KARPAGAM ACADEMY OF HIGHER EDUCATION
(Deemed to be University Under section 3 of UGC act 1956)
COIMBATORE-641021
FACULTY OF ENGINEERING
DEPARMENT OF CIVIL ENGINEERING

Semester : V– (2017-2018)

15BECE502 FOUNDATION ENGINEERING 3 0 0 3 100

OBJECTIVE:

- To explain the analysis of sheet pile wall under different support conditions
- To explain overall stability analysis of well foundation
- To explain fundamentals of soil dynamics and its application to machine foundation analysis including codel provisions
- To explain problems related to expansive soils and solution to overcome
- To explain the concept of slope stability analysis for various slope conditions including graphical methods

UNIT I **9**

Site Investigation And Selection Of Foundation: Scope and Objectives – Methods of exploration - Borings for Exploration – Wash boring and rotatory drilling – Depth of boring - Sampling – Representative and undisturbed sampling – sampling techniques – Split spoon sampler, Thin tube sampler, Stationary piston sampler – Penetration tests (SPT and SCPT) – Core cutter method, its significances and applications- Selection of foundation based on soil condition.

UNIT II **9**

Shallow Foundation: Introduction – Location and depth of foundation –– bearing capacity of shallow foundation on homogeneous deposits – Terzaghi's formula and BIS formula – factors affecting bearing capacity – problems - Bearing Capacity from insitu tests (SPT, SCPT and plate load) –Settlement – Components of settlement – Methods of minimizing settlement, differential settlement - subsoil stabilization - codal provisions .

UNIT III **9**

Footings and Rafts: Types of foundation – Raft foundation - Deep foundations – Dewatering system –- Contact pressure distribution below footings & raft - Isolated and combined footings – Types – proportioning - Mat foundation – Types – use - proportioning – Floating foundation.

UNIT IV **9**

Piles: Types of piles and their function – Factors influencing the selection of pile – Load Carrying capacity of single pile in granular and cohesive soil - Static formula - dynamic formulae (Engineering News and Hiley's) – Negative skin friction –- Settlement of pile groups – Under reamed piles .

UNIT V

Retaining Walls: Plastic equilibrium in soils –Types of Retaining Wall – Active and Passive states – Rankine's theory – cohesionless and cohesive soil –Coloumb's wedge theory – Graphical methods (Rebhann and Culmann) - Pressure on the wall due to line load – Stability of Retaining walls.Introduction to Geo textiles – applications.

TOTAL HRS:45

TEXT BOOKS

Sl.No	Title of Book	Author of Book	Publisher	Year of Publishing
1	Soil Mechanics and Foundations	Punmia B.C	Laximi Publications Pvt. Ltd., New Delhi	2012

REFERENCES

Sl.No	Title of Book	Author of Book	Publisher	Year of Publishing
1	Basic and Applied Soil Mechanics	GopalRanjan and Rao A.S.R.	Wile Eastern Ltd., New Delhi, India	2012
2	Analysis and Design of Structures – Limit state Design	Swami saran	Oxford IBH Publishing Co- Pvt. Ltd., New Delhi	2012
3	Foundation Engineering Standard	Varghese P C	Publishers Distributors New Delhi	2005
4	Soil Mechanics and Foundations Engineering	Arora K.R	Published by A.K Jain, New Delhi	2012

WEBSITES:

- http://www.icivilengineer.com
- http://www.engineeringcivil.com/
- http://www.aboutcivil.com/
- http://www.engineersdaily.com

COURSE OUTCOMES

On completion of the course, the students will be able to:

- Analyse and design any kind of sheet pile wall system including coffer dam.
- Analyse and design well foundation including complete stability analysis.
- Estimate soil parameters under dynamic conditions including machine foundations.
- Design a suitable foundation system for any kind of problematic soils.
- Analyse the stability of any kind of slope by using both theoretical and graphical methods.

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15BECE502 / FOUNDATION ENGINEERING
LECTURE PLAN

Number of credits : 3
Contact hours : 4 hours per week
Lecturer : Ms.A.Nisha Devi
Semester : V– (2017-2018)
Course Type : Core

Lecture	Hours	Topics to be Covered	Text / Reference	Page No	Remarks
Unit-I (SITE INVESTIGATION AND SELECTION OF FOUNDATION)					
1	1	Introduction to foundation engineering,	**T1,T2,R1**		
2	1	Scope and objectives, methods of exploration, borings for exploration	**T1,R1**		
3	1	Wash boring and rotatory drilling, depth of boring	**T1,R1**		
4	1	Sampling, representative and undisturbed sampling	**T1,R1**		
5	1	Sampling techniques, split spoon sampler, thin tube sampler	**T1,R1**		
6	1	Stationary piston sampler	**T1,R1**		
7	1	Penetration tