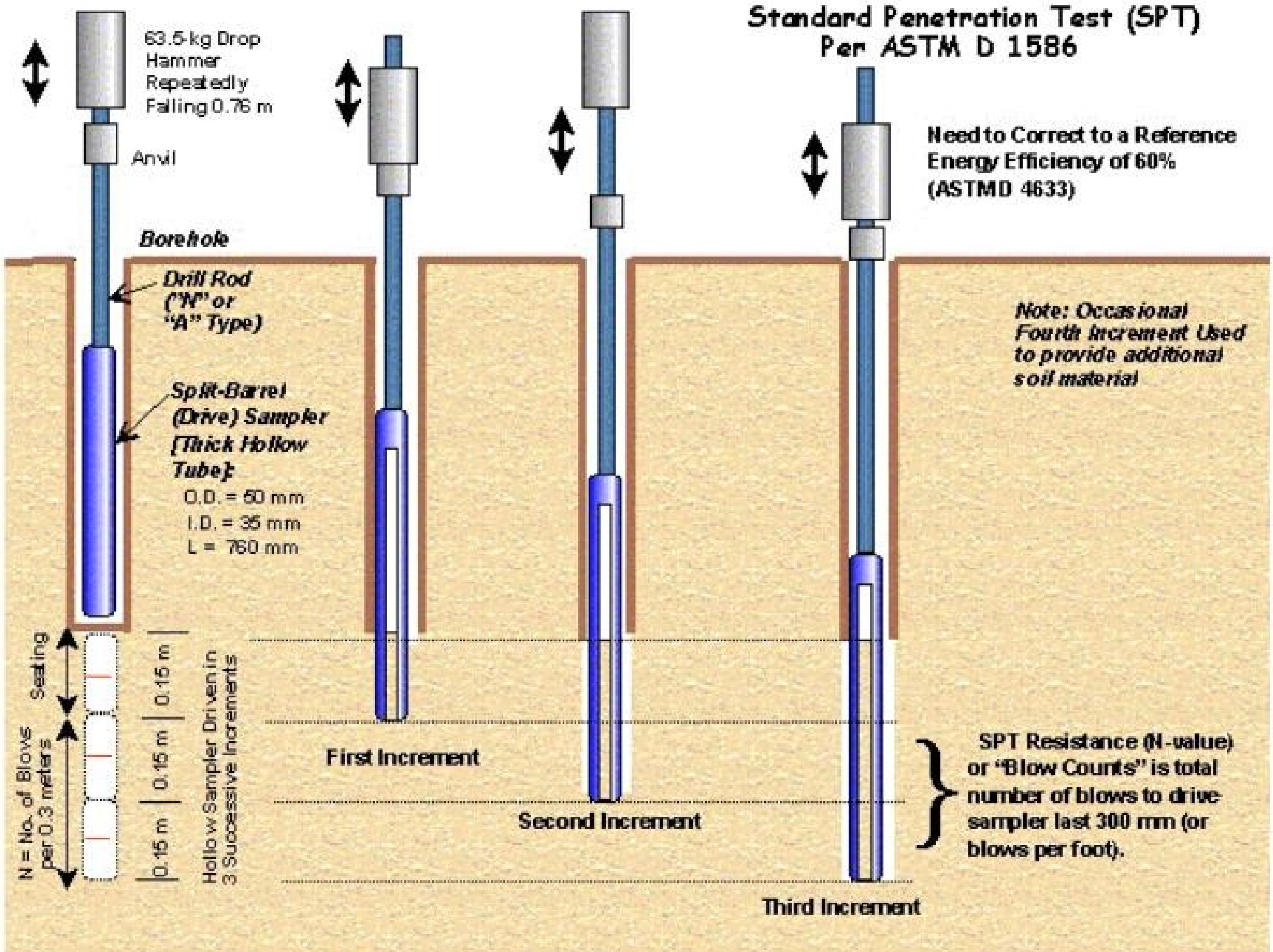



***Fig.Standard penetration test arrangment***


Standard Penetration Test (SPT)
Per ASTM D 1586
63.5-kg Drop Hammer Repeatedly Falling 0.76 m
Anvil
Need to Correct to a Reference Energy Efficiency of 60% (ASTMD 4633)
Borehole
Drill Rod ("N" or "A" Type)
Split-Barrel (Drive) Sampler [Thick Hollow Tube]:
O.D. = 50 mm
I.D. = 35 mm
L = 760 mm
Note: Occasional Fourth Increment Used to provide additional soil material
Seating
N = No. of Blows per 0.3 meters
0.15 m
0.15 m
0.15 m
Hollow Sampler Driven in 3 Successive Increments
First Increment
Second Increment
Third Increment
SPT Resistance (N-value) or "Blow Counts" is total number of blows to drive sampler last 300 mm (or blows per foot).

# Procedure

- The bore hole is advanced to desired depth and bottom is cleaned.
- Split spoon sampler is attached to a drill rod and rested on bore hole bottom.
- Driving mass is dropped onto the drill rod repeatedly and the sampler is driven into soil for a distance of 450 mm.
- The number of blow for each 150 mm penetration are recorded
- The number of blows for the **last two 150 mm** penetration are added together and reported as N-value for the depth of bore hole.

- The split spoon sampler is recovered, and sample is collected from split tub so as to preserve moisture content and sent to the laboratory for further analysis.
- SPT is repeated at every **750 mm or 1500 mm interval for** larger depths.
- NB : Under the following conditions the penetration is referred to as refusal and test is halted/stopped.

a) 50 blows are required for any 150 mm penetration

b)100 blows are required for last 300 mm penetration

c) 10 successive blows produce no advancement

# Precautions during SPT

- ➢ The height of the free fall **Must be 750 mm**
- ➢ The fall of hammer must be:
  - ✓Free
  - ✓Frictionless and Vertical
  - ✓Cutting shoe of the sampler must be free from **wear & tear**
  - ✓The bottom of the bore hole must be cleaned to collect **undisturbed** sample

- The blow count (N) may be corrected by field conditions such as,
  - ✓ Energy used for driving the rod into the soil ($E_m$),
  - ✓ Variations in the test apparatus ($C_s$ and $C_R$),
  - ✓ Size of drilling hole ($C_B$)
- The values of Em, Cs, CR, and CB depend on the SPT equipment.
- Many of the correlations developed based on hammer that have an efficiency of 60%, the results of other hammer should be corrected to this efficiency factor. Thus

$$N_{60} = \frac{E_m C_B C_S C_R}{0.6} N$$

# Cont.…

$C_B$ = Bore hole diameter correction

$C_S$ =Sampler correction from table

$C_R$ = Rod length correction

Em= W*h

Where W is weight or mass of hammer and h is height of fall

❑ Relationship between the N value and the consistency and the undrained shear strength of cohesive soil was also developed as:

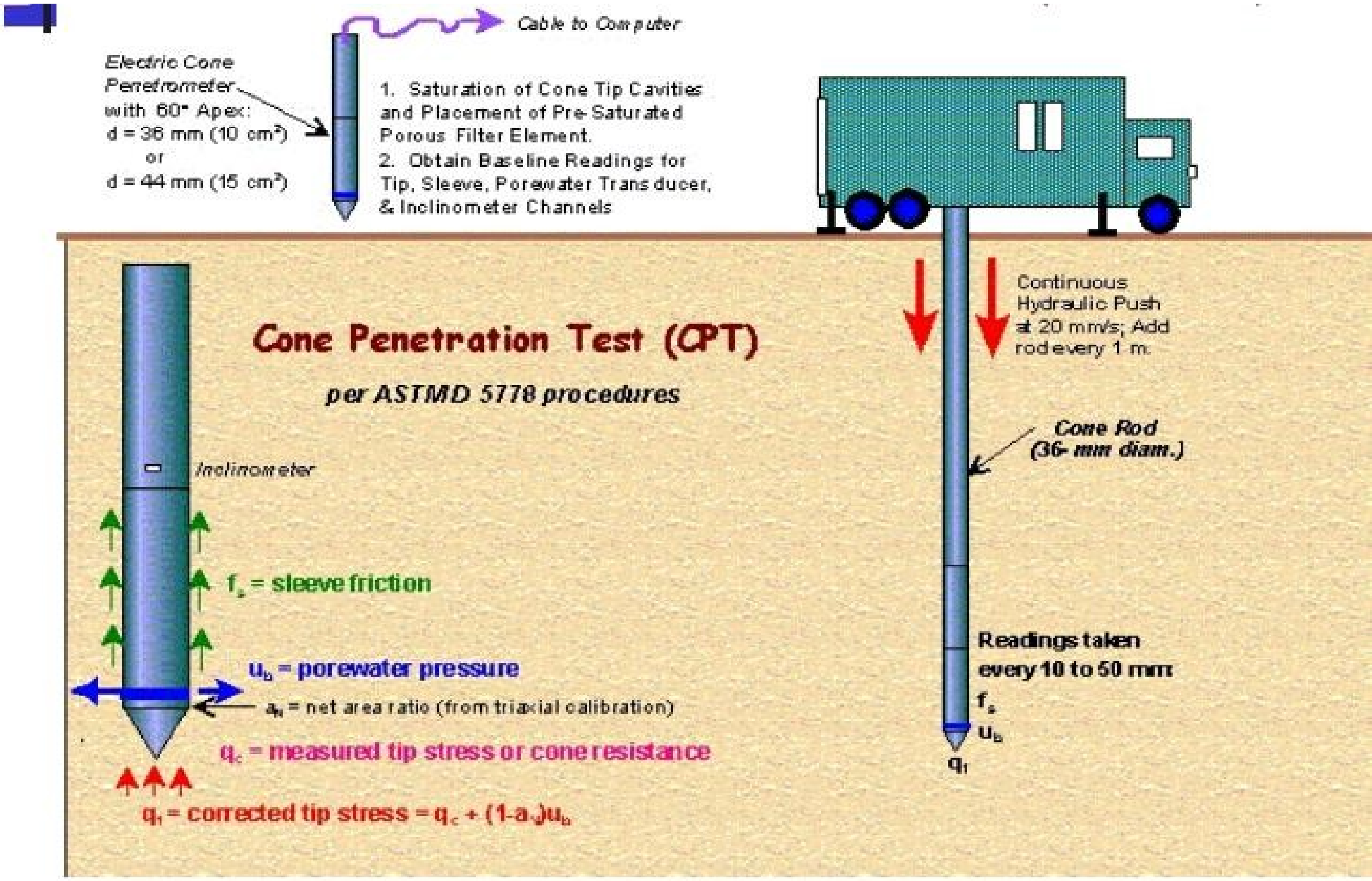
| SPT N (blows/300 mm) | Relative Density (%) | Internal friction angle | State of packing |
|---|---|---|---|
| 4 | 20 | 30 | Very loose |
| 4 – 10 | 20 – 40 | 30 – 35 | Loose |
| 10 – 30 | 40 – 60 | 35 – 40 | Compact |
| 30 – 50 | 60 – 80 | 40 – 45 | Dense |
| > 50 | > 80 | 45 | Very dense |

not corrected for overburden

$$c_u = 6.25.N \quad \text{in kPa}$$

| N | $c_u$ (kPa) | consistency | visual identification |
|---|---|---|---|
| 0-2 | 0 - 12 | very soft | Thumb can penetrate > 25 mm |
| 2-4 | 12-25 | soft | Thumb can penetrate 25 mm |
| 4-8 | 25-50 | medium | Thumb penetrates with moderate effort |
| 8-15 | 50-100 | stiff | Thumb will indent 8 mm |
| 15-30 | 100-200 | very stiff | Can indent with thumb nail; not thumb |
| >30 | >200 | hard | Cannot indent even with thumb nail |

# 2. Cone Penetration Test(CPT)



- Cone penetration test carried out by **mechanically or hydraulically pushing a cone into the ground at a constant speed (10-25mm/sec) while measuring the tip resistance and friction.**

- The cone penetration test measures **the tip resistance** (designated as qc and the friction resistance qf.

- Friction ratio (fr) represents the ratio between the friction resistance and the cone resistance in percentage which is very useful in the estimation of soil type.

$$fr=qf/qc$$

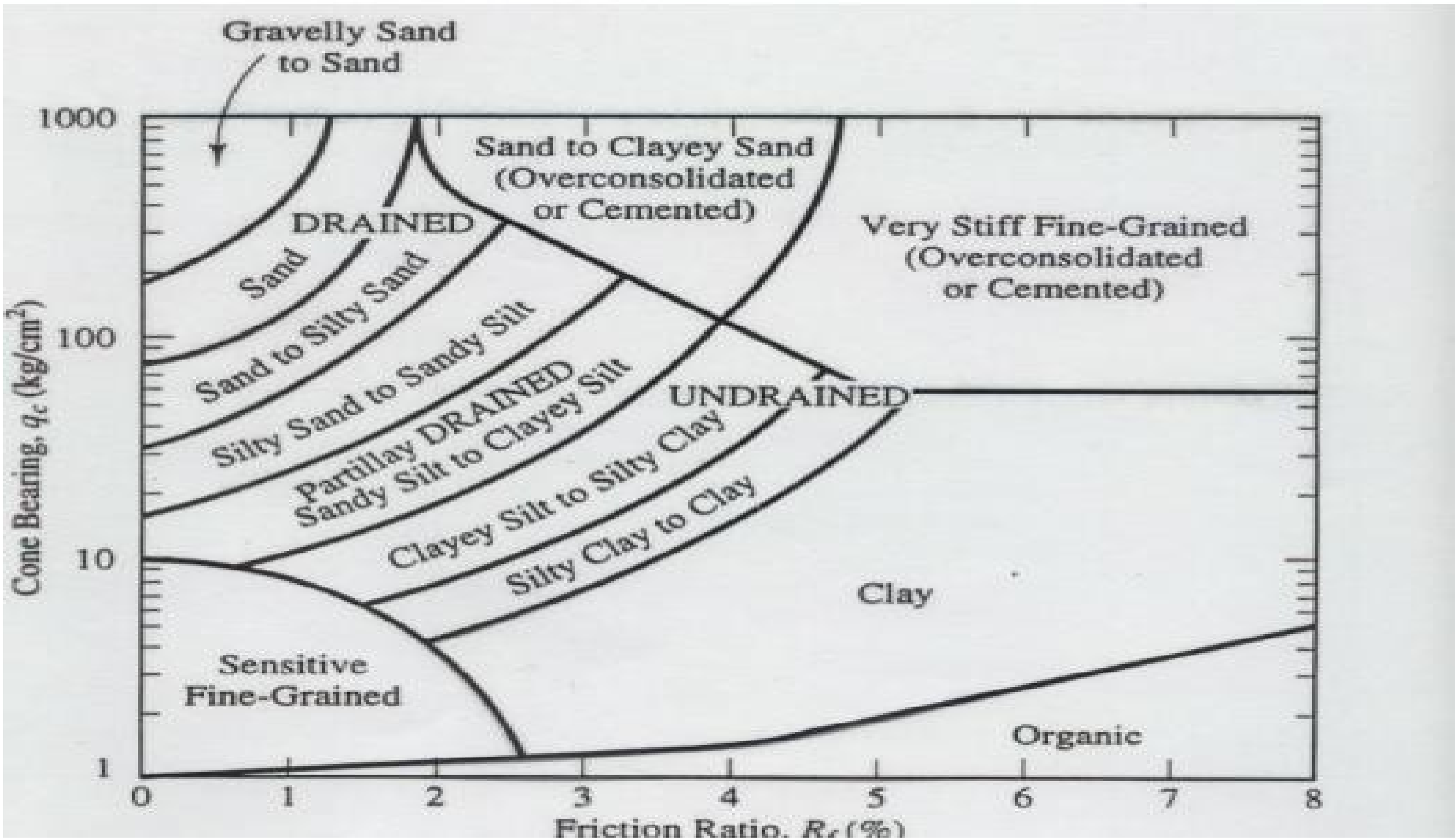
- The cone penetration resistance can be related to the **undrained shear strength** ($c_u$) of cohesive soil by the following equation:

$$c_u = \frac{q_c - \sigma'_0}{N_k}$$

In which $\sigma'_0$ is the overburden pressure and $N_k$ is the cone factor which ranges from 15 to 20 depending on the type cone used.

| Type of clay | Cone factor |
|---|---|
| Normally consolidated | 11 to 19 |
| Overconsolidated | |
| At shallow depths | 15 to 20 |
| At deep depths | 12 to 18 |

- Another correlation based on CPT data Modulus of elasticity is equal to **2.5 – 3.5 $q_c$.** Other correlations relate the results of cone penetration test with the **N value** from Standard penetration test.



# Advantages of CPT over SPT

- Provides much better reliability

- versatility

# Disadvantages over SPT

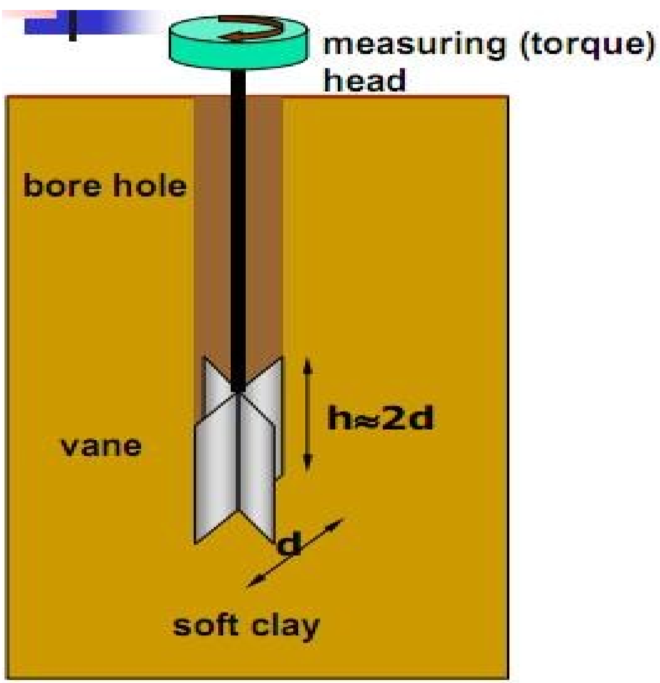
- Doesn’t give a sample

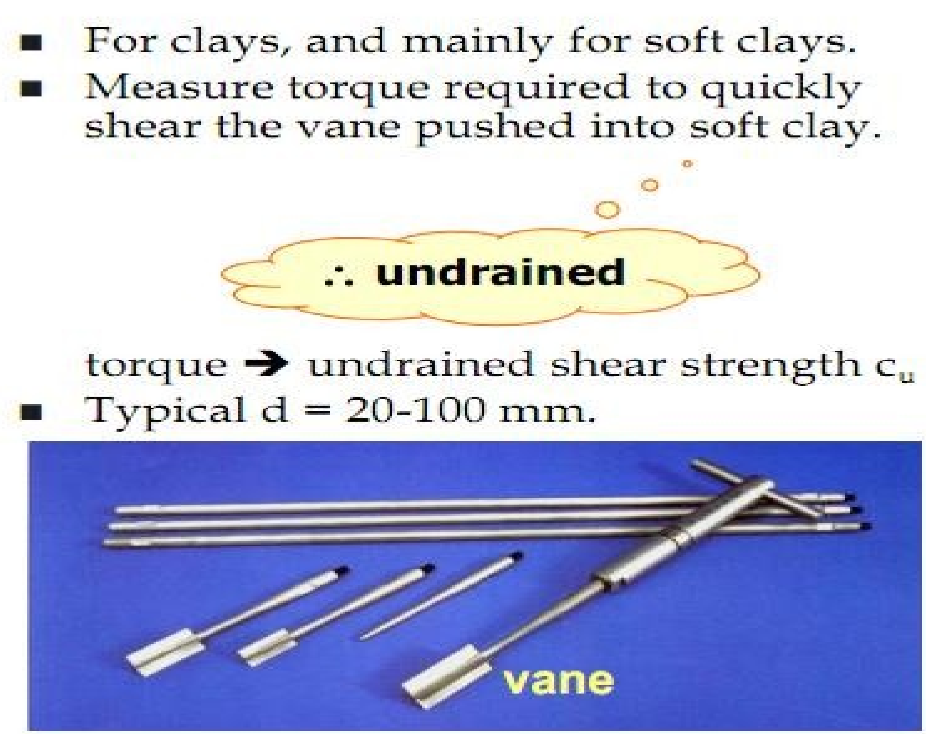
- Will not work with soil with gravel

- Need to assemble a special rig

# 3. VANE SHEAR TEST (VST)

- Vane shear test is commonly used to measure the shear strength and sensitivity of clay. The equipment consists of four-bladed rectangular vane, rotating rod, and measuring device.



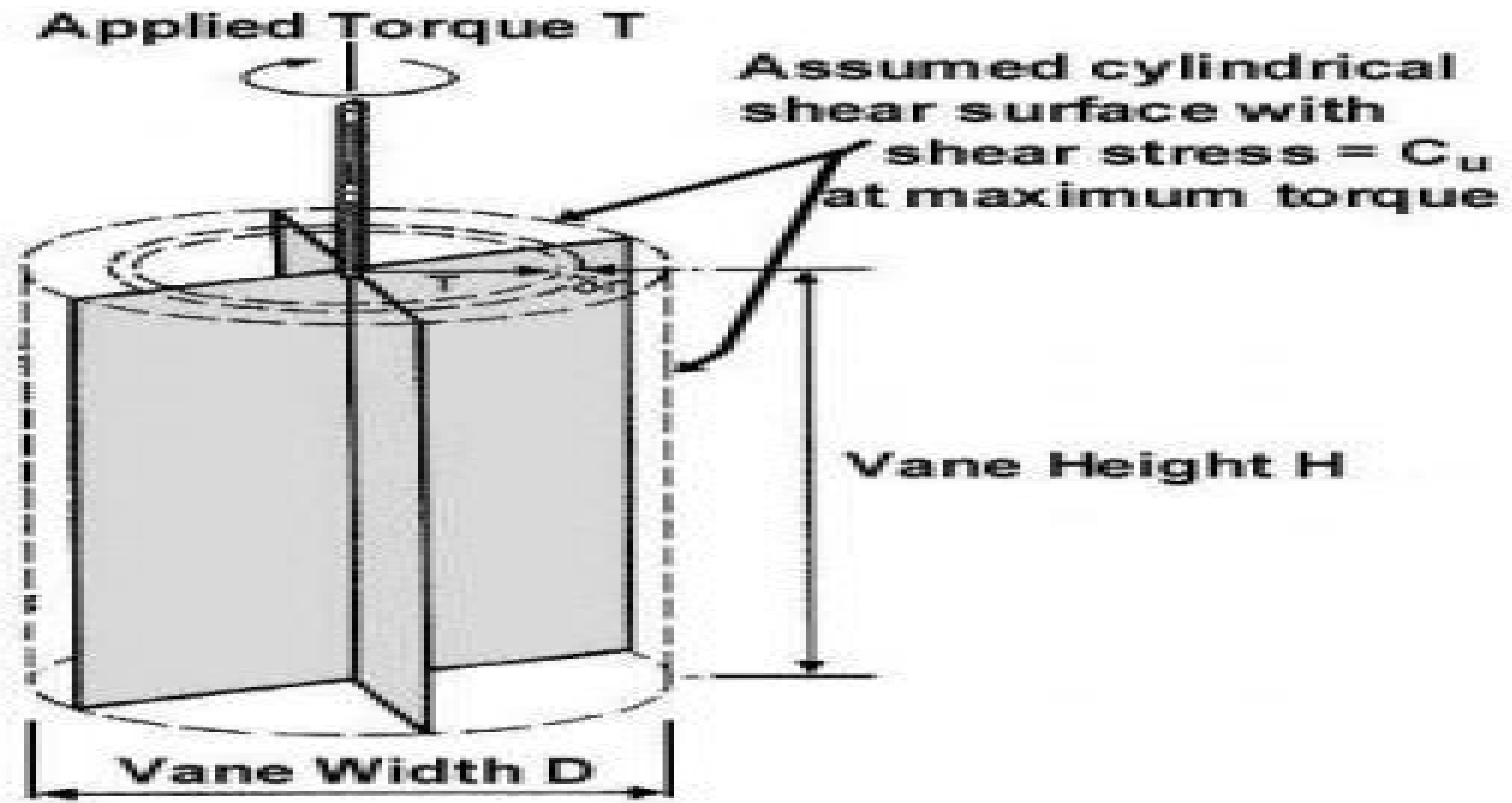
**Cont.…**

- The test is carried out in a borehole or directly pushing the vane into the ground.
- The vane rod is then rotated at a rate of 6-12 degree/min, while the torque is read at interval of 30 seconds.
- After maximum torque is achieved, the vane is rotated at a higher rate to obtain the remolded strength of the soils
- Measure parameters include the peak torque ($T_{peak}$), and residual torque ($T_{res}$).

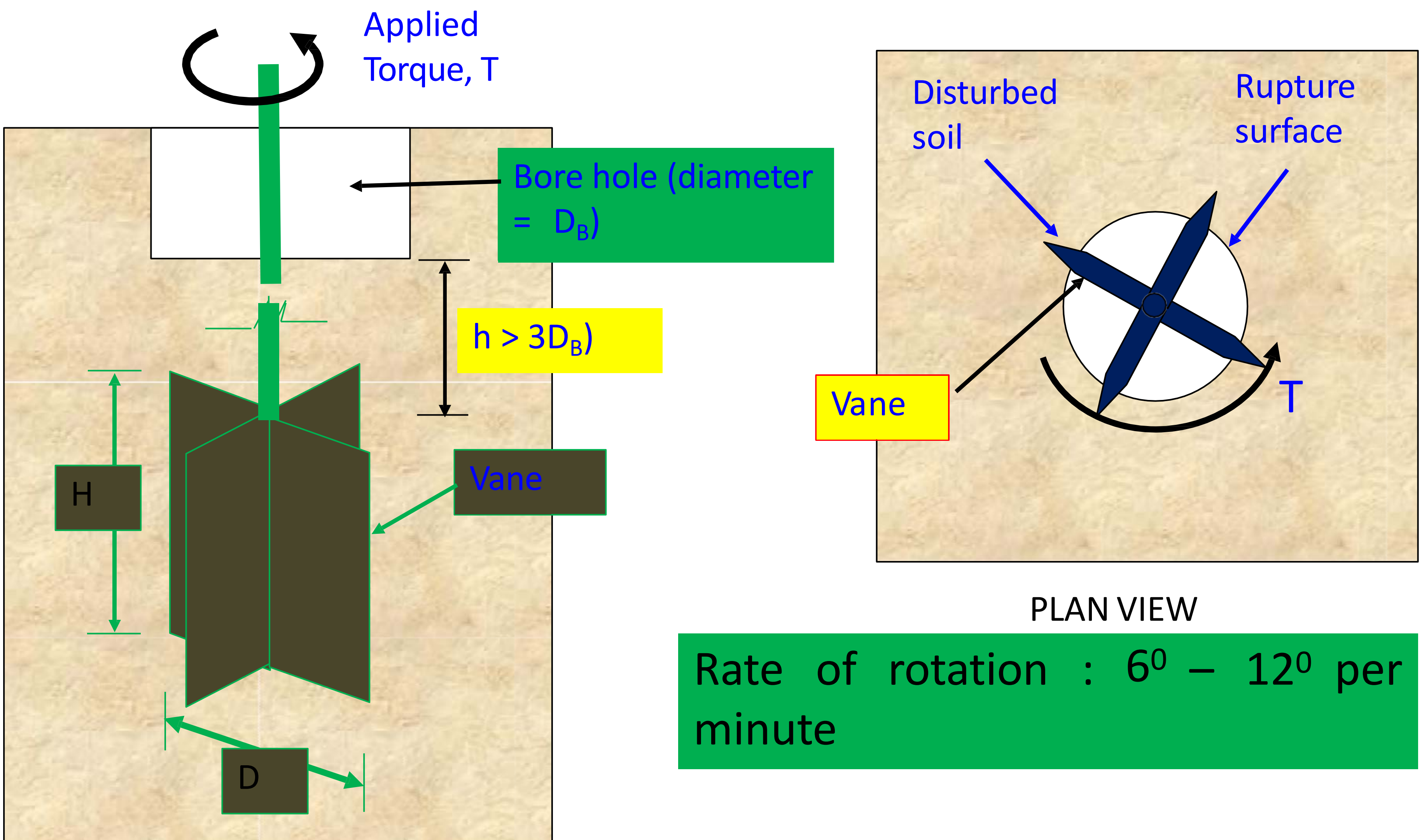
**Vane test** is done in **two** ways:

## 1.Field vane test

## 2.Laboratory vane test



# GENERAL PROCEDURE OF VANE SHEAR TEST ON FIELD




PLAN VIEW

Rate of rotation : $6^0 - 12^0$ per minute

# How to Measure Torque on Field

Cont….

- The theoretical formula for relating the results of vane shear test to the shear strength parameters of the soil is :

$$c_u = \frac{T}{\pi\left[\left(\frac{d^2H}{2}\right)+\left(\frac{d^3}{6}\right)\right]}$$

- Where: cu is the undrained shear strength of soil, T is the maximum torque, d is the diameter of the vane, and h is the height of the vane.

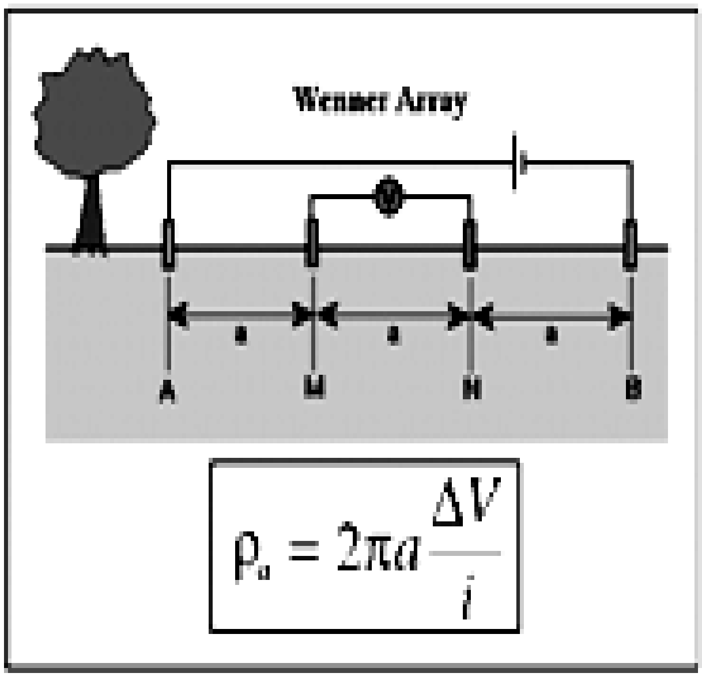
**For H = 2.D**

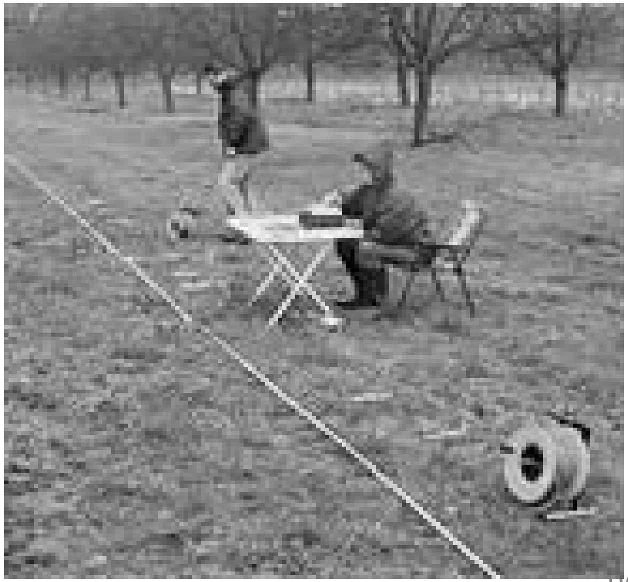
$$c_u = 0.273\,\frac{T}{D^3}$$

# Geophysical methods

❖ Geophysical methods: Indicate general boundaries of drastically dissimilar layers

- ➢ Seismic refraction method
- ➢ Electrical resistivity method (Wenner and schulumberger array).



- Geophysics, as the name indicates is the study of the earth by making use of the established principles of physics.
- Geophysical investigations involve methods of study made on the surface to acquire the subsurface details
- Some of the advantages of geophysical study are:
  - These investigations provide quick, inexpensive easy and fairly reliable information, though indirect
  - Their relevance lie in the concrete and cost-effective benefits it delivers

- The geophysical methods provide nondestructive in situ measurements
- Large areas can be investigated in a reasonably short period
- Field work is no laborious as the instruments used in the field are simple, mostly portable and can be operated easily
- Cost-effectiveness: It does not require chemicals and such

# Some important Geophysical applications in Engineering

- ✓Depth to bedrock or thickness of overburden
- ✓Bed rock profiling
- ✓Fault and fracture mapping
- ✓Mine working locating
- ✓Underground utility locating
- ✓Estimation of aquifer conditions
- ✓Location of GWT and
- ✓Estimating the density of soils etc..

# Foundation on expansive soils

- In black cotton soil and other expansive type of soils, building often cracks due to relative ground movements.
- This is caused by alternate swelling and shrinkage of the soil due to changes in its moisture content.
- They Expand greatly when saturated with water so increase volume
- Expansion is largely due to chemical attraction of water molecules between layers of clay minerals.
- Expansive clays can also dry out and decrease in volume

## Cont…

- Clay Minerals has the following chemical compositions

1. Kaolinite

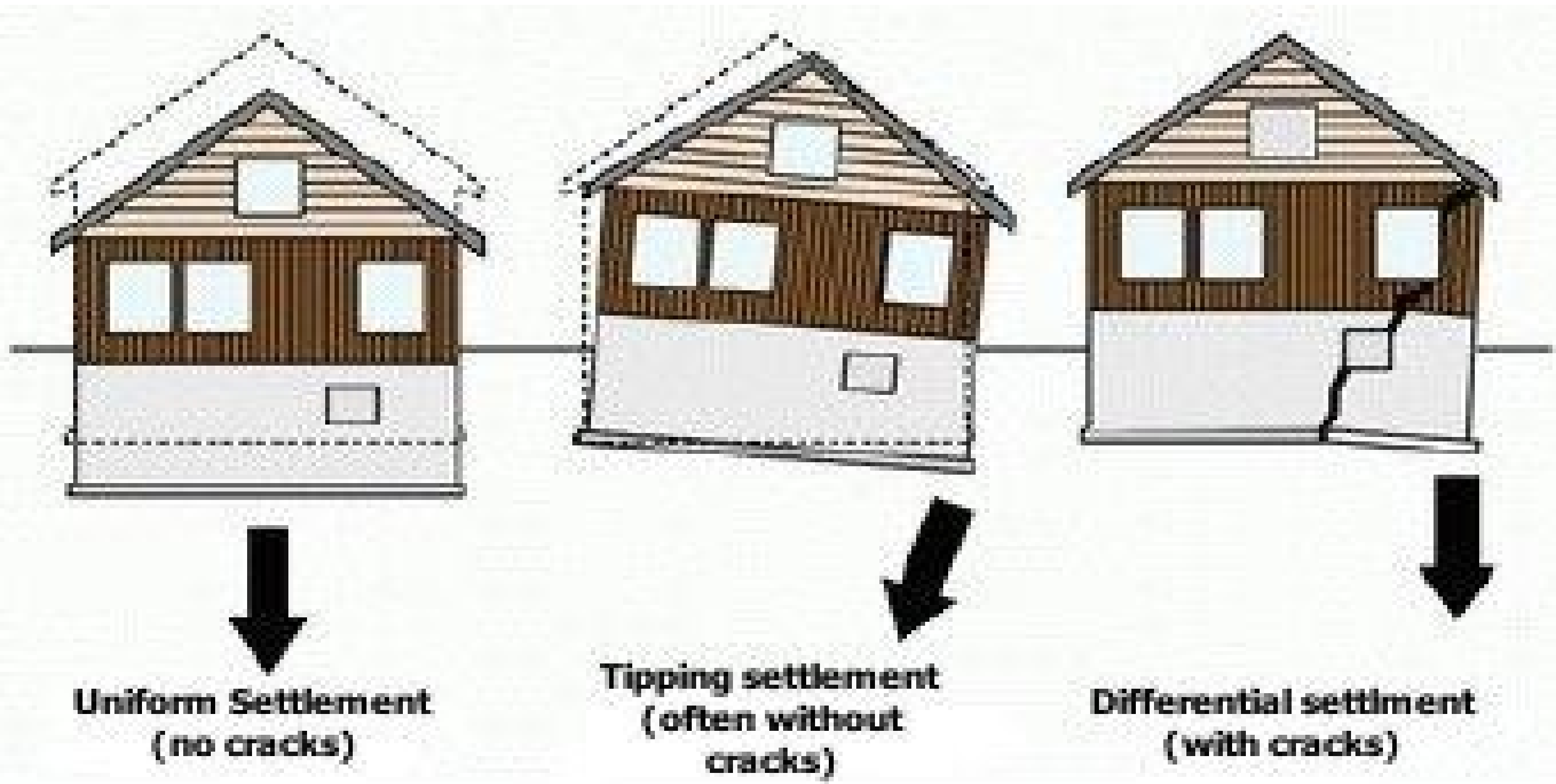
2. Illite

3. Montmorillonite

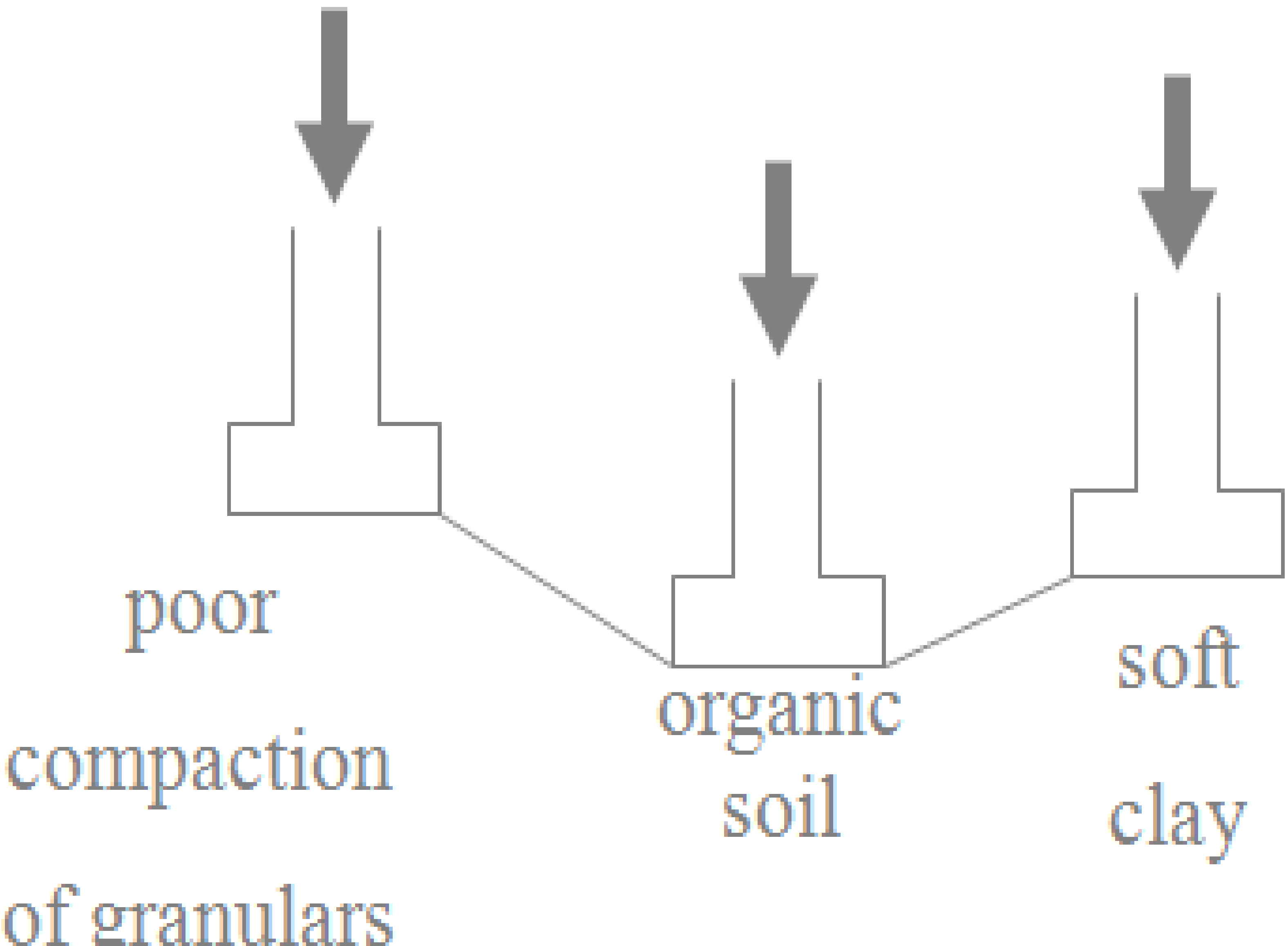
- A potentially expansive soil is not necessarily damaging **unless it is subjected to moisture changes**, which may result from seasonal climatic changes.

# Engineering problems

- Foundations are affected by engineering properties and characteristic of the soil (swelling and shrinking of soil)



# Differential Settlement



Cont….

# Methods of Identifying and preventing damages Due to Expansive soils

- ❖ When expansive soils are exposed to change in moisture or in load equilibrium any structure build on them will be subject to additional stresses and strains.
- ❖ Specially the uplift pressure will subject the buildings to either uniform but in most cases to differential uplift.
- ❖ The additional stresses and strains are often displayed as cracks on buildings, roads and pavements.
- ❖ Light structures are the most exposed to damage as they do not have the necessary counter weight to contain the uplift pressure .

# Cont…

- ❖Heavy buildings tend to exert more counter pressure
- ❖In buildings cracks due to expansive soils may often displayed as:
  - ✓Vertical and diagonal cracks
  - ✓ Horizontal separation
  - ✓Stacking of doors and windows etc.

# Some Controlling Mechanisms

- ✓ Moisture content control through drainage
- ✓ Compaction control
- ✓ Soil Replacement
- ✓ Dead load pressure more than swelling pressure
- ✓ Mat Foundation Flexible or rigid mat may be used
- ✓ Drilled pile /Pier Foundation if expansive is too thick
- ✓ Enlarged base

# BEARING CAPACITY OF Shallow Foundations

*By: Gamachis A.*

*2020*

# *Objective of the chapter*

- When you complete this chapter you should be able to:
  - ✓ Calculate the bearing capacity of soils.
  - ✓ Differentiate the different methods of bearing capacity equation.
  - ✓ Determine the bearing capacity for eccentrically loaded footings.

# 1.1 Introduction

- A foundation, is a structure designed to transfer loads from a superstructure to the soil underneath the superstructure.
- In general, foundations are categorized into two groups, namely,
    1. Shallow foundations and
    2. Deep foundations.

## 1.2 Some Basic Definition

**1) Ultimate Bearing Capacity ($q_u$)** :

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

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

It is the net increase in pressure at the base of foundation that cause shear failure of the soil.

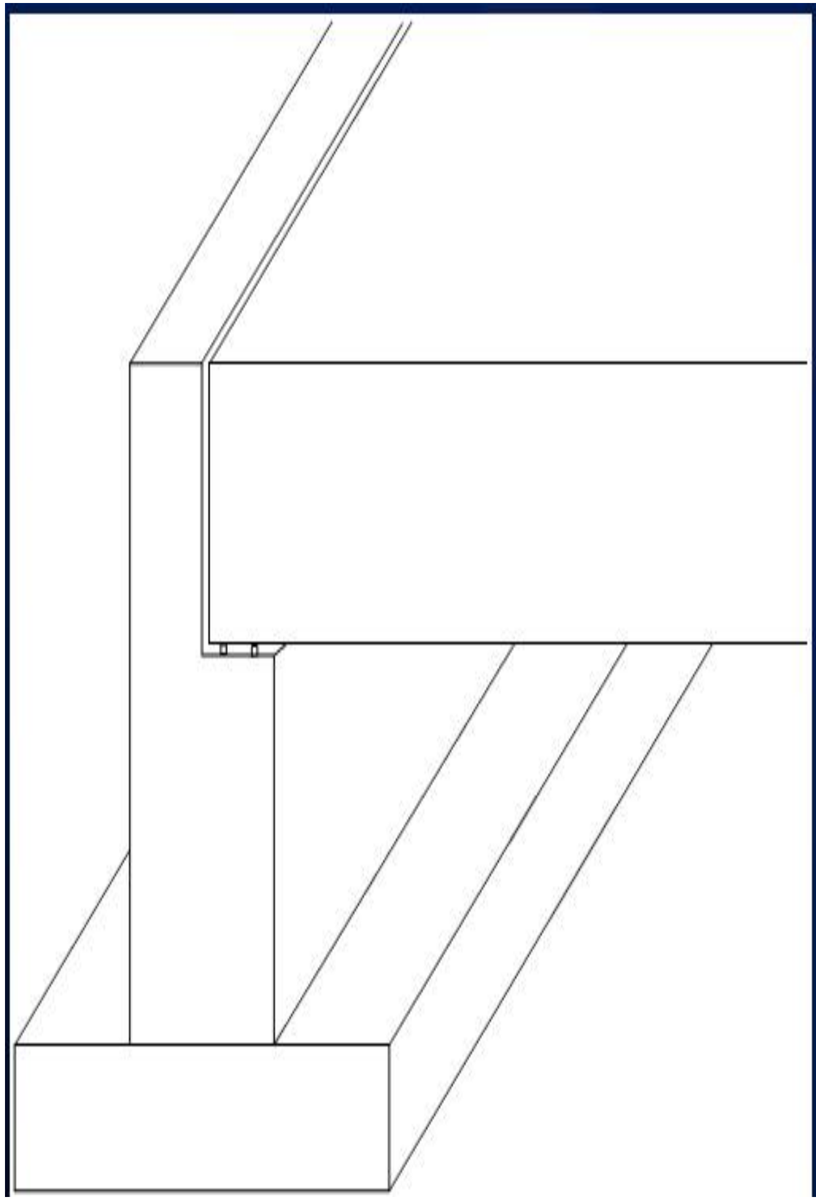
$\Rightarrow$ Thus, $q_{nu} = q_u - \gamma D_f$ (overburden pressure)

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

It is the net soil pressure which can be safely applied to the soil considering only shear failure.

$\Rightarrow$ Thus, $\mathbf{q_{ns} = q_{nu} / [FS]}$

# Strip footing



# Cont'd...

4) Gross Safe Bearing Capacity ($q_s$) :

It is the maximum pressure which the soil can carry safely without shear failure. $\Rightarrow q_s = q_{nu} / FS + \gamma D_f$

5) Net Safe Settlement Pressure ($q_{np}$) :

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

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

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

Thus,

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

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

# 1.3 Modes of shear Failure

- Vesic (1973) classified shear failure of soil under a foundation base into three categories depending on the type of soil & location of foundation.

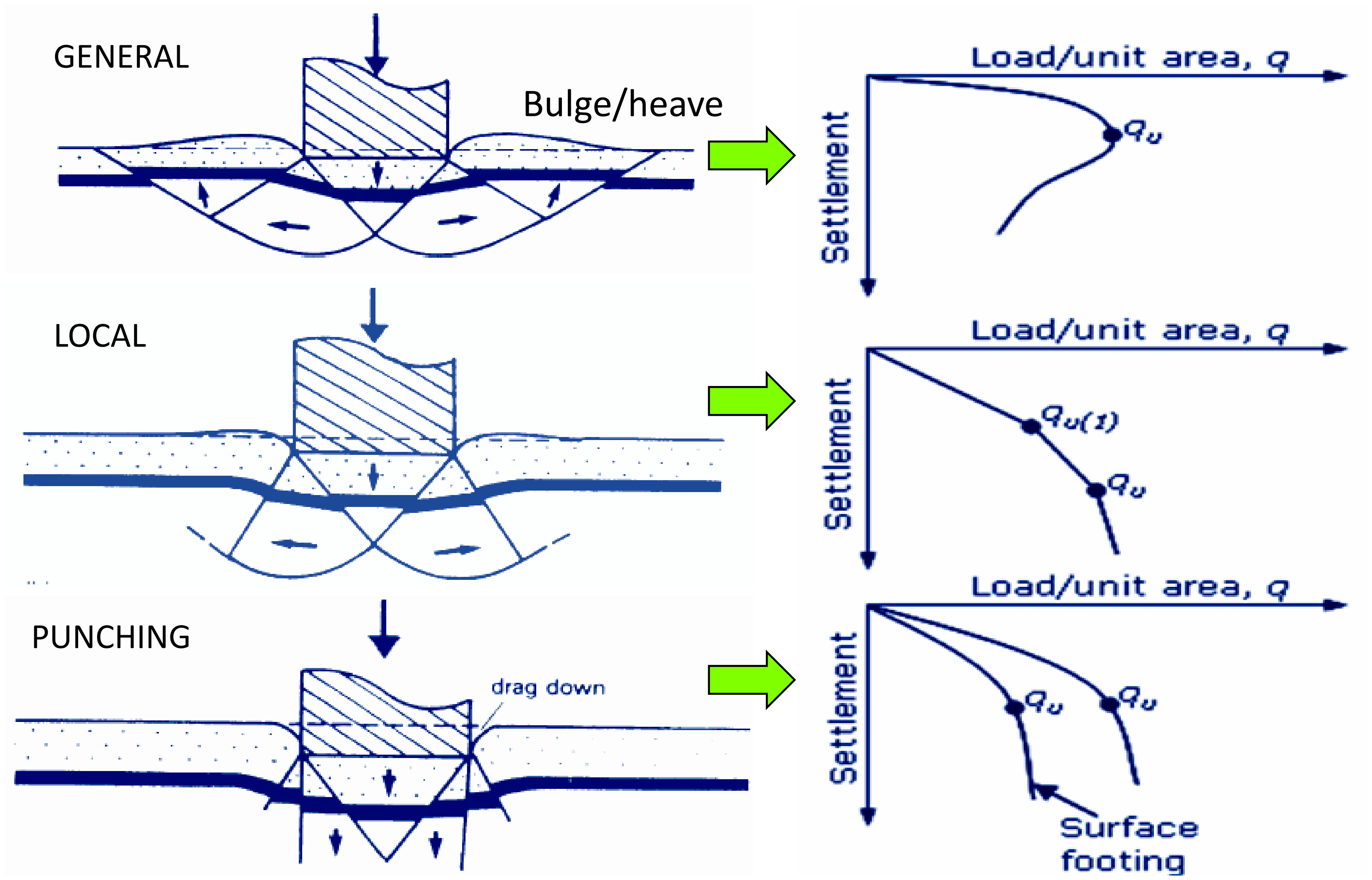
**a) General Shear Failure**

Most common type of shear failure; occurs in dense sand and stiff clay soil under strip footing.

**b) Local Shear Failure** Intermediate b/n general and punching shear failure. This is common in sand and clays of medium compaction. failure surface will gradually extend outward from the foundation but will not reach the ground surface as shown by the solid segment

**c) Punching Shear Failure** Occurs in very loose sands weak clays.

# Cont'd...


GENERAL
Bulge/heave
Load/unit area, q
Settlement
$q_u$
LOCAL
Load/unit area, q
Settlement
$q_{u(1)}$
$q_u$
PUNCHING
drag down
Load/unit area, q
Settlement
$q_u$
$q_u$
Surface footing

# Bearing capacity failure

**FIGURE 6.4** Transcona grain elevator failure.

# 2.1 Ultimate Bearing Capacity Equations

1. Prandtl (1920) developed the equation by assuming

- ✓ The footing as frictionless
- ✓ Ignoring the weight of the soil at the failure zone

✓For pure cohesive soil ($\phi = 0$)

$\Rightarrow$ For the footing at the surface

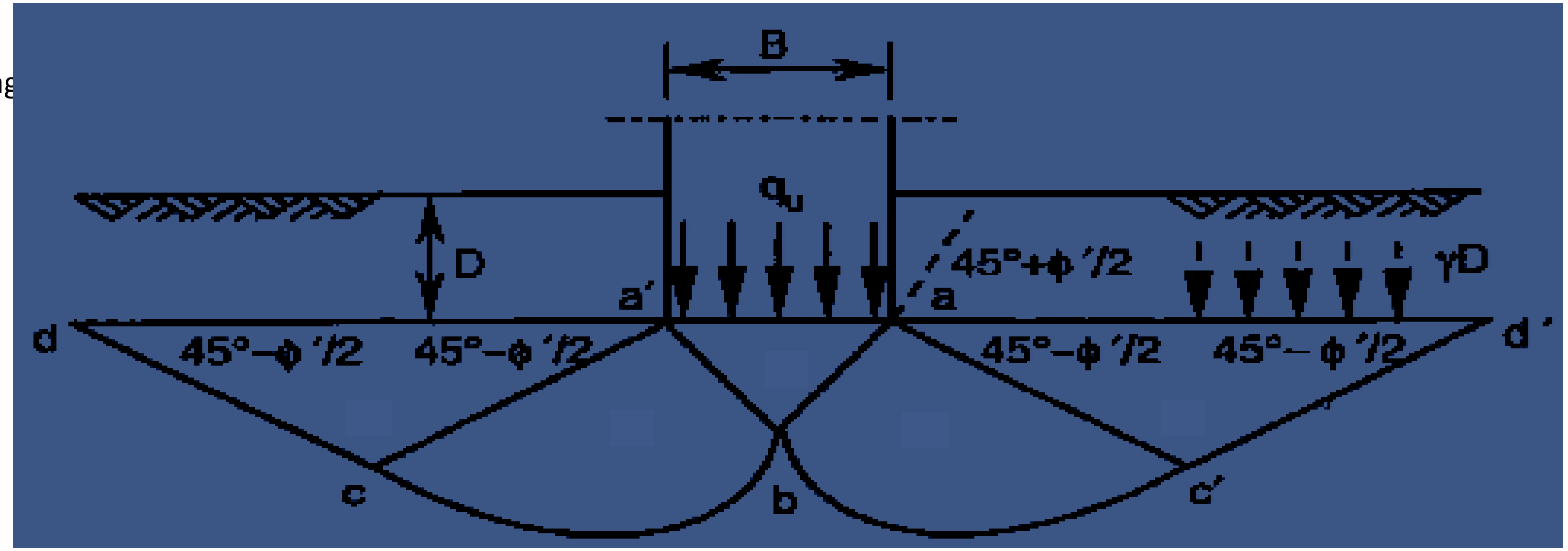
$\Rightarrow$ For the footing at a certain depth f $q_u = 5.14c_u$ nd surface

$\Rightarrow$ His assumptio $q_u = 5.14c_u + \gamma D_f$ and therefore the equation is never used in practical design, but it was a beginning.

# 2.1.1 Terzaghi bearing capacity equation

- **Terzhagi (1943)** improved the Prandtl equation to include the roughness of the footing and the weight of the failure zone.
- The failure mechanism in a $c'$, $\phi'$ soil is given in fig. below

➢ Terzag



# Terzhagi's ultimate bearing capacity equations

- Strip (or long) footing
- Square footing:
- Circular footing:

$$q_u = c' N_c + \gamma D N_q + 0.5 B \gamma N_\gamma$$

$$q_u = 1.3 c' N_c + \gamma D N_q + 0.4 B \gamma N_\gamma$$

$$q_u = 1.3 c' N_c + \gamma D N_q + 0.3 B \gamma N_\gamma$$

Where

➢ $N_c$, $N_q$ and $N_\gamma$ are called the bearing

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

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

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

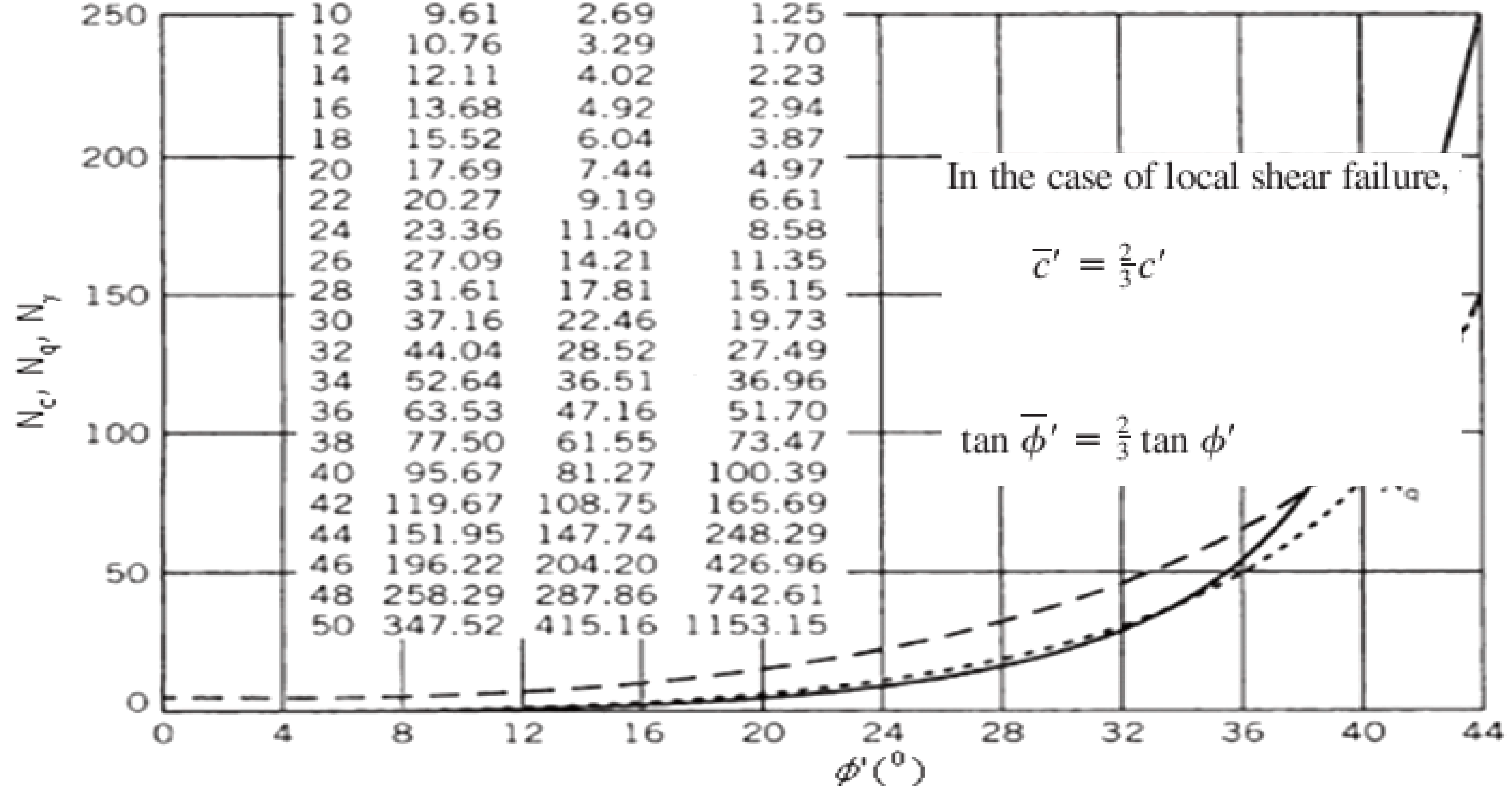
$$K_{p\gamma} = (8\phi'^2 - 4\phi' + 3.8)\tan^2(60^0 + \phi'/2)$$

➢ In the undrained conditions ($c_u$ and $\phi_u = 0$):

$$N_q = 1 \qquad N_c = 5.71 \qquad N_\gamma = 0$$

# Terzaghi Bearing capacity coefficients

| $\phi°$ | $N_c$ | $N_q$ | $N_\gamma$ |
|---|---|---|---|
| 0 | 5.71 | 1.00 | 0.00 |
| 2 | 6.30 | 1.22 | 0.18 |
| 4 | 6.97 | 1.49 | 0.38 |
| 6 | 7.73 | 1.81 | 0.62 |
| 8 | 8.60 | 2.21 | 0.91 |
| 10 | 9.61 | 2.69 | 1.25 |
| 12 | 10.76 | 3.29 | 1.70 |
| 14 | 12.11 | 4.02 | 2.23 |
| 16 | 13.68 | 4.92 | 2.94 |
| 18 | 15.52 | 6.04 | 3.87 |
| 20 | 17.69 | 7.44 | 4.97 |
| 22 | 20.27 | 9.19 | 6.61 |
| 24 | 23.36 | 11.40 | 8.58 |
| 26 | 27.09 | 14.21 | 11.35 |
| 28 | 31.61 | 17.81 | 15.15 |
| 30 | 37.16 | 22.46 | 19.73 |
| 32 | 44.04 | 28.52 | 27.49 |
| 34 | 52.64 | 36.51 | 36.96 |
| 36 | 63.53 | 47.16 | 51.70 |
| 38 | 77.50 | 61.55 | 73.47 |
| 40 | 95.67 | 81.27 | 100.39 |
| 42 | 119.67 | 108.75 | 165.69 |
| 44 | 151.95 | 147.74 | 248.29 |
| 46 | 196.22 | 204.20 | 426.96 |
| 48 | 258.29 | 287.86 | 742.61 |
| 50 | 347.52 | 415.16 | 1153.15 |




In the case of local shear failure,

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

$$\tan \bar{\phi}' = \tfrac{2}{3}\tan \phi'$$

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

where

$$N_c = \tan\phi'(K_c + 1) \tag{16.5}$$

$$N_q = K_q \tan\phi' \tag{16.6}$$

$$N_\gamma = \tfrac{1}{2}\tan\phi'(K_\gamma \tan\phi' - 1) \tag{16.7}$$

The terms $N_c$, $N_q$, and $N_\gamma$ are, respectively, the contributions of cohesion, surcharge, and unit weight of soil to the ultimate load-bearing capacity. It is extremely tedious to evaluate $K_c$, $K_q$, and $K_\gamma$. For this reason, Terzaghi used an approximate method to determine the ultimate bearing capacity, $q_u$. The principles of this approximation are the following.

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

$$q_u = q_\gamma = \tfrac{1}{2}\gamma BN_\gamma \tag{16.8}$$

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

$$q_u = q_c = c'N_c \tag{16.9}$$

3. If $\gamma = 0$ (weightless soil) and $c' = 0$, then

$$q_u = q_q = qN_q \tag{16.10}$$

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

$$q_u = q_c + q_q + q_\gamma = c'N_c + qN_q + \tfrac{1}{2}\gamma BN_\gamma \tag{16.11}$$

Equation (16.11) is referred to as *Terzaghi's bearing capacity equation*. The terms $N_c$, $N_q$, and $N_\gamma$ are called the *bearing capacity factors*. The values of these factors are given in Table 16.1.

**Table 16.1** Terzaghi's Bearing Capacity Factors—$N_c$, $N_q$ and $N_\gamma$—Eqs. (16.11), (16.12), and (16.13), respectively

| $\phi'$ (deg) | $N_c$ | $N_q$ | $N_\gamma^a$ | $\phi'$ (deg) | $N_c$ | $N_q$ | $N_\gamma^a$ |
|---|---|---|---|---|---|---|---|
| 0 | 5.70 | 1.00 | 0.00 | 26 | 27.09 | 14.21 | 9.84 |
| 1 | 6.00 | 1.10 | 0.01 | 27 | 29.24 | 16.90 | 11.60 |
| 2 | 6.30 | 1.22 | 0.04 | 28 | 31.61 | 17.81 | 13.70 |
| 3 | 6.62 | 1.35 | 0.06 | 29 | 34.24 | 19.98 | 16.18 |
| 4 | 6.97 | 1.49 | 0.10 | 30 | 37.16 | 22.46 | 19.13 |
| 5 | 7.34 | 1.64 | 0.14 | 31 | 40.41 | 25.28 | 22.65 |
| 6 | 7.73 | 1.81 | 0.20 | 32 | 44.04 | 28.52 | 26.87 |
| 7 | 8.15 | 2.00 | 0.27 | 33 | 48.09 | 32.23 | 31.94 |
| 8 | 8.60 | 2.21 | 0.35 | 34 | 52.64 | 36.50 | 38.04 |
| 9 | 9.09 | 2.44 | 0.44 | 35 | 57.75 | 41.44 | 45.41 |
| 10 | 9.61 | 2.69 | 0.56 | 36 | 63.53 | 47.16 | 54.36 |
| 11 | 10.16 | 2.98 | 0.69 | 37 | 70.01 | 53.80 | 65.27 |
| 12 | 10.76 | 3.29 | 0.85 | 38 | 77.50 | 61.55 | 78.61 |
| 13 | 11.41 | 3.63 | 1.04 | 39 | 85.97 | 70.61 | 95.03 |
| 14 | 12.11 | 4.02 | 1.26 | 40 | 95.66 | 81.27 | 116.31 |
| 16 | 12.86 | 4.45 | 1.52 | 41 | 106.81 | 93.85 | 140.51 |
| 16 | 13.68 | 4.92 | 1.82 | 42 | 119.67 | 108.75 | 171.99 |
| 17 | 14.60 | 5.45 | 2.18 | 43 | 134.58 | 126.50 | 211.56 |
| 18 | 15.12 | 6.04 | 2.59 | 44 | 161.95 | 147.74 | 261.60 |
| 19 | 16.56 | 6.70 | 3.07 | 45 | 172.28 | 173.28 | 325.34 |
| 20 | 17.69 | 7.44 | 3.64 | 46 | 196.22 | 204.19 | 407.11 |
| 21 | 18.92 | 8.26 | 4.31 | 47 | 224.55 | 241.80 | 512.84 |
| 22 | 20.27 | 9.19 | 5.09 | 48 | 258.28 | 287.85 | 650.67 |
| 23 | 21.75 | 10.23 | 6.00 | 49 | 298.71 | 344.63 | 831.99 |
| 24 | 23.36 | 11.40 | 7.08 | 50 | 347.50 | 416.14 | 1072.80 |
| 25 | 25.13 | 12.72 | 8.34 | | | | |

[a] *$N_\gamma$ values from Kumbhojkar (1993)*

## 2.1.2 Meyerhof's Bearing Capacity equation

- Meyerhof (1951) developed a bearing capacity equation by extending Terzhagi's failure mechanism and taking into account the effects of <u>footing shape</u>, <u>load inclination</u> and <u>footing depth</u> by adding the corresponding factors of *s*, *d*, & *i*.
- For a rectangular footing of *L* by *B* (*L* > *B*) and inclined load:
- For vert
- The bea

$$q_u = c' N_c s_c i_c d_c + \gamma D N_q s_q i_q d_q + 0.5 B \gamma N_\gamma s_\gamma i_\gamma d_\gamma$$

- In the undrained conditions ($c_u$ a

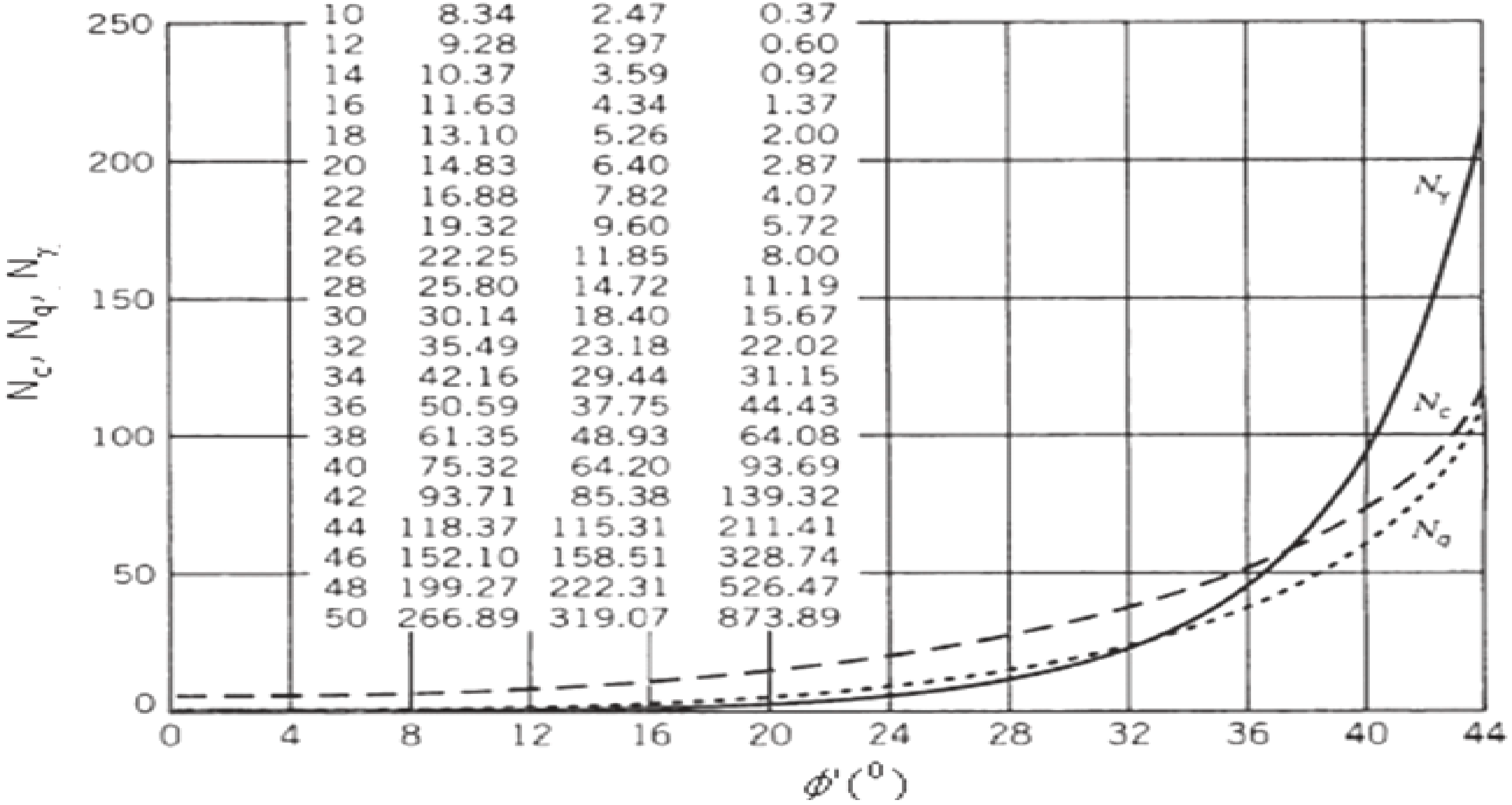
$$N_\gamma = (N_q - 1)\tan(1.4\phi')$$

$$N_q = \exp(\pi \tan\phi')\tan^2(45 + \phi'/2) \qquad N_c = \cot\phi'(N_q - 1)$$

$$N_q = 1 \qquad N_c = 5.71 \qquad N_\gamma = 0$$

# Meyerhof's bearing capacity coefficients

| $\phi°$ | $N_c$ | $N_q$ | $N_\gamma$ |
|---|---|---|---|
| 0 | 5.14 | 1.00 | 0.00 |
| 2 | 5.63 | 1.20 | 0.01 |
| 4 | 6.19 | 1.43 | 0.04 |
| 6 | 6.81 | 1.72 | 0.11 |
| 8 | 7.53 | 2.06 | 0.21 |
| 10 | 8.34 | 2.47 | 0.37 |
| 12 | 9.28 | 2.97 | 0.60 |
| 14 | 10.37 | 3.59 | 0.92 |
| 16 | 11.63 | 4.34 | 1.37 |
| 18 | 13.10 | 5.26 | 2.00 |
| 20 | 14.83 | 6.40 | 2.87 |
| 22 | 16.88 | 7.82 | 4.07 |
| 24 | 19.32 | 9.60 | 5.72 |
| 26 | 22.25 | 11.85 | 8.00 |
| 28 | 25.80 | 14.72 | 11.19 |
| 30 | 30.14 | 18.40 | 15.67 |
| 32 | 35.49 | 23.18 | 22.02 |
| 34 | 42.16 | 29.44 | 31.15 |
| 36 | 50.59 | 37.75 | 44.43 |
| 38 | 61.35 | 48.93 | 64.08 |
| 40 | 75.32 | 64.20 | 93.69 |
| 42 | 93.71 | 85.38 | 139.32 |
| 44 | 118.37 | 115.31 | 211.41 |
| 46 | 152.10 | 158.51 | 328.74 |
| 48 | 199.27 | 222.31 | 526.47 |
| 50 | 266.89 | 319.07 | 873.89 |

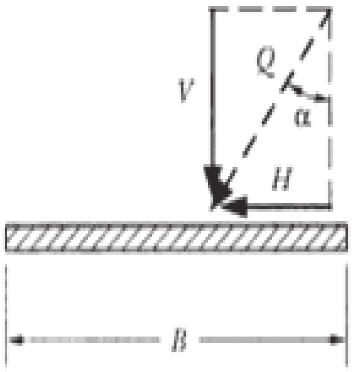


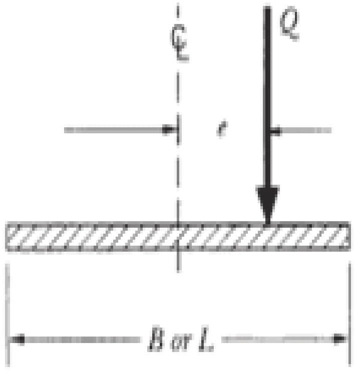
# The shape, inclination and depth factors

| | Shape | Depth | Inclination |
|---|---|---|---|
| Any $\phi'$ | $s_c = 1 + 0.2K_p \dfrac{B}{L}$ | $d_c = 1 + 0.2\sqrt{K_p}\dfrac{D}{B}$ | $i_c = i_q = \left(1 - \dfrac{\alpha^0}{90^0}\right)^2$ |
| For $\phi' = 0$ | $s_q = s_\gamma = 1$ | $d_q = d_\gamma = 1$ | $i_\gamma = 0$ |
| For $\phi' \geq 10^0$ | $s_q = s_\gamma = 1 + 0.1K_p \dfrac{B}{L}$ | $d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\dfrac{D}{B}$ | $i_\gamma = \left(1 - \dfrac{\alpha^0}{\phi'^0}\right)^2$ |

$K_p = \tan^2\left(45 + \dfrac{\phi'}{2}\right)$,  $\alpha$ =angle of resultant measured from vertical axis.

when triaxial $\phi'$ is used for plane strain, adjust $\phi'$ to obtain $\phi' = \left(1.1 - 0.1\dfrac{B}{L}\right)\phi'_{\text{triaxial}}$

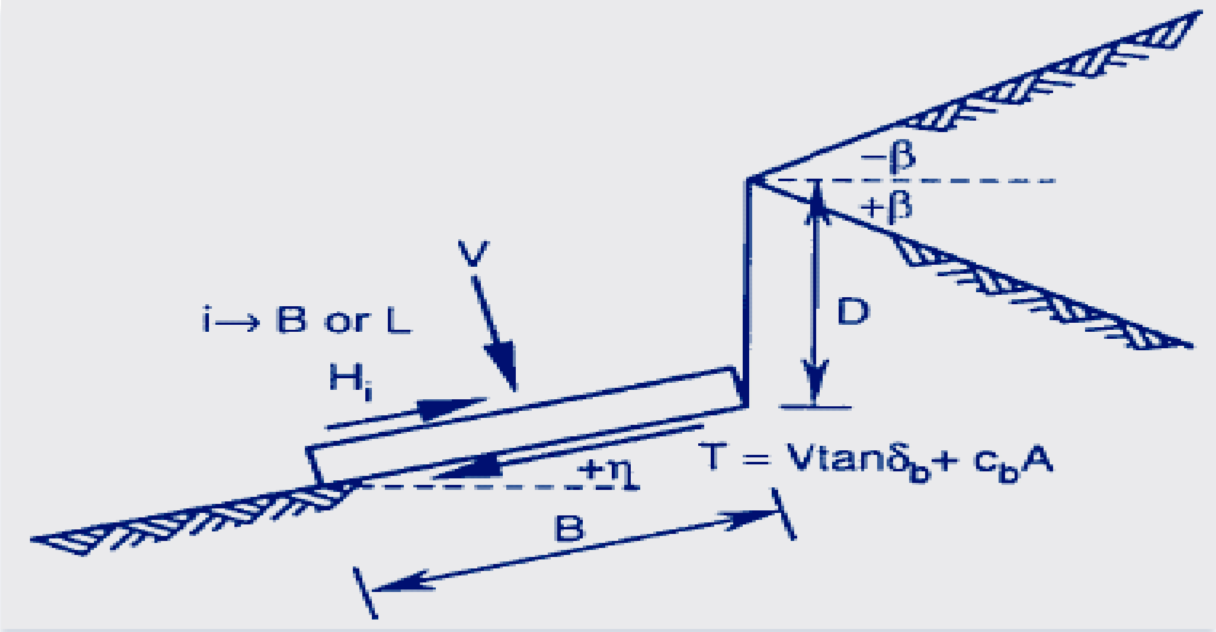


## 2.1.3 Hansen's Bearing Capacity Equation

- Hansen (1961) extended Meyerhof's solutions by considering the effects of sloping ground surface and tilted base as well as modification of $N_\gamma$ and other factors.
- For a rectangular footing of $L$ by $B$ ($L > B$) and inclined ground surface, base and load:

$$q_u = c' N_c s_c d_c i_c b_c g_c + \gamma D N_q s_q d_q i_q b_q g_q + 0.5 B \gamma N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$



In the special case of a horizontal ground surface,

$$q_u = c' N_c s_c d_c i_c b_c + \gamma D N_q s_q d_q i_q b_q + 0.5 B \gamma N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma$$

- The bearing capacity factors $N_c$ and $N_q$ are identical with Meyerhof's factors. But $N_\gamma$ :- $N_\gamma = 1.5(N_q - 1)\tan\phi$
- Failure can take place either along the long side or along the short side of the footing so that Hansen proposed that

1.Shape factors is given as

$$s_{c,B} = 1 + \frac{N_q}{N_c} \cdot \frac{B}{L} i_{c,B} \qquad s_{q,B} = 1 + \frac{B}{L} i_{q,B} \cdot \sin\phi' \qquad s_{\gamma,B} = 1 - 0.4 \frac{B}{L} i_{\gamma,B} \geq 0.6$$

$$s_{c,L} = 1 + \frac{N_q}{N_c} \cdot \frac{L}{B} i_{c,L} \qquad s_{q,L} = 1 + \frac{L}{B} i_{q,L} \cdot \sin\phi' \qquad s_{\gamma,L} = 1 - 0.4 \frac{L}{B} i_{\gamma,L} \geq 0.6$$

❖For $c_u$, $\phi_u$=0 soil

$$s_{c,B} = 0.2 \frac{B}{L} i_{c,B} \qquad s_{c,L} = 0.2 \frac{L}{B} i_{c,L}$$

- The inclination factors are:

$$i_{c,i} = i_{q,i} - \frac{1 - i_{q,i}}{N_q - 1}$$

$$i_{q,i} = \left(1 - \frac{0.5H_i}{V + Ac_b \cot\phi'}\right)^{\alpha_1}$$

$$i_{\gamma,i} = \left(1 - \frac{0.7H_i}{V + Ac_b \cot\phi'}\right)^{\alpha_2}$$

- For the Tilted base:-

$$i_{\gamma,i} = \left[1 - \frac{(0.7 - \eta^0/450^0)H_i}{V + Ac_b \cot\phi'}\right]^{\alpha_2}$$

- *Where*

⇒ *A = is the area of the footing base*

⇒ $\eta^0$ *= angle of inclination of the base of the footing.*

⇒ $\alpha_1$ *and* $\alpha_2$ *are in the range of* $2 \le \alpha_1 \le 5$ $2 \le \alpha_2 \le 5$

# Cont…

⇒ *H and V are horizontal and vertical component of the total load.*

⇒ $\delta_b$ *= is the angle of friction between the base of footing and soil.*

⇒ $c_a$ *= is the adhesion between footing and soil.*

For $c_u$, $\phi_u=0$ soil: $i_{c,i} = 0.5 - 0.5\sqrt{1 - H_i / Ac_b}$

⇒ In the above equations, *B* and *L* may be replaced by their effective values (*B'* and *L'*) expressed as :

$L' = L - 2e_L$ and $B' = B - 2e_B$

Where $e_L$ and $e_B$ represent the eccentricity along L and B directions

## *Cont…*

The depth factors are expressed in two sets

$\Rightarrow$ For $D/B \le 1$ & $D/L \le 1$:

$$d_{c,B} = 1 + 0.4 \cdot {}^{D}\!/_{B}$$

$$d_{q,B} = 1 + 2\tan\phi'(1-\sin\phi')^2 \cdot {}^{D}\!/_{B}$$

$$d_{c,L} = 1 + 0.4 \cdot {}^{D}\!/_{L}$$

$$d_{q,L} = 1 + 2\tan\phi'(1-\sin\phi')^2 \cdot {}^{D}\!/_{L}$$

$\Rightarrow$ For $D/B > 1$ & $D/L > 1$:

$$d_{c,B} = 1 + 0.4 \cdot \tan^{-1}\left({}^{D}\!/_{B}\right)$$

$$d_{q,B} = 1 + 2\tan\phi'(1-\sin\phi')^2 \cdot \tan^{-1}({}^{D}\!/_{B})$$

$$d_{c,L} = 1 + 0.4 \cdot \tan^{-1}\left({}^{D}\!/_{L}\right)$$

$$d_{q,L} = 1 + 2\tan\phi'(1-\sin\phi')^2 \cdot \tan^{-1}({}^{D}\!/_{L})$$

$$d_{\gamma} = 1$$

For $c_u$, $\phi_u$=0 soil:

$$d_{c,B} = 0.4 \cdot D/B \qquad d_{c,L} = 0.4 \cdot D/L$$

1. For the sloping ground and tilted base:-

The ground factors "$g_i$"

$$g_c = 1 - \beta^0/147^0 \qquad g_q = g_\gamma = (1 - 0.5\tan\beta)^5$$

The base factors "$b_i$"

$$b_c = 1 - \eta^0/147^0 \qquad b_q = e^{-2\eta\tan\phi'} \qquad b_\gamma = e^{-2.7\eta\tan\phi'}$$

For $c_u$, $\phi_u$ soil

$$g_c = \beta^0/147^0 \qquad b_c = \eta^0/147^0$$

# General Bearing Capacity Equations

- Generalized Bearing Capacity Equation
  - This equation accounts for
    - Rectangular foundation
    - Shear strength of overburden
    - Inclined loads
    - Shallow foundation
  - To account for this the following factors were included in the previous equations

# Cont'd...

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

In this equation:

$c'$ = cohesion

$q$ = effective stress at the level of the bottom of the foundation

$\gamma$ = unit weight of soil

$B$ = width of foundation (= diameter for a circular foundation)

$F_{cs}, F_{qs}, F_{\gamma s}$ = shape factors

$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors

$F_{ci}, F_{qi}, F_{\gamma i}$ = load inclination factors

$N_c, N_q, N_\gamma$ = bearing capacity factors

# *2.2 A comparative summary of the three bearing capacity equations*

1. Terzaghi's equation is widely used, because it is some what simpler than Meyerhof's and Hansen's.
2. Practitioners use Terzaghi's equations for a very cohesive soil and D/B < 1.

❖ However, Terzaghi's equations have the following major drawbacks:

  - ✓ Shape, depth and inclination factors are not considered.
  - ✓ Terzaghi's equations are suitable for a concentrically loaded horizontal footing but are not suitable for eccentrically loaded footings that are very common in practice.

## *Cont…*

- ✓ The equations are generally conservative than Meyerhof's and Hansen's.
- ❖ That is why Currently, Meyerhof's and Hansen's equations are more widely used than Terzaghi's. Both are applicable to more general conditions.
- ❖ Hansen's is, however, used when the base is tilted or when the footing is on a slope and for $D/B > 1$.

## 2.3 Effects of Groundwater Table on Bearing Capacity

- For all the bearing capacity equations, you will have to make
  - Some adjustments for the groundwater condition.
- ✓ The term " $\gamma D$ " refers to the vertical stress of the soil above the base of the footing.
- ✓ The term "$\gamma B$" refers to the vertical stress of a soil mass of thickness *B*, below the base of the footing. Here we have three condition for ground water effect ,so check which one of the three groundwater situations is applicable to your project.

***Situation 1:***

Groundwater level at a depth B below the base of the footing. In this case no modification of the bearing capacity equations is required.

# Cont…

**Situation 2:**

Groundwater level within a depth *B* below the base of the footing.

If the groundwater level is at a depth *z* below the base, such that *z* < *B*. Then

The term $\gamma B \rightarrow \gamma z + \gamma'(B - z)$ or $\gamma_{sat} z + \gamma'(B - z)$

*Soil is saturated above ground water level*

The term $\gamma D$ remains unchanged.

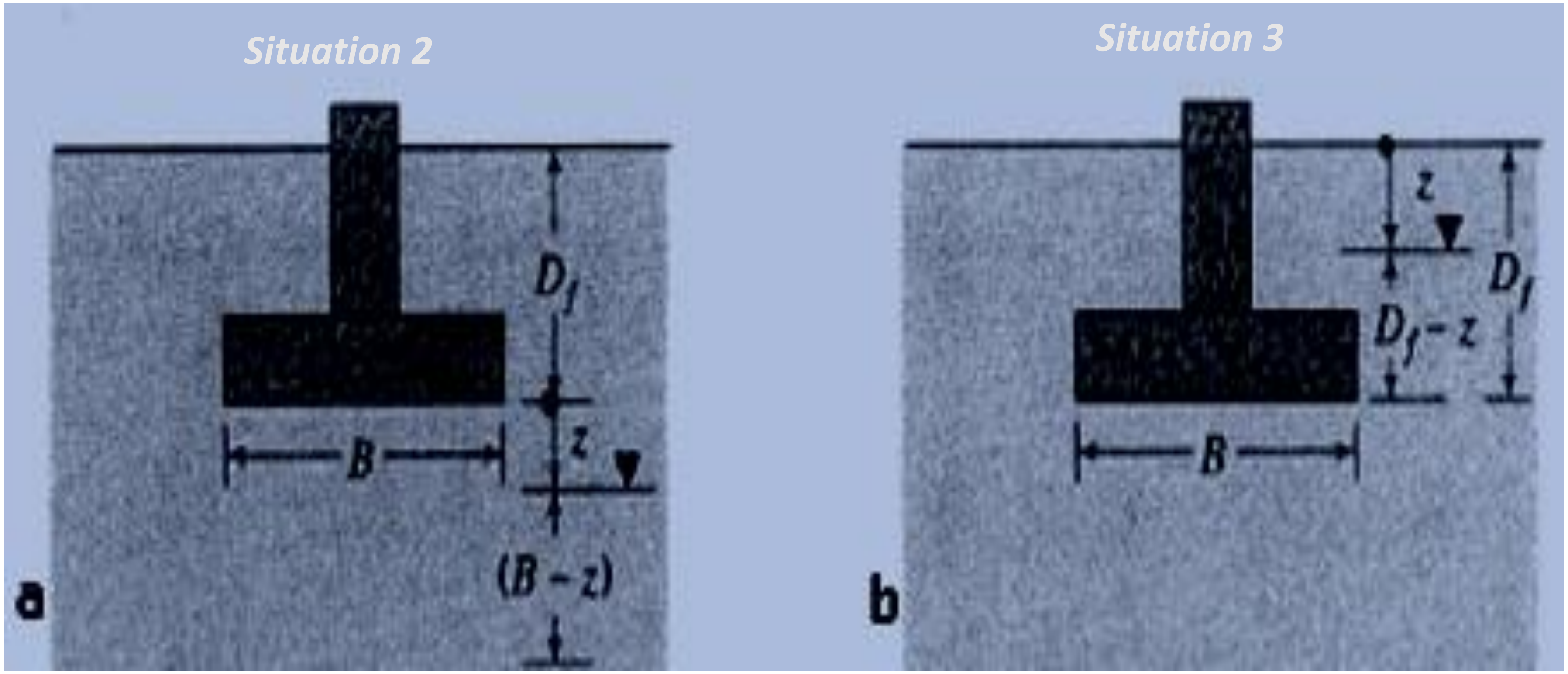
**Situation 3:**

Groundwater level within the embedment depth. If the groundwater is at a depth *z* within the embedment such that *z* < *D*

# Cont…

⇒Then the term $\gamma B \rightarrow \gamma z + \gamma'(B-z)$ or $\gamma_{sat} z + \gamma'(B-z)$

⇒The term $\gamma B$ is changed in to $\gamma' B$.



## 2.4 Allowable bearing capacity and factor of safety

The allowable bearing capacity "$q_a$" is calculated by dividing the ultimate bearing capacity by a factor.

The *FS* is intended to compensate for

1) Assumptions made in developing the bearing capacity equations.
2) Soil variability.
3) Inaccurate soil data, and
4) Uncertainties of loads.

- The magnitude of *FS* applied to the ultimate bearing capacity may be between 2 and 3.
- The allowable bearing capacity is

$$q_a = \frac{q_u}{FS}$$

If the maximum applied foundation stress

# Cont'd…

⇒ If the maximum applied foundation stress is known and the dimension of the footing is also known. By replacing "$q_a$" with $(\sigma)_{max}$ then FS is given by:-

$$FS = \frac{q_u}{(\sigma_a)_{\max}}$$

## 2.5 Eccentric Loads

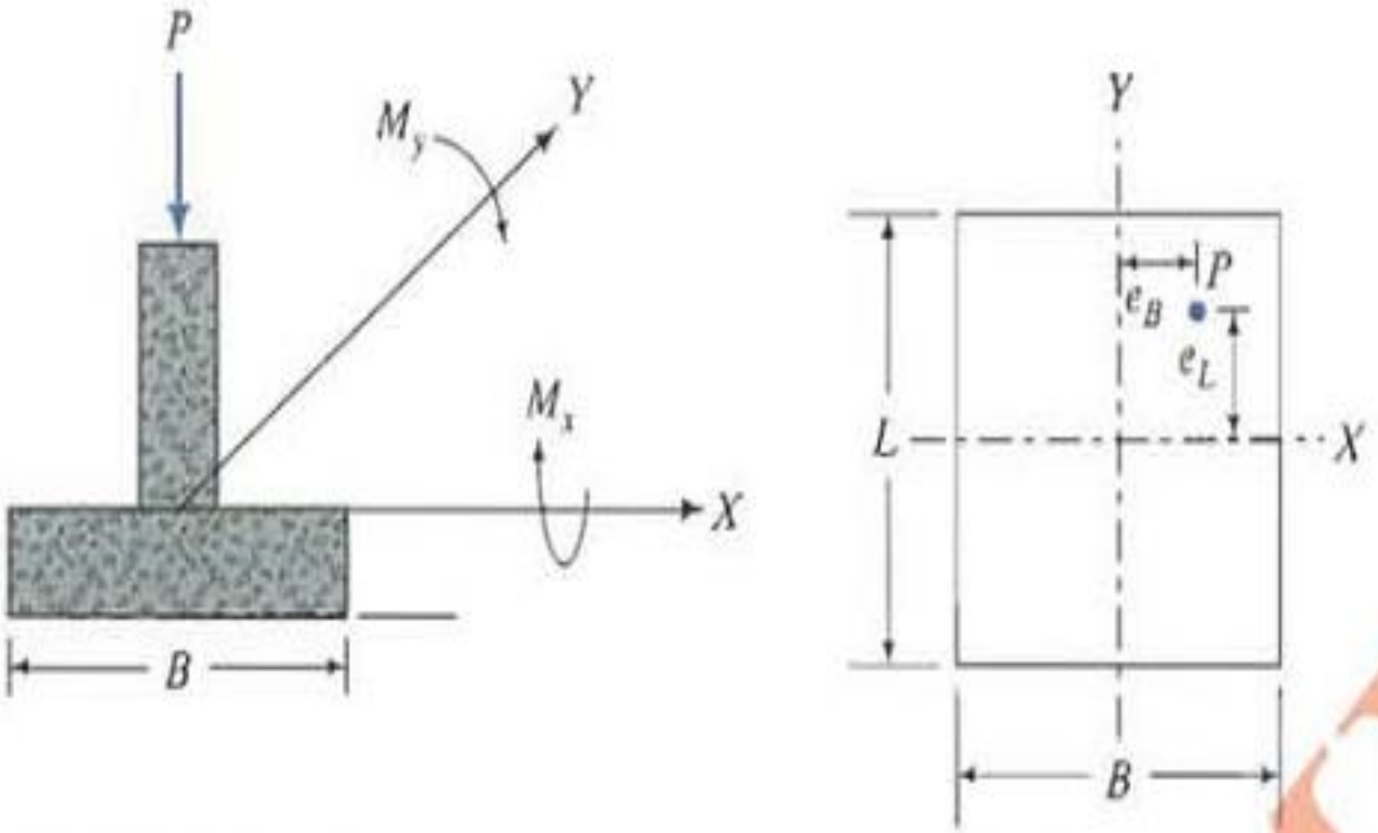
- Meyerhof (1963) proposed an approximate method for loads that are located off-centered (or eccentric loads).
- He modified base area or dimensions of the footing pad, as

$$\Rightarrow B' = B - 2e_B \text{ and } L' = L - 2e_L$$

Where $e_B$ and $e_L$ are eccentricity along B and L.

# Cont'd.....





if the maximum applied foundation stress

# Cont'd...

Where $e_B = \dfrac{M_y}{P}$ and $e_L = \dfrac{M_x}{P}$

$M_x$ = moment in X-direction.

$M_y$ = moment in Y-direction.

p = vertical load.

The maximum and minimum vertical stresses

⇒ Along the *x* axis are:

$$\sigma_{\max} = \frac{P}{BL}\left(1 + \frac{6e_B}{B}\right) \qquad \sigma_{\min} = \frac{P}{BL}\left(1 - \frac{6e_B}{B}\right)$$

⇒ Along the *y* axis are:

$$\sigma_{\max} = \frac{P}{BL}\left(1 + \frac{6e_L}{B}\right) \qquad \sigma_{\min} = \frac{P}{BL}\left(1 - \frac{6e_L}{B}\right)$$

Since the tensile strength of soils = 0, it should always be > 0.
Therefore, $e_B < B/6$ & $e_L < L/6$.

❖ The bearing capacity equations are modified for eccentric loads by replacing *B* with *B'*.

If the maximum applied foundation stress

# *Cont'd…*

- The ultimate bearing capacity for footings with eccentricity, using either the Meyerhof or Hansen equations, is found in *either of two ways:*

**Method 1.** Use either the Hansen bearing-capacity equation  with the following adjustments:

*a. Use B' in the $\gamma B N_\gamma$ term.*

*b. Use B' and L' in computing the shape factors.*

*c. Use actual B and L for all depth factors.*

- The computed ultimate bearing capacity [$q_{ult}$] *is then reduced to an allowable value $q_a$ with* an appropriate safety factor SF as

$$\Rightarrow q_a = q_{ult} / SF \quad \text{and} \quad P_a = q_a B'L'$$

if the maximum applied foundation stress

# Cont'd...

**Method 2.** Use the Meyerhof general bearing-capacity equation and a reduction factor *Re used as:-*

$$q_{ult(design)} = q_{ult(computed)} * R_e$$

*Where reduction factor :-*

⇒For cohesive soil:-

$$R_e = 1 - \frac{2e}{B}$$

⇒For cohesion less soil and for 0 < *e/B < 0.3.*

$$R_e = 1 - \sqrt{\frac{e}{B}}$$

⇒In practice the *e/B ratio is seldom greater than 0.2.if it is equal to 0.5 the foundation will be unstable or Area (A')is at corner.*

- ***Alternatively, use the Meyerhof equation with B' and L' for shape and depth factors & B' in the 0.5$\gamma B'N_\gamma$ term.***

# Cont'd.....

### 23.21. ECCENTRICALLY LOADED FOUNDATIONS

Foundations are sometimes subjected to moments in addition to the loads (Fig. 23.19). The distribution of footing pressure is not uniform in this case. It is a case of bending combined with thrust, treated in the mechanics of materials. The maximum and minimum pressures are given by

$$q_{max} = \frac{Q}{B \times L} + \frac{M}{I}(B/2)$$

and $$q_{min} = \frac{Q}{B \times L} - \frac{M}{I}(B/2)$$

where $I$ = moment of inertia ( $= LB^3/12$), $Q$ = total vertical load (gross), $M$ = moment on the foundation, $B$ = width of footing, $L$ = length of footing.

Taking the eccentricity $e$ as $M/Q$, the above equations become

$$q_{max} = \frac{Q}{BL}(1 + 6\,e/B) \quad \ldots[23.63(a)]$$

and $$q_{min} = \frac{Q}{BL}(1 - 6\,e/B) \quad \ldots[23.63(b)]$$

The maximum pressure $q_{max}$ should be less than the safe gross bearing capacity.

**Meyerhof's Method**

The factor of safety of eccentrically loaded foundations against bearing capacity failure can be determined using the method given by Meyerhof (1953), as explained below

(1) Determine the eccentricity of the load, along the width as,

$$e_b = M_b/P$$

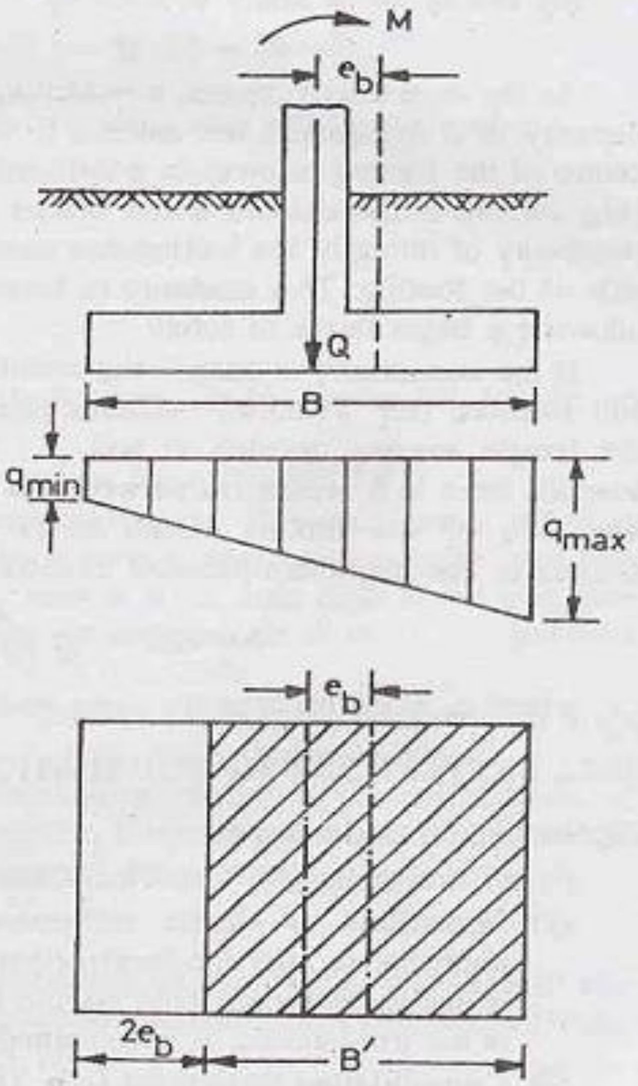
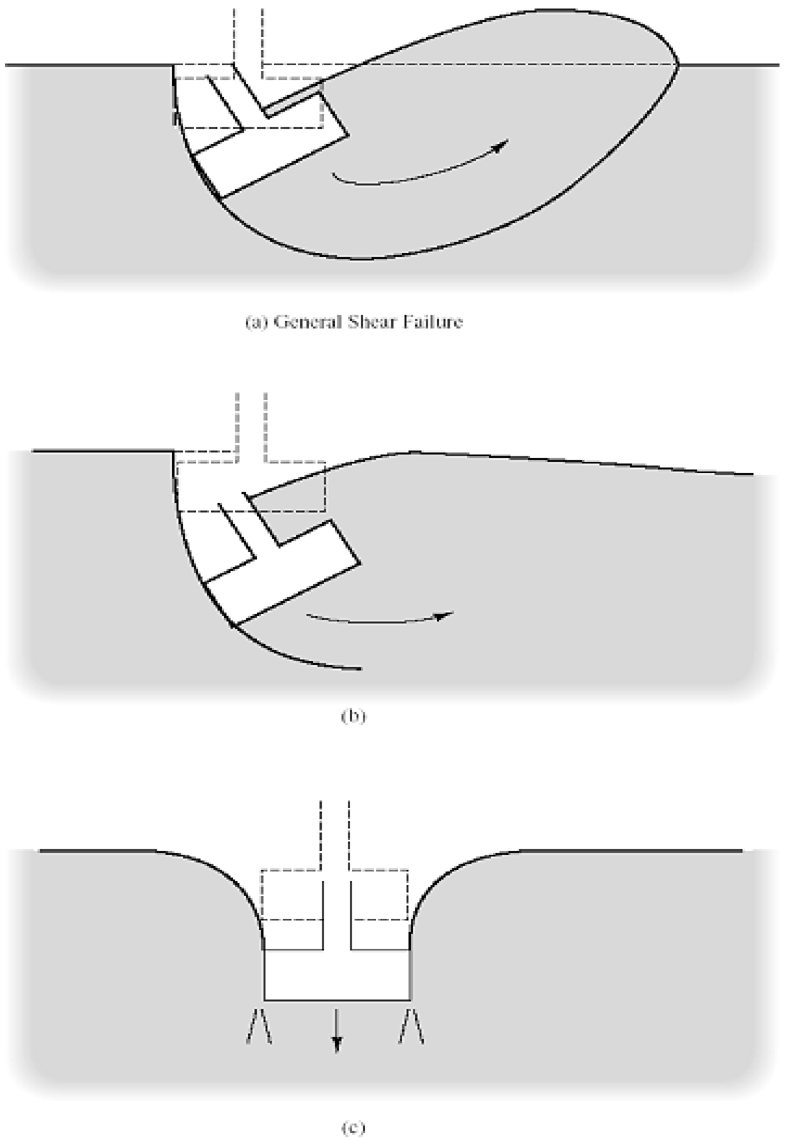



Fig. 23.19. Eccentricaly loaded footing.

# Summary of chapter two

- Define ultimate bearing capacity of soils under foundation structure.
- What is factor of safety regarding bearing capacity of the soils?
- Define and sketch failure modes of bearing capacity of soils.
- Discuss the reason of failure in bearing capacity of foundations.
- What are the factors affects ultimate bearing capacity?
- A bearing capacity failure is defined as a foundation failure that occurs when the shear stresses in the soil exceed the shear strength of the soil. Types of failure

# Cont'd…




## Bearing Capacity Failure

a) General Shear Failure

Most common type of shear failure; occurs in strong soils and rocks

b) Local Shear Failure

Intermediate between general and punching shear failure

c) Punching Shear Failure

Occurs in very loose sands and weak clays

# Cont'd…

- List ultimate bearing capacity equations and discuss their drawn backs, and comparisons among them.
- Make modification for groundwater conditions on bearing capacity.
- What is the advantage of field test method of determination of bearing capacity and list the methods used for bearing capacity determination with some sketch and procedure.

# Cont'd...

- **Terzaghi's** Bearing Capacity Equation
  - Assumption
    - General Shear Failure
    - Strip Foundation(continous)
    - Shallow Foundation ($D_f/B<1$)
    - Homogenous Layer Extends to great depth
    - Rough, rigid foundation
    - Overburden soil has no shear strength
    - C-$\varphi$ soil
  - Terzaghi Assumed the failure surface to be composed of
    - Elastic Wedge
    - Prandtl's radial Surface
    - Rankin's Passive wedge

# Cont'd…

- Meyerhof's Bearing Capacity Equation
  - Assumption
    - General Shear Failure
    - Strip Foundation (continuous)
    - Homogenous Layer Extends to great depth
  - Meyerhof Assumed the failure surface to be composed of
    - Elastic Wedge
    - Prandtl's radial Surface
    - Mixed Shear Zone

# Eccentrically Loaded Spread Footings

- The ensuing "load" on the column, and subsequently on the footing, due to supported beams from several spans, can be a combination of a vertical load and moments as shown in figure below.

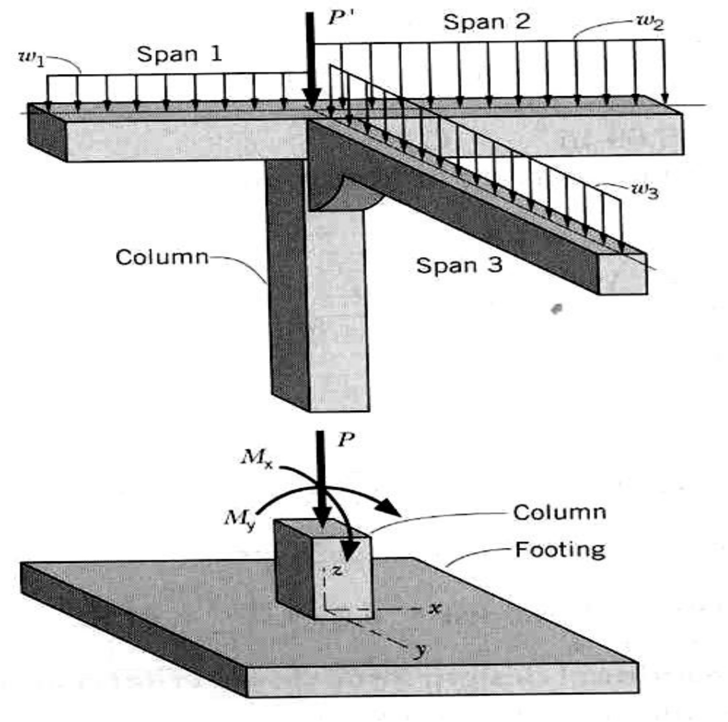



**Figure** Example of a loading condition that may induce eccentric loading in two directions.

Thank you!!!!!

# CHAPTER THREE

## Elastic Settlement of Shallow foundations

*By Gamachis A.*

# INTRODUCTION

- The increase of stress in soil layers due to the load imposed by various structures at the foundation level will always be accompanied by some strain($\nabla l/L$), which will result in the **settlement of the structures.**
- **Foundation settlement** is the shifting of the foundation (and the structure built upon it) into the soil.
- This can cause damage to the structure. Whether the soil is moist or dry is central to predicting the amount of settlement to expect in a given foundation.
- Areas with moist soils will have more foundation settlement than dry areas.

Cont.…

- The idea is that as water is squeezed out from the soil, the structure will shift according to the empty spaces the water left. The more water, the more shift.
- Where foundation settlement occurs at roughly the same rate throughout all portions of a building, it is termed **uniform settlement.**
- Settlement that occurs at differing rates between different portions of a building is termed **differential settlement**.
- When all parts of a building rest on the same kind of soil, and the loads on the building and the design of its structural system are uniform throughout, differential settlement is normally **not a**

Generally **Settlement** in a structure refers to the distortion or **disruption** of parts of a building due to

- Unequal compression of its foundations;
- Shrinkage like that which occurs in timber-framed buildings as the frame adjusts its moisture content; or
- Undue loads being applied to the building after its initial construction.

**Principal criteria for foundation design**

When designing foundations, two principal criteria must be satisfied:

1. Maintaining Stability
2. Limiting Settlement

**Permissible settlement:** The maximum limit of settlement, that a structure can **tolerate without damaging its structural integrity** or function, is known as the permissible settlement.

Therefore, the allowable bearing capacity $\mathbf{q_{all}}$ should be the smaller of the following two

$$q_{\text{all}} = \begin{cases} \dfrac{q_u}{\text{FS}} \\ \text{or} \\ q_{\text{allowable settlement}} \end{cases}$$

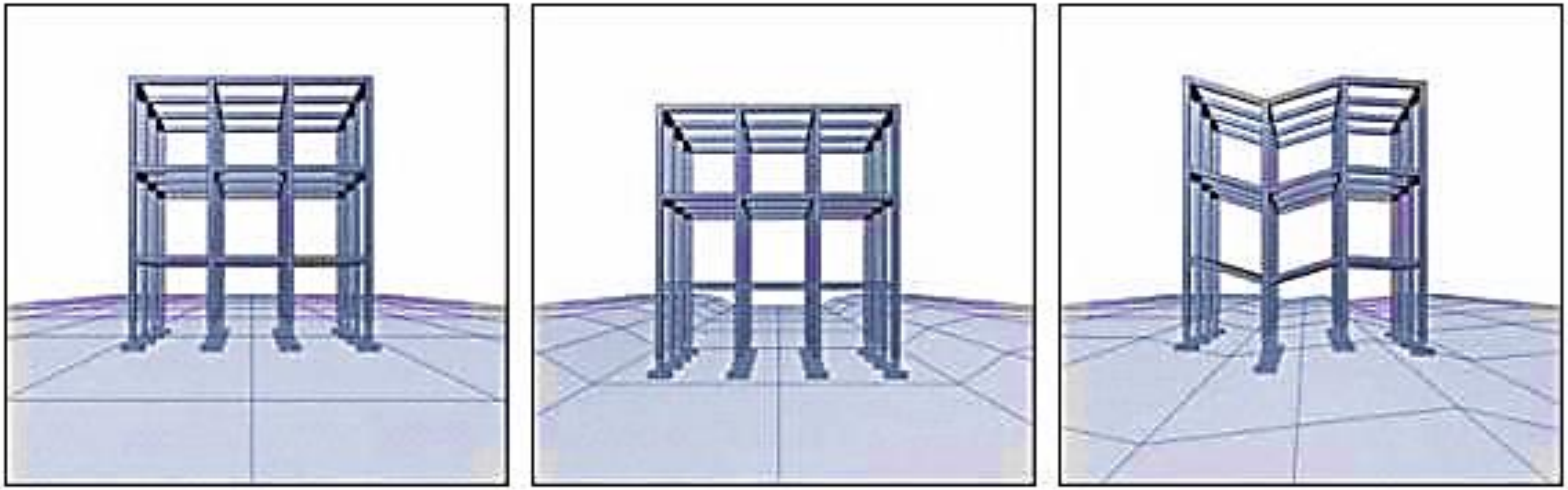
The safe pressure that does not cause **bearing failure**

The safe pressure that does not cause unacceptable settlement

# Types of settlements

From **structural consideration** there are two types of settlements:

Uniform and differential settlements



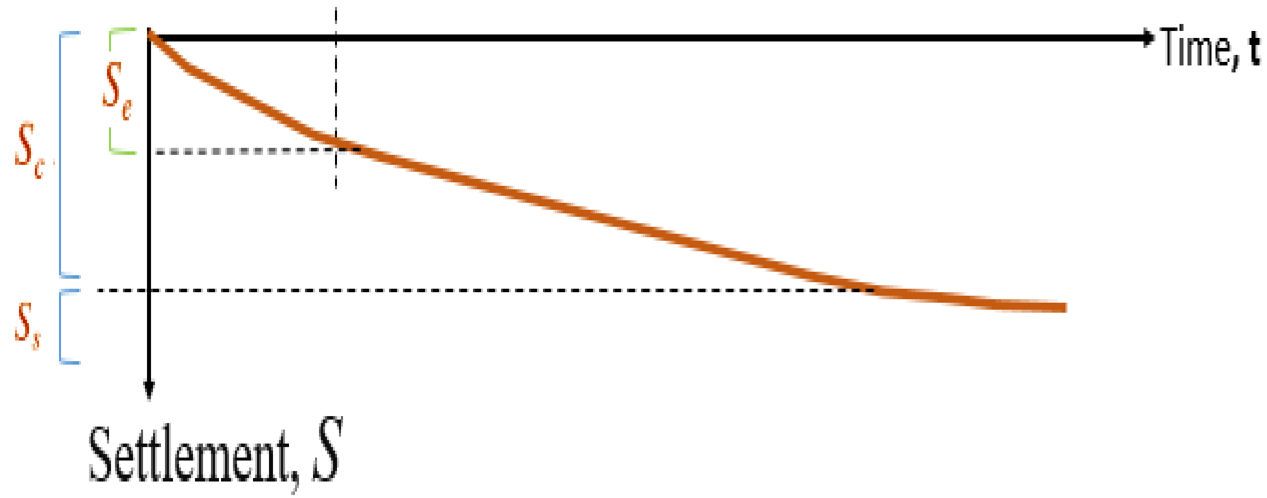
(*a*) *Building before settlement occurs* (*b*) *Uniform settlement* (*c*) *Differential settlement*

From **geotechnical** consideration, there are three types of settlements:

Immediate or Elastic Settlement: Occurs immediately after the construction. This is computed using elasticity theory (Important for Granular soil and mostly during construction time)

Primary Consolidation: Due to gradual dissipation of pore pressure induced by external loading and consequently expulsion/squeezing out of water from the soil mass, hence volume change. (Important for Inorganic clays)

Secondary Consolidation: Occurs at constant effective stress with volume change due to rearrangement of particles. (Important for Organic soils).



# Cont….

So, generally, the **types of settlements are**

### Immediate settlement ($S_e$)

- elastic deformation with no change in water content
- occurs rapidly during the application of load
- occurs in sandy, silty and clayey soils

### Primary consolidation settlement ($S_c$)

- decrease in voids volume as pore-water is squeezed out of the soil
- occurs slowly according to the permeability
- only significant in saturated clays and silts

### Secondary compression or creep ($S_s$)

- due to gradual changes in the particulate structure of the soil
- occurs very slowly, long after the primary consolidation is completed
- most significant in soft organic soils

# The total settlement of a foundation can then be given as:

$$S_T = S_e + S_c + S_s$$

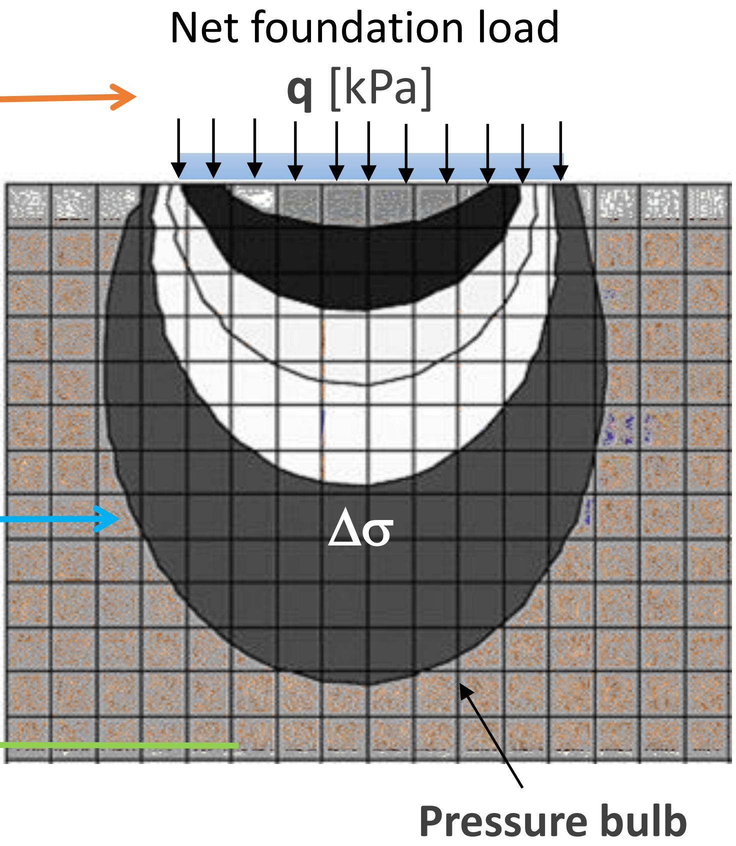
# Things required to calculate settlements

Methods used for settlement calculations usually require to know the followings:

**Foundation** (net load **q**, type, dimensions (B,L), Rigidity, …)

**Vertical stress increase $\Delta\sigma$** in soil caused by foundation load

**Soil profile & parameters**: e.g. E, $\nu$, $C_c$, $C_R$, …



# General about elastic soil parameters and stress distributions

- Elastic: the material which can turn back to its original shape/postion.
- Stress=applied force per unit area
- Strain=change in L/Original L
- Poisons ratio=Axial deformation to lateral deformation
- E or young's modulus is the measure of resistance of the soil against………..
- Or E is the slope of stress to strain relation ship studied by theory of elasticity.
- G,K are other important properties of materials
- So, the very important soil parameters of soil to know the deformation and settlement of soils are E and v.
- Net Foundation stress distribution decreases along the depth .but if we

# EBCS Recommendations

- The EBCS recommends a permissible total settlement of

  50mm for sand and 75mm for clay and

- Angular distortion ( differential settlement) not more than 1/500.

# Cont….

In this chapter, we will deal mainly about:

- Types of footings
- Contact Pressure and Settlement Profile
- Calculation of Settlement based on general theory of Elasticity
- Calculation of Elastic Settlement of saturated clay
- Calculation of Elastic Settlement of sandy soil by using strain influence factor

# Types of footings

- There are two types of footings: **Rigid** and **Flexible** footings
- The basic difference between the rigid footing and flexible footing is that, the flexible footing **undergoes differential settlement** while a rigid footing will undergo uniform settlement i.e. at every point settlement will be same in case of rigid footing while in flexible footing it will vary.
- The rigid footing is assumed to be infinitely rigid which means that whole footing will settle as a rigid element.

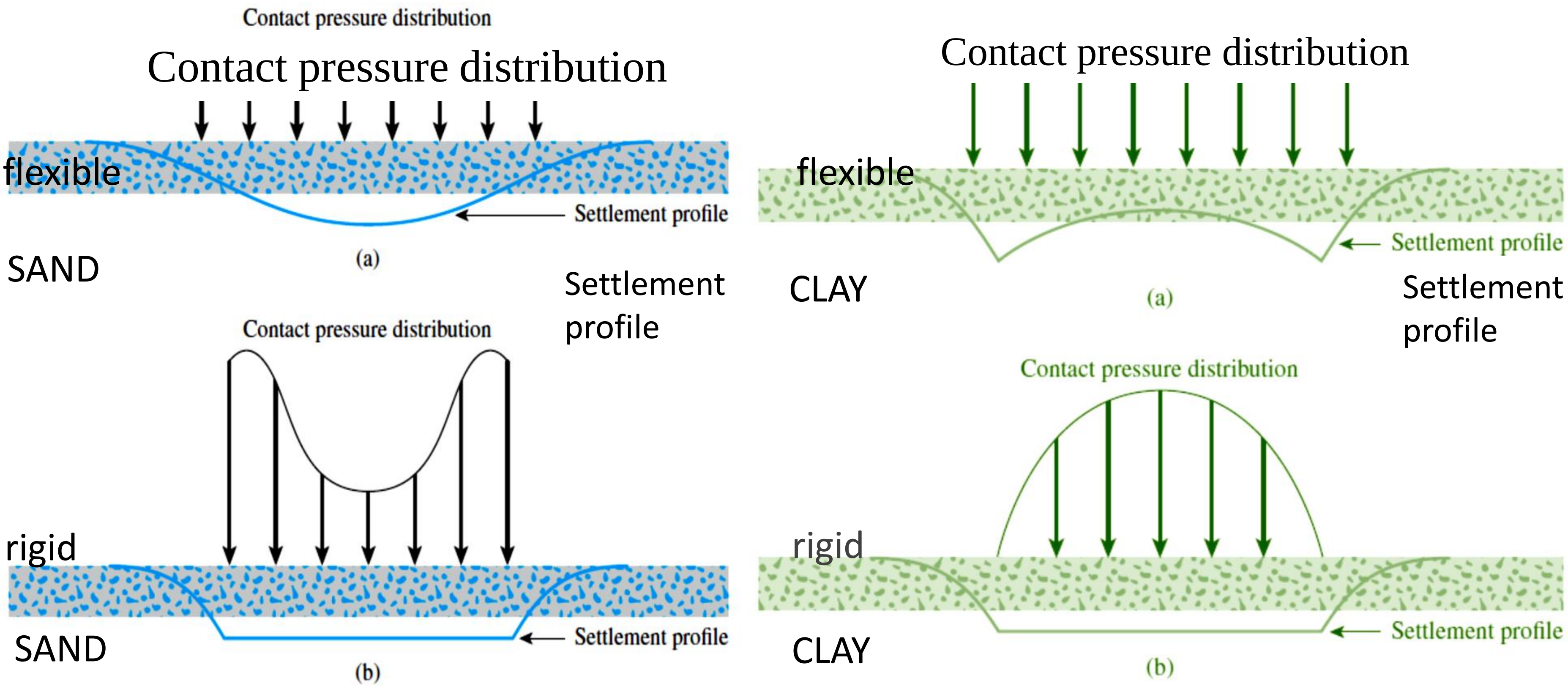
# Cont.…

- There will not be any curvature(no bending) along its length or width even if it experiences the concentrated loading and hence the pressure distribution beneath such footing remains linear.
- In flexible footing, the footing is considered to have some degree of flexibility and hence upon application of partial pressure or concentrated load the footing bends. As the footing attains the bending curvature, the soil beneath the footing base experiences non linear pressure distribution.

# Contact Pressure and Settlement Profile

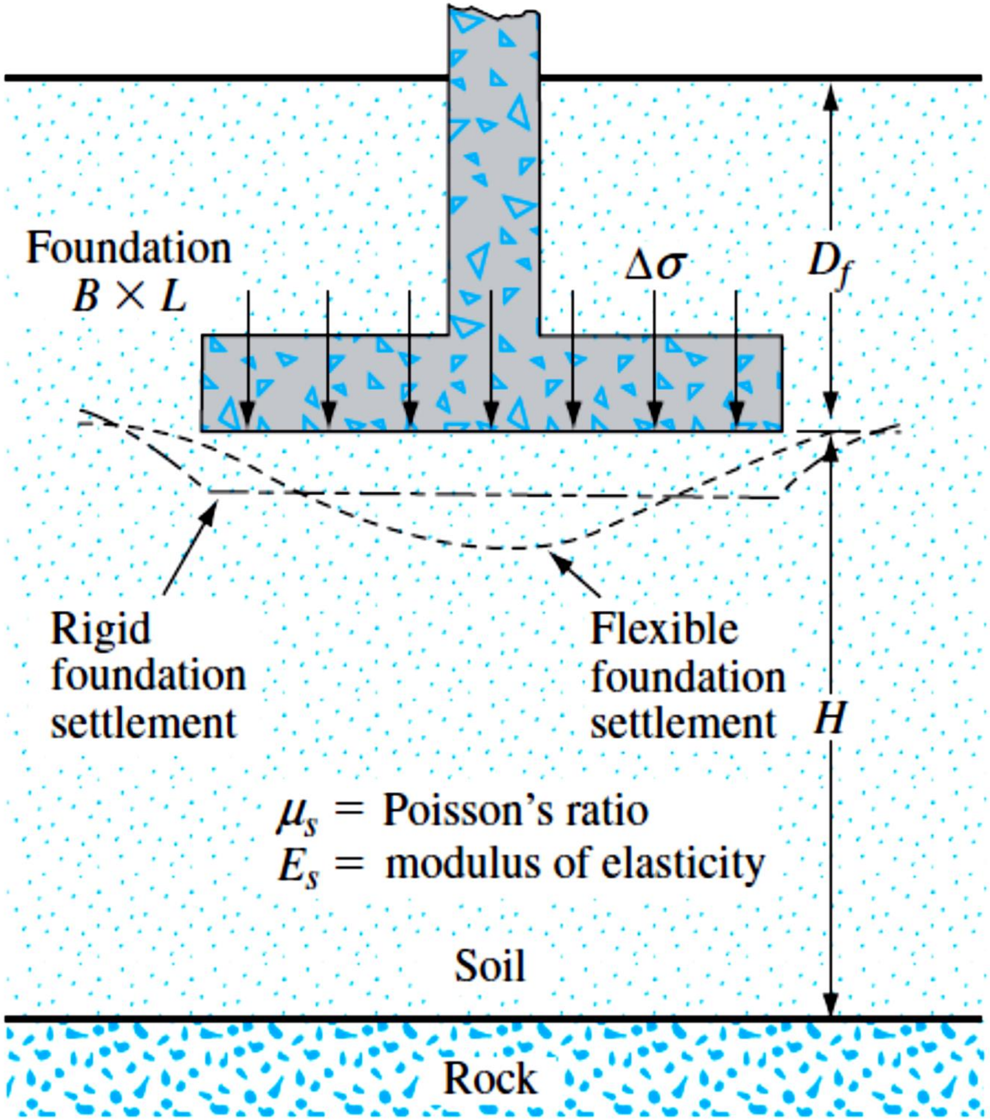
The contact pressure distribution and settlement profile under the foundation are not uniform and will depend on:

1. Flexibility of the foundation (flexible or rigid).
2. Type of soil (clay, silt, sand, or gravel).



# Settlement based on theory of Elasticity

For flexible shallow foundation subjected to a net force per unit area:



# Settlement based on theory of Elasticity

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

where $q$ = net applied pressure on the foundation
$\mu_s$ = Poisson's ratio of soil
$E_s$ = average modulus of elasticity of the soil under the foundation, measured from $z$ = 0 to about $z = 4B$
$B' = B/2$ for center of foundation (= $B$ for corner of foundation)

$I_s$ = shape factor (Steinbrenner, 1934) $= F_1 + \dfrac{1-2\mu_s}{1-\mu_s} F_2$

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

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

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

# Cont….

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

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

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

$\alpha'$ = a factor that depends on the location below the foundation where settlement is being calculated

## Cont….

To calculate settlement at the center of the foundation, we use

$$\alpha' = 4$$

$$m = \frac{L}{B} \qquad \text{and} \qquad n = \frac{H}{\left(\frac{B}{2}\right)}$$

To calculate settlement at a corner of the foundation,

$$\alpha' = 1$$

$$m = \frac{L}{B} \qquad \text{and} \qquad n = \frac{H}{B}$$

Cont….

The variations of $F_1$ and $F_2$ with $m$ and $n$ are given Tables 4 and 5.

Based on the works of Fox (1948), the variations of depth factor $I_f$ for $\mu_s = 0.3$ and 0.4 and $L/B$ have been determined by Bowles (1987) and are given in Table 6. Note that $I_f$ is not a function of $H/B$.

Table 4

Table 5

Table 6

Table 4. Variation of $F_1$ with $m$ and $n$

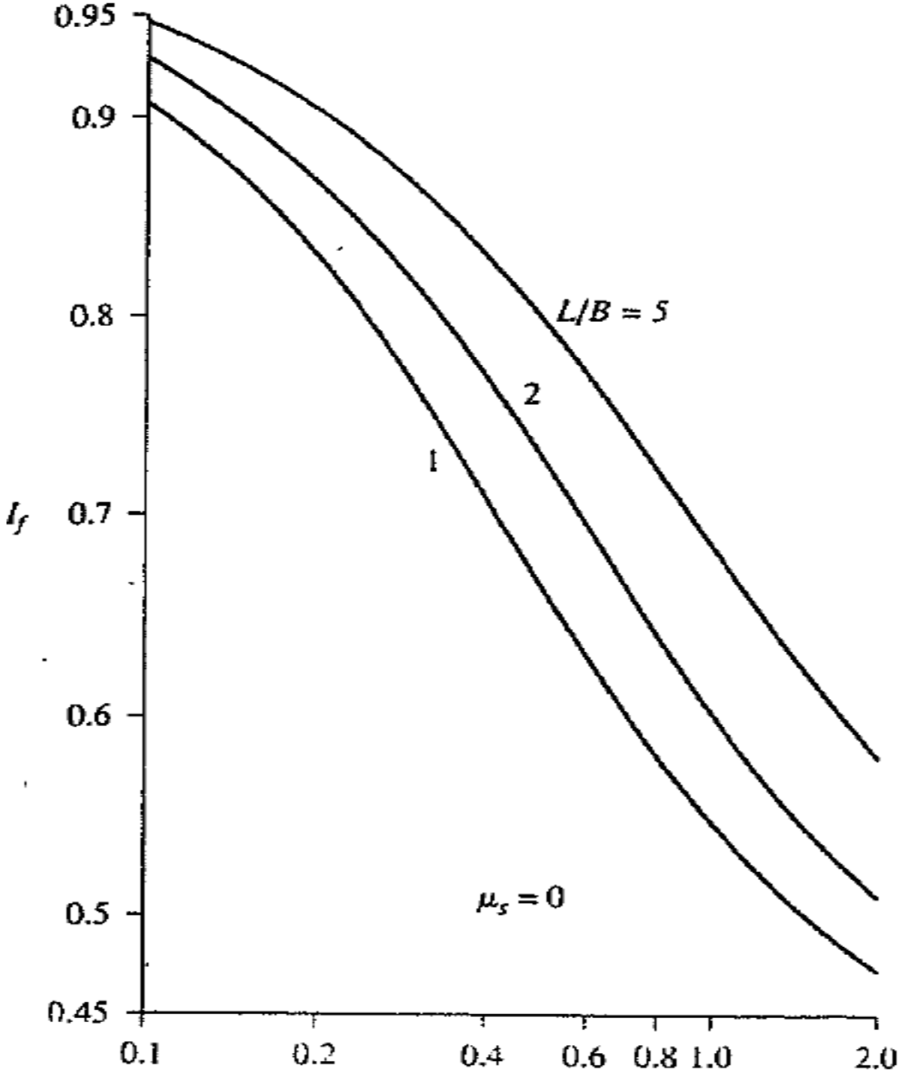
| $n$ | $m$ = 4.5 | 5.0 | 6.0 | 7.8 | 8.0 | 9.0 | 10.0 | 25.0 | 50.0 | 100.0 |
|---|---|---|---|---|---|---|---|---|---|---|
| 5.25 | 0.569 | 0.568 | 0.564 | 0.560 | 0.556 | 0.553 | 0.550 | 0.537 | 0.534 | 0.534 |
| 5.50 | 0.584 | 0.583 | 0.579 | 0.575 | 0.571 | 0.568 | 0.585 | 0.551 | 0.549 | 0.548 |
| 5.75 | 0.597 | 0.597 | 0.594 | 0.590 | 0.586 | 0.583 | 0.580 | 0.565 | 0.583 | 0.562 |
| 6.00 | 0.611 | 0.610 | 0.608 | 0.604 | 0.601 | 0.598 | 0.595 | 0.579 | 0.576 | 0.575 |
| 6.25 | 0.623 | 0.623 | 0.621 | 0.618 | 0.615 | 0.611 | 0.608 | 0.592 | 0.589 | 0.588 |
| 6.50 | 0.635 | 0.635 | 0.634 | 0.631 | 0.628 | 0.625 | 0.622 | 0.605 | 0.601 | 0.600 |
| 6.75 | 0.646 | 0.647 | 0.646 | 0.644 | 0.641 | 0.637 | 0.634 | 0.617 | 0.613 | 0.612 |
| 7.00 | 0.656 | 0.658 | 0.658 | 0.656 | 0.653 | 0.650 | 0.647 | 0.628 | 0.624 | 0.623 |
| 7.25 | 0.666 | 0.669 | 0.669 | 0.668 | 0.665 | 0.662 | 0.659 | 0.640 | 0.635 | 0.634 |
| 7.50 | 0.676 | 0.679 | 0.680 | 0.679 | 0.676 | 0.673 | 0.670 | 0.651 | 0.646 | 0.645 |
| 7.75 | 0.685 | 0.688 | 0.690 | 0.689 | 0.687 | 0.684 | 0.681 | 0.661 | 0.656 | 0.655 |
| 8.00 | 0.694 | 0.697 | 0.700 | 0.700 | 0.698 | 0.695 | 0.692 | 0.672 | 0.666 | 0.665 |
| 8.25 | 0.702 | 0.706 | 0.710 | 0.710 | 0.708 | 0.705 | 0.703 | 0.682 | 0.676 | 0.675 |
| 8.50 | 0.710 | 0.714 | 0.719 | 0.719 | 0.718 | 0.715 | 0.713 | 0.692 | 0.686 | 0.684 |
| 8.75 | 0.717 | 0.722 | 0.727 | 0.728 | 0.727 | 0.725 | 0.723 | 0.701 | 0.695 | 0.693 |
| 9.00 | 0.725 | 0.730 | 0.736 | 0.737 | 0.736 | 0.735 | 0.732 | 0.710 | 0.704 | 0.702 |
| 9.25 | 0.731 | 0.737 | 0.744 | 0.746 | 0.745 | 0.744 | 0.742 | 0.719 | 0.713 | 0.711 |
| 9.50 | 0.738 | 0.744 | 0.752 | 0.754 | 0.754 | 0.753 | 0.751 | 0.728 | 0.721 | 0.719 |
| 9.75 | 0.744 | 0.751 | 0.759 | 0.762 | 0.762 | 0.761 | 0.759 | 0.737 | 0.729 | 0.727 |
| 10.00 | 0.750 | 0.758 | 0.766 | 0.770 | 0.770 | 0.770 | 0.768 | 0.745 | 0.738 | 0.735 |
| 20.00 | 0.878 | 0.896 | 0.925 | 0.945 | 0.959 | 0.969 | 0.977 | 0.982 | 0.965 | 0.957 |
| 50.00 | 0.962 | 0.989 | 1.034 | 1.070 | 1.100 | 1.125 | 1.146 | 1.265 | 1.279 | 1.261 |
| 100.00 | 0.990 | 1.020 | 1.072 | 1.114 | 1.150 | 1.182 | 1.209 | 1.408 | 1.489 | 1.499 |

Table 5. Variation of $F_2$ with $m$ and $n$

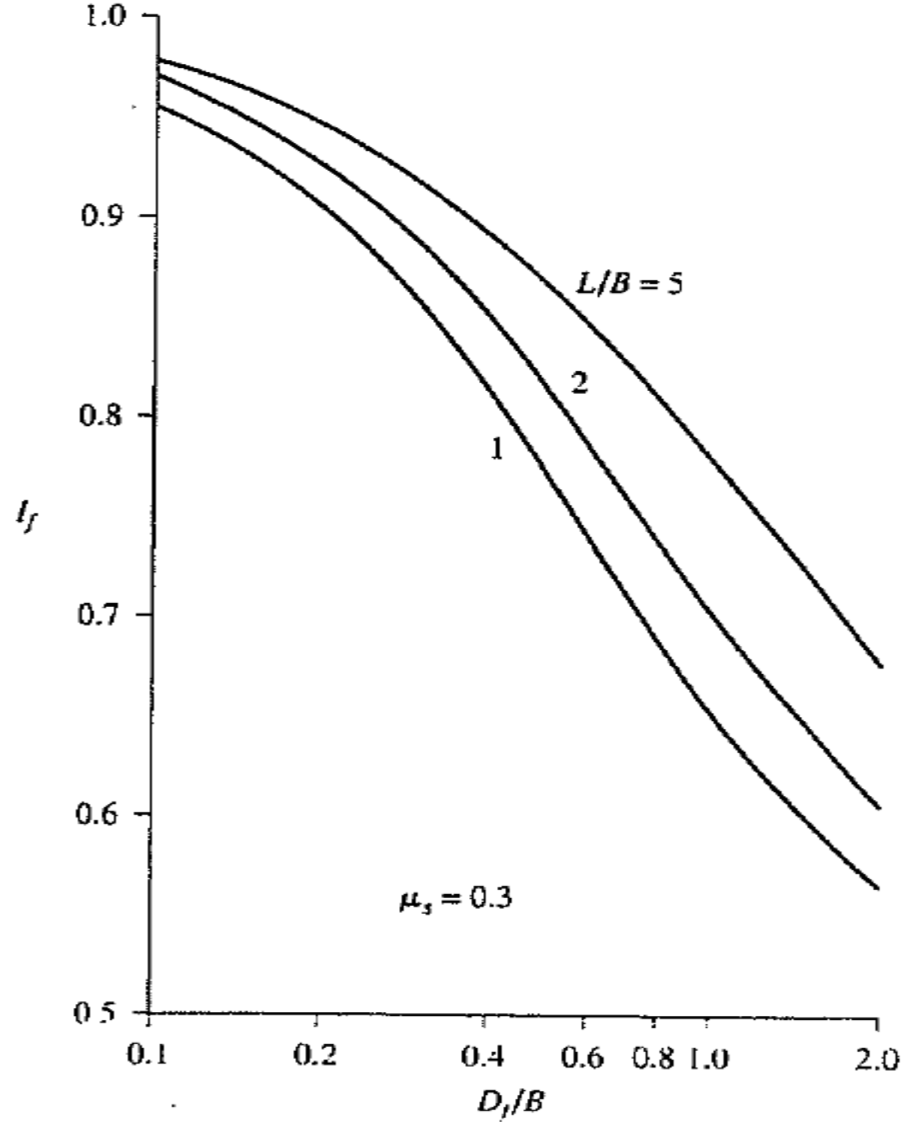
| $n$ | $m$ = 4.5 | 5.0 | 6.0 | 7.0 | 8.0 | 9.0 | 10.0 | 25.0 | 50.0 | 100.0 |
|---|---|---|---|---|---|---|---|---|---|---|
| 5.25 | 0.102 | 0.108 | 0.118 | 0.126 | 0.131 | 0.136 | 0.139 | 0.154 | 0.156 | 0.157 |
| 5.50 | 0.099 | 0.106 | 0.116 | 0.124 | 0.130 | 0.134 | 0.138 | 0.154 | 0.156 | 0.157 |
| 5.75 | 0.097 | 0.103 | 0.113 | 0.122 | 0.128 | 0.133 | 0.136 | 0.154 | 0.157 | 0.157 |
| 6.00 | 0.094 | 0.101 | 0.111 | 0.120 | 0.126 | 0.131 | 0.135 | 0.153 | 0.157 | 0.157 |
| 6.25 | 0.092 | 0.098 | 0.109 | 0.118 | 0.124 | 0.129 | 0.134 | 0.153 | 0.157 | 0.158 |
| 6.50 | 0.090 | 0.096 | 0.107 | 0.116 | 0.122 | 0.128 | 0.132 | 0.153 | 0.157 | 0.158 |
| 6.75 | 0.087 | 0.094 | 0.105 | 0.114 | 0.121 | 0.126 | 0.131 | 0.153 | 0.157 | 0.158 |
| 7.00 | 0.085 | 0.092 | 0.103 | 0.112 | 0.119 | 0.125 | 0.129 | 0.152 | 0.157 | 0.158 |
| 7.25 | 0.083 | 0.090 | 0.101 | 0.110 | 0.117 | 0.123 | 0.128 | 0.152 | 0.157 | 0.158 |
| 7.50 | 0.081 | 0.088 | 0.099 | 0.108 | 0.115 | 0.121 | 0.126 | 0.152 | 0.156 | 0.158 |
| 7.75 | 0.079 | 0.086 | 0.097 | 0.106 | 0.114 | 0.120 | 0.125 | 0.151 | 0.156 | 0.158 |
| 8.00 | 0.077 | 0.084 | 0.095 | 0.104 | 0.112 | 0.118 | 0.124 | 0.151 | 0.156 | 0.158 |
| 8.25 | 0.076 | 0.082 | 0.093 | 0.102 | 0.110 | 0.117 | 0.122 | 0.150 | 0.156 | 0.158 |
| 8.50 | 0.074 | 0.080 | 0.091 | 0.101 | 0.108 | 0.115 | 0.121 | 0.150 | 0.156 | 0.158 |
| 8.75 | 0.072 | 0.078 | 0.089 | 0.099 | 0.107 | 0.114 | 0.119 | 0.150 | 0.156 | 0.158 |
| 9.00 | 0.071 | 0.077 | 0.888 | 0.097 | 0.105 | 0.112 | 0.118 | 0.149 | 0.156 | 0.158 |
| 9.25 | 0.069 | 0.075 | 0.086 | 0.096 | 0.104 | 0.110 | 0.116 | 0.149 | 0.156 | 0.158 |
| 9.50 | 0.068 | 0.074 | 0.085 | 0.094 | 0.102 | 0.109 | 0.115 | 0.148 | 0.156 | 0.158 |
| 9.75 | 0.066 | 0.072 | 0.083 | 0.092 | 0.100 | 0.107 | 0.113 | 0.148 | 0.156 | 0.158 |
| 10.00 | 0.065 | 0.071 | 0.082 | 0.091 | 0.099 | 0.106 | 0.112 | 0.147 | 0.156 | 0.158 |
| 20.00 | 0.035 | 0.039 | 0.046 | 0.053 | 0.059 | 0.065 | 0.071 | 0.124 | 0.148 | 0.156 |
| 50.00 | 0.014 | 0.016 | 0.019 | 0.022 | 0.025 | 0.028 | 0.031 | 0.071 | 0.113 | 0.142 |
| 100.00 | 0.007 | 0.008 | 0.010 | 0.011 | 0.013 | 0.014 | 0.016 | 0.039 | 0.071 | 0.113 |

**Table 6** Variation of $I_f$ with $L/B$ and $D_f/B$

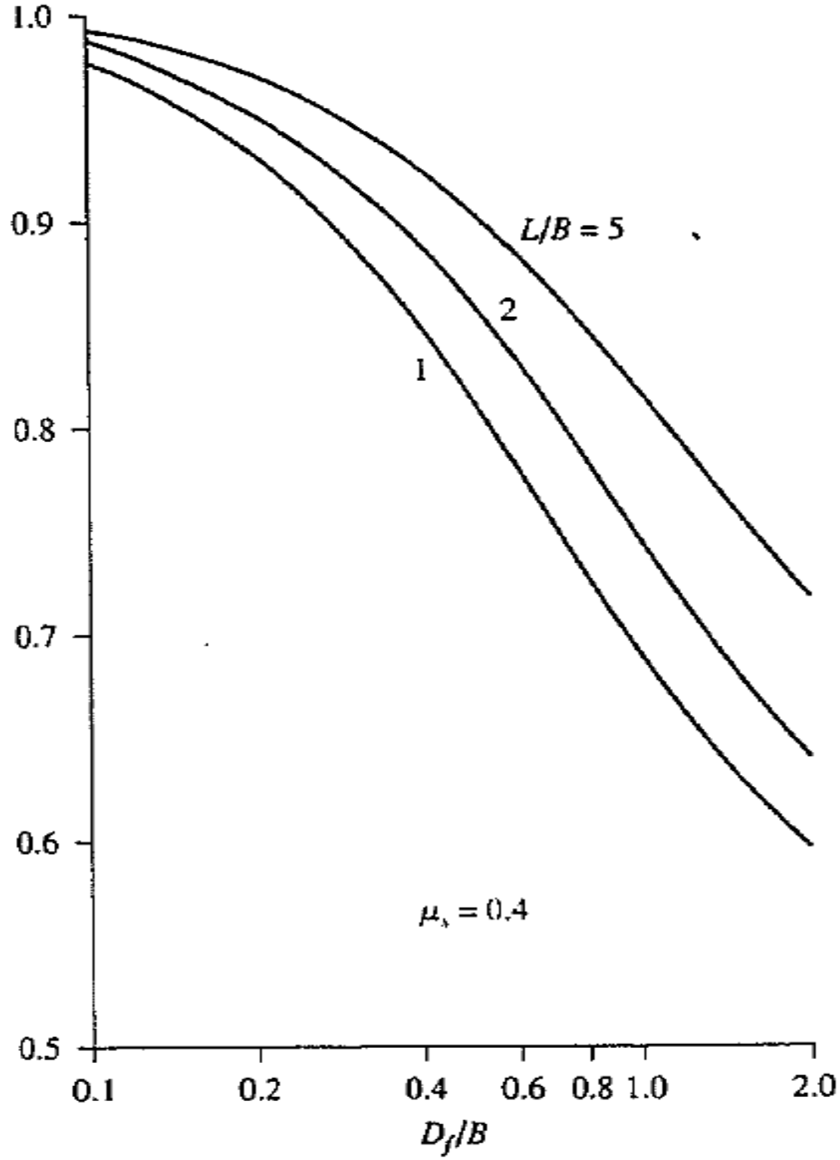
| | | $I_f$ | | |
|---|---|---|---|---|
| $L/B$ | $D_f/B$ | $\mu_s = 0.3$ | $\mu_s = 0.4$ | $\mu_s = 0.5$ |
| 1 | 0.5 | 0.77 | 0.82 | 0.85 |
| | 0.75 | 0.69 | 0.74 | 0.77 |
| | 1 | 0.65 | 0.69 | 0.72 |
| 2 | 0.5 | 0.82 | 0.86 | 0.89 |
| | 0.75 | 0.75 | 0.79 | 0.83 |
| | 1 | 0.71 | 0.75 | 0.79 |
| 5 | 0.5 | 0.87 | 0.91 | 0.93 |
| | 0.75 | 0.81 | 0.86 | 0.89 |
| | 1 | 0.78 | 0.82 | 0.85 |


0.95
0.9
0.8
0.7
0.6
0.5
0.45
$I_f$
$L/B = 5$
2
1
$\mu_s = 0$
0.1
0.2
0.4
0.6
0.8
1.0
2.0
$D_f/B$

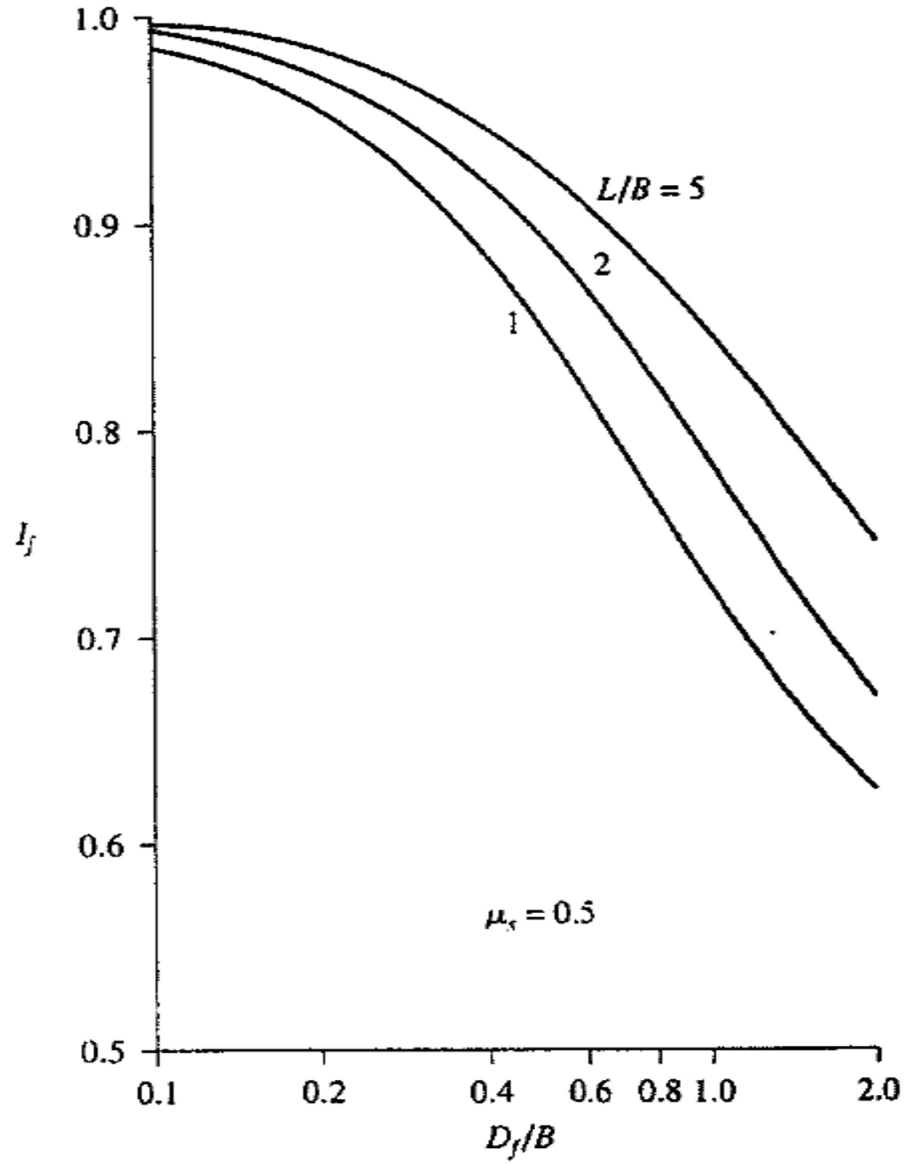
(a)


1.0
0.9
0.8
0.7
0.6
0.5
$I_f$
$L/B = 5$
2
1
$\mu_s = 0.3$
0.1
0.2
0.4
0.6
0.8
1.0
2.0
$D_f/B$

(b)


1.0
0.9
0.8
0.7
0.6
0.5
0.1
0.2
0.4
0.6
0.8
1.0
2.0
$D_f/B$
$L/B = 5$
2
1
$\mu_s = 0.4$

(c)

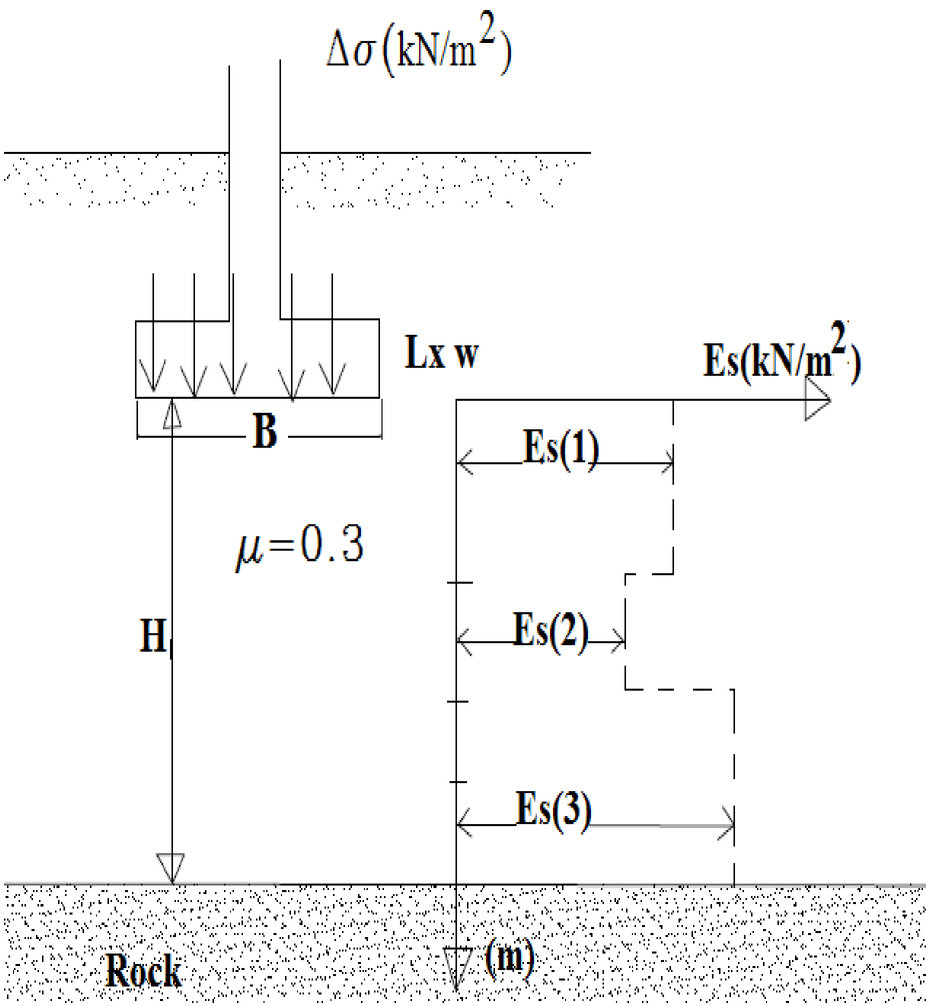

1.0
0.9
0.8
0.7
0.6
0.5
$I_f$
0.1
0.2
0.4
0.6
0.8
1.0
2.0
$D_f/B$
$L/B = 5$
2
1
$\mu_s = 0.5$

(d)

# Cont….

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

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




where:

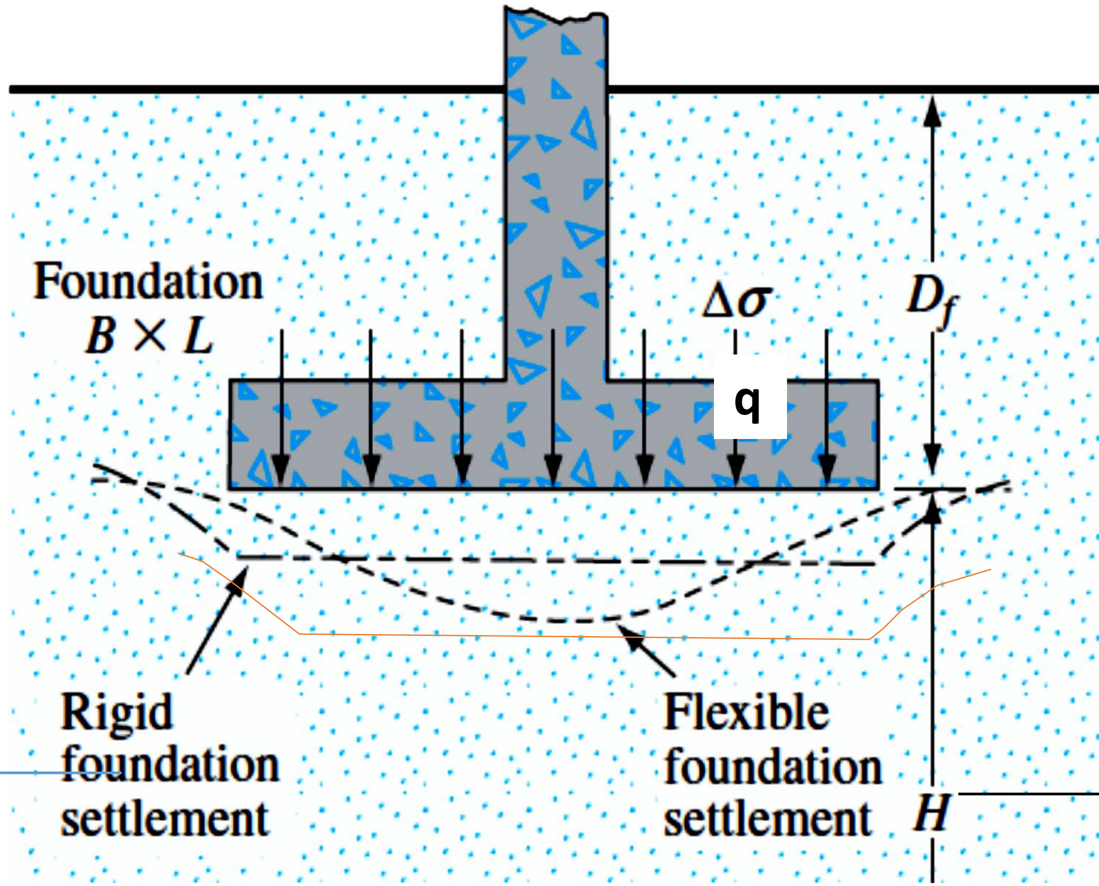
$E_{s(i)}$ soil modulus of elasticity within a depth Dz.

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

**Table 7** Representative Values of the Modulus of Elasticity of Soil

| Soil type | $E_s$ (kN/m²) | $E_s$ (lb/in.²) |
|---|---|---|
| Soft clay | 1,800–3,500 | 250–500 |
| Hard clay | 6,000–14,000 | 850–2,000 |
| Loose sand | 10,000–28,000 | 1,500–4,000 |
| Dense sand | 35,000–70,000 | 5,000–10,000 |

Thick foundation




**Thin** foundation

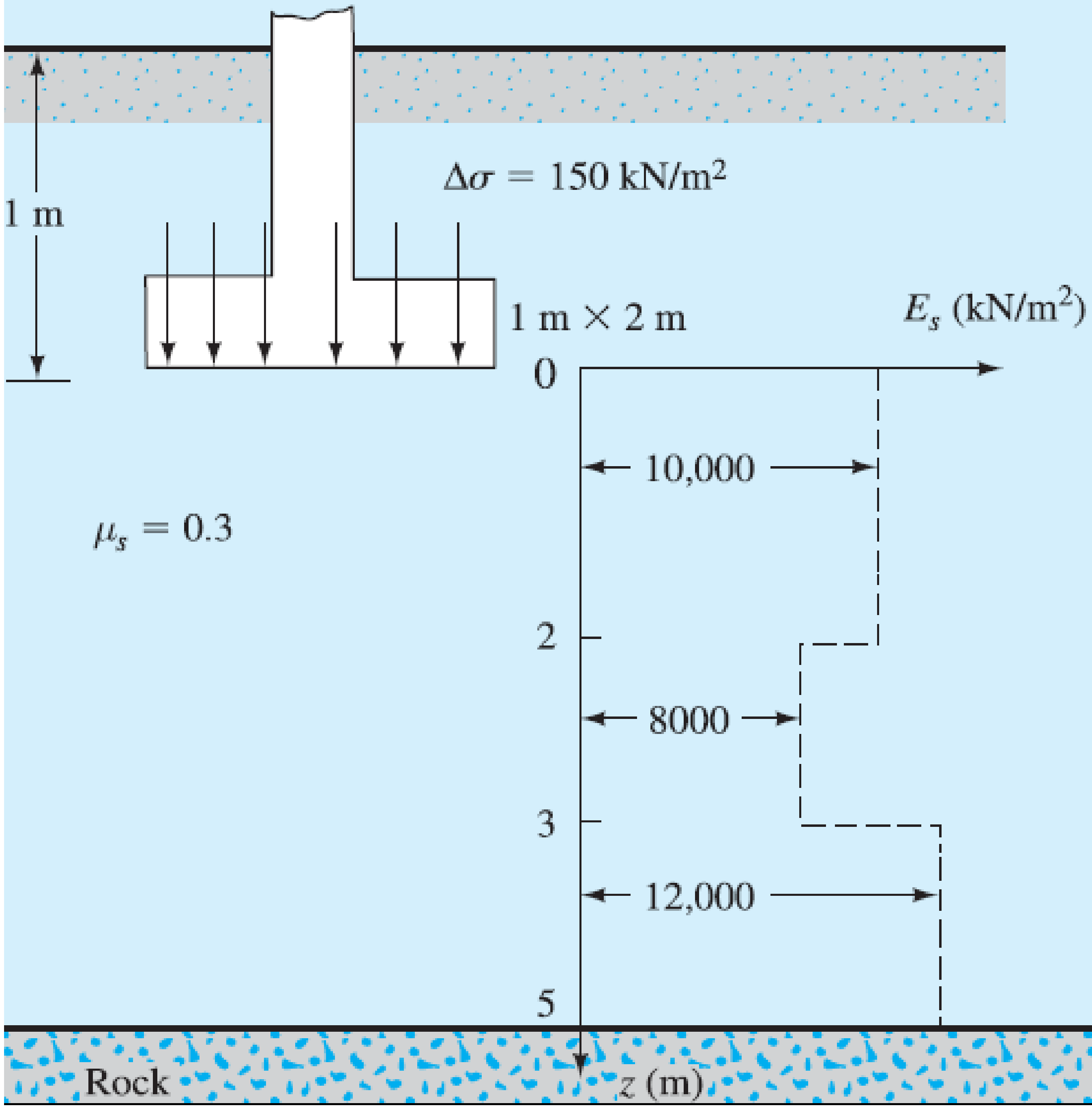
$$S_e = q(\alpha' B')\frac{1-\mu_s^2}{E_s} I_s I_f$$

To rigid

$$S_{e(rigid)} = 0.93 S_{e(flexible)}$$

## Example 3.1

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



# Solution

- Given: $B$=1m and $L$=2m. Note that $\tilde{z}$=5m=5B.

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

$$\frac{(10{,}000*2) + (800*1) + (12{,}000*2)}{5} = 10{,}400 \text{kN/m}^2$$

For the *center of the foundation*

$$\alpha' = 4$$

$$m = \frac{L}{B} = 2/1 = 2 \quad \text{and} \quad n = \frac{H}{\left(\frac{B}{2}\right)} = \frac{5}{1/2} = 10$$

From Tables 4 and 5, $F1$= 0.641 and $F2$ =0.031.

# Solution

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

- $I_s = 0.641 + \frac{1-2*0.3}{1-0.3}(0.031) = 0.66$

$\frac{D_f}{B} = 1$ , $\frac{L}{B} = 2$, $\mu_S = 0.3$ from table 6 $I_f = 0.71$

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

$S_e = (150)(4*1/2)\frac{1-0.3^2}{10400}(0.66 * 0.71) = \mathbf{12.3}\boldsymbol{mm}$

- $\boldsymbol{S_{e\ (rigid)}}$ = 0.93*12.3 = ***11.44mm***

# Elastic Settlement of saturated clay

The average vertical immediate displacement under a flexible area carrying a uniform pressure q is given by

$$\boxed{s_e = A_1 A_2 \frac{q_0 B}{E_s}}$$

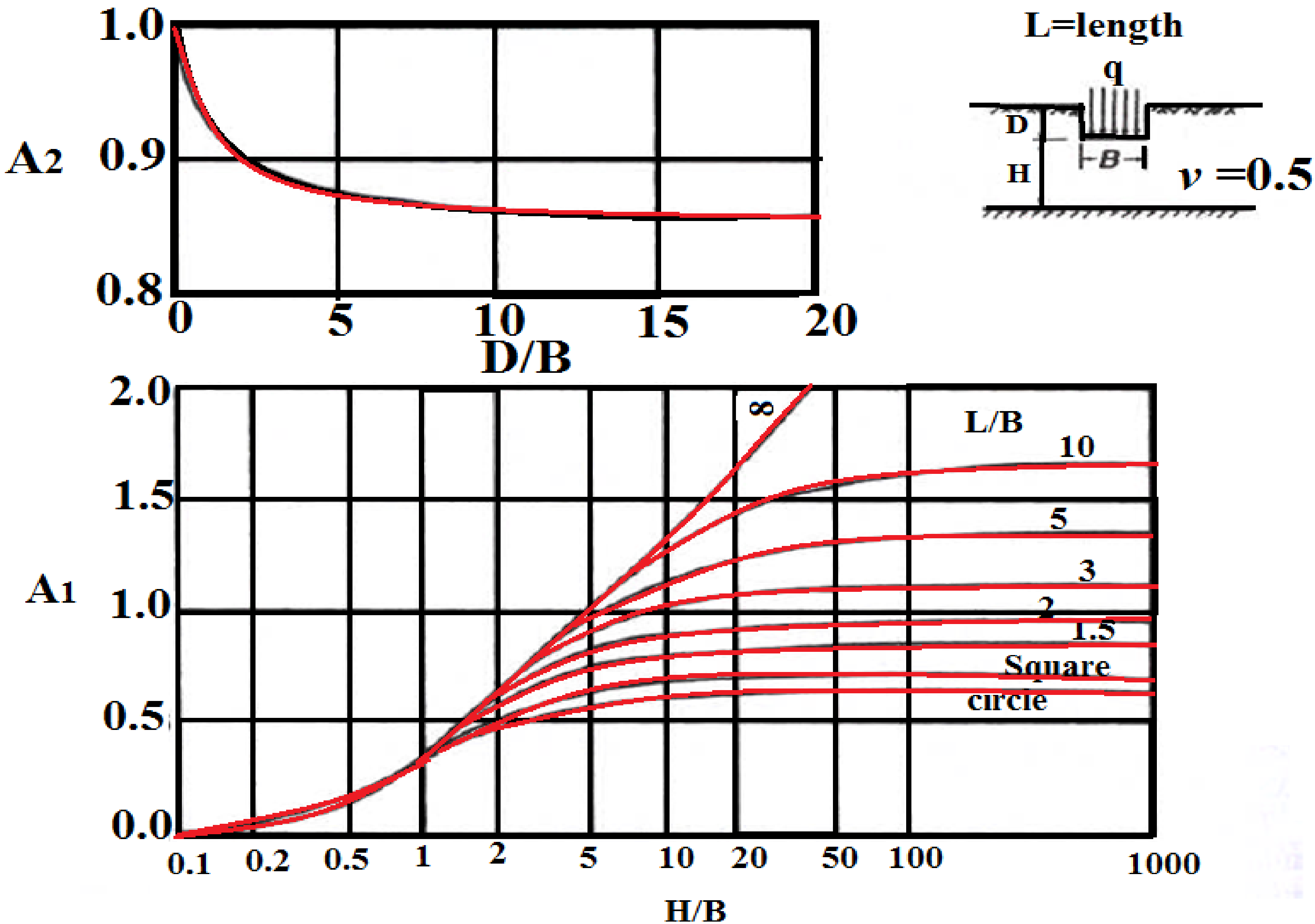
where

$A_1$ depends on the shape of the loaded area

$A_2$ depends on the depth of footing

The above solutions for vertical displacement are used mainly to estimate the immediate settlement of foundation on saturated clays; such settlement occurs under undrained conditions, the appropriate value of Poisson's ratio being 0.5. The value of the undrained modulus $E_s$ is therefore required.

## Elastic Settlement of saturated clay

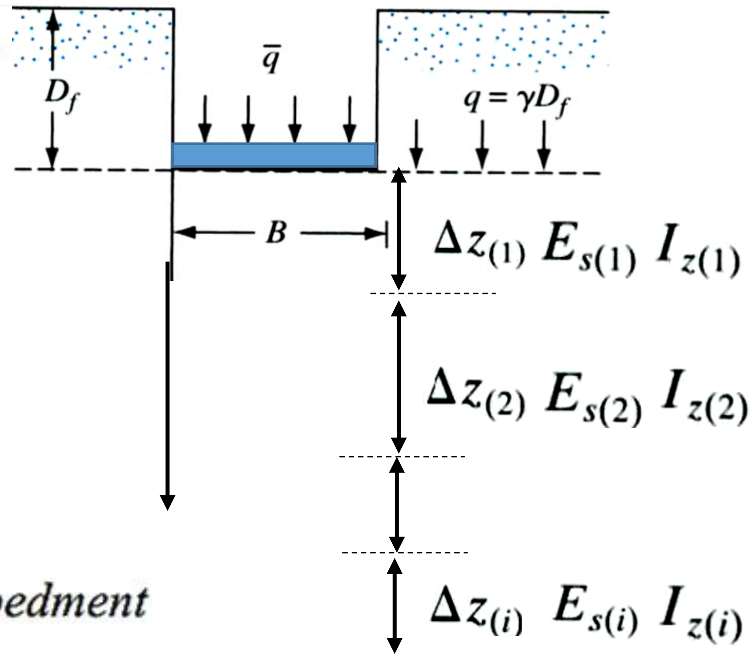



**Example 2**

# Elastic Settlement of sandy soil: use of strain influence factor

## (Settlement in granular soils by using CPT values)

***The settlement of granular soils can also be evaluated by the use of semiemperical strain influence factor proposed by Schmertmann (1978).***




$$\boxed{S_e = C_1 C_2 (\bar{q} - q) \sum_{0}^{z_2} \frac{I_z}{E_s} \Delta z}$$

$I_z =$ Strain influence factor ***(changes with depth)***

$C_1 =$ *a correction factor for the depth of foundation embedment*

$C_2 =$ *a correction factor to account for creep in soil*

$\bar{q} =$ *stress at the level of the foundation* ***(applied by the foundation)***

$q = \gamma D_f$ = effective stress at the base of the foundation

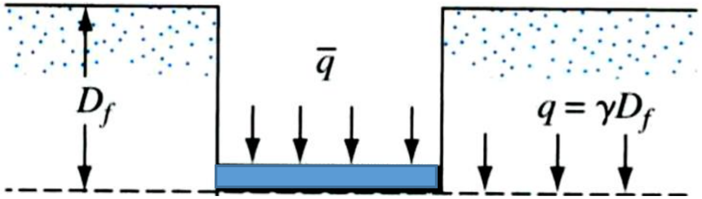
$E_s$ = modulus of elasticity of soil ***(changes with depth)***

# Elastic Settlement of sandy soil: use of strain influence factor

**Correction factor for depth ($C_1$) and correction factor for creep ($C_2$) are given by:**

$$C_1 = 1 - 0.5\left(\frac{q}{\bar{q} - q}\right)$$

$$C_2 = 1 + 0.2\log\left(\frac{\text{time in years}}{0.1}\right)$$




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

$q = \gamma D_f$ = effective stress at the base of the foundation

# ➤ Elastic Settlement of sandy soil: use of strain influence factor

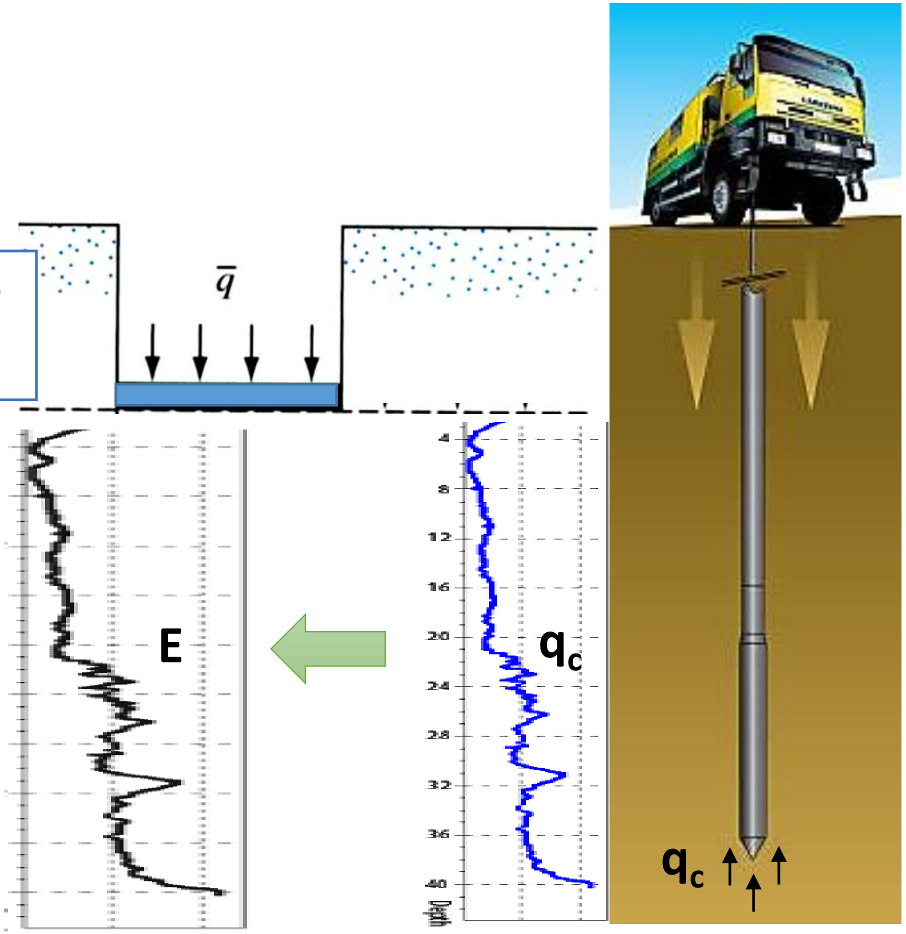
$E_s$ = modulus of elasticity of soil

*is obtained from CPT test*

***Es=2.5qc for square or circular foundation***
***Es=3.5qc for strip foundation***

$1 < L/B < 10 \rightarrow$ *by interpolation we can find:*

$$E = \left[2.5 + \left(\frac{L}{B} - 1\right)\frac{1}{9}\right] q_c$$




$q_c$ = cone penetration resistance

# Elastic Settlement of sandy soil: use of strain influence factor

**Note**

Can use $N_{SPT}$ to estimate $q_c$ if no CPT data available. This is a common and generally conservative procedure, but there is significant uncertainty due to the variability of the SPT (some prefer to use $q_c$ to estimate N). The ratio ($q_C$ / $N_{SPT}$) increases as the mean grain size increases.

$q_c = C_3 N$ [tsf]

$q_c \boxed{= 95.76 \times C_3 N \text{ [kPa]}}$

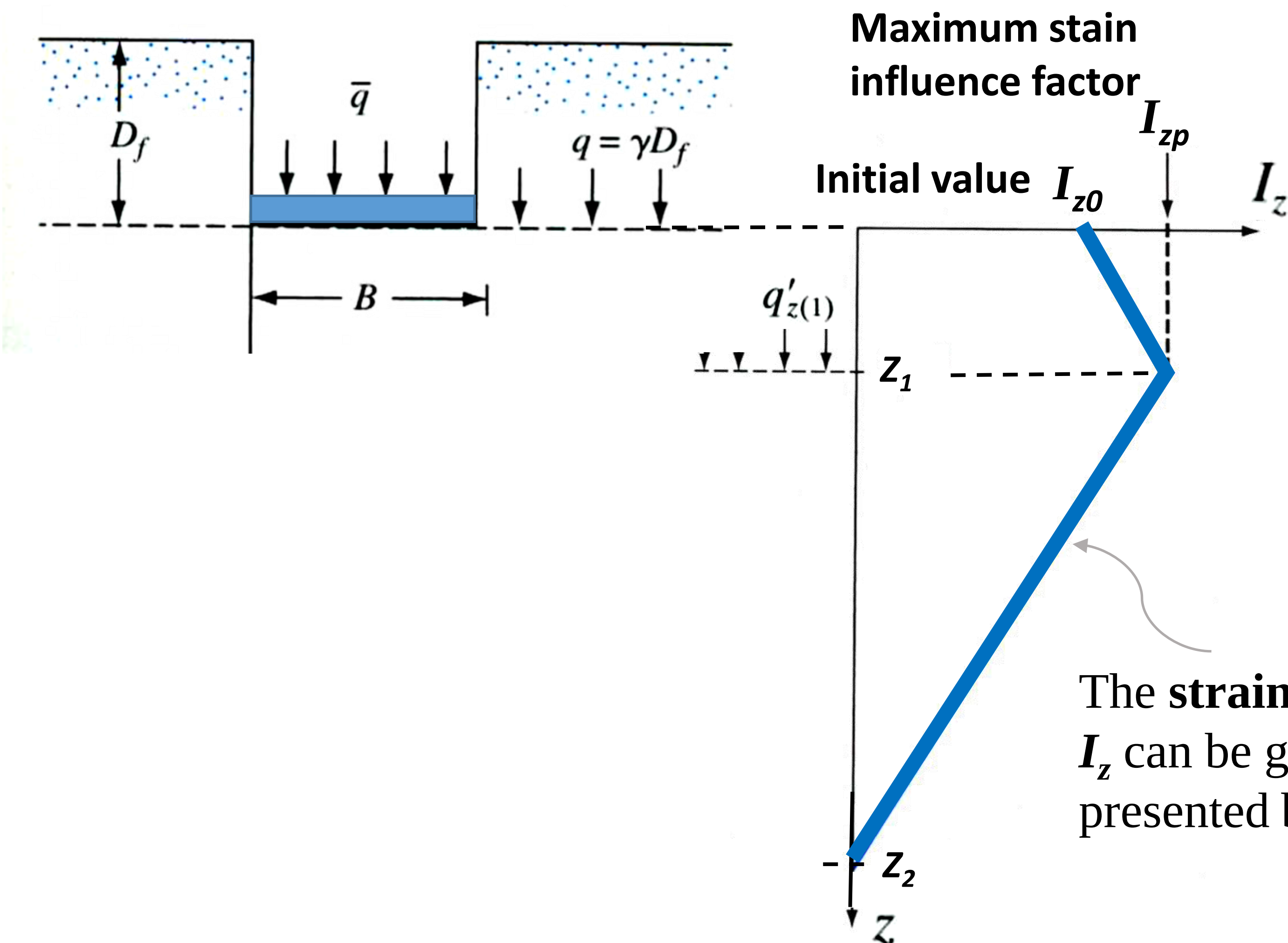
$q_c$ = Dutch cone penetration resistance tsf [x 95.76 = kPa]

N = uncorrected SPT blow counts [blow/ft or blow/0.3m]

$C_3$ = a material constant that estimated as follows:

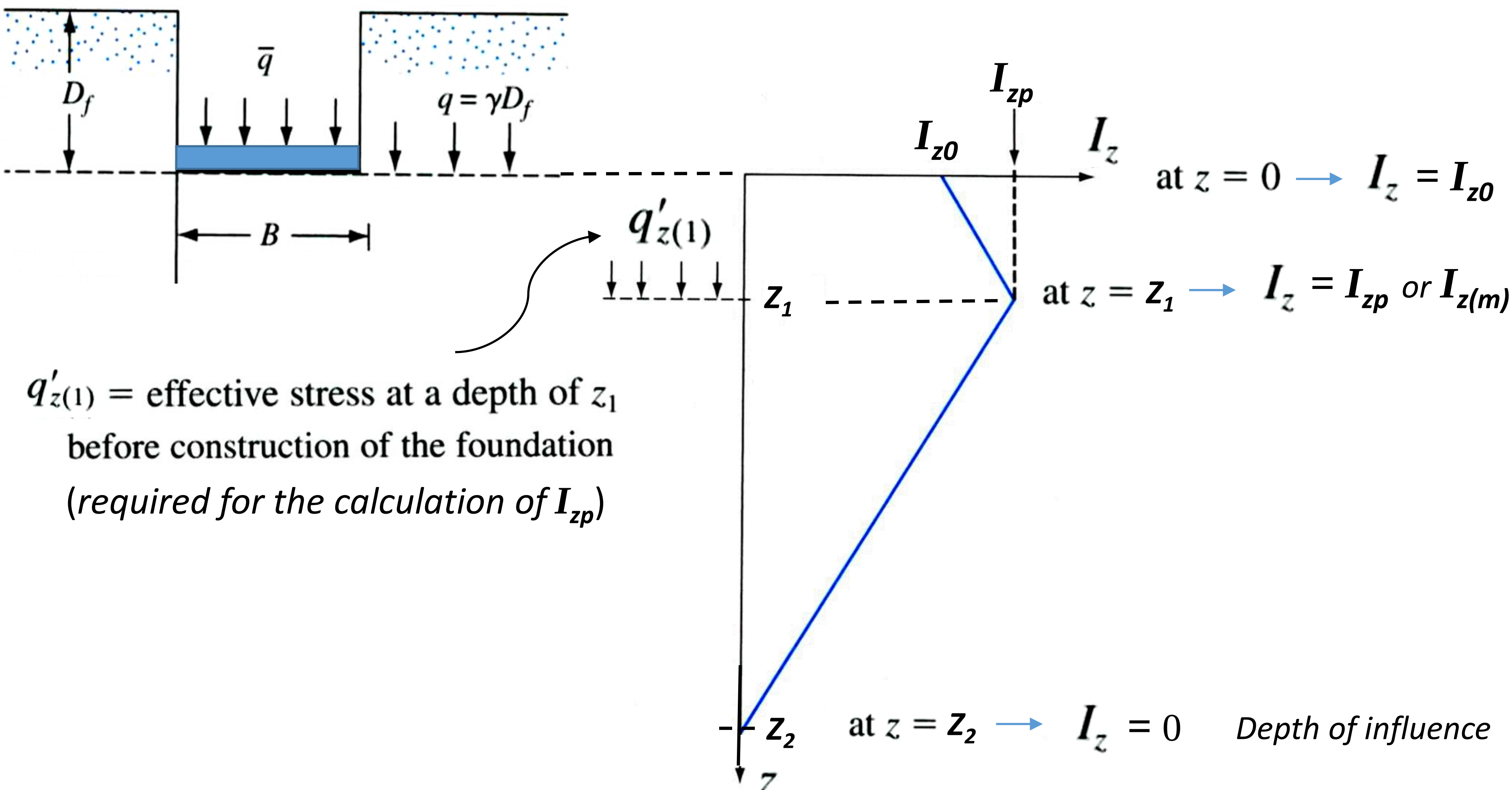
| Soil Type | $q_c / N = C_3$ |
|---|---|
| Clay | 1 |
| Clay to Silty Clay | 1.5 |
| Silty Clay to Clayey Silt and Clay-Silt-Sand mix | 2 |
| Clayey Silt to Sandy Silt | 2.5 |
| Sandy Silt to Silty Sand | 3 |
| Silty Sand to Sand | 4 |
| Sand | 5 |
| Sand to Gravelly Sand | 6 |

# Elastic Settlement of sandy soil: use of strain influence factor




The **strain influence factor** $I_z$ can be graphically presented by this diagram

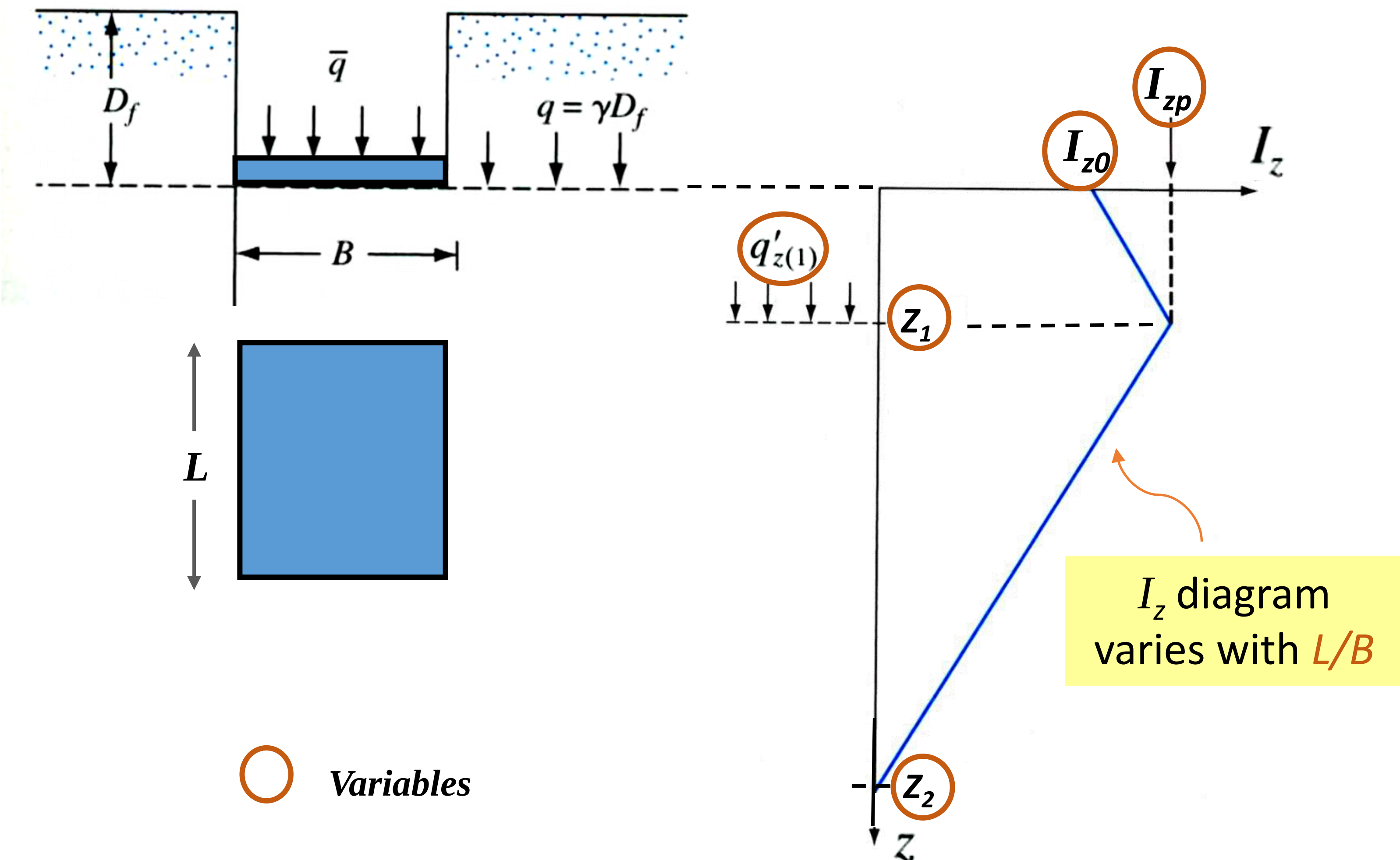
# Elastic Settlement of sandy soil: use of strain influence factor




$q'_{z(1)}$ = effective stress at a depth of $z_1$ before construction of the foundation

(*required for the calculation of $I_{zp}$*)

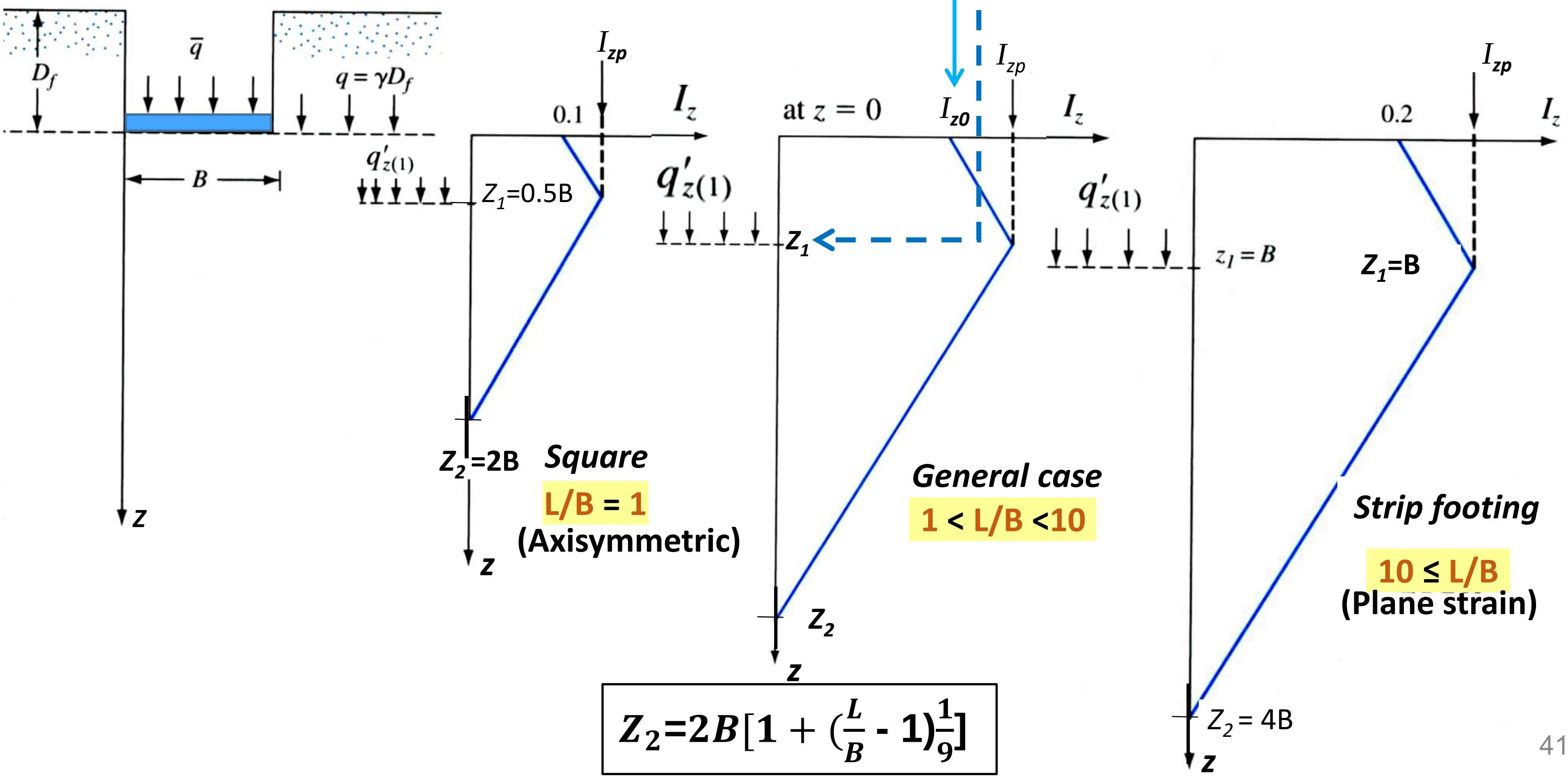
# Elastic Settlement of sandy soil: use of strain influence factor



# Elastic Settlement of sandy soil: use of strain influence factor

$$Z_1 = B[0.5 + (\frac{L}{B} - 1)\frac{0.5}{9}]$$

$$I_{Zo} = 0.1 + (\frac{L}{B} - 1)\frac{0.1}{9}$$




$$Z_2 = 2B[1 + (\frac{L}{B} - 1)\frac{1}{9}]$$

# Elastic Settlement of sandy soil: use of strain influence factor

- **The variation of the strain influence factor with depth. for square or circular foundations,**

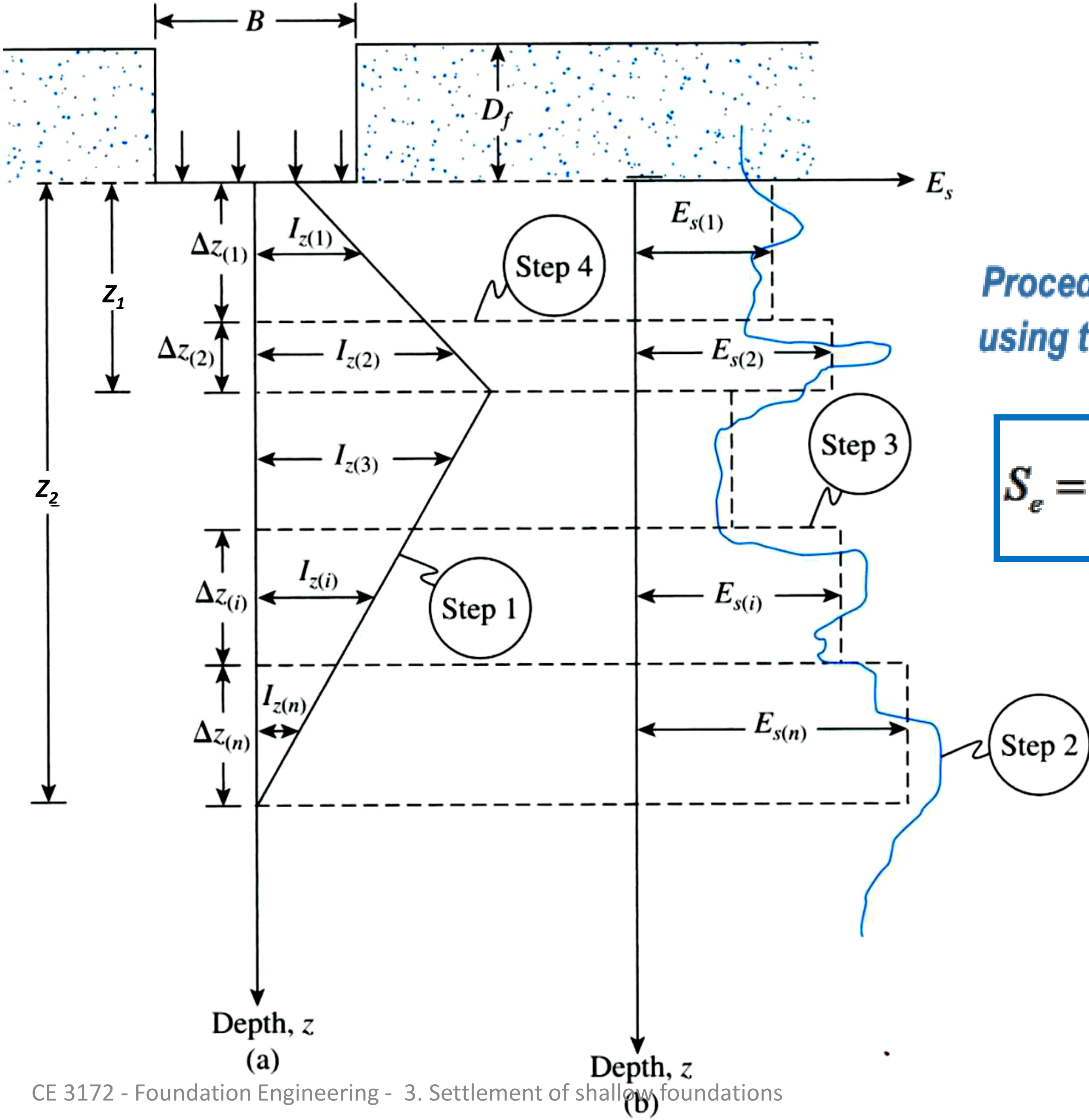
- **$I_z$ = 0.1 at z = 0**
- **$I_z$ = $I_{zp}$ at z = 0.5B**
- **$I_z$ = 0 at 2 = 2B**

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{\overline{q} - q}{q`_{z(1)}}}$$

- **Similarly, for foundations with L/B ≥ 10**

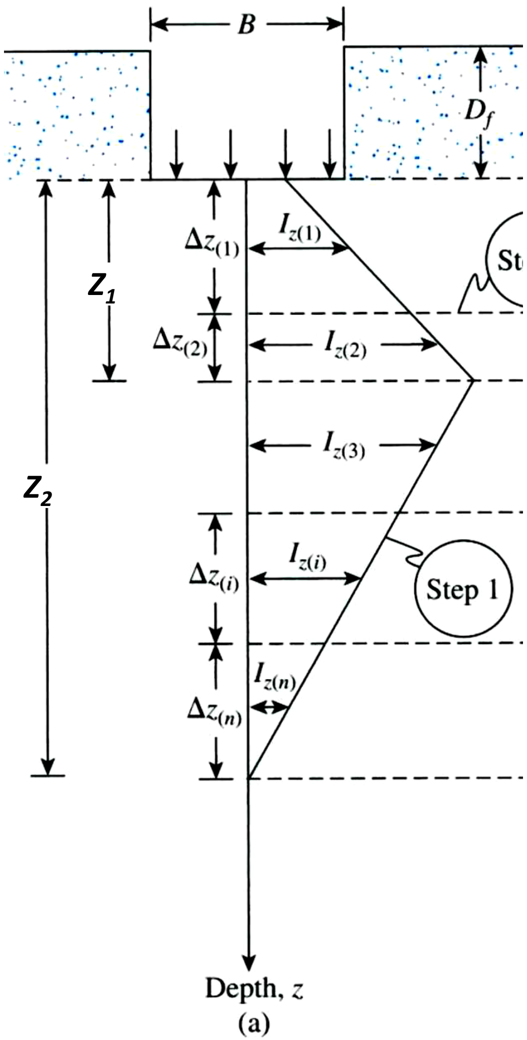
- **$I_z$ = 0.2 at z = 0**
- **$I_z$ = $I_{zp}$ at z = B**
- **$I_z$ = 0 at z = 4B**

## Procedure for calculation of $S_e$ using the strain influence factor




$$S_e = C_1 C_2 (\bar{q} - q) \sum_{0}^{z_2} \frac{I_z}{E_s} \Delta z$$

# Elastic Settlement of sandy soil: use of strain influence factor



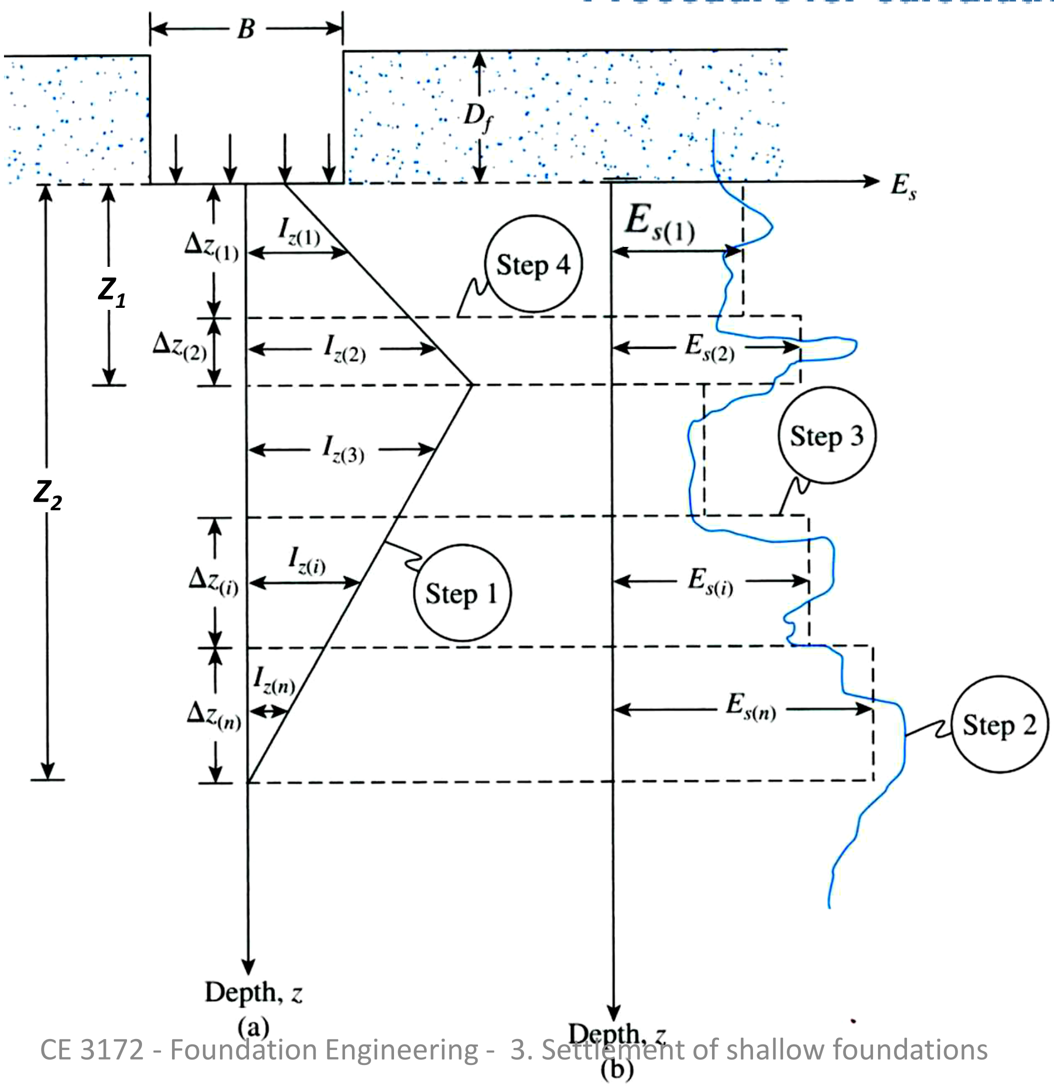

**Procedure for calculation of $S_e$ using the strain influence factor**

**Step1: Plot the variation of $I_z$ with depth**

| 1. Square L/B = 1 | | 2. General case 1 ≤ L/B ≤10 | | 3. Strip 10 ≤ L/B | |
|---|---|---|---|---|---|
| $z$ | $I_z$ | $z$ | $I_z$ | $z$ | $I_z$ |
| 0 | 0.1 | 0 | $0.1+\left(\frac{L}{B}-1\right)\frac{0.1}{9}$ | 0 | 0.2 |
| 0.5B | $I_{zp}$ | $B\left[0.5+\left(\frac{L}{B}-1\right)\frac{0.5}{9}\right]$ | $I_{zp}$ | B | $I_{zp}$ |
| 2B | 0 | $2B[1+(\frac{L}{B}-1)\frac{1}{9}]$ | 0 | 4B | 0 |

# ➤ Elastic Settlement of sandy soil: use of strain influence factor

*Procedure for calculation of $S_e$ using the strain influence factor*




***Step1:** Plot the variation of $I_z$ with depth*

***Step2:** Plot the actual variation of $E_s$ of soil with depth. $E_s$ can be calculated from SPT or CPT results*

***Step3:** Approximate the actual variation of $E_s$ in to number of layers of soil having constant $E_s$*

***Step4:** Divide the soil layer from z=0 to $z=z_2$ into number of layers which will depend on breaking in continuity in $I_z$ and $E_s$ diagrams*

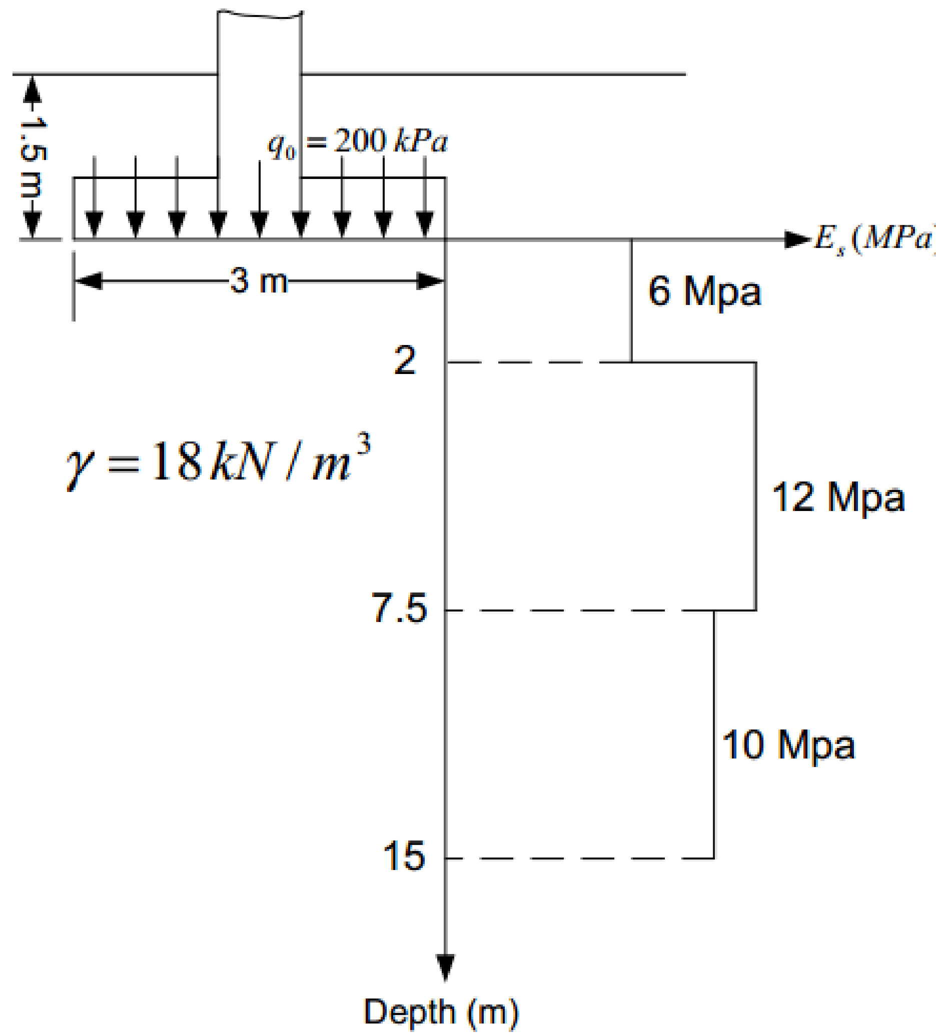
***Step5:** Prepare a table to obtain* $\sum \frac{I_z}{E_s}\Delta z$

***Step6:** Calculate $C_1$ and $C_2$*

# Example 3

**A 3 m wide strip foundation on a deposit of sand layer is shown along with the variation of modulus of elasticity of the soil (Es). The unit weight of sand is 18 kN/m3.**

**Calculate the elastic settlement of foundation using the strain influence factor. Assume there is a creep over a period of 10 years.**



**Solution**

**First step is to plot the variation of strain influence factor $I_z$ with depth (to scale).**

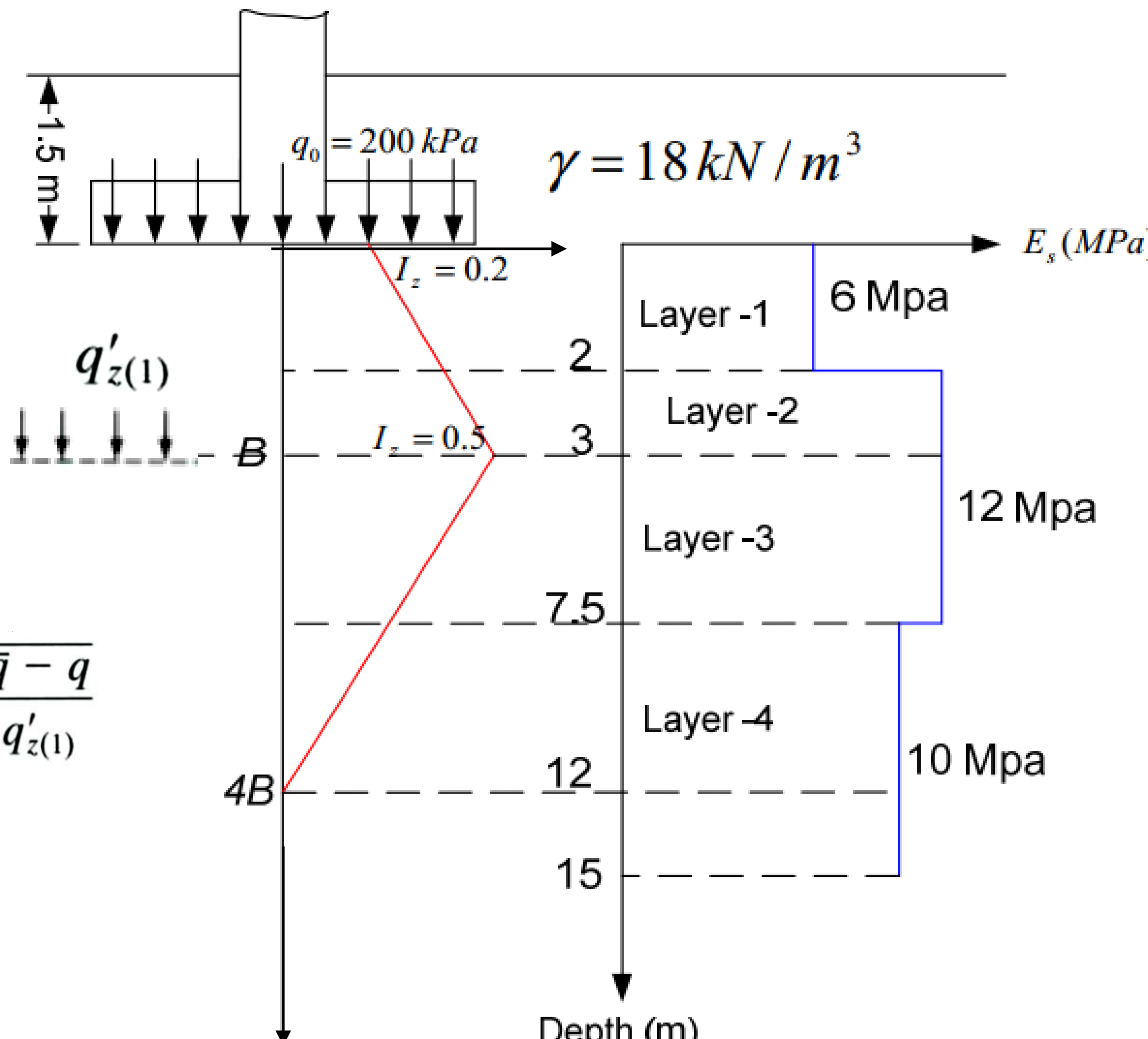
| Strip footing $10 \le L/B$ | |
|---|---|
| **Z** | $I_z$ |
| 0 | 0.2 |
| B=3 | $I_{zp}$ |
| 4B=12 | 0 |

$I_{zp} = 0.5 + 0.1\sqrt{\dfrac{\bar{q} - q}{q'_{z(1)}}}$

$\bar{q} = 200\ kPa$

$q = \gamma D_f = 18 \times 1.5 = 27\ kPa$

$q'_{z(1)}$ = 18 x (1.5 + 3) = ....



# ➢ Elastic Settlement of sandy soil: use of strain influence factor

**Solution (cont..)**

$E_s$ profile is given.

We divide the soil into a number of layers depending on the variation of $I_z$ and $E_s$ values with depth.

Then prepare the following table:

| Layer No | $\Delta z$ [m] | $E_s$ [kPa] | Z to the middle of layer | $I_z$ at the middle of layer | $\sum \frac{I_z}{E_s}\Delta z$ |
|---|---|---|---|---|---|
| 1 | 2.0 | 6000 | 1.0 | 0.3 | 0.0001 $[m^3/kN]$ |
| 2 | 1.0 | 12000 | 2.5 | 0.45 | 0.0000375 |
| 3 | 4.5 | 12000 | 5.25 | 0.375 | 0.000141 |
| 4 | 4.5 | 10000 | 9.75 | 0.125 | 0.0000563 |

$$\sum \frac{I_z}{E_s}\Delta z\,[m^3/kN] = 0.000334\,m^3/kN$$

# Elastic Settlement of sandy soil: use of strain influence factor

**Solution (cont..)**

$$\bar{q} = 200\ kPa$$

$$q = \gamma D_f = 18\times 1.5 = 27\ kPa$$

$$C_1 = 1-0.5\left(\frac{q}{\bar{q}-q}\right) = 1-0.5\times\frac{27}{200-27} = 0.922$$

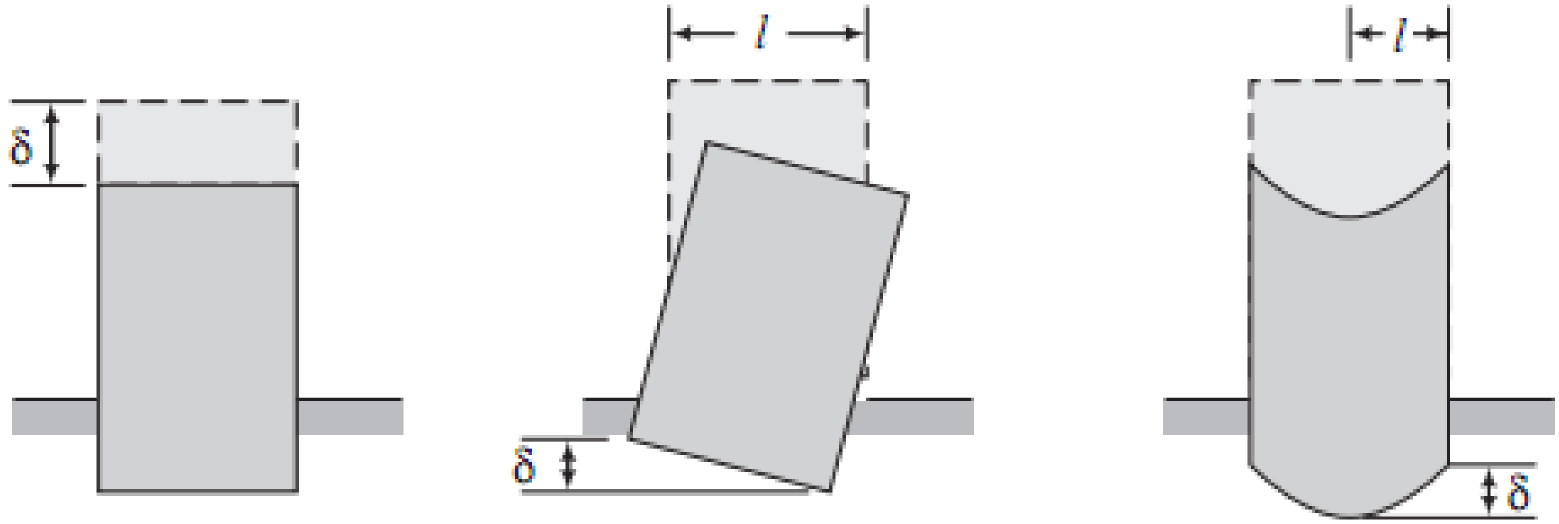
$$C_2 = 1+0.2\log\left(\frac{time\ in\ years}{0.1}\right) = 1+0.2\log\left(\frac{10}{0.1}\right) = 1.4$$

$$\sum \frac{I_z}{E_s}\Delta z\ [m^3/kN] \quad = 0.000334\, m^3/kN$$

$$S_e = C_1 C_2(\bar{q}-q)\sum_0^{z_2}\frac{I_z}{E_s}\Delta z = 0.922\times 1.4\times(200-27)\times 0.000334 = 0.075\ m = 75\, mm$$

# REVISSION

- ➢Foundation settlement can be
- ❖Rigid(uniform)
- ❖Tilt or distortion-caused by differential settlement
- ❖Non uniform settlement




(*a*) Uniform settlement (*b*) Tilt or distortion (*c*) Nonuniform settlement

- ❖Example: For Building on shallow foundations Angular distortion is limited to 1/150 to 1/250.

**Example:** Two shallow footings are located at 12m on center of a multistory building. The vertical, uniform settlements of the two footings are 25 mm and 38 mm, respectively. The footings rest on a clay soil.

❖Calculate the angular distortion.

❖Is the angular distortion satisfactory to reduce wall cracking?

**Solution**

D / settlement=13mm and Angular distortion=13/12000=1/923

To reduce cracking for end bays , from tables, the angular distortion should ideally be between 1/1000 to 1/1400 so not satisfactory.

- **Consolidation** is the time-dependent settlement of soils resulting from the expulsion of water from the soil pores.
- **Primary consolidation** is the change in volume of a i  ne-grained soil caused by the expulsion of water from the voids and the transfer of stress from the excess pore water pressure to the soil particles.
- **Secondary compression** is the change in volume of a i ne-grained soil caused by the adjustment of the soil fabric (internal structure) after primary consolidation has been completed
- **Past maximum vertical effective stress, $\sigma'_{zc}$**,  is the maximum vertical effective stress that a soil was subjected to in the past

- **Normally consolidated soil** is one that has never experienced vertical effective stresses greater than its current vertical effective stress ($\sigma'zo= \sigma'zc,$ ).

- **Over consolidated soil** is one that has experienced vertical effective stresses greater than its existing vertical effective stress ($\sigma'zo<\sigma'zc,$ ).

- **Over consolidation ratio, OCR,** is the ratio by which the current vertical effective stress in the soil was exceeded in the past ($OCR = \sigma'zc/ \sigma'zo$)

- **Compression index, Cc,** is the slope of the normal consolidation line in a plot of the logarithm of vertical effective stress versus void ratio.\

- **Unloading/reloading index or recompression index, Cr** , is the average slope of the unloading/reloading curves in a plot of the logarithm of vertical effective stress versus void ratio.

**Ultimate net bearing capacity (qu)** is the maximum pressure that the soil can support above its current overburden pressure.

**Ultimate gross bearing capacity (qult)** is the sum of the ultimate net bearing capacity and the overburden pressure above the footing base.

So, generally in this chapter we have to design:

- ❖Safe bearing capacity of the soil
- ❖Settlement of shallow foundation
- ❖Size of shallow foundation to satisfy bearing capacity

NB:There are **two design principles** followed

**load and resistance factored design(LRFD):**in this cases the loads are multiplied by load factors as **P=1.2DL+1.6LL**

We will use also performance factor w/c is set for d/t tests.

**Allowable stress design(ASD):** in this cases the loads are multiplied by load factors as **P=DL+LL**

There are also two limiting conditions

**Total stress analysis(TSA):**for short term condition and the shear strength parameter is undrain shear strength(su)

$$\text{TSA:}\quad \boxed{q_u = 5.14 s_u s_c d_c}$$

**Effective stress analysis(ESA):**for long term condition and the shear strength parameter is peak friction angle(ϕ'p)

$$\text{ESA:}\quad \boxed{q_u = \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma}$$

# Gazetas equation to calculate settlements

There are three types of settlements

- Elastic/ immediate
- Primary consolidation
- Secondary consolidation

**Gazetas et al. (1985)** considered an arbitrarily shaped rigid footing embedded in a deep homogeneous soil and proposed the following general equation for the elastic settlement of sand and clay materials:

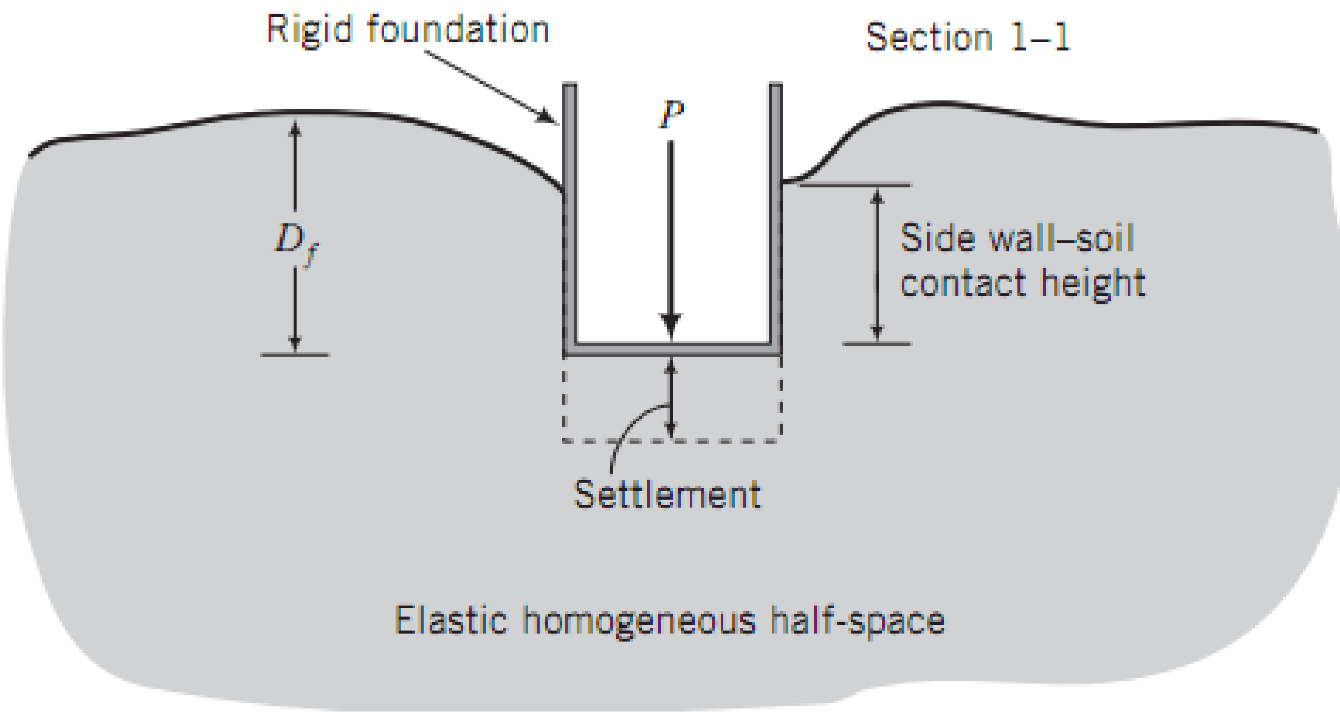
$$\rho_e = \frac{P}{E_u L}(1 - \nu_u^2)\mu_s \mu_{emb} \mu_{wall}$$

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

$$\mu_s = 0.45\left(\frac{A_b}{4L^2}\right)^{-0.38}$$

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

$$\mu_{wall} = 1 - 0.16\left(\frac{A_w}{A_b}\right)^{0.54}$$



❖Ab is the actual area of the base of the foundation and

❖Aw is the actual area of the wall in contact with the embedded portion of the footing. The length and width of the circumscribed rectangle are 2L and 2B, respectively.

❖The dimensionless shape parameter, Ab/4L^2 has the values for common footing geometry shown in Table

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

The Elastic Settlement of rigid footing embedded in homogeneous sandy soil is modified and given as:

$$\boxed{\rho_e = \frac{q_s B_r (1 - \nu_u^2)}{E_u} I_s \mu'_{emb}}$$

at center of foundation

$$I_s = 0.62 \ln (L/B) + 1.12$$

where

$$\mu'_{emb} = 1 - 0.08 \frac{D_f}{B_r}\left(1 + \frac{4B_r}{3L_r}\right)$$

where $B_r$ and $L_r$ are the actual width and length, respectively.

❖If wall friction is neglected, then µwall=1

NB: Lr&Br are actual length and width and L=Lr/2,B=Br/2

The Immediate Settlement for clay when 90% of applied load is distributed

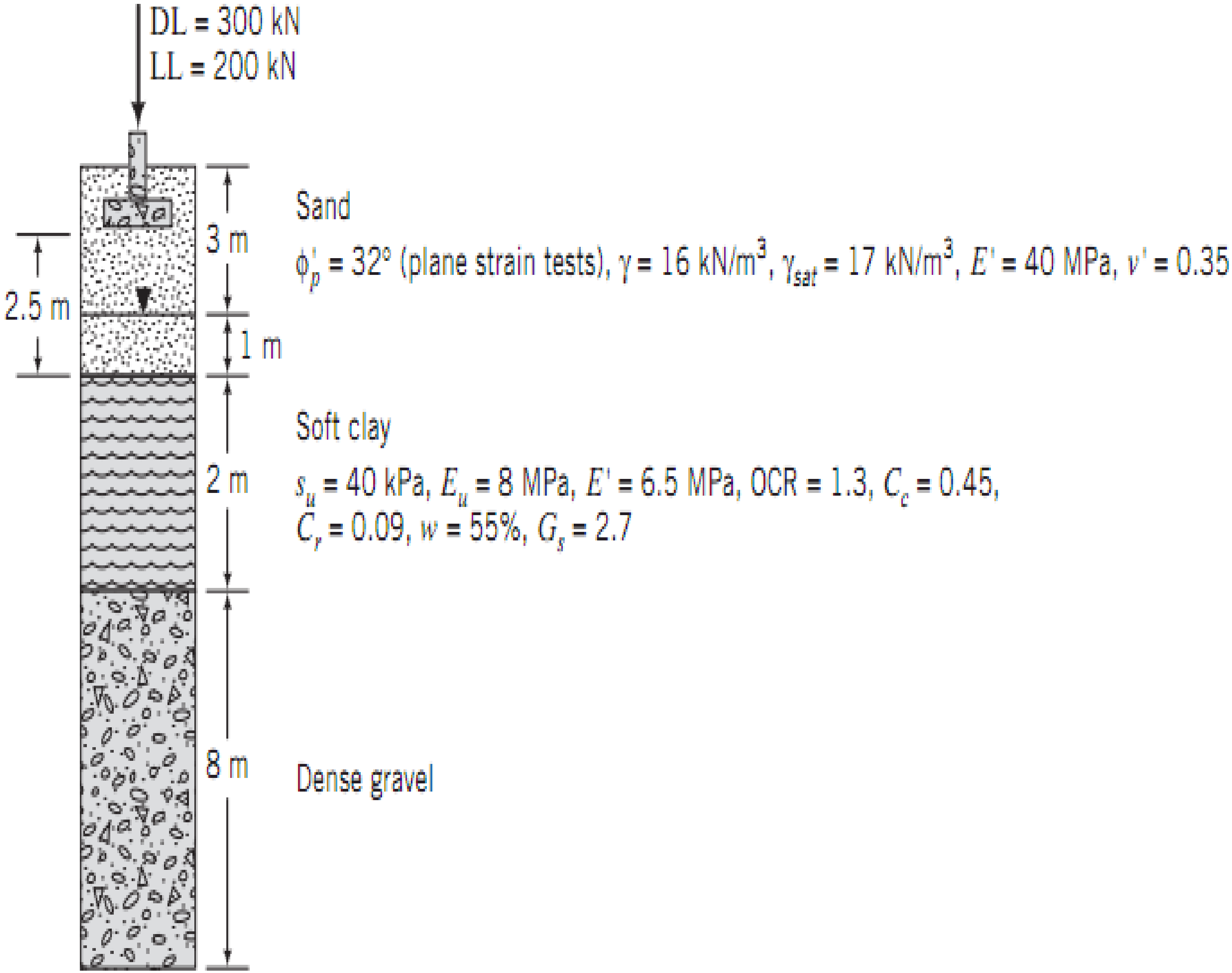
$$\rho_e = \frac{P(1 - \nu_u^2)}{E_u L} \mu_s \mu'_{emb}$$

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

# EXAMPLE: Design a Square Footing

- Determine the size of a square footing to carry a dead load of **300 kN** and a live load of **200 kN** using LRFD.
- The soil at the site is shown below. The tolerable total settlement is **20 mm.** The groundwater level is **3 m below the ground surface** and the depth of the footing is **1.5 m.** However, the groundwater level is expected to seasonally rise to the surface.
- You may assume that the clay layer is **thin so that one-dimensional consolidation takes place.** The performance factor is 0.8.

# Cont....



DL = 300 kN
LL = 200 kN
2.5 m
3 m
1 m
2 m
8 m
Sand
$\phi'_p$ = 32° (plane strain tests), $\gamma$ = 16 kN/m$^3$, $\gamma_{sat}$ = 17 kN/m$^3$, $E'$ = 40 MPa, $\nu'$ = 0.35
Soft clay
$s_u$ = 40 kPa, $E_u$ = 8 MPa, $E'$ = 6.5 MPa, OCR = 1.3, $C_c$ = 0.45,
$C_r$ = 0.09, $w$ = 55%, $G_s$ = 2.7
Dense gravel

# Solution

- ❖The presence of the soft clay layer gives a clue that settlement may govern the design. In this case, we should determine the width required to satisfy settlement and then check the bearing capacity.

- ❖One can assume a width, calculate the settlement, and reiterate until the settlement criterion is met.

- ❖Since this is a multilayer soil profile, it is more accurate to calculate the increase in vertical stress for multilayer soils.

- ❖However, for simplicity, we will use Boussinesq's method for a uniform soil.

**Step 1:** Assume a width and a shape.

Try $B = 3$ m, and assume a square footing.

**Step 2:** Calculate the elastic settlement.

**Sand**

Neglect side wall and embedment effects; that is, $\mu_{wall} = \mu_{emb} = 1$.

$$\mu'_{emb} = 1 - 0.08\frac{D_f}{B_r}\left(1 + \frac{4B_r}{3L_r}\right) = .906$$

$$I_s = 0.62 \ln (L/B) + 1.12 = 0.62 \ln (1) + 1.12 = 1.12$$

$$\rho_e = \frac{q_s B_r[1 - (\nu')^2]}{E'} I_s \mu'_{emb} = \frac{P[1 - (\nu')^2]}{E' L_r} I_s \mu'_{emb}$$

$$: \quad = \frac{500(1 - 0.35^2)}{40 \times 10^3 \times 3} \times 1.12 \times .906 = 3.3 \times 10^{-3} = 3.3 \text{ mm}$$

**Clay**

Find the equivalent footing size at the top of the clay layer. Let $z_1$ be the depth from the base of the footing to the top of the clay layer.

Equivalent width and length of footing at top of clay: $B + z_1 = 3 + 2.5 = 5.5$ m.

Df=1.5m+2.5m=4m

$$\mu_{emb} = 1 - 0.04\frac{D_f}{B}\left[1 + \frac{4}{3}\left(\frac{A_b}{4L^2}\right)\right] = 1 - 0.04\frac{4}{(5.5/2)}\left(1 + \frac{4}{3} \times 1\right) = 0.86$$

$$\mu_s = 0.45$$

For immediate settlement in clays, use undrained conditions with $\nu = \nu_u = 0.5$.

$$\rho_e = \frac{P(1 - \nu_u^2)}{E_u L}\mu_s \mu'_{emb} = \frac{500(1 - 0.5^2)}{8000 \times (5.5/2)} \times 0.45 \times 0.86 = 6.6 \times 10^{-3} = 6.6 \text{ mm}$$

**Step 3:** Calculate the consolidation settlement of the clay.

$$e_o = wG_s = 0.55 \times 2.7 = 1.49$$

$$\gamma_{sat} = \frac{G_s + e_o}{1 + e_o}\gamma_w = \frac{2.7 + 1.49}{1 + 1.49}9.8 = 16.5 \text{ kN/m}^3$$

Calculate the current vertical effective stress (overburden pressure) at the center of the clay layer.

$$\sigma'_{zo} = 3 \times 16 + 1(17 - 9.8) + 1(16.5 - 9.8) = 61.9 \text{ kPa}$$

Calculate the stress increase at the center of the clay layer ($z = 3.5$ m).

$$\frac{Z}{B} = \frac{3.5}{3} = 1.17$$

According d/t literatures from figures Iz=0.27

$$\sigma_{ap} = \frac{p}{B^2} = \frac{500}{3^2} = 55.6 \text{ kPa}$$

$$\Delta\sigma_z = \sigma_{ap} \times I_z = 55.6 \times 0.27$$

$$\sigma'_{zo} + \Delta\sigma_z = 61.9 + 15 \approx 77 \text{ kPa}$$

$$\sigma'_{zc} = \text{OCR} \times \sigma'_{zo} = 1.3 \times 61.9 = 80.5 \text{ kPa} > \sigma'_{zo} + \Delta\sigma_z (= 77 \text{ kPa})$$

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

$$= \frac{2}{1 + 1.49} \times 0.09 \log \frac{77}{61.9} = 6.9 \times 10^{-3} \text{ m} = 6.9 \text{ mm}$$

**Step 4:** Find the total settlement.

Total settlement: $\rho = (\rho_e)_{sand} + (\rho_c)_{clay} + \rho_{pc}$

$$= 3.3 + 6.6 + 6.9 = 16.8 \text{ mm} < 20 \text{ mm}$$

**Step 5:** Check bearing capacity.

The groundwater table is less than $B = 3$ m below the footing base, so groundwater effects must be taken into account.

## Step 6: Check the critical height

- If the thickness, H1, of the soil below the footing in the top layer is greater than Hcr, the failure surface will be confined in the top layer and it is sufficiently accurate to calculate the bearing capacity based on the properties of the soil in the top layer.
- Otherwise, the failure surface would be influenced by the bottom layer and may extend into it.

$$H_{cr} = \frac{Br}{2\cos\left(45° + \frac{\phi'_p}{2}\right)} \exp\left[A \tan \phi'_p\right]$$

$$A = \left(45° - \frac{\phi'_p}{2}\right) = \left(45° - \frac{32}{2}\right)\frac{\pi}{180} = 0.506 \text{ rad}$$

$$H_{cr} = \frac{3}{2\cos\left(45° + \frac{32}{2}\right)} \exp\left[0.506 \tan 32\right] = 4.24 \text{ m}$$

The height of soil below the footing to the top of the soft clay is 2.5 m < $H_{cr}$. Therefore, you must consider the bearing capacity of the soft layer.

**ESA (sand)**

$$d_\gamma = 1, \quad s_q = 1 + \frac{Br'}{Lr'} \tan \phi'_p = 1 + \tan 32° = 1.62$$

$$s_\gamma = 1 - 0.4 \frac{Br'}{Lr'} = 0.6$$

$$d_q = 1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1}\left(\frac{D_f}{Br'}\right) = 1 + 2 \tan 32° (1 - \sin 32°)^2 \left[\tan^{-1}\left(\frac{1.5}{3}\right) \times \frac{\pi}{180}\right] = 1.13$$

$$N_q = e^{\pi \tan 32°} \tan^2\left(45 + \frac{32}{2}\right) = 23.2; \quad N_q - 1 = 23.2 - 1 = 22.2$$

$$N_\gamma = 0.1054 \exp\left(9.6 \times 32 \times \frac{\pi}{180}\right) = 22.5$$

Calculate the bearing capacity for the worst-case scenario—groundwater level at surface, i.e., $\gamma = \gamma' = 17 - 9.8 = 7.2\ \text{kN/m}^3$.

$$q_u = \gamma D_f (N_q - 1) s_q d_q + 0.5(\gamma B) N_\gamma s_\gamma d_\gamma$$

$$= (7.2 \times 1.5 \times 22.2 \times 1.62 \times 1.13) + (0.5 \times 7.2 \times 3 \times 22.5 \times 0.6 \times 1.0) = 585\ \text{kPa}$$

$$q_{ult} = q_u + \gamma D_f = 585 + 1.5 \times 7.2 = 596\ \text{kPa}$$

$$Q_{ult} = q_{ult} A = 596 \times 3^2 = 5364\ \text{kN}$$

$$\omega Q_{ult} = 0.8 \times 5364 = 4291\ \text{kN}$$

$$P_{uf} = 1.25\ \text{DL} + 1.75\ \text{LL} = (1.25 \times 300) + (1.75 \times 200) = 725\ \text{kN} < 4291\ \text{kN}; \quad \text{okay}$$

**TSA (clay)**

Check the bearing capacity of the clay.

Equivalent footing width $= B + z_1 = 3 + 2.5 = 5.5$ m.

$$(\Delta\sigma)\text{max} = \text{P}/[(\text{B+Z})(\text{B+Z})] = \frac{500}{5.5^2} = 16.5\ \text{kPa}$$

$$q_u = 5.14 s_u s_c d_c$$

$$s_c = 1 + 0.2\frac{B'}{L'} = 1.2$$

$d_c = 1.0$ because the footing is assumed to be a surface footing on the clay.

$$q_u = 5.14 \times 40 \times 1.2 \times 1.0 = 247 \text{ kPa}$$

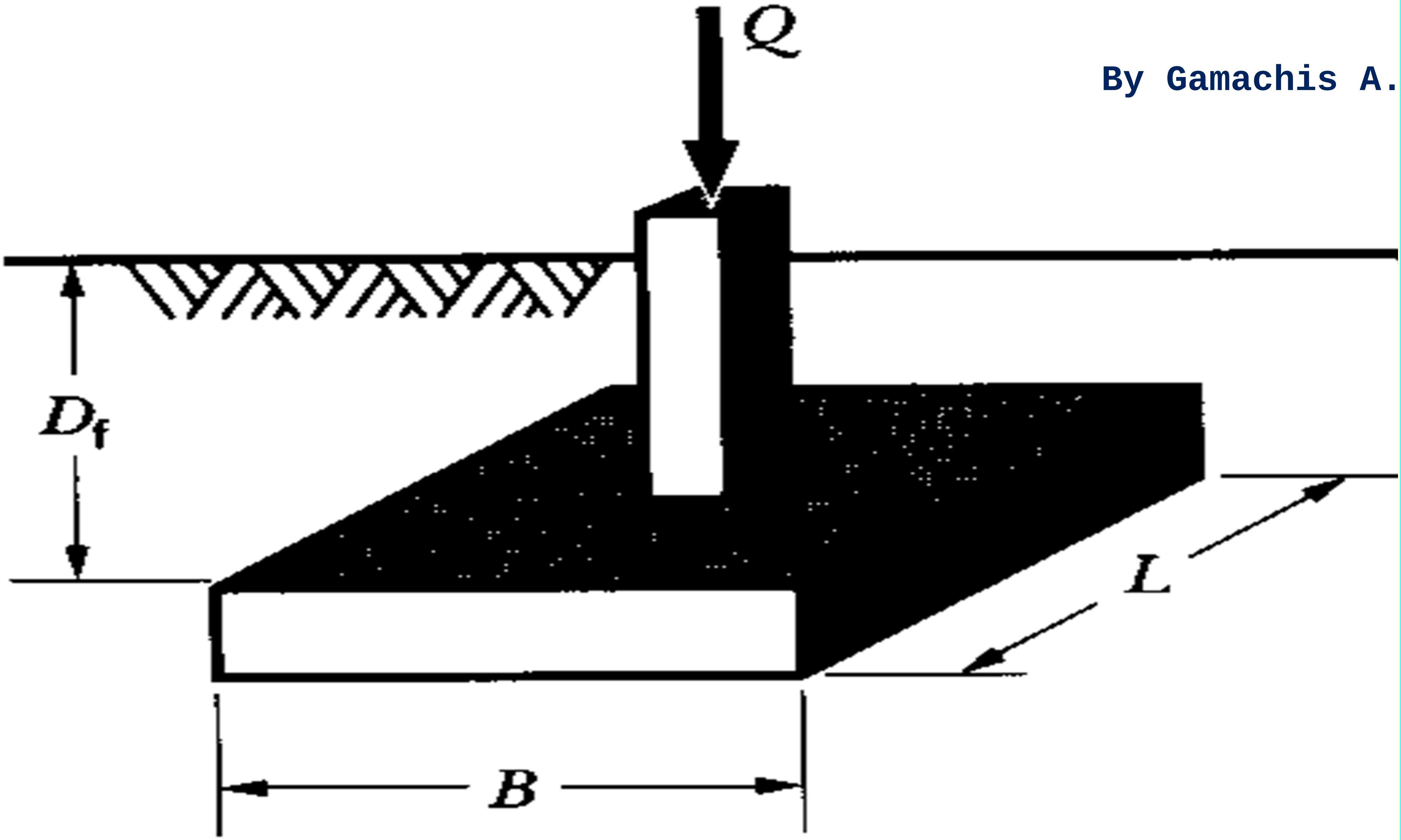
$$\omega Q_{ult} = 0.8 \times 247 \times 5.5^2 = 5977 \text{ kN} > 725 \text{ kN}; \quad \text{okay}$$

Settlement governs the design.

# END OF CHAPTER THREE

# DESIGN OF SHALLOW FOUNDATION

By Gamachis A.



# Introduction

**General Requirements of Foundations:**

For a satisfactory performance, a foundation must satisfy the following three basic criteria:

- Location and depth criterion.
- Shear failure criterion or bearing capacity.
- Settlement.

**1. Location and Depth Criterion:**

- As a general rule, any foundation should be placed at a depth where the soil stratum is adequate from the point of view of bearing capacity and settlement criteria.

# Introduction

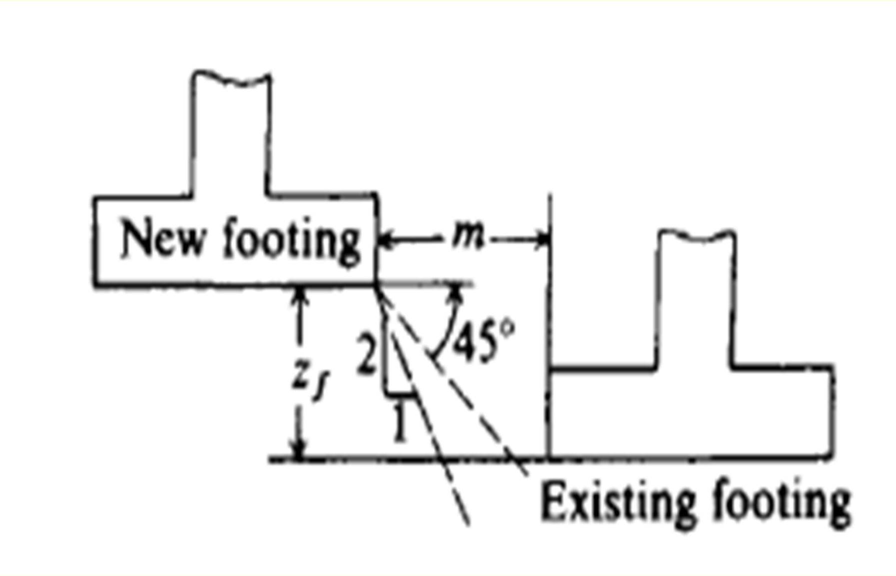
**Minimum Requirements:**

- A foundation should be located at a minimum depth of 50cm below natural ground surface.
- The foundation must be placed below the zone of volume change, where volume change is expected. For example, in areas where there is ***expansive soil*** the foundation should be taken below the active zone.
- Foundations for structures in a river have to be protected from the ***scouring action*** of the flowing-stream. The depth of foundation for a bridge pier or any similar structure must be sufficiently below the deepest scour level.

# Foundations Near Existing Structures:

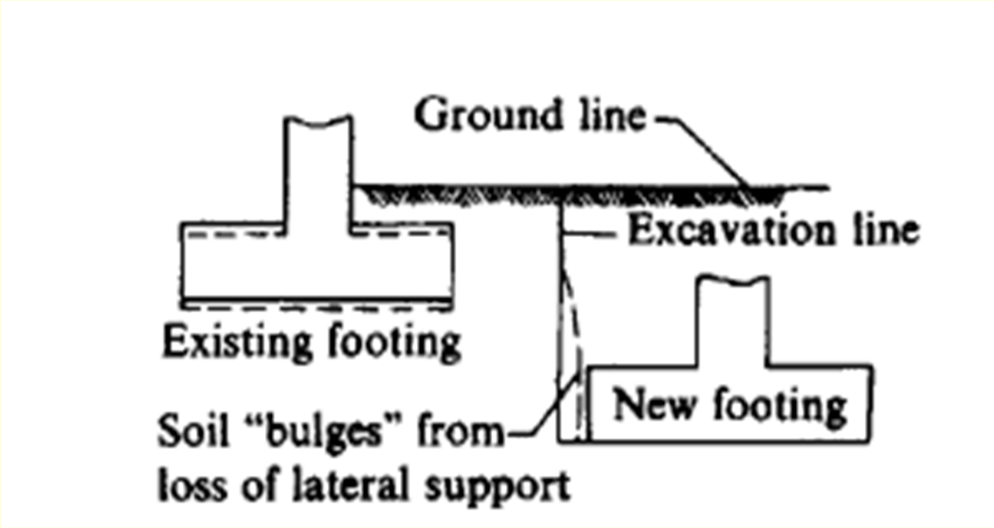
- When footings are to be placed adjacent to existing structure, the line from the base of the new footing to the bottom edge of the existing footing should be **45º** Or less with the horizontal plane. The distance **m** should be greater than $Z_f$ .

$$z_f = \frac{2c}{(SF)\gamma\sqrt{K}} - \frac{q_o}{(SF)\gamma}$$



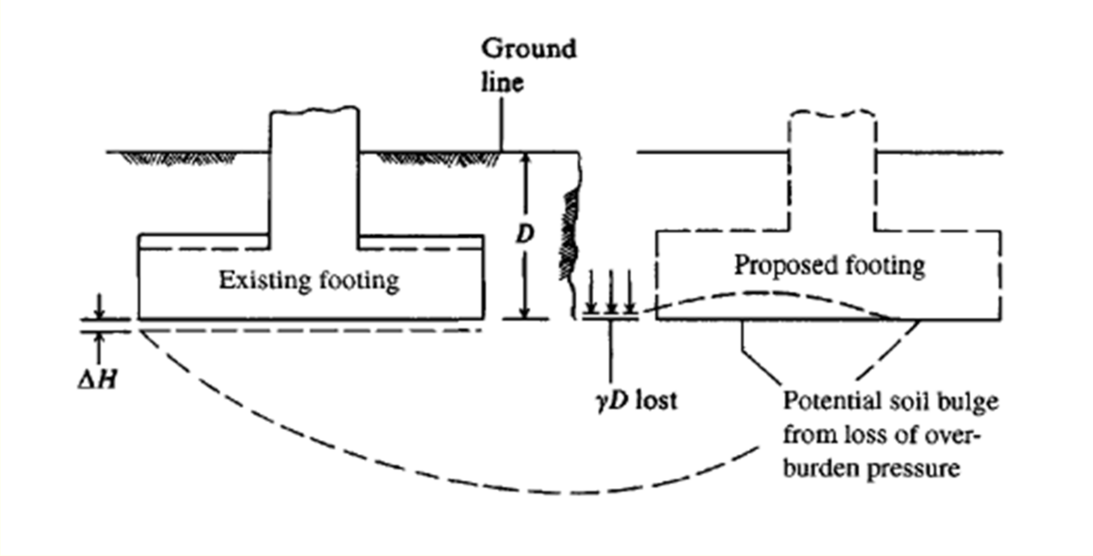
# Foundations Near Existing Structures…

- Conversely, the Figure below indicates that if the new footing is lower than the existing footing, there is a possibility that the soil may flow laterally from beneath the existing footing. This may increase the amount of excavation somewhat but, more importantly, may result in settlement cracks in the existing building. This problem is difficult to analyze; however, an approximation of the safe depth $Z_f$ may be made for C-$\phi$ soil.



# Foundations Near Existing Structures…

- In the figure below illustrate how a problem can develop if the excavation for the foundation of the new structure is too close to the existing building. In this case the $qN_q$ term of the bearing capacity equation is lost, for most foundations below the ground surface this is a major component of the bearing capacity.



# Foundation Classifications:

Foundations may be classified based on where the load is carried by the ground, according to Terzaghi:

- ***Shallow foundations*:** are footings, or mats. The depth is generally $D/B \leq 1$.

- ***Deep foundations*:** piles, drilled piers, or drilled caissons. $Lp \geq 1$

# Factors on selecting foundation type

- The choice of the appropriate type of foundation is affected by:
  1) Type of superstructure to be supported: function and load that it transfers to the foundation.
  2) Subsurface condition and/or type of soil.
  3) Cost of foundation.
- A safe foundation design provides for a suitable factor against,
  A. Shear failure of the soil: by not exceeding the bearing capacity of the soil.
  B. Excessive settlement (both uniform and differential settlement)

# Shallow Foundations:

- Shallow/spread footings are the most widely used type among all foundations because they are usually more economical.
- Construction of footings requires a least amount of equipment and skill and no heavy or special equipment is necessary.
- Shallow foundations are usually used when the soil at a shallow depth has adequate capacity to support the load of the superstructure. For reasons of economy, shallow foundations are the first choice unless they are considered inadequate.

# Shallow Foundations…

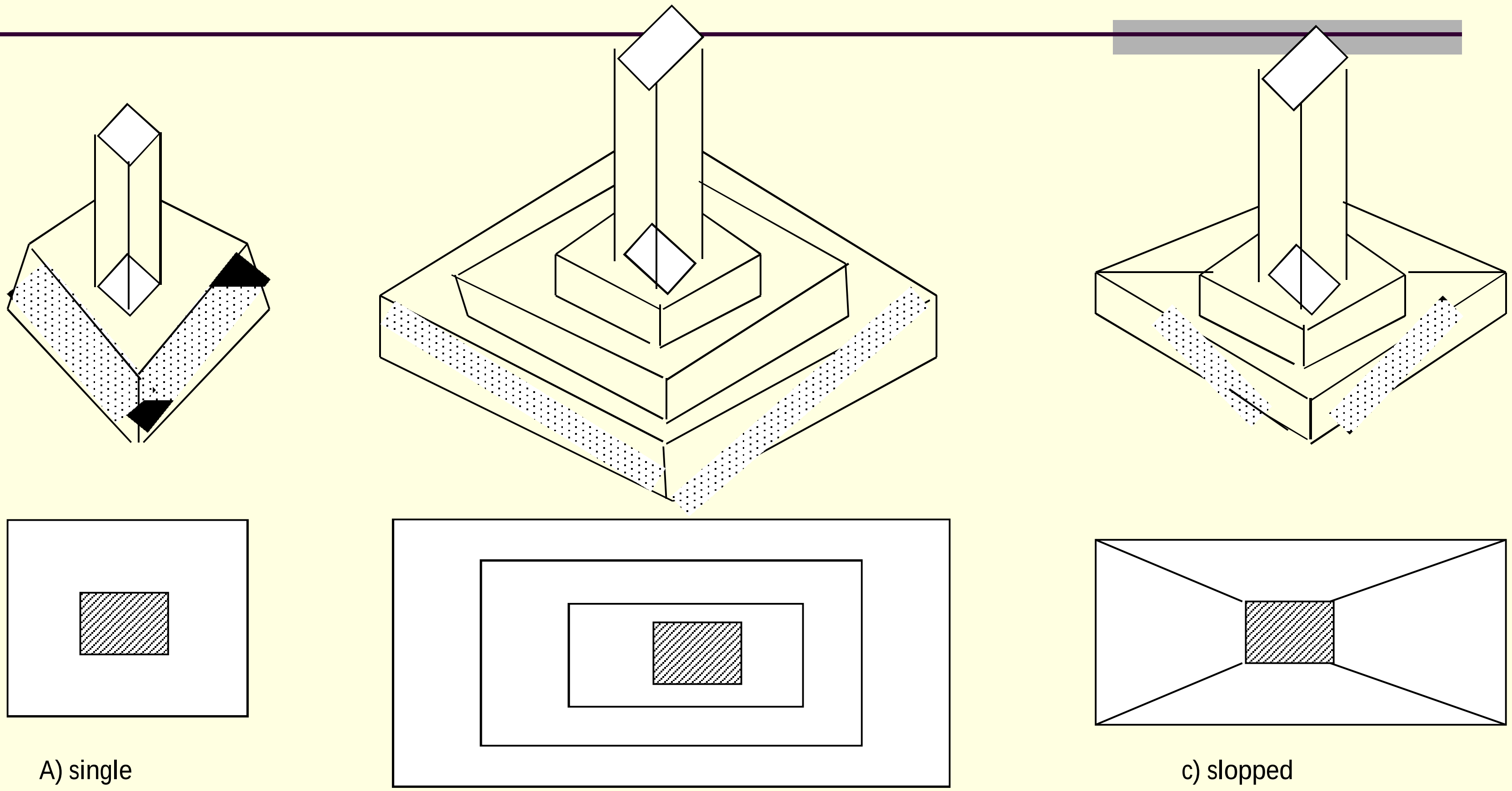
**Spread footing**

- spread the super-imposed load of column or wall over a larger area
- spread footings support either a column or wall

Types of spread footings:

- Single footing
- Stepped footing
- Sloped footing

# Shallow Foundations…



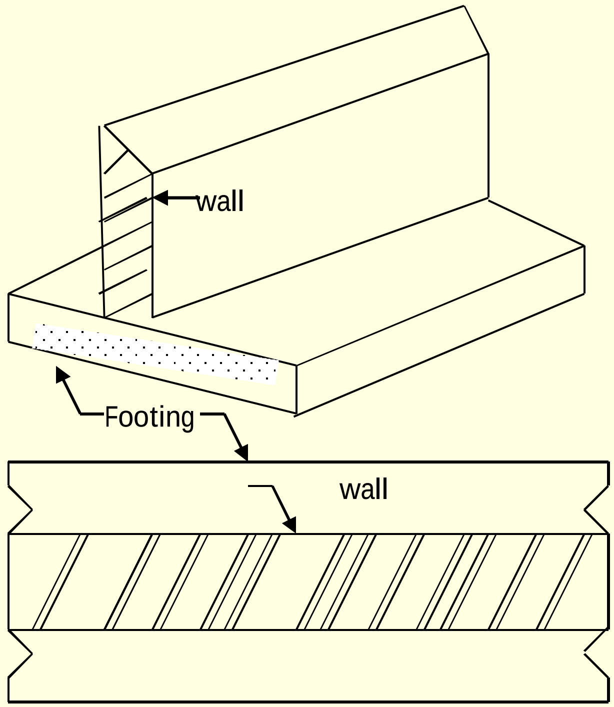
A) single
footing

b) stepped footing

c) slopped

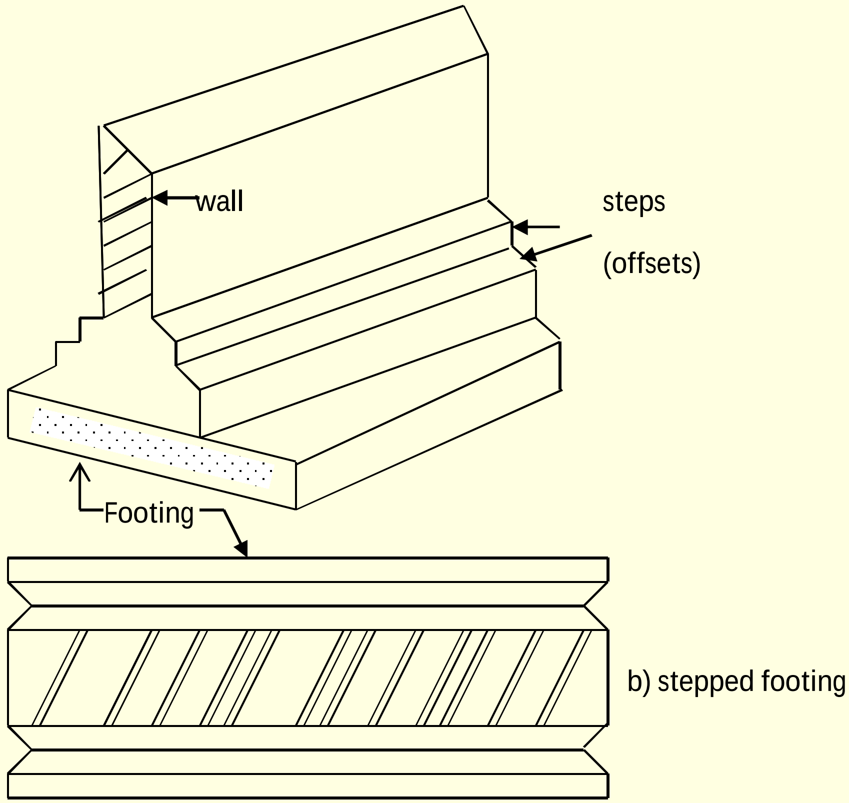
footing

# Shallow Foundations…




a) simple Footing

# Shallow Foundations:



# Footings:

- Footings belong to shallow foundation and their purpose is to transmit the load from the structure to soil or rock.
- Footings that support a single column, referred to as: **Isolated or spread footings:** can be a square, rectangular or circular in shape depending on the relative magnitude of the moments Mx and My from the superstructure.
- Footings that support two or more columns are classified as ***combined footings.*** *Rectangular*, *Trapezoidal* and *strap combined footings* are special versions of combined footings, patterned to meet certain conditions or restrictions.

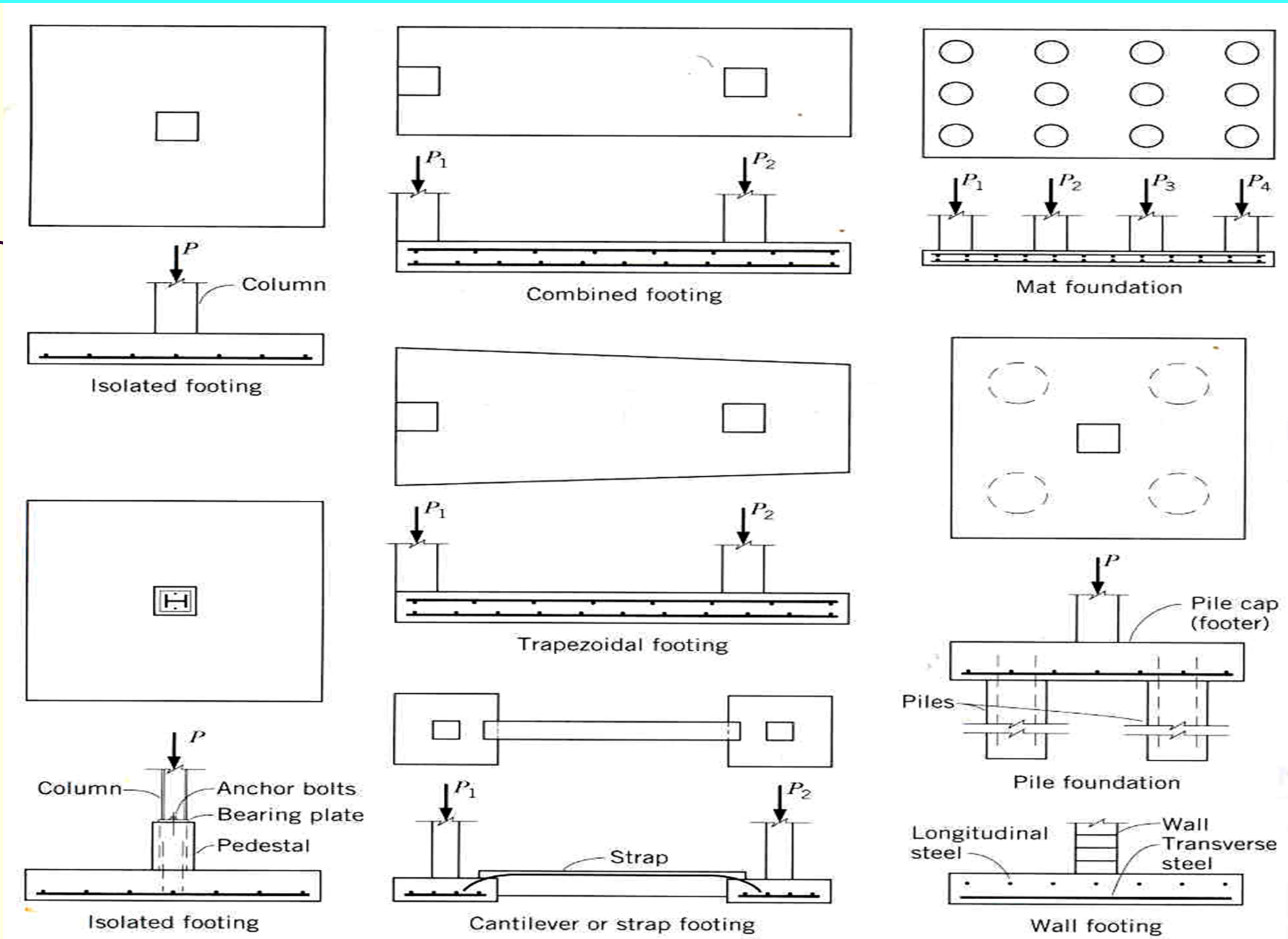



**Figure** Typical configurations for various types of footings

# Footings…

- Concrete is almost always the material used in footings. It is strong, durable, and is a convenient, economical construction material, workable and adaptable to field construction and requirements.
- Concrete footings may be plain or reinforced, with reinforcement running in one (one way) or two (two way) directions, depending on the direction of flexure.

# Footings…

- Footing shapes usually vary with specific requirements and design needs.
- For spread or isolated footings, square shapes are common and usually most economical, but rectangular shapes are used if space is limited in one direction, or when loads are eccentric in one direction.
- The typically desired objective is to select the footing shape that makes the soil pressure (bearing pressure) as uniform as possible.
- Furthermore, footings may be of uniform thickness or may be sloped or stepped. Stepped or sloped footings are most commonly used to reduce the quantity of concrete away from the column where the bending moments are small and when the footing is not reinforced.

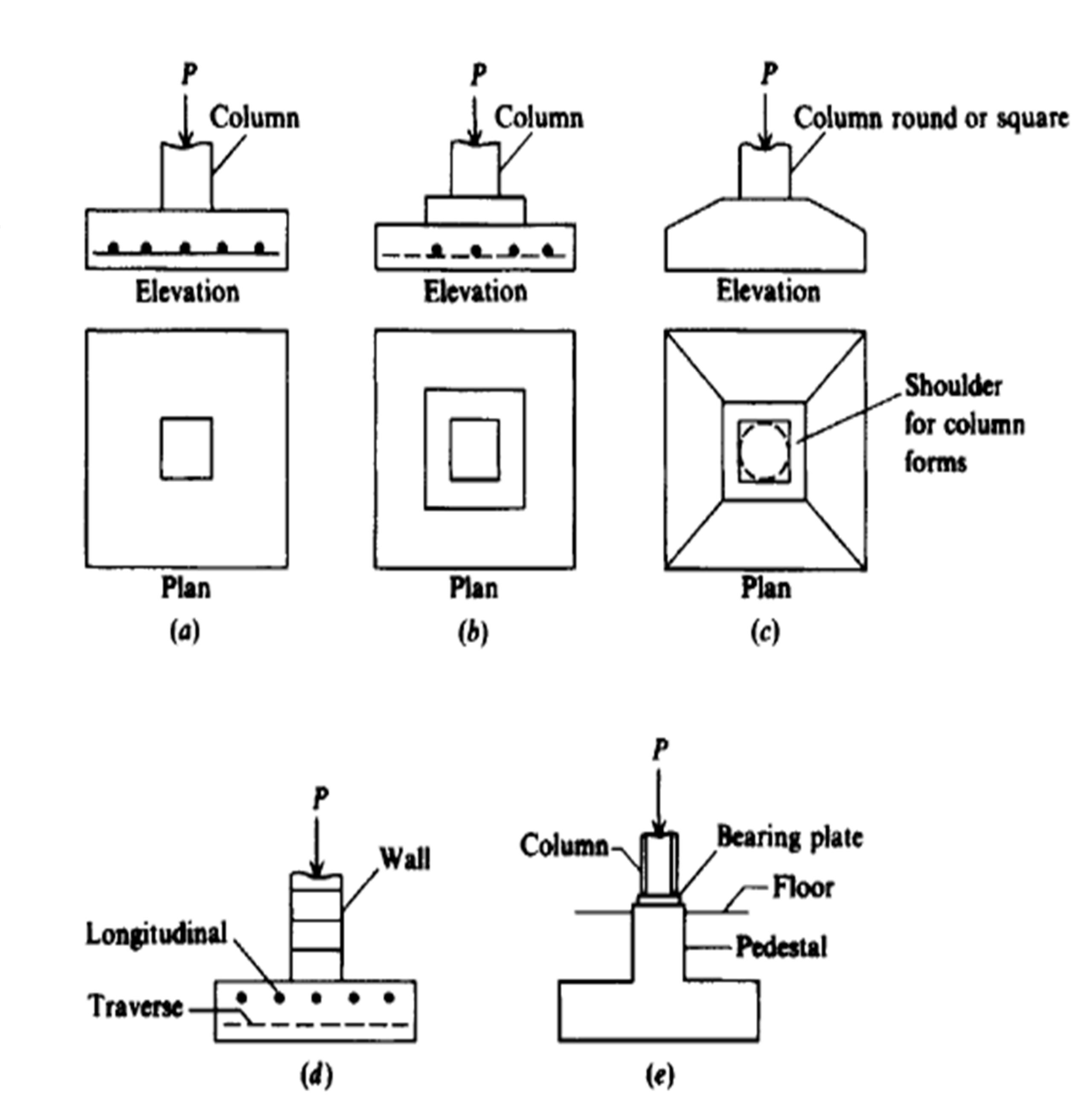



**Figure:** Typical footings.

(a) spread footings;
(b) stepped footing;
(c) sloped footing;
(d) Wall footing;
(e) footing with pedestal.

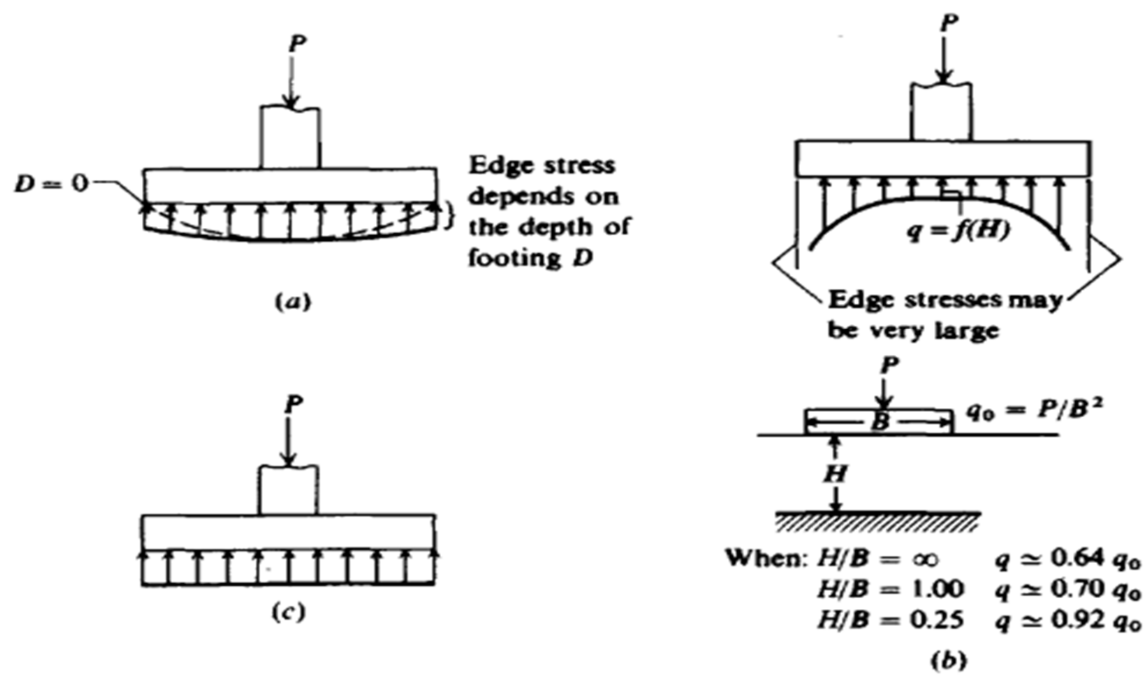
# ASSUMPTIONS USED IN FOOTING DESIGN

- Theory of Elasticity analysis [Borowicka (1963)] and observations [Schultze (1961), Barden(1962)] indicate that the stress distribution beneath symmetrically loaded footings is not uniform. The actual stress distribution depends on both footing rigidity and base soil.
- For footings on <u>loose sand</u> the grains near the edge tend to displace laterally, whereas the interior soil is relatively confined. This difference results in a pressure diagram qualitatively shown in Fig.[a].
- Fig [b] is the theoretical pressure distribution for the general case of rigid footings on any material. The high edge pressure may be explained by considering that edge shear must occur before any settlement can take place.

# ASSUMPTIONS…

- The pressure distribution beneath most footings will be rather indeterminate because of the interaction of the footing rigidity with the soil type, state, and time response to stress. For this reason it is common practice to use the linear pressure distribution of Fig.[c ] beneath spread footings.

**Figure:** Probable pressure distribution beneath a rigid footing.
(a) On a cohesionless soil;
(b) Generally for cohesive soils;
(c) usual assumed linear distribution.



# STRUCTURAL DESIGN OF SPREAD FOOTINGS

- The allowable soil pressure controls the plan *(B X L) dimensions of a spread footing.*
- Shear stresses usually control the footing thickness *D. Two-way action shear always controls* the depth for centrally loaded square footings.
- Wide-beam shear may control the depth for rectangular footings when the *L/B ratio is greater than about 1.2 and may control for other L/B ratios when there are overturning or eccentric loadings.*

# STRUCTURAL DESIGN…

- The punching resistance, $V_{Rd1}$

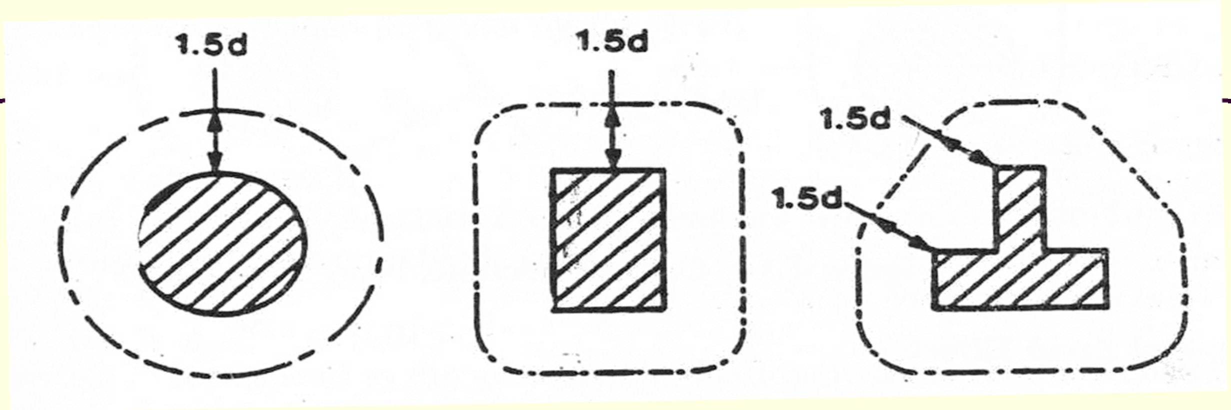
$$V_{Rd1} = 0.25 f_{ctd} k_1 k_2 u d$$

where

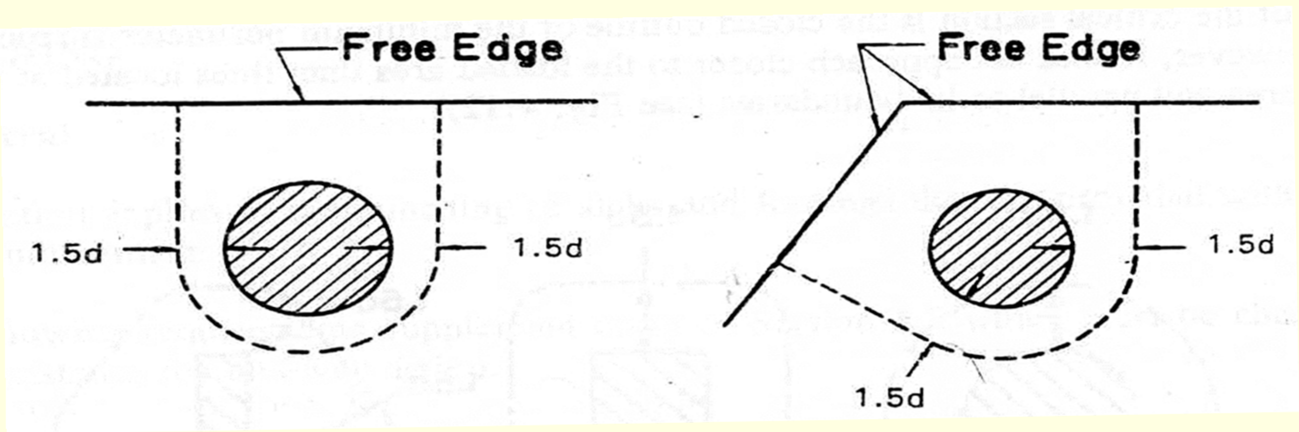
$$f_{ctd} = 0.21 f_{ck}^{2/3} / \gamma_c \qquad d_{av} = \left(d_x + d_y\right)/2$$

$$k_1 = 1 + 50 \rho_e \leq 2.0 \qquad \rho_e = \left(\rho_{ex} + \rho_{ey}\right)^{1/2} \leq 0.015$$

$$k_2 = 1.6 - d_{av} \geq 1.0$$




(a) Critical section remote from a free edge




(a) Critical sections near free edges

**Figure** Critical sections for punching shear

# STRUCTURAL DESIGN…

- **Equation of Punching Shear**

  Observing figure above:

$$P_u = v_c du + A_p q_u$$

$$u = 2\left(x + y + 1.5\pi d\right) \quad ; \quad A_p = xy + 2(1.5dx) + 2(1.5dy) + \pi(1.5d)^2$$

$$P_u = BLq_u \; - \; from\,equilibrium\,of\,entire\,footing.$$

Expanding and rearranging terms, it becomes.

$$3\pi\left(v_c + 0.75q_u\right)d^2 + \left(2v_c + 3q_u\right)\left(x + y\right)d - \left(BL - xy\right)q_u = 0$$

$$v_c = 0.25\, f_{ctd}\, k_1 k_2$$

# STRUCTURAL DESIGN…

- **Equation for wide beam shear**

$$v_c db = q_u l_1 b \quad \Rightarrow \quad d = \frac{q_u l_1}{v_c}$$

Where, $l_1$ is the distance from d distance to the periphery of footing.

For $v_c$ determination $\rho_e$ is replaced by $\rho_x$ and $d_{av}$ replaced by $d_x$.

Hence, one proceeds to solve for **d** using either of the above shear equations, then check for the other shear stress, and subsequently to complete the design.

# STRUCTURAL DESIGN…

- In one-way footings and two way square footings reinforcement shall be distributed uniformly across the entire width of footing.
- In two-way rectangular footings, reinforcement shall be distributed as follows:
  - Reinforcement in longer direction shall be distributed uniformly across the entire width of footing.

# STRUCTURAL DESIGN…

- For reinforcement in the short direction, a portion of the total reinforcement given by equation below shall be distributed uniformly over a band width *(centered on center line of column or pedestal) equal to* the length of the short side of footing. The remainder of the reinforcement required in the short direction shall be distributed uniformly outside the center band width of the footing.

$$\frac{\text{Reinforcem ent in band width}}{\text{Total reinforcem ent in short direction}} = \frac{2}{\beta + 1}$$

Where, $\beta$ is the ratio of long side to short side of footing.

# Anchorage Length

- In Reinforced concrete members, the flexural strength relies on the transfer of tensile force, commonly known as ***Bond***, between the longitudinal reinforcement and the surrounding concrete.
- Clearly, the greater the length that the bars are embedded in the concrete, the greater the bond b/n the two materials.
- The application of loads to a reinforced concrete member leads to bending of the member which in turn, results in tensile forces being developed in the reinforcement.

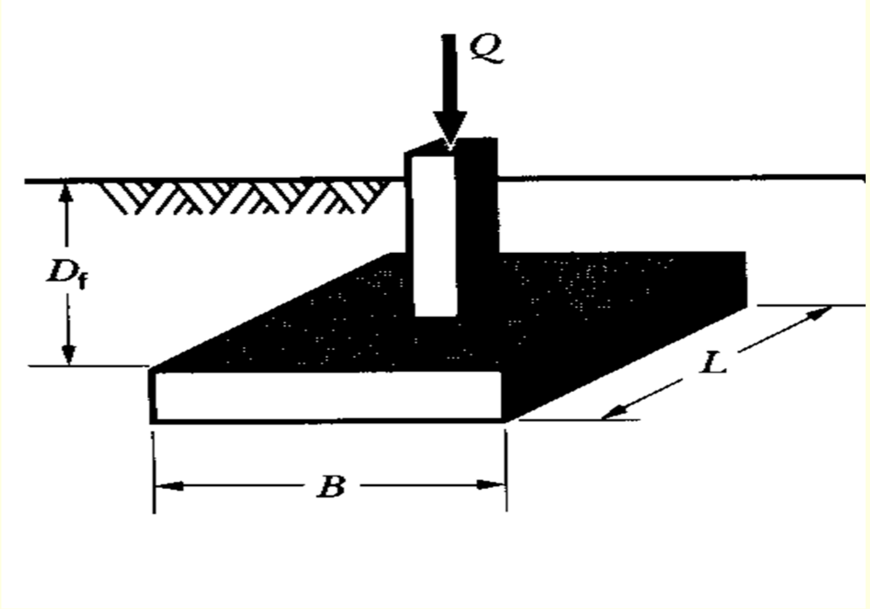
# Anchorage Length…

- If the anchorage bond b/n the bars and concrete is sufficient, the full strength of reinforcement can be utilized.
- If, however, the bond is insufficient, the bar will pull out of the concrete, the tensile force will drop to zero and the member will fail.
- The anchorage length, $l_b$, is the length of reinforcement required to develop sufficient anchorage bond so that the full strength of the reinforcement can be used.
- Basic anchorage length:

$$l_b = \frac{\phi}{4}\frac{f_{yd}}{f_{bd}}$$

# Design Procedure for Concentrically Loaded Isolated Footing

- ***Given*:** Column dimensions and reinforcement; column loads (LL, DL); $f_{ck}$ for footing and column; $f_{yk}$ for footing and column; allowable bearing capacity, $q_a$.



# Design Procedure…

1) Find $P_u = 1.2DL + 1.6LL$ (Self wt. and backfill usually absent).
2) Determine B and L of footing; . $A = \frac{(DL + LL)}{q_a}$ For a unique solution, B or L is fixed.
3) Find $q_u = \frac{P_u}{BL}$ (Ultimate bearing pressure beneath footing).
4) Assume trial effective depth, **d**, of footing for determination of flexural reinforcement.
5) Check **d** for punching shear and wide beam shear.
6) If step (5) is not fulfilled increase **d** and repeat starting from step (4).

# Cont'd…

**TABLE 1.2**
**Factored load combinations for determining required strength *U* in the ACI Code**

| Condition[a] | Factored Load or Load Effect $U$ |
|---|---|
| Basic[b] | $U = 1.2D + 1.6L$ |
| Dead plus Fluid[b] | $U = 1.4(D + F)$ |
| Snow, Rain, Temperature, | $U = 1.2(D + F + T) + 1.6(L + H)$ 1 $0.5(L_r$ or $S$ or $R)$ |
| and Wind | $U = 1.2D + 1.6(L_r$ or $S$ or $R) + (1.0L$ or $0.8W)$ |
| | $U = 1.2D + 1.6W + 1.0L + 0.5(L_r$ or $S$ or $R)$ |
| | $U = 0.9D + 1.6W + 1.6H$ |
| Earthquake | $U = 1.2D + 1.0E + 1.0L + 0.2S$ |
| | $U = 0.9D + 1.0E + 1.6H$ |

[a] Where the following represent the loads or related internal moments or forces resulting from the listed factors: $D$ = dead load; $E$ = earthquake; $F$ = fluids; $H$ = weight or pressure from soil; $L$ = live load; $L_r$ = roof live load; $R$ = rain; $S$ = snow; $T$ = cumulative effects of temperature, creep, shrinkage, and differential settlement; $W$ = wind.

[b] The ACI Code includes $F$ or $H$ loads in the load combinations. The "Basic" load condition of $1.2D + 1.6L$ reflects the fact that most buildings have neither $F$ nor $H$ loads present and that $1.4D$ rarely governs design.

# Design procedure …

7) Calculate the anchorage length and reinforcement distribution.
8) Select the appropriate dowels based on the anchorage length and lap length.
9) Complete a design drawing showing all details (footing dimensions, reinforcement size, spacing cover, etc..)

**EXAMPLE**

# Eccentrically Loaded Spread Footings

- The ensuing “load” on the column, and subsequently on the footing, due to supported beams from several spans, can be a combination of a vertical load and moments as shown in figure below.

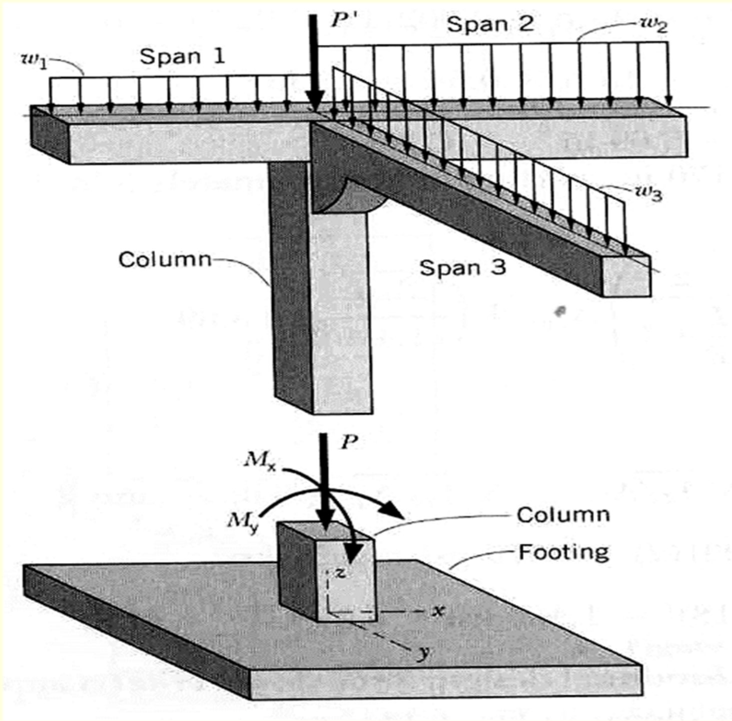
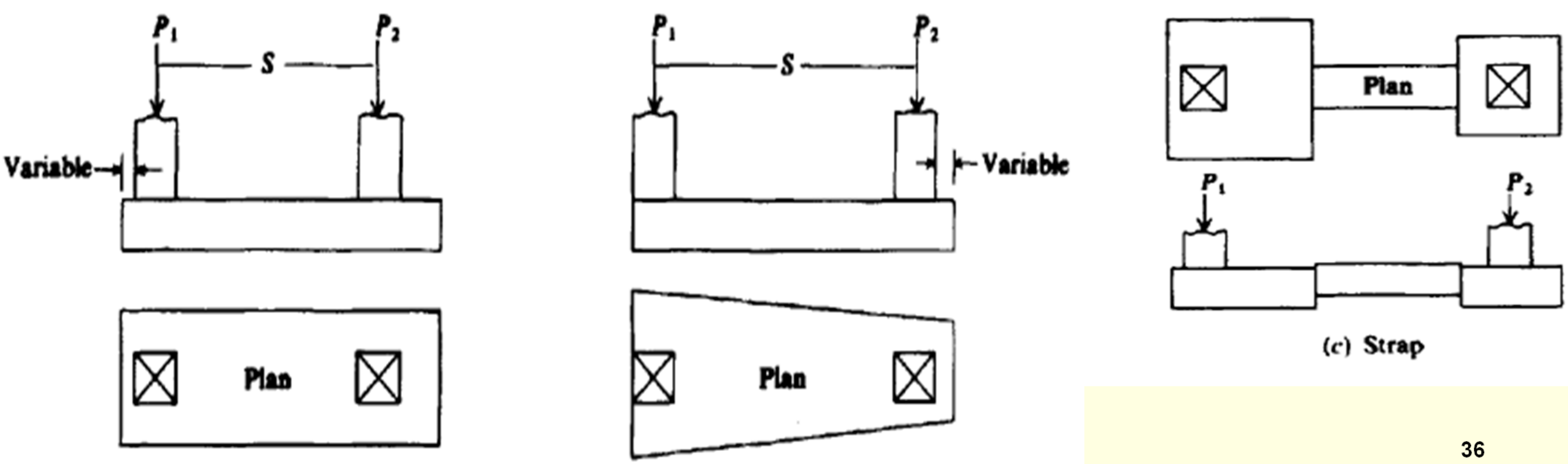



**Figure** Example of a loading condition that may induce eccentric loading in two directions.

# Combined Footings

- When a footing supports a line of two or more columns, it is called a combined footing.
- A combined footing may have either rectangular or trapezoidal shape or be a series of pads connected by narrow rigid beams called a strap footing.




(*a*) Rectangular (*b*) Trapezoid (*c*) Strap

# Combined Footings…

**Combined footings are used when:**

A. Columns are closely spaced and design/proportioning of isolated footings results in an overlap of footing areas and/or,

B. When there is a property line/boundary line/restriction and there exists a column along the boundary line and use of isolated footing is not possible.

# Combined Footings…

**Rectangular Combined footings** are used:

- When case(a) is encountered and the spacing between the columns is less than 6m-7m and/or,
- When case (b) is encountered and the outer column, which is the one along the boundary line, carries a larger load as compared to the inner column (the one to be combined with the outer column).

# Combined Footings…

- **Trapezoidal combined footings:** are used when case (b) is encountered and the inner column carries a larger load as compared to the load carried by the column along the boundary line.

# Design Procedure for Rectangular Combined Footing

- Generally, it is assumed that the rectangular footing is a rigid member, thus, the pressure is linear. The approach yields a rather conservative design; the moments are somewhat larger than those obtained by treating the footing as a beam on an elastic foundation.
- ***Given***: Typically included in the given part of the problem are column data (loads, sizes, reinforcement, location, and spacing), soil bearing, concrete strength *($f_{ck}$),* and grade of reinforcement *($f_{yk}$).*
- ***Objective***: The goal is to determine footing dimensions (width, length, thickness), steel reinforcement (bar sizes, spacing, placement, details, dowels), and relevant details for construction.

# The Problem:

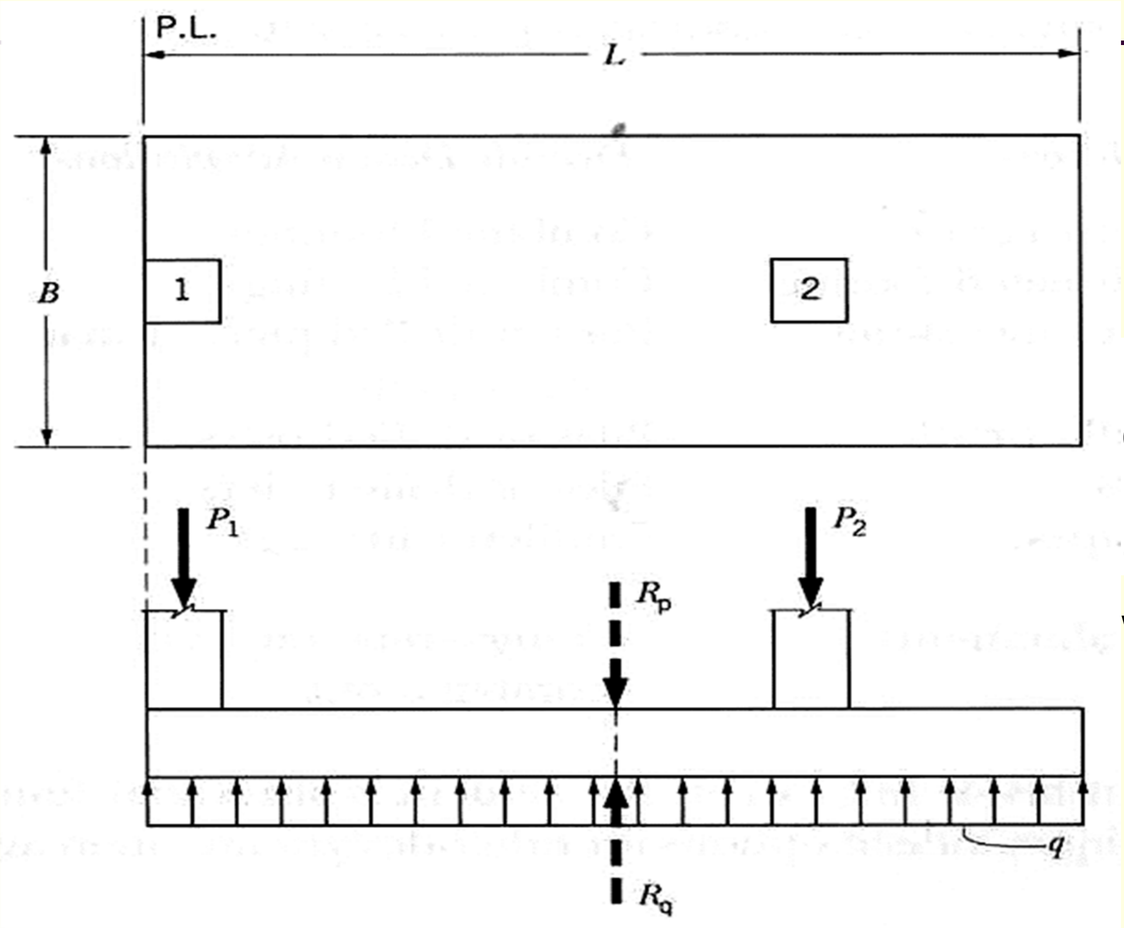



**Figure** Rectangular shaped combined footing. For uniform $q$, the resultant of the applied load is collinear with that of the soil pressure $q$.

# Design…

- ***Procedure*:** The design is predicated on the assumption that the footing is rigid and that the soil pressure is uniform. The following explanation may illustrate the procedure:
- ***Step* 1:** Convert the column loads to ULS loads via

$P_u$ = 1.3(D.L.) + 1.6(L.L.). Then convert the allowable soil pressure to ULS pressure via $q_u = (P_{lu} + P_{2u})\, q_a / (P_1 + P_2)$.

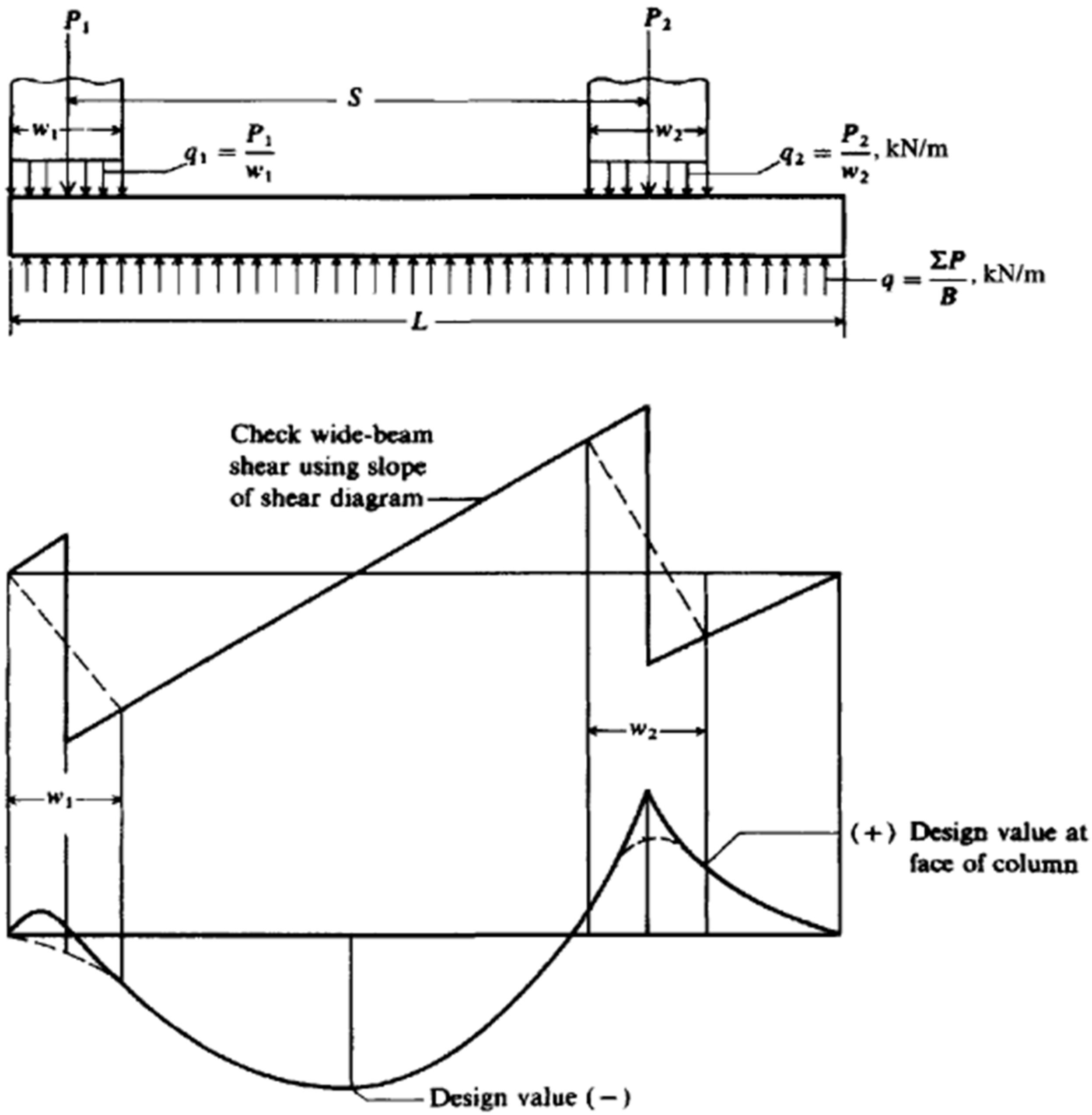
- ***Step* 2:** Determine the footing length *(L)* and width *(B).* First determine the location of the load resultant distance

(X). This point coincides with the midpoint of *L*, thus yielding the value for *L*. *B* is then determined from:

$$B = \sum P_u \Big/ L q_u$$

# Design…

- ***Step* 3:** Draw shear and moment diagrams. The footing is treated as a beam, loaded with a uniform soil pressure (upward) and column loads (downward), which are treated as concentrated loads.



$P_1$
$P_2$
$S$
$w_1$
$w_2$
$q_1 = \frac{P_1}{w_1}$
$q_2 = \frac{P_2}{w_2}$, kN/m
$q = \frac{\Sigma P}{B}$, kN/m
$L$
Check wide-beam shear using slope of shear diagram
$w_2$
$w_1$
(+) Design value at face of column
Design value (−)

# Design…

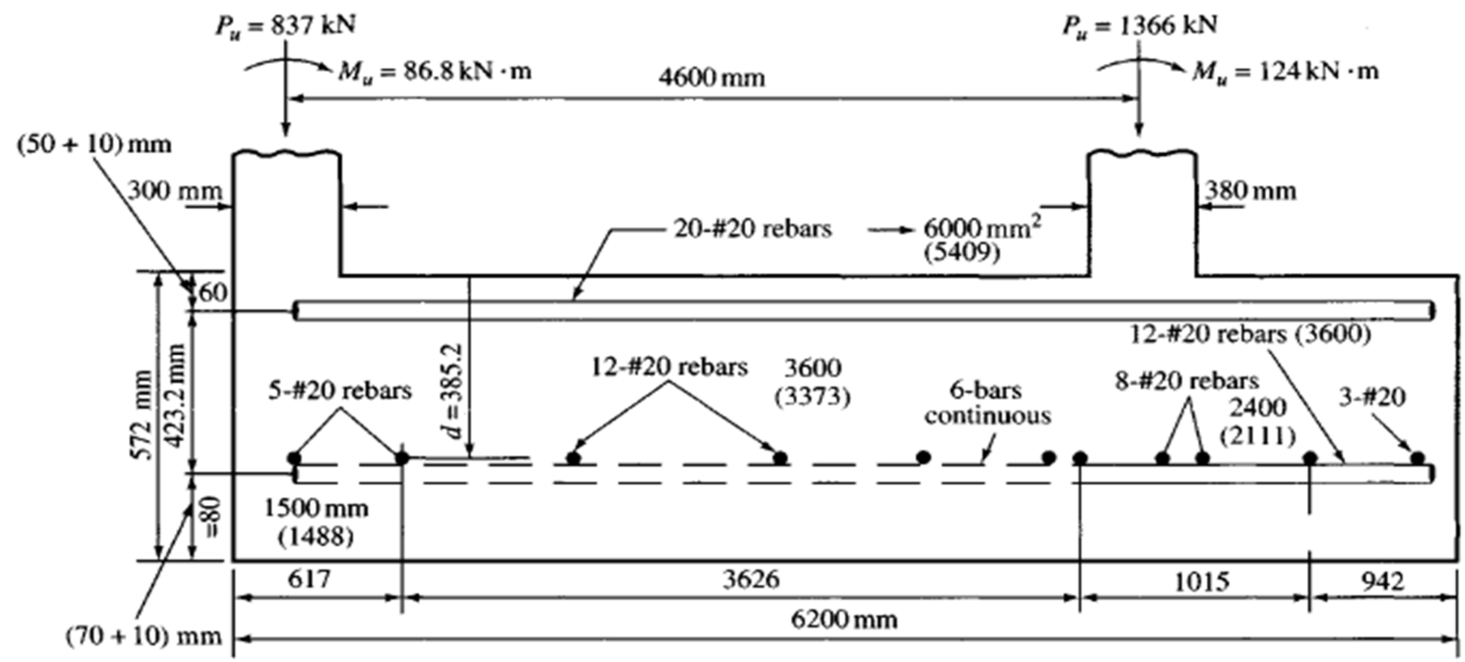
- ***Step* 4:** Determine the flexural reinforcing steel based on reasonable assumption of footing depth. The longitudinal (flexural) steel is designed using the critical moments (negative and positive) from the moment diagram. Thus, typically, combined footings will have longitudinal steel at both top and bottom of the footing.
- ***Step* 5:** Check footing depth based on shear. Critical sections are at *1.5d* for diagonal tension (or punching shear) and at the *d* for a wide beam, the same as for spread footings. The critical section for wide-beam shear is investigated only at one point (max. shear). For punching shear, however, an investigation of a three- or four-sided zone for each column may have to be done.

# Design…

- ***Step* 6:** Determine the steel in the short direction. The steel in the transverse direction is-determined based on an equivalent soil pressure *q'* and subsequent moment, for each column. Even for stiff footings, it is widely accepted that the soil pressure in the proximity of the columns is larger than that in the zone between columns. Thus, for design, we account for this phenomenon by assuming an empirical effective column zone width of s. The soil pressure in this zone, *q'*, is calculated as $q' = P_u / B_s$, where $P_u$ is the ULS column load, *B* the footing width and s an equivalent width of footer strip for the column in question. Commonly, the value of s is taken as the width of the column (in the longitudinal direction) plus about *0.75d* on each side of that column.

# Design…

- ***Step* 7:** Evaluate dowel steel. The requirements are the same as for spread footings.
- ***Step* 8:** Provide a drawing showing final design. This drawing is to show sufficient detail from which one may construct.



Final design

# EXAMPLE

# Trapezoid-shaped Footings

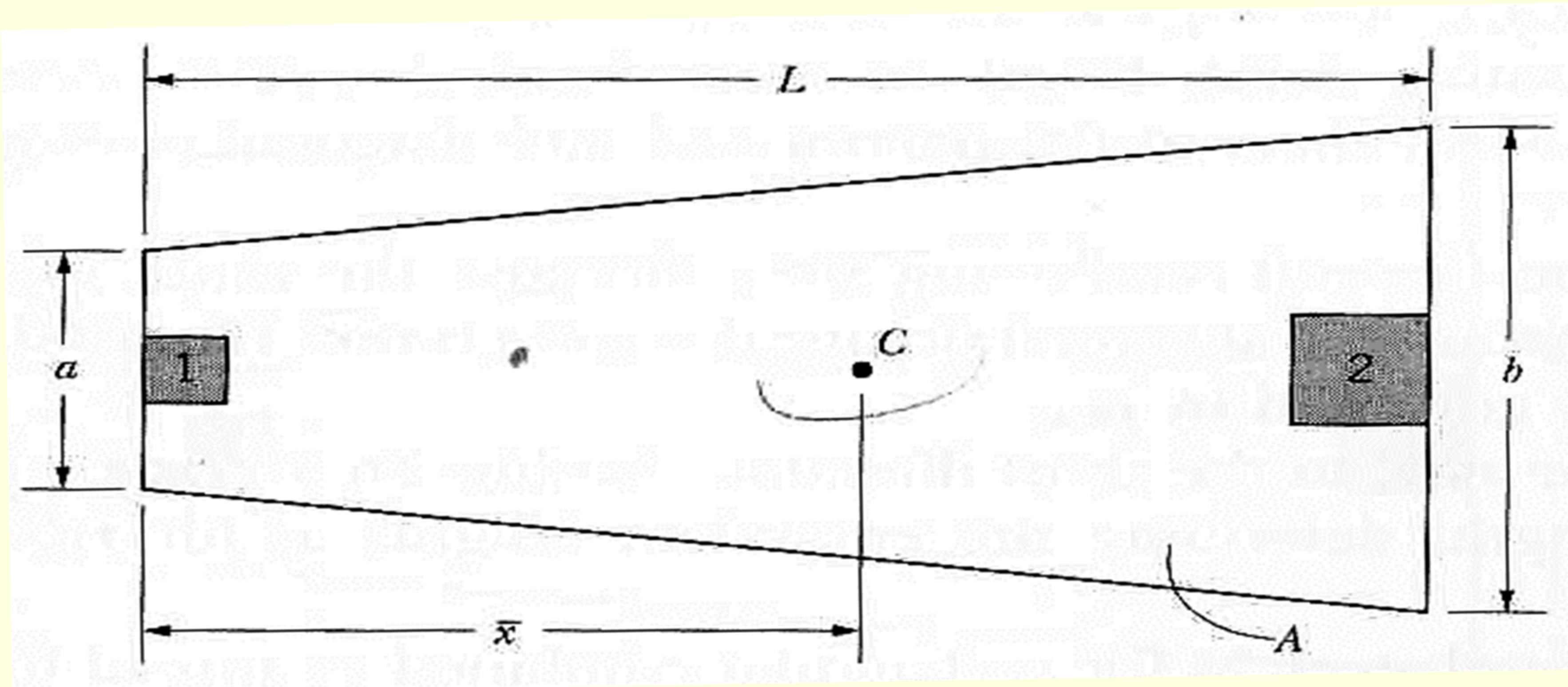



**Figure** Trapezoid-shaped footing

# Trapezoidal…

The area, A, is:

$$A = (a+b)L/2$$

$$\bar{x} = \sum Ax / \sum A, \ weget$$

$$\bar{x} = \frac{(aL)(L/2) + [(b-a)(L/2)]2L/3}{aL + (b-a)L/2}$$

$$\bar{x} = (L/3)(a+2b)/(a+b)$$

# Trapezoidal…

- ***Given***: Included in the given data are column information (loads, sizes, location, and spacing), length of footing *(L)*, soil bearing values *($q_a$)*, concrete strength *($f_{ck}$)*, and grade of reinforcement *($f_{yk}$)*
- ***Objective***: The goal is to determine footing dimensions (width, thickness), steel reinforcement (bar sizes, spacing, placement, details, dowels), and relevant details for construction.
- ***Procedure***: The design is predicated on the assumption that the footing is rigid and that the soil pressure is uniform. The basic steps are:

# Trapezoidal…

- ***Step* 1**: Convert the column loads to ultimate loads via $P_u$ = 1.3(DL) + 1.6(LL); then convert the allowable soil pressure to ultimate; that is, $q_u = (P_{ul} + P_{u2})\, q_a / (P_l + P_2)$.
- ***Step* 2**: Determine dimensions *a* and *b* via simultaneous solutions of two independent equations.

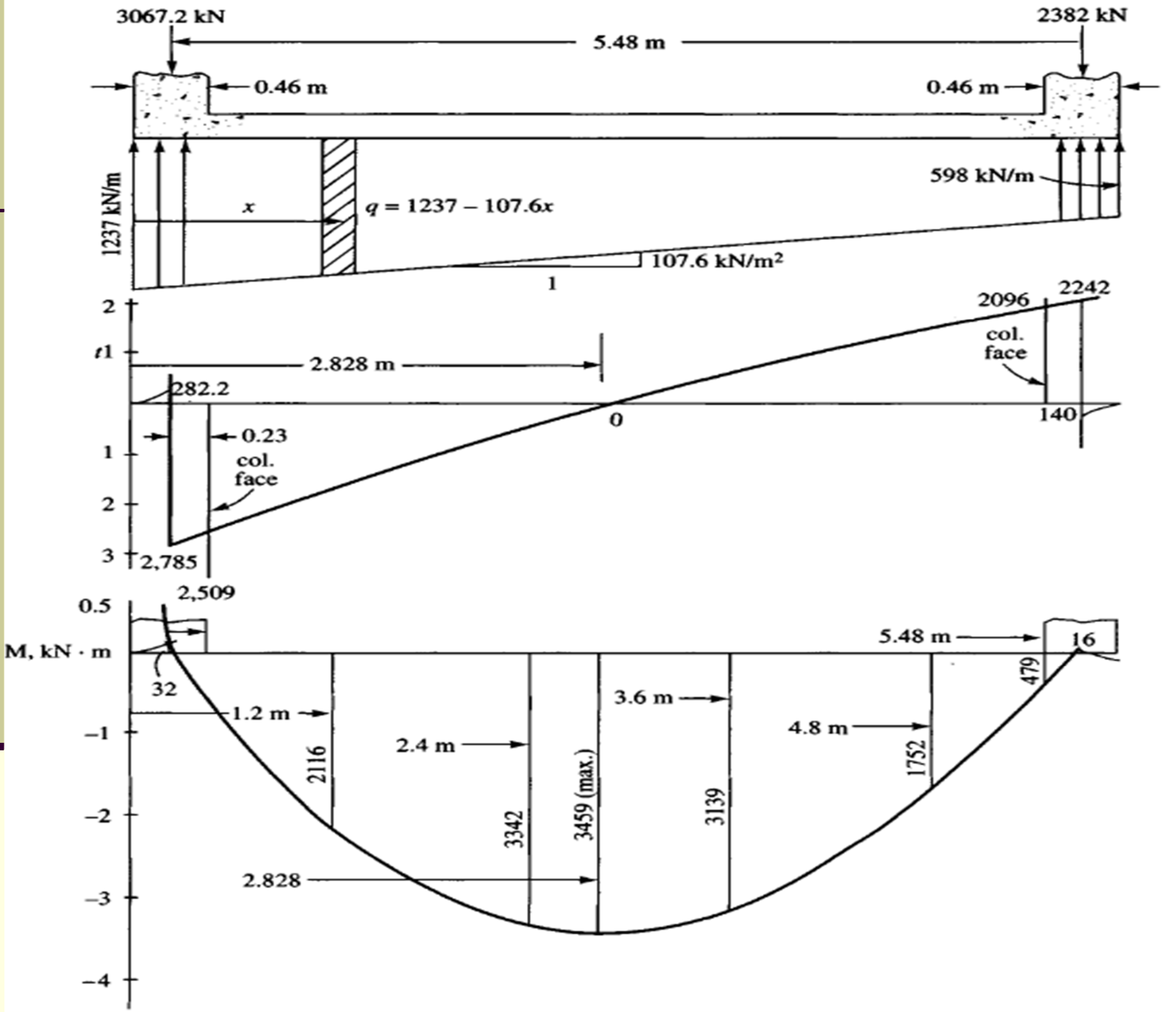
$$A = (a+b)L/2$$

$$\bar{x} = (L/3)(a+2b)/(a+b)$$

Thus, we solve for *a* and *b.*

# Trapezoidal…

- ***Step* 3**: Draw the shear and moment diagrams. The footing is treated as a beam, loaded with a uniform soil pressure (upward) and column loads (downward), which are treated as concentrated loads. Note that while the pressure is uniform, the pressure force for-unit length varies with the width [e.g., at the narrow end, the load is $a(q_u)$; and $b(q_u)$ at the wide end, etc.].
- ***Step* 4**: Determine footing depth based on shear (Use $\rho_{min}$ and $\rho_e$ = 0.015 for $k_1$ in wide beam shear and punching shear respectively). Critical sections are usually checked for wide-beam shear at the narrow end and punching shear at the wide end.


3067.2 kN
5.48 m
2382 kN
0.46 m
0.46 m
598 kN/m
1237 kN/m
x
q = 1237 – 107.6x
107.6 kN/m²
1
2
t1
2.828 m
2096
2242
col.
face
282.2
0.23
col.
face
0
140
1
2
3
2,785
2,509
0.5
M, kN · m
32
1.2 m
–1
2116
2.4 m
3342
3459 (max.)
3.6 m
3139
4.8 m
1752
5.48 m
479
16
–2
2.828
–3
–4

# Trapezoidal…

- ***Step* 5**: Determine the flexural reinforcing steel. Because the width varies, it is advisable to determine $-A_s$ at several points; the same is now required for $+A_s$ since it is typically governed by $\rho_{min}$.
- ***Step* 6**: Determine the steel in the short direction. Assume an average length for the cantilever length; determine the equivalent lengths as for rectangular footings.
- ***Step* 7**: Determine dowel steel, as for rectangular combined or spread footings.
- ***Step* 8**: Provide a drawing with details for construction. Here some judgment is necessary to accommodate the steel arrangement in view of the variable width along the footing.

# Strap Footings

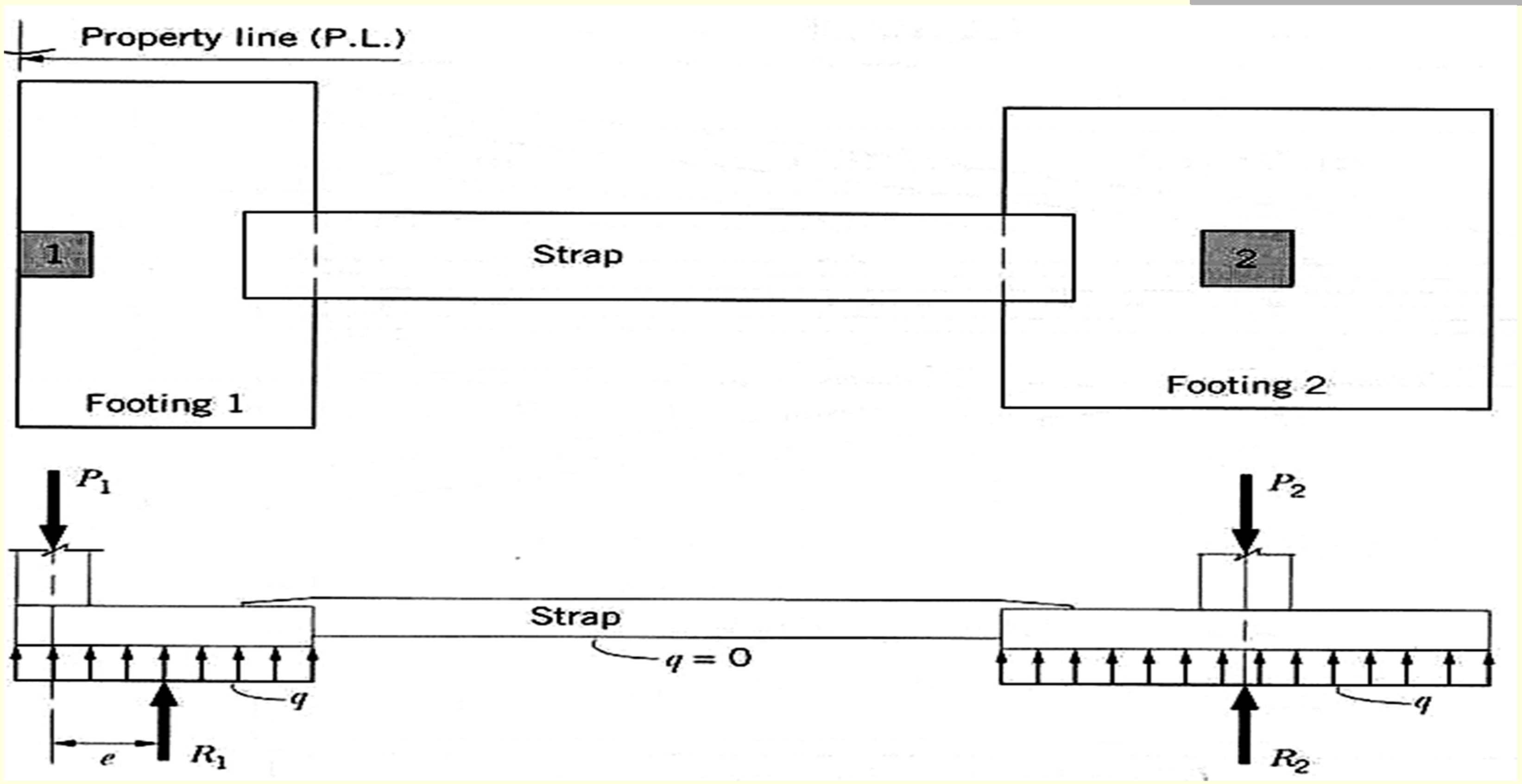



**Figure** Typical configuration of a strap footing

# Strap Footings…

- ***Given*** Typically, included in the given part of the problem are column data (loads, sizes, reinforcement, location, and spacing), allowable soil bearing, *q.,* concrete strength $(f_c)$, and grade of reinforcement $(f_y)$.
- ***Objective*** The goal is to (a) determine the footing dimensions (length, width, and thickness) proportioned such that the soil pressure is reasonably uniform and differential settlement is minimal, (b) design the strap, (c) design the footings, and (d) show a drawing with pertinent details for construction purposes.

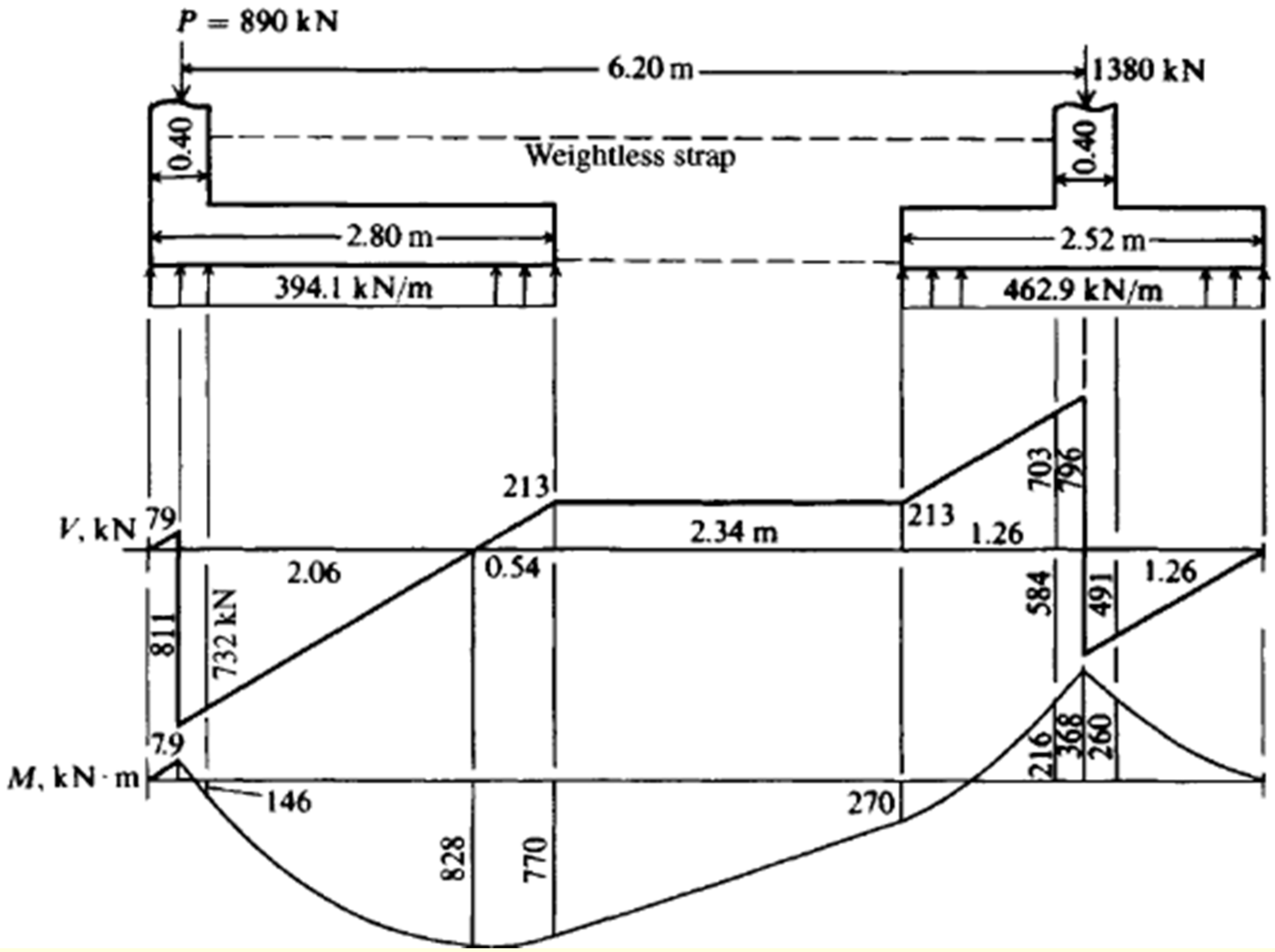
# Strap Footings…

- ***Procedure****:* The design assumes no soil pressure under the strap (other than that necessary to support the weight of the strap; hence, the weight of the strap is negated). The footings are designed as isolated footings subjected to column loads and strap reactions.
- ***Step* 1** (a) Convert to $P_u$ and $q_u$, as previously described. (b) Try a value for $e$. This establishes the position of $R_1$; subsequently, this influences the ratio of $L_l$ and $B_l$ . An adjustment in $e$ may be warranted if $L_l/\ B_l$ appear unreasonable. (c) From equilibrium (i.e., $\sum M = 0$ and $\sum F_y = 0$), determine the values for $R_l$ and $R_2$.

# Strap Footings…

- ***Step* 2** Determine footing dimensions, L and *B.* Note that *q* will be uniform when *R* coincides with the centroid of that footing. Also, for minimum differential settlement, *q* should be the same for both footings.
- ***Step* 3** Draw the shear *(V)* and moment *(M)* diagrams.

# Strap Footings…



# Strap Footings…

- ***Step* 4** Design the strap as a beam. Use maximum, M in the section between footings. Affix the strap to the footings to effectively prevent footing rotation.
- ***Step* 5** Design the footings as spread (isolated) footings with reinforcement in both directions including -$A_s$ steel to accommodate the negative moment. Some special assessment for the transverse steel near column 1 is recommended.
- ***Step* 6** Provide the final drawing showing details for construction.

# Thank you

# CHAPTER FOUR

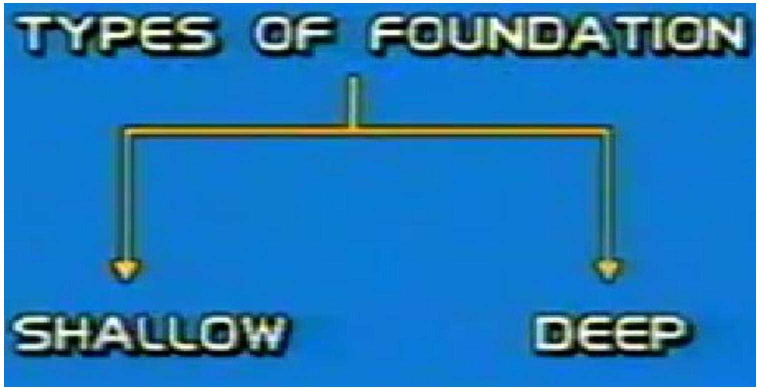
# DESIGN OF DEEP FOUNDATION

# INTRODUCTION

## what is Foundation?

Foundation is:-

- Part of a building,
- Usually below the ground,
- Transfers and distributes the weight of the super structure onto the ground such that the compressive stresses do not exceed the bearing capacity of the soil.



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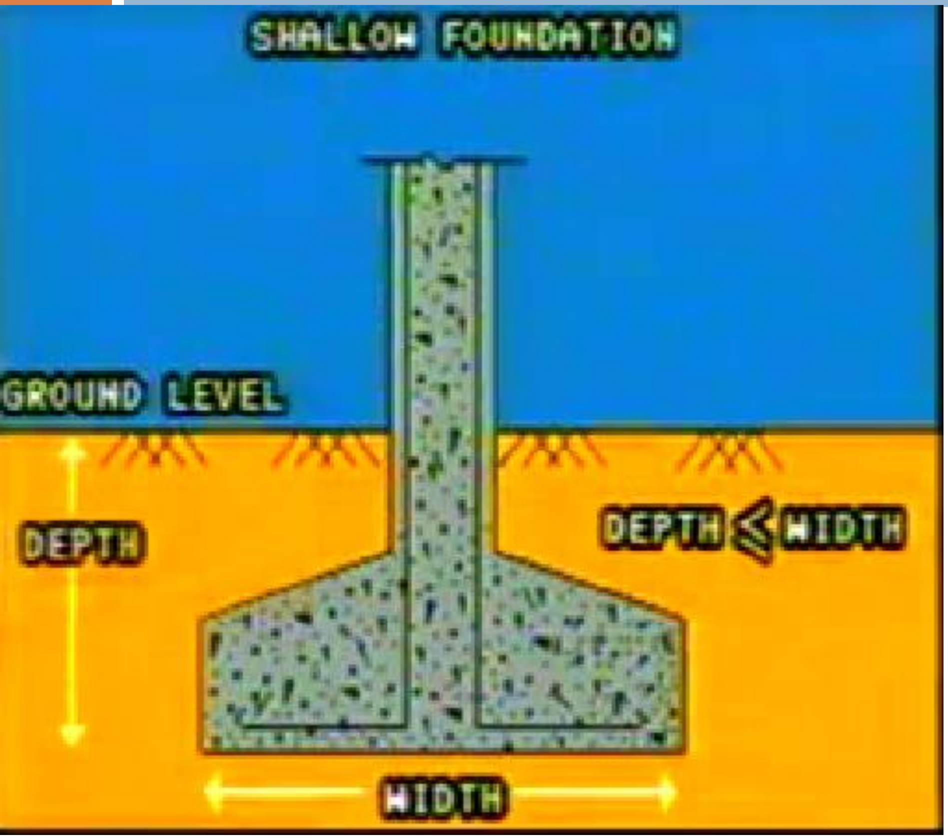
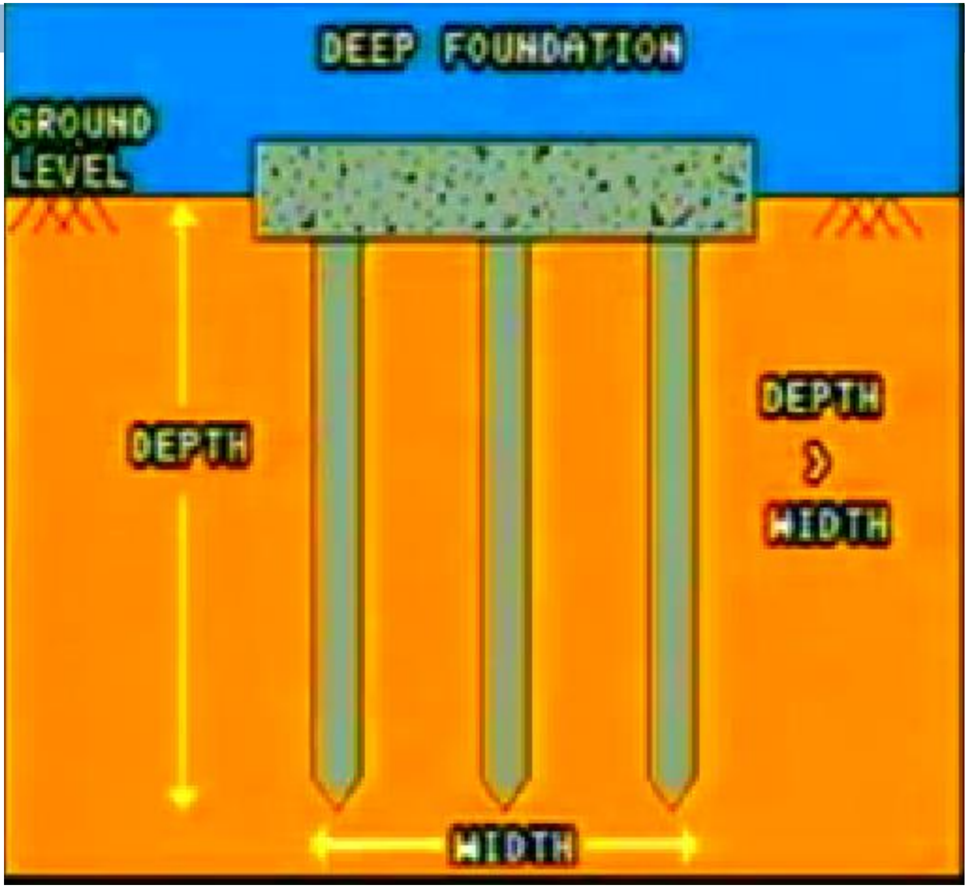

SHALLOW FOUNDATION
GROUND LEVEL
DEPTH
DEPTH ≤ WIDTH
WIDTH


DEEP FOUNDATION
GROUND LEVEL
DEPTH
DEPTH
>
WIDTH
WIDTH

# Piles Foundation

- Piles are structural members that are made of steel, concrete, timber or combination of them.
- Despite the cost, the use of piles often is necessary to ensure structural safety.
- They are used to build **pile foundations**, which are deep and which cost more than shallow foundations.

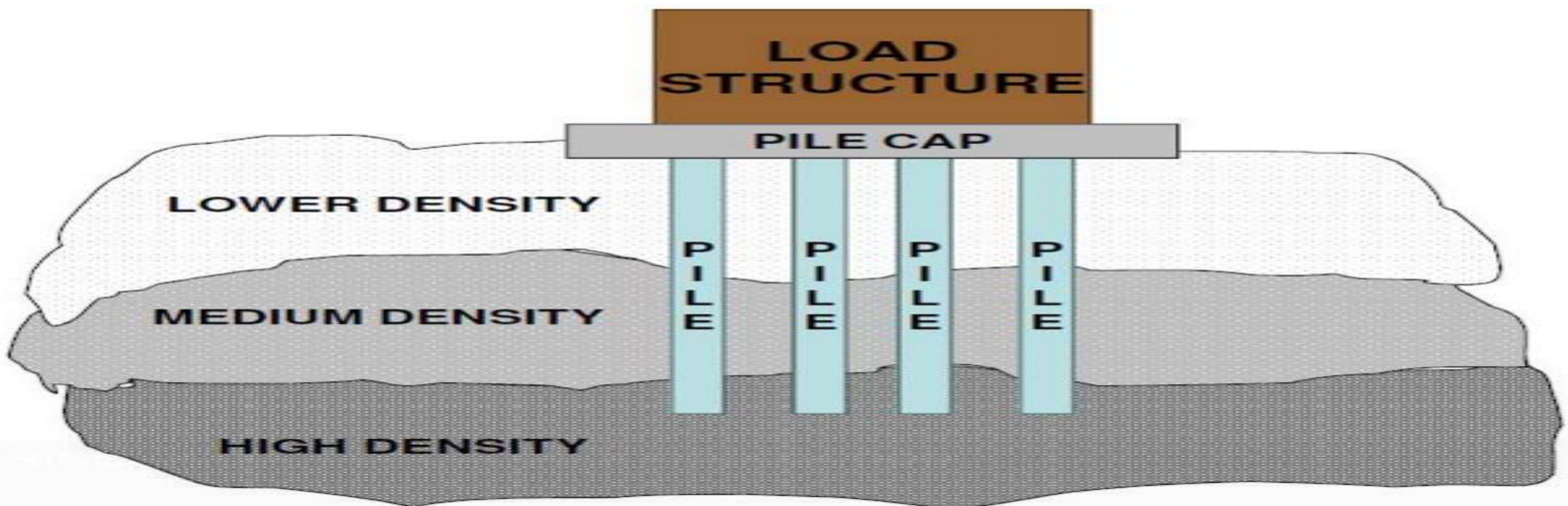
# PILE FOUNDATION

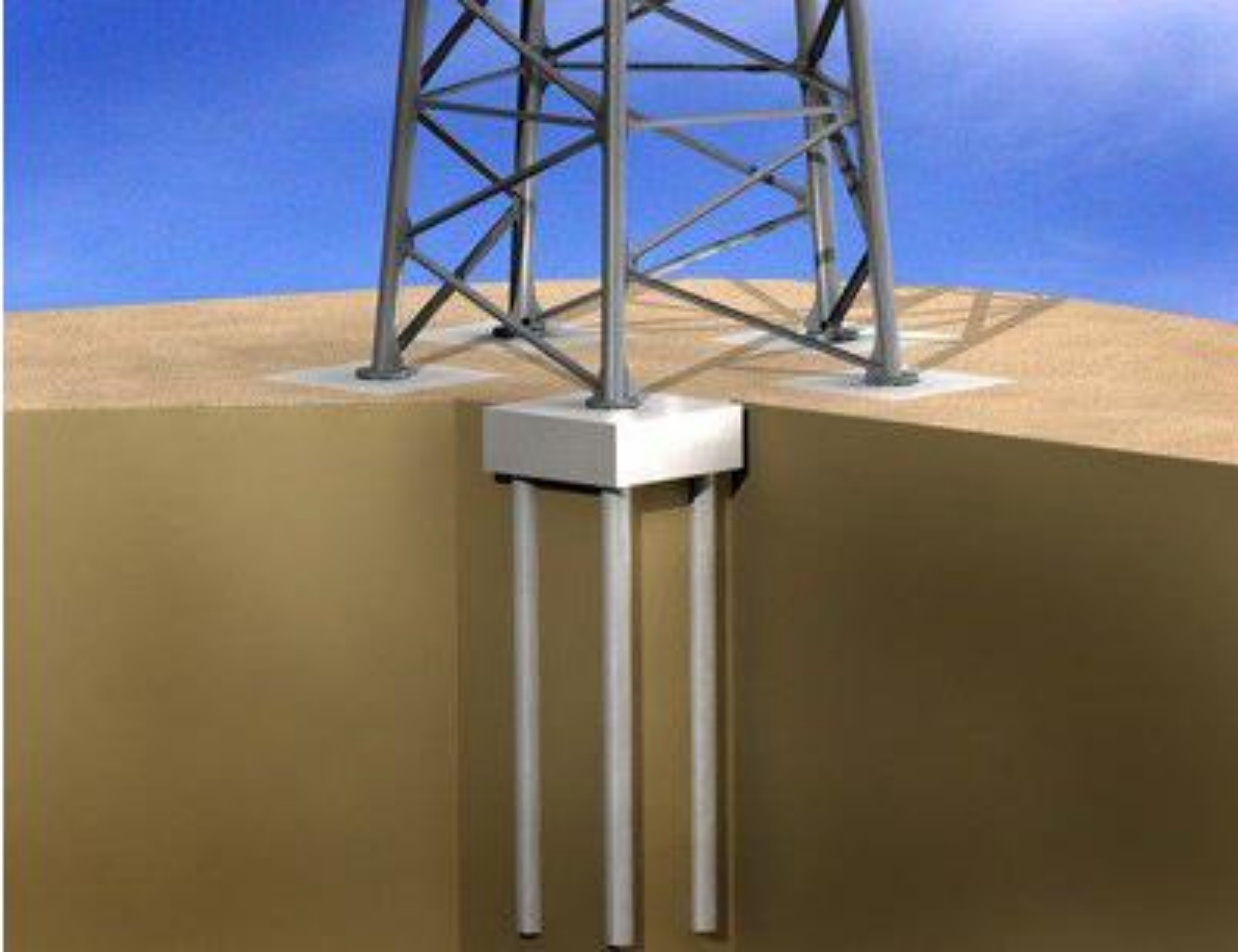
- A **pile foundation** is a substructure supported by piles.
- Unless the ground condition is rocks, for heavy construction and multi-storied buildings, the bearing capacity of soil at shallow depth may not be satisfactory for the loads on the foundation.
- In such cases, pile foundation has to be provided.
- The deeper the pile, or support pole goes, the more stable the structure should be. but cost should considered.

❖There are two basic parts of pile foundation:

❖The pile and

❖The pile cap.





# pile cap

- The pile cap serves as the **base of the structure**. It's similar to a spread foot in this way in that it can support a slab, a wall, or a structural column.
- The pile cap distributes the applied load to the individual piles which, in turn,. transfer the load to the bearing ground.
- However, it is different in that it puts all of the weight on a pile or a group of piles.

# Functions of piles

- As other types of foundations, the purpose of pile foundations is:
  - To transmit the buildings loads to the foundations and the ground soil layers whether these loads vertical or inclined
  - To control the settlements which can be accompanied by surface foundations etc.
  - To increase the factor of safety for heavy loads buildings
- The selection of type of pile foundation is based on site investigation report.

# Cont….

- Site investigation report suggests:
- The need of pile foundation,
- Type of pile foundation to be used,
- Depth of pile foundation to be provided.
- The cost analysis of various options for use of pile foundation should be carried out before selection of pile foundation types.

## CONDITIONS THAT REQUIRE USE OF PILE FOUNDATION?

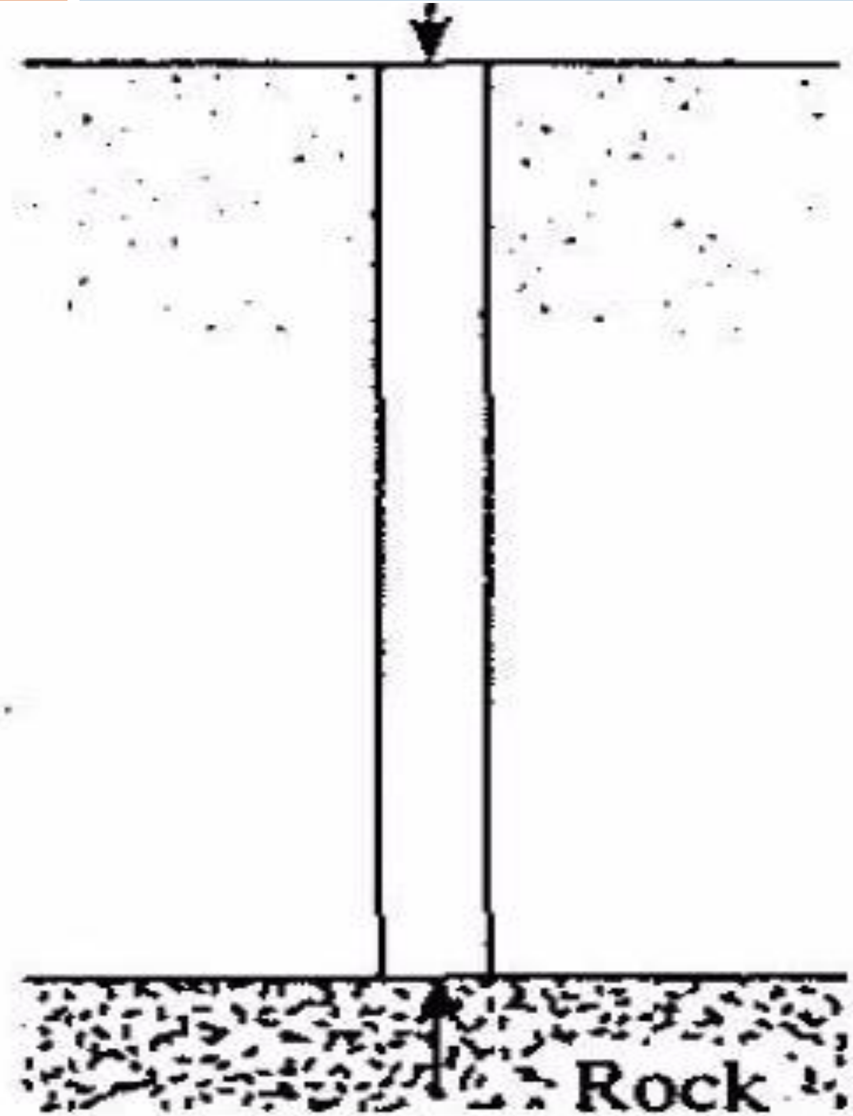
- Some of the conditions that require pile foundations are:
  - Compressible or weak upper soil layer
  - Presence of horizontal forces
  - Presence of expansive soils
  - Subjected to uplifting forces
  - Soil erosion

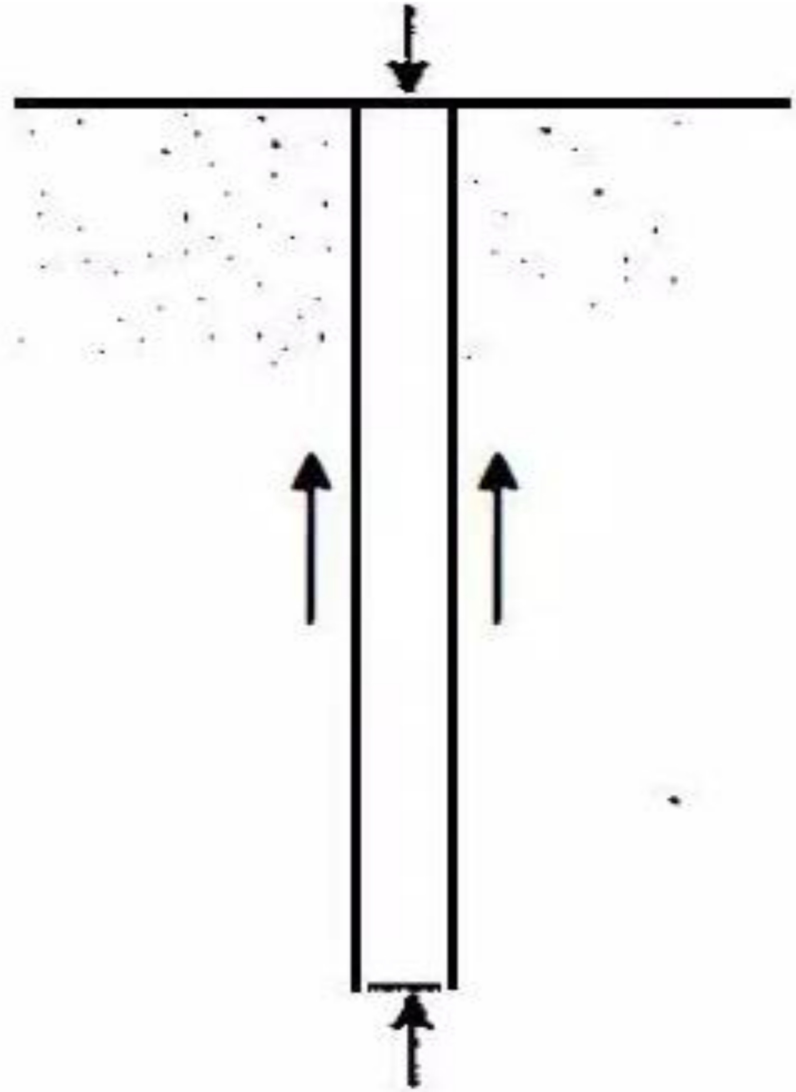
**Let us see one by one!**

# I. COMPRESSIBLE OR WEAK UPPER SOIL LAYER

- When one or more upper soil layers are highly compressible and too weak to support the load transmitted by the superstructure, piles are used to transmit the load to underlying **bedrock or a stronger soil layer**.
- When bedrock is not encountered at a reasonable depth below the ground surface, piles are used to transmit the structural load to the soil gradually.
- The **resistance to the applied structural load** is derived mainly from the **frictional resistance** developed at the soil-pile interface.

# Cont…

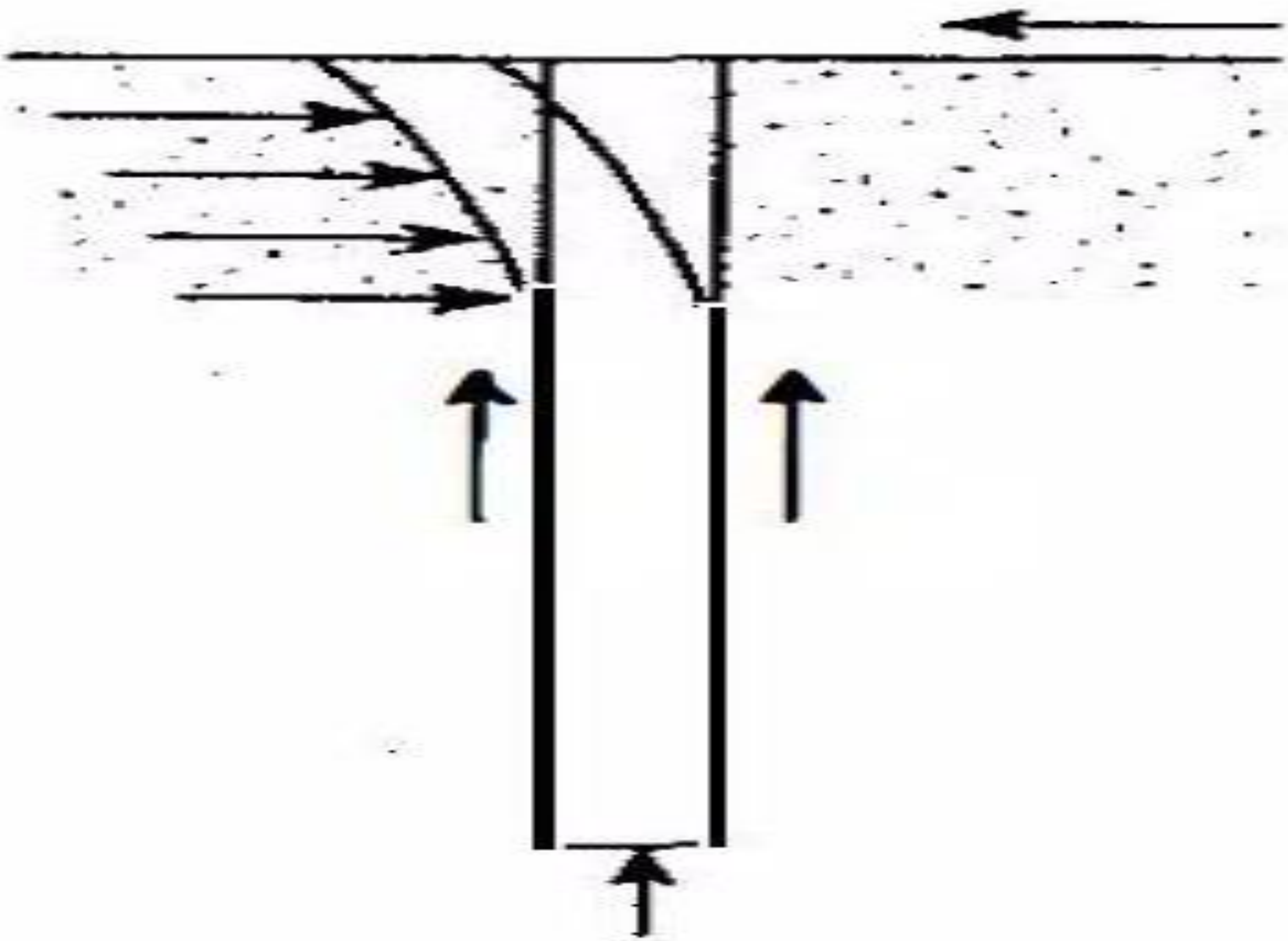


# II. PRESENCE OF HORIZONTAL FORCES

- When subjected to horizontal forces , pile foundations resist by **bending**, while still supporting the vertical load transmitted by the superstructure.
- This type of situation is generally encountered in the design and construction of **earth-retaining structures** and **foundations of tall structures** that are subjected to **high wind** or **earthquake** forces.

# Cont…

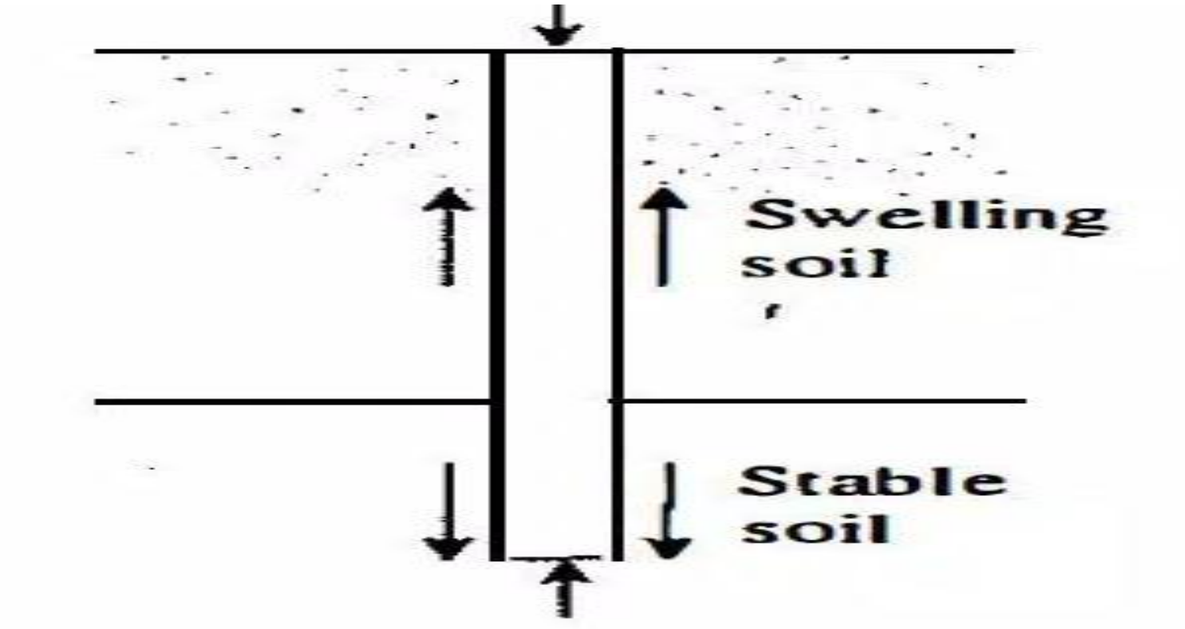


## III. PRESENCE OF EXPANSIVE SOILS

- In many cases, expansive and collapsible soils may be present at the site of a proposed structure. These soils may extend to a great depth below the ground surface.
- Expansive soils **swell and shrink** as their moisture content increases and decreases, and the pressure of the swelling can be considerable.
- If shallow foundations are used in such circumstances, the structure may suffer considerable damage. However, pile foundations may be considered as an alternative when piles are extended beyond the **active zone**, which is where swelling and shrinking occur.

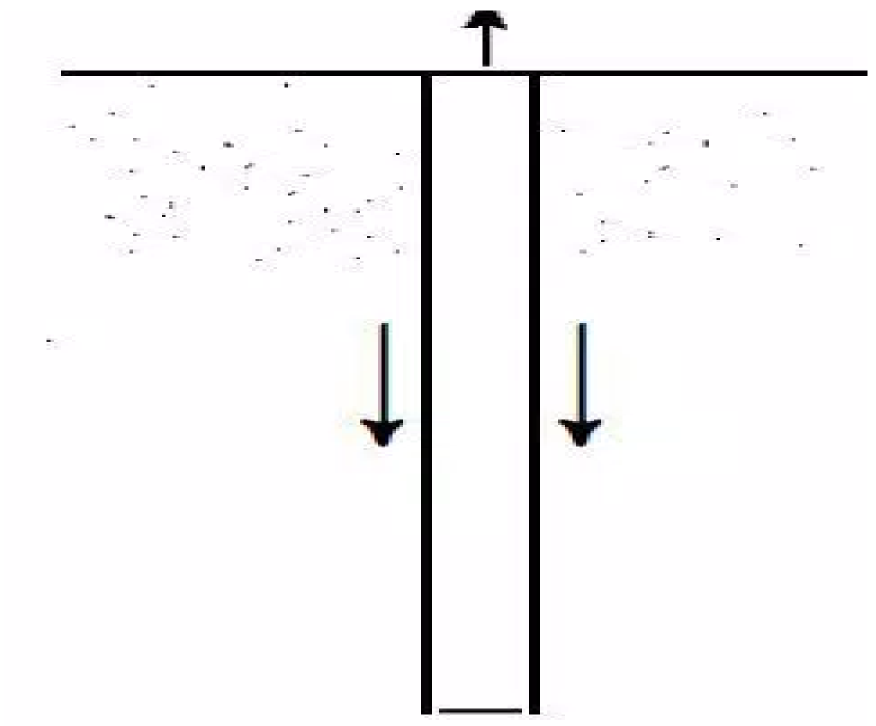
# Cont…

- A sudden decrease in the void ratio of soil induces large settlements of structures supported by shallow foundations. In such cases, pile foundations may be used in which the piles are extended into stable soil layers beyond the zone where moisture will change.



# IV. SUBJECTED TO UPLIFTING FORCES

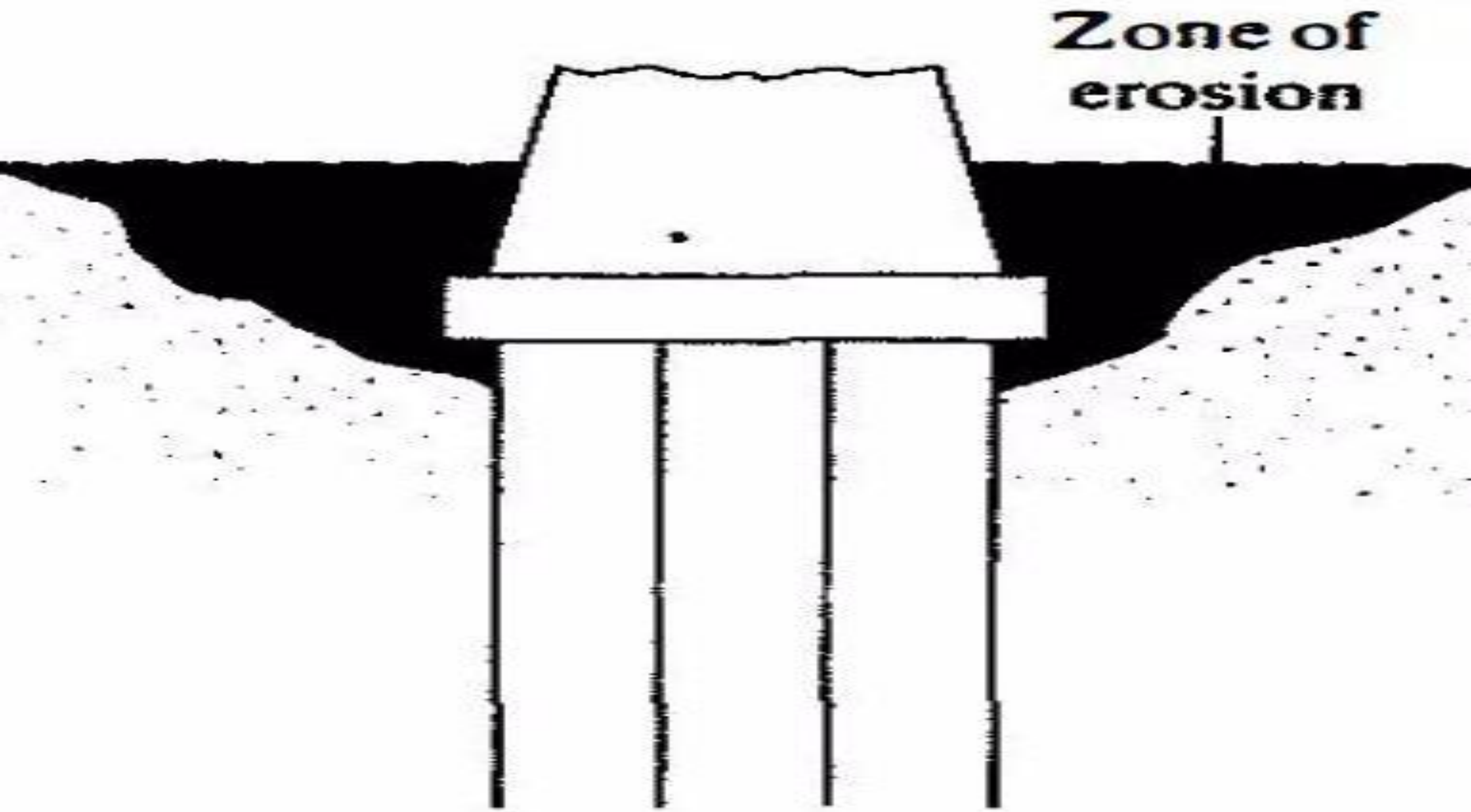
- The foundations of some structures, such as transmission towers, offshore platforms, and basement mats below the water table, are subjected to uplifting forces. Piles are sometimes used for these foundations to resist the uplifting force.



# V. SOIL EROSION

- Bridge abutments and piers are usually constructed over pile foundations to avoid the loss of bearing capacity that a shallow foundation might suffer because of soil erosion at the ground surface.
- The design and analysis of pile foundations may thus be considered somewhat of an art as a result of the uncertainties involved in working with some subsoil conditions.

# Cont…



# Types of Piles

A. Based on the material the are made, Piles are made from :

- Concrete
- Steel
- Timber or
- Composite

# Concrete piles

- **Advantage:**
  - Relatively cheap
  - It can be easily combined with concrete superstructure
  - Corrosion resistant
  - It can bear hard driving
- **Disadvantage:**
  - Difficult to transport
  - Difficult to achieve desired cut of

# STEPS OF CONCRETE PILE CONSTRUCTION

- Excavate the bore hole using basic equipment called boring rig which is equipped with pocket to bore and remove excavated material
- The pocket is attached with telescopic- Kelly to kept vertical movement of pocket in the borehole.
- The reinforcement cage for the cast institute pile is fabricated in the fabrication and transported.
- Insert the reinforcement cage in to the borehole mechanically lifted.
- Send the concrete using hopper and pipe.


SINGLE PILE


GROUP OF PILES


PILE CAP CONSTRUCTION


COLUMN CONSTRUCTION

FINALLY PILES CARRYING HEAVY LOADS

# Steel Piles

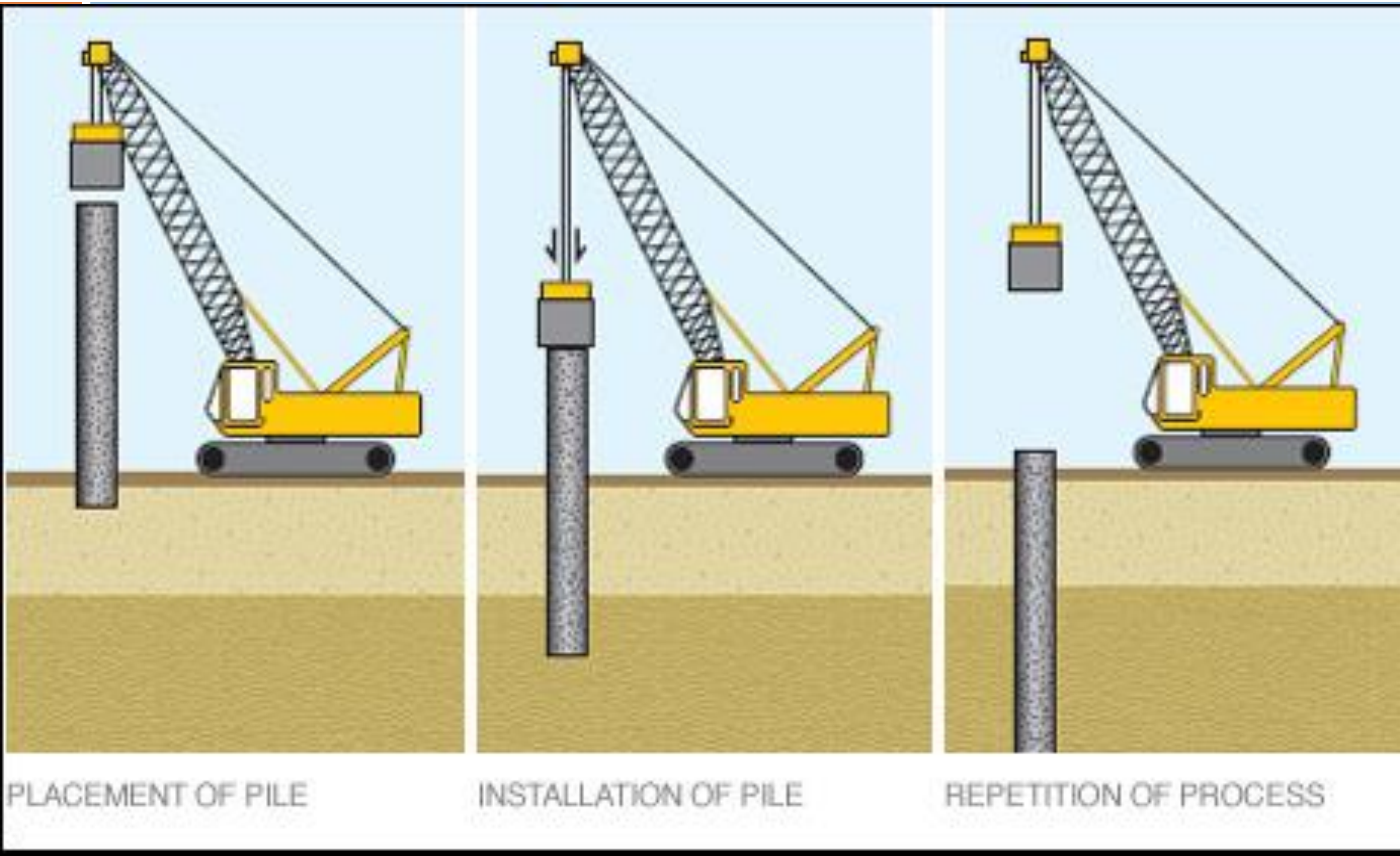
„ **Advantage:**

- Relatively less hassle during installation and easy to achieve cutoff level.
- High driving force may be used for fast installation
- Good to penetrate hard strata
- Load carrying capacity is high

- **Disadvantage:**

- Relatively expensive
- Noise pollution during installation
- Corrosion
- Bend in piles while driving

# Installation of precast piles piles



# TIMBER PILES

- Timber piles are made of-trees driven with small end as a point.
- Maximum length: 35 m; optimum length: 9 - 20m

**Advantages:**

- Comparatively low initial cost
- Permanently submerged piles are resistant to decay
- Easy to handle
- Best suited for friction piles in granular material.

# Cont…

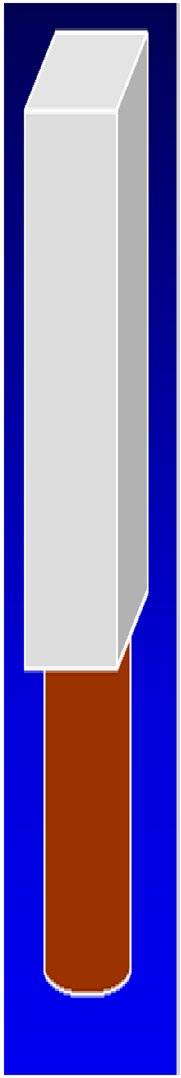
**Disadvantages of using timber piles:**

- Difficult to splice,
- Vulnerable to damage in hard driving,
- Vulnerable to decay unless treated with preservatives (If timber is below permanent Water table it will apparently last forever),
- If subjected to alternate wetting & drying, the useful life will be short,
- Partly embedded piles or piles above Water table are susceptible to damage from wood borers and other **insects unless treated.**


AUG

# Composite piles

- In general, a composite pile is made up of **two** or **more sections** of different materials or different pile types.
- The upper portion could be eased cast-in-place concrete combined with a lower portion of timber, steel or concrete filled steel pipe pile.
- These piles have limited application and arc employed under special conditions.



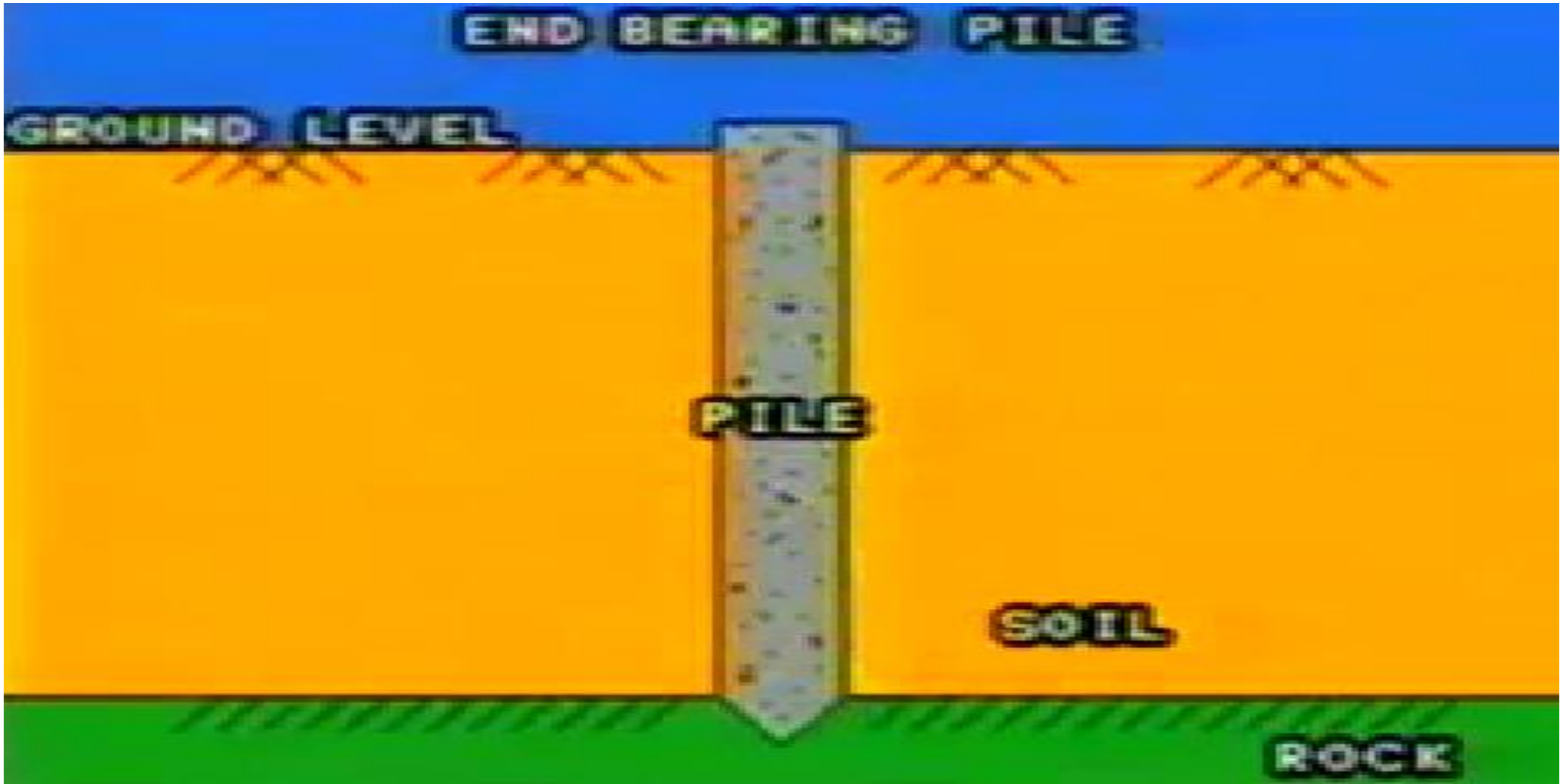
## cont…

B. Piles based on the effect on the soil can be Classified in to:

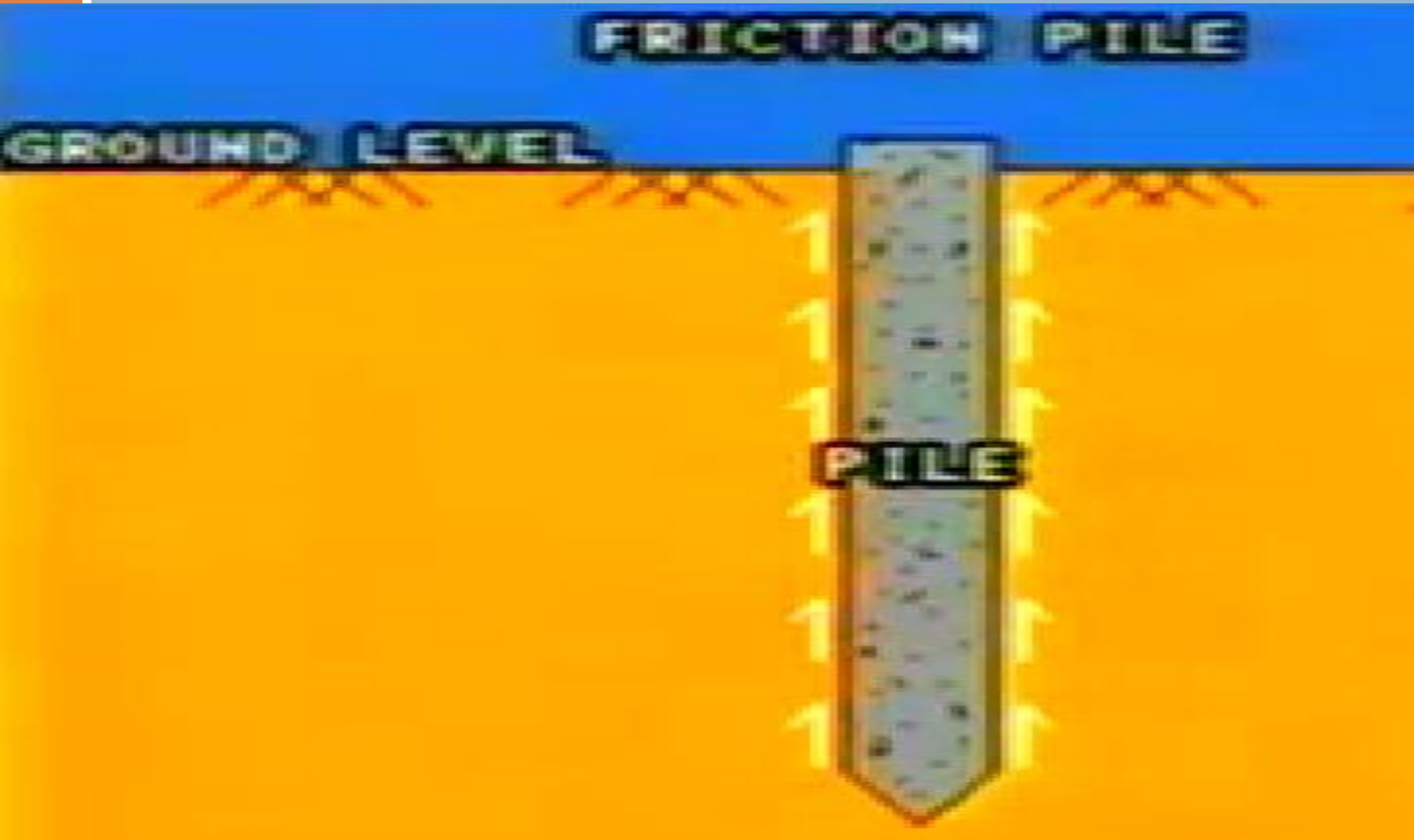
1. Driven piles are considered to be displacement piles
2. Bored piles (Replacement piles) which are generally considered to be non displacement piles.

C. Pile based mode of **load transformations**

1.End bearing piles $Q_u=Q_p$



# 2.Friction piles Qu=Qs



# 3.Combined end bearing and friction piles

- Qu=Qp + Qs

# LOAD CARYING CAPACITY

The methods are categorized as

- Static method
- Dynamic formulas
- In situe penetration test.
- Pile load test

# STATIC METHODS FOR DRIVEN PILES IN SAND

The ultimate capacity of single pile driven in to sand is calculated by:

Qu=Qp + QS

Where:

$$Q_p = q_p A_p$$
$$Q_s = f_s A_s$$

Qu=ultimate failure load

Qp=ultimate bearing capacity of soil at tip of pile

QS=resisting capacity of soil on pile wall or friction.

Ap=area of tip pile

Fs=average unit skin friction b/n sand and pile surface.

As= effective surface area of the pile in contact with soil.

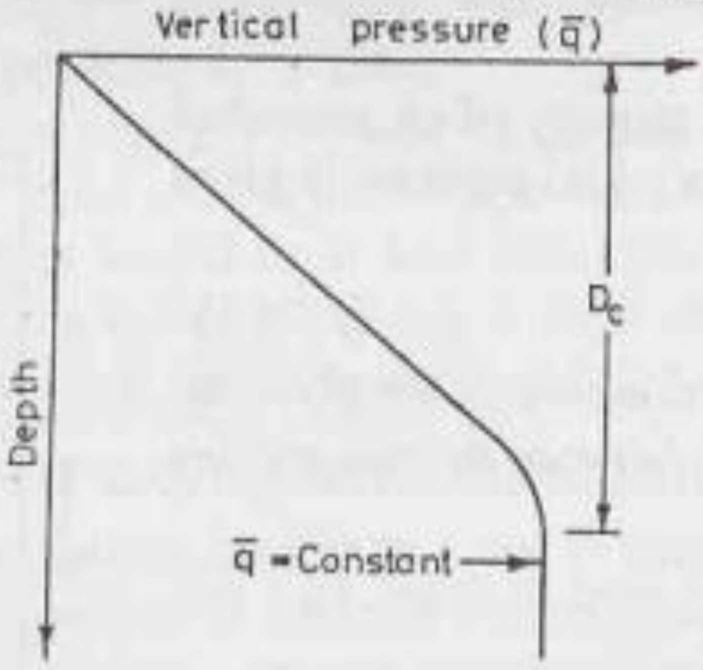
**Methods for determination of $Q_p$.** The ultimate bearing capacity ($q_p$) of the soil at the pile tip can

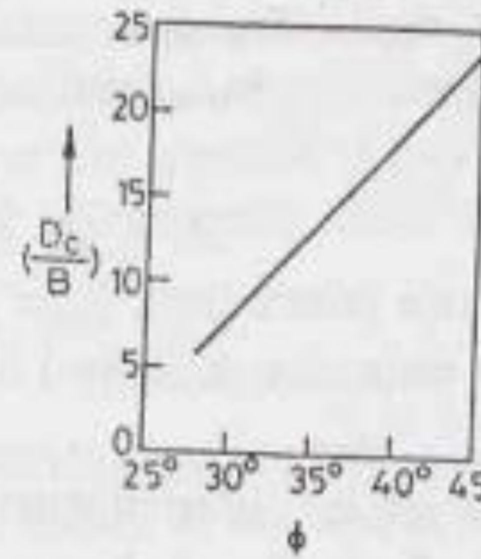
For sandy soils,

$$q_p = \bar{q} N_q + 0.4 \gamma B N_\gamma$$

$\bar{q}$ = effective vertical pressure at the pile tip, $B$ = pile tip width (or diameter),
$\gamma$ = unit weight of the soil in the zone of the pile tip.
$N_q$ and $N_\gamma$ = bearing capacity factors for deep foundations.

$$q_p = \bar{q} N_q$$



# Methods of determination of $Q_s$

$$\sigma_h = K\bar{\sigma}_v$$

where $K$ = earth pressure coefficient, $\bar{\sigma}_v$ = effective vertical pressure at that depth.
Thus unit skin friction ($f_s$) acting at any depth can be written as

$$f_s = \sigma_h \tan\delta \qquad \text{or} \qquad f_s = K\bar{\sigma}_v \tan\delta$$

where $\tan\delta$ = coefficient of friction between sand and the pile material.

| *Pile Material* | $\delta$ | $K$ *(loose sand)* | $K$ *(dense sand)* |
|---|---|---|---|
| Steel | 20° | 0.50 | 1.0 |
| Concrete | 0.75 $\phi$ | 1.0 | 2.0 |
| Timber | 0.67 $\phi$ | 1.5 | 4.0 |

As stated earlier, the effective vertical pressure ($\bar{\sigma}_v$) increases with depth only upto the critical depth.

Below the critical depth, the value of $\bar{\sigma}_v$ remains constant.

The frictional resistance ($Q_s$) can be expressed as

$$Q_s = \sum_{i=1}^{n} K\,(\bar{\sigma}_v)_i\,\tan\delta\ \ (A_s)_i$$

where $n$ = number of layers in which the pile is installed,

$(\bar{\sigma}_v)_i$ = effective normal stress in ith layer,

$(A_s)_i$ = surface area of the pile in ith layer,

Eq. 25.9 (*a*) can be written as

$$Q_s = \sum_{i=1}^{n} K \tan\delta\ (\text{area of } \bar{\sigma}_v \text{ diagram}) \times \text{pile perimeter}$$

The ultimate load for the pile (Eq. 25.1) can be written as

$$Q_u = Q_p + Q_s$$

or

$$Q_u = \bar{q}\,N_q A_p + \sum_{i=1}^{n} K\,(\bar{\sigma}_v)_i \tan\delta\ \ (A_s)_i$$

# STATIC METHOD FOR DRIVEN PILES IN SATURATED CLAY

for the determination of the load-carrying capacity of driven piles in saturated clay.

The point resistance ($Q_p$) can be expressed as

$$Q_p = q_p A_p$$

$$Q_p = q_p A_p$$

where $q_p$ is the unit point resistance, equal to the ultimate bearing capacity ($q_u$) of the soil.

For cohesive soils ( $\phi = 0$), the ultimate bearing capacity is found from the following equation, which is similar to that for a shallow foundation.

$$q_u = cN_c + qN_q$$

As $N_q = 1.0$ for $\phi = 0$, the above equation becomes

$$q_u = cN_c + q$$

Therefore, $$Q_p \text{ (gross)} = (cN_c + q) A_p$$

or $$Q_p \, (net) = cN_c A_p$$

In above equations, $c$ is the cohesion of the clay in the zone surrounding the pile tip, and $N_c$ is the bearing capacity factor for the deep foundation.

The value of $N_c$ depends upon the $D/B$ ratio and it varies from 6 to 9.0. A value of $N_c = 9.0$ is generally used for the piles. In the case of short piles ($D/B \leq 5.0$), the value of $N_c$ is reduced to the values proposed

The skin resistance ($Q_s$) of the pile can be expressed as

$$Q_s = c_a A_s$$

where $\alpha$ is the adhesion factor and $\bar{c}$ is the average cohesion along the shaft length.

The value of $\alpha$ depends upon the consistency of the clay. For normally consolidated clays, the value of $\alpha$ is taken as unity. According to IS : 2911—1979, the value of $\alpha$ can be taken as unity for soils having soft

# Cont…

- So for cohesive soils, the ultimate load can be determined by adding the point resistance and shaft resistance.i.e

$$Q_u = cN_c A_p + \alpha \bar{c} A_s$$

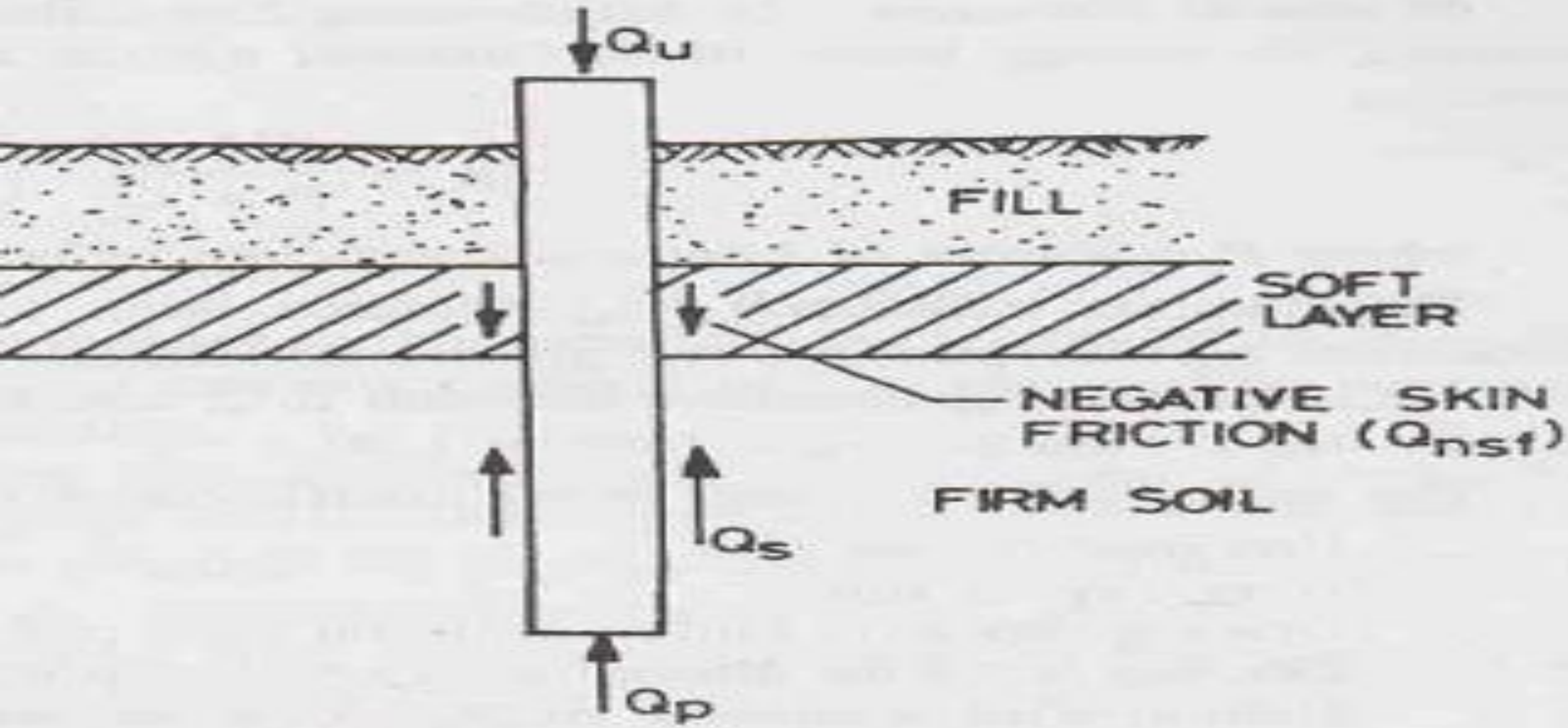
- And

$$Q_{all} = Q_u / FS$$

# NEGATIVE SKIN FRICTION

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

Negative skin friction develops when a soft or loose soil surrounding the pile settles after the pile has been installed. The negative skin friction occurs in the soil zone which moves downward relative to the pile.



# cont…

owever, the direction is downwards.

The net ultimate load-carrying capacity of the pile

$$Q_u' = Q_u - Q_{nsf}$$

where $Q_{nsf}$ = negative skin friction,

$Q_u'$ = net ultimate load.

## DYNAMIC FORMULAE

**Engineering News Record Formula.**

The dynamic formulae are based on the assumption that the kinetic energy delivered by the hammer during driving operation is equal to the work done on the pile. Thus cont…

$$W h \eta_h = R \times S$$

where $W$ = weight of hammer (kN), $h$ = height of ram drop (cm), $\eta_h$ = efficiency of pile hammer, $R$ = pile resistance (kN), taken equal to $Q_u$, and $S$ = pile penetration per blow (cm).

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

where $S$ = penetration of pile per hammer blow. It is generally based on the average penetration obtained from the last few blows (cm), $C$ = constant (For drop hammer, $C$ = 2.54 cm and for steam hammer, $C$ = 0.254 cm)

# REFFERENCE


- *Dr. Swami Saran "Soil Dynamics and Machine Foundations First Edition 1999*
- *Principles of soil dynamics Braja M.das*
- *Arora soil mechanics*
- *Advanced foundation engineering handout*

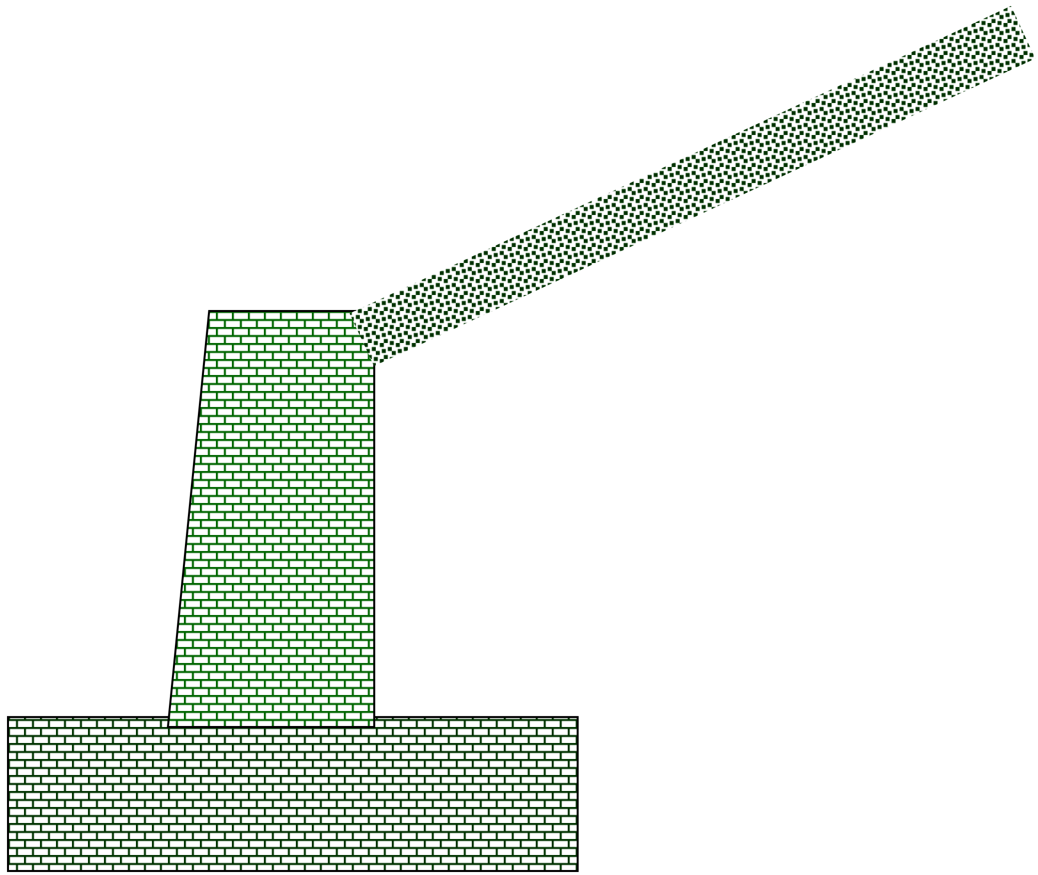
THANK YOU !

# 5.Analysis and Proportioning of Retaining walls

- Retaining walls are structures used to provide stability of earth or other material where conditions disallow the mass to assume its natural slope.
- Common Types of retaining walls

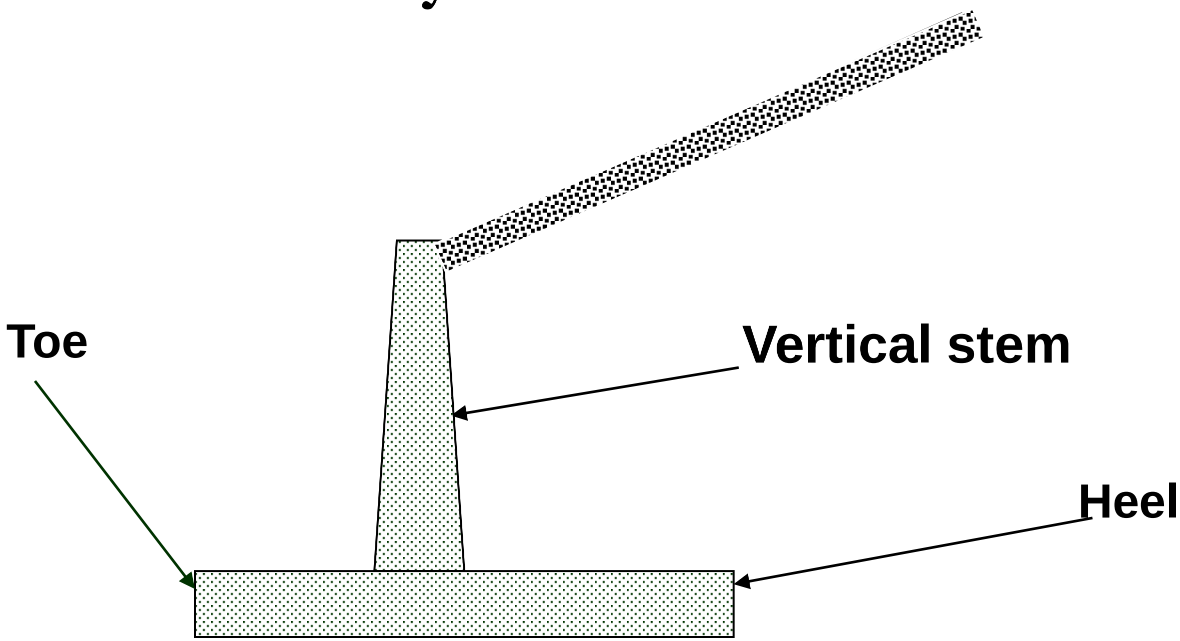
**1.Gravity walls:**

- made of plain concrete or stone masonry.
- depends upon its weight for stability.
- trapezoidal in section with the base projecting beyond the face and back of the wall.
- no tensile stress in any portion of the wall.
- economically used for walls less than 6m high.



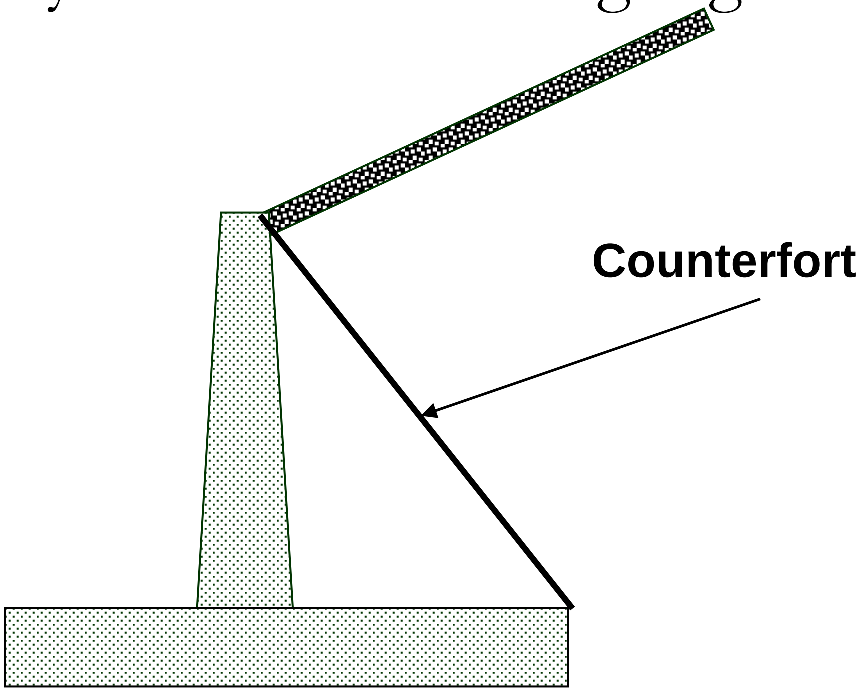
## 2. Cantilever walls

- made of reinforced concrete material.
- inverted T-shaped in section with each projecting acts as a cantilever.
- economically used for walls 6 to 7.5m high.



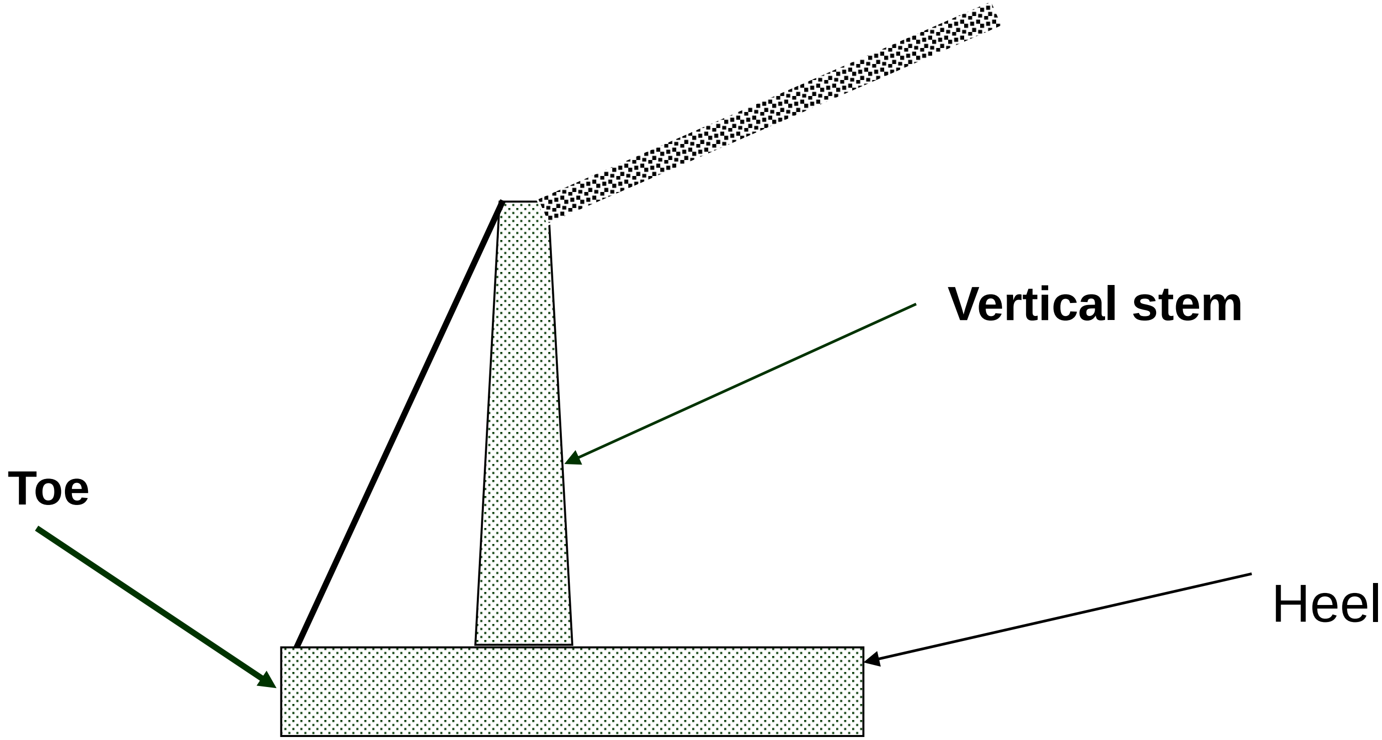
# 3. Counterfort walls:

- made of reinforced concrete materials
- consists of cantilever wall with vertical brackets known as counterfort placed behind face of wall
- ordinarily used for walls height greater than 6.0m



# 4. Buttress walls

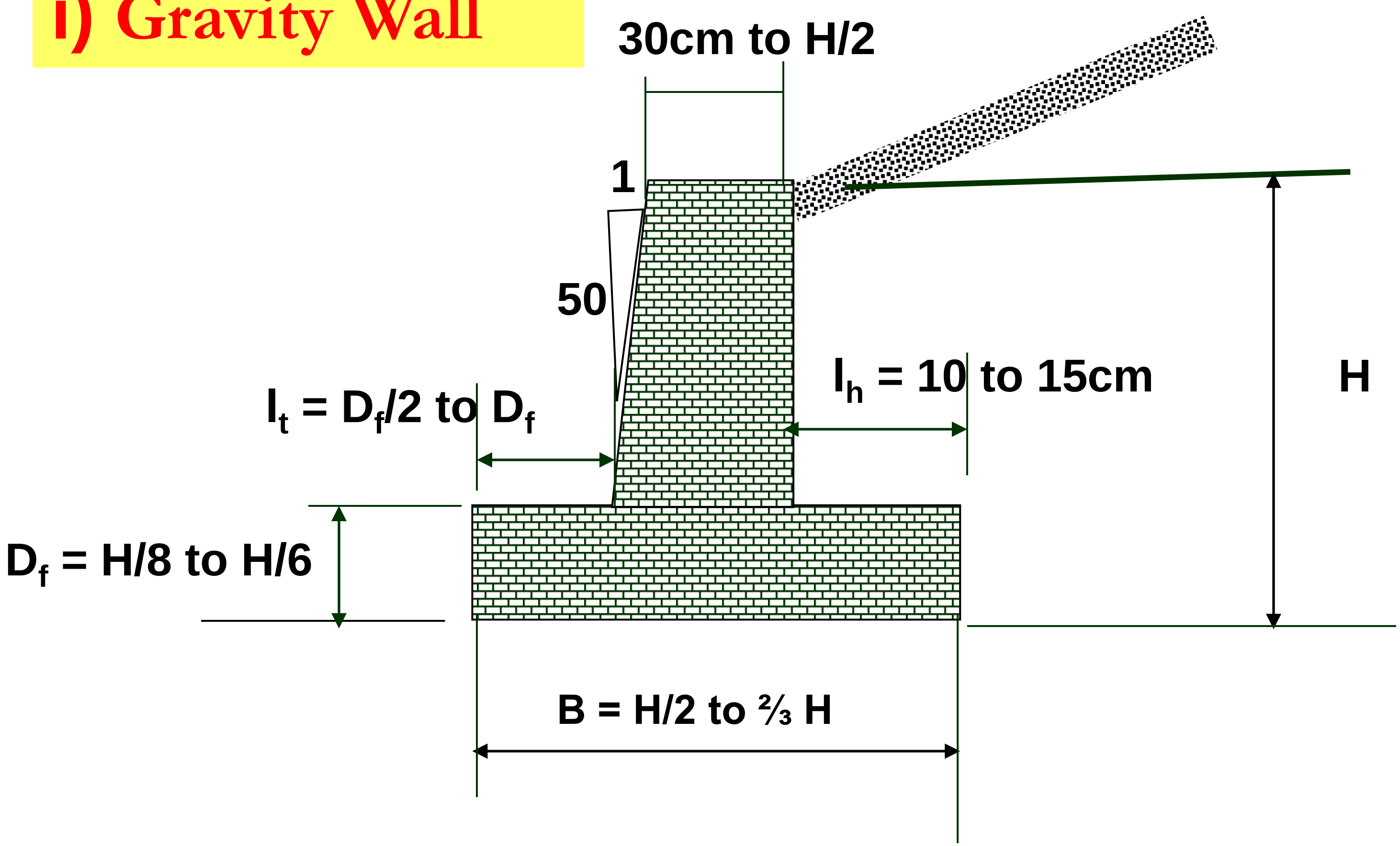
- same as counterfort except that the vertical brackets are on the opposite side of the backfill



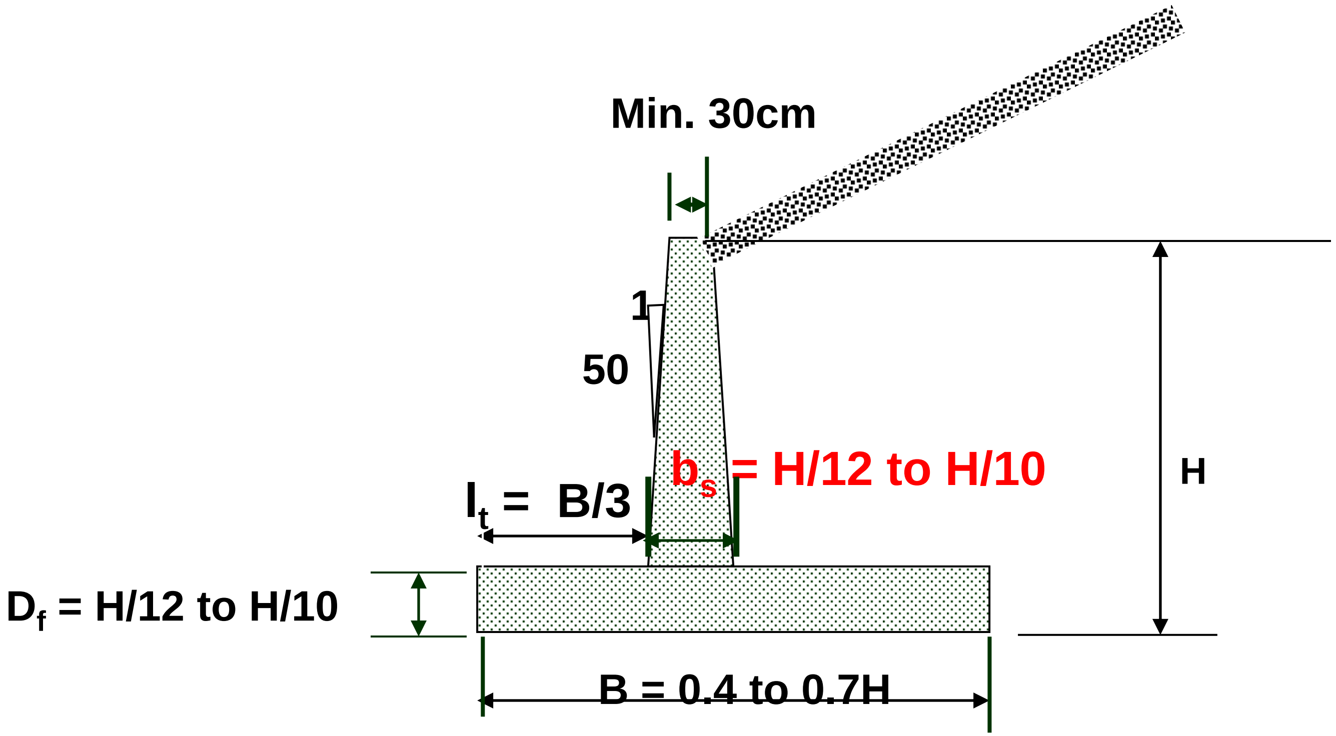
# Common Proportions of Retaining walls

- The usual practice in the design of retaining walls is to assign tentative dimensions and then check for the overall stability of the structure.

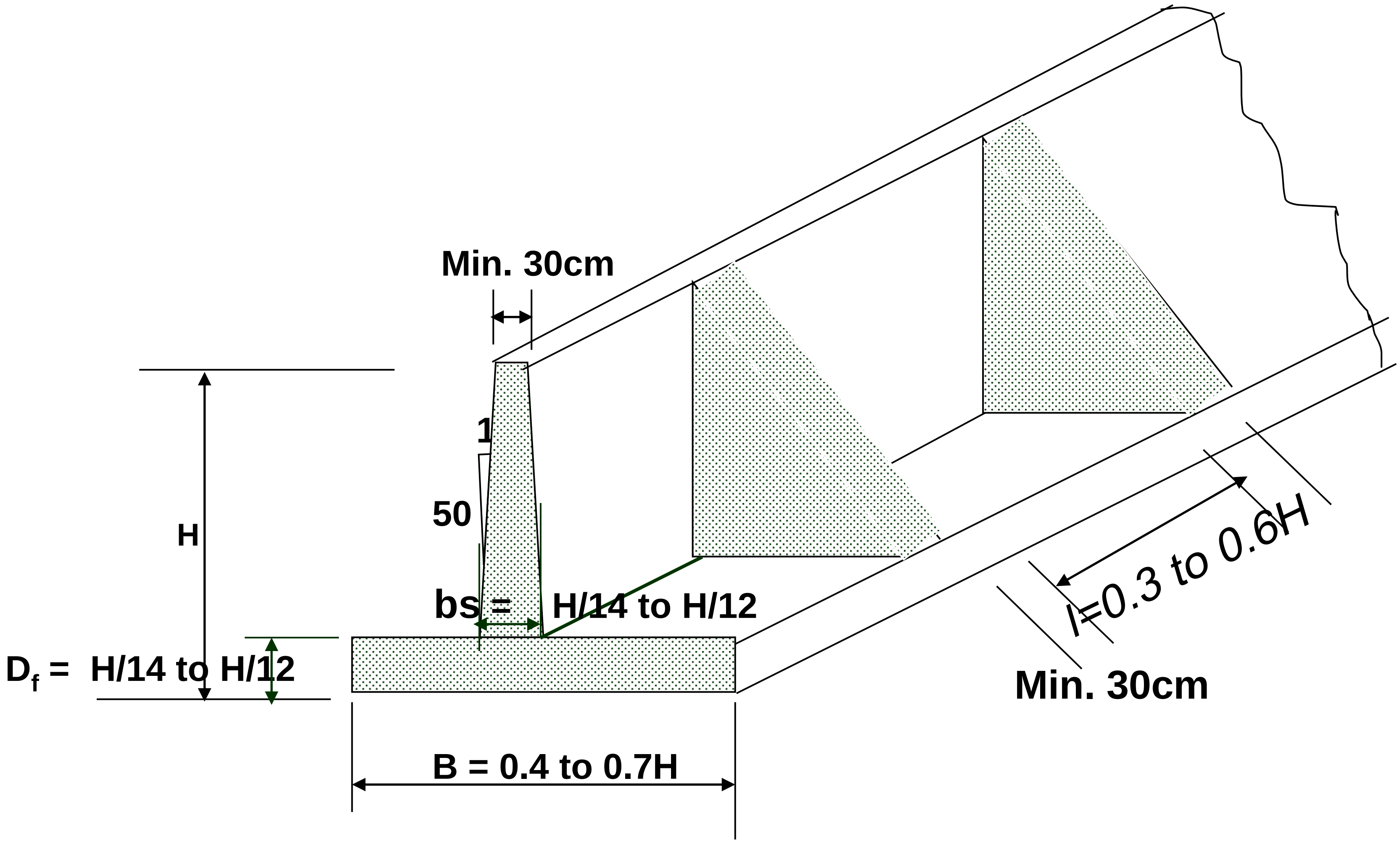
## i) Gravity Wall



# ii) Cantilever wall



# iii) Counterfort wall



# Forces on Retaining Walls

- The forces that should be considered in the design of retaining walls include
  - Active and passive earth pressures
  - Dead weight including the weight of the wall and portion of soil mass that is considered to act on the retaining structure
  - Surcharge including live loads, if any
  - Water pressure, if any
  - Contact pressure under the base of the structure

# Stability of Retaining Walls

- Retaining walls should be designed to provide adequate stability against sliding, overturning, foundation bearing failure and overall or deep foundation failure.

## 1.Sliding stability

**Factor of safety =** $$\frac{Horizontal\ resisting\ force}{Horizontal\ sliding\ force} = \frac{F_R}{P_{Ah}}$$

$$\text{Factor of safety} \geq 1.5 \text{ for granular soils}$$
$$\text{Factor of safety} \geq 2.0 \text{ for cohesive soils}$$

## 2. Overturning Stability

$$\text{Factor of safety} = \frac{\textit{Sum of moments to resist overturning}}{\textit{Sum of overturning moments}} = \frac{M_r}{M_o}$$

**Factor of safety ≥ 1.5 for granular backfill**
**Factor of safety ≥ 2.0 for cohesive backfill**

**If the line of action of the resultant force on wall acts within the middle third width of the base, wall is safe against overturning .**

## 3. Foundation stability

$$q_t = \frac{Rv}{B}\left(1 \pm \frac{6e}{B}\right)$$

Where, e= eccentricity of $R_v$

$q_t \leq q_{all}$, $q_{all} = q_{ult}/F.S$

F. S = Factor of safety = 2 and 3 for granular and cohesive soils, respectively.

# Deep foundation failure ( Overall stability)

- If layer of weak soil is located within a depth of about 1 ½ times the height of the retaining wall the overall stability of retaining wall should be investigated.
- E.g. using Swedish circle method.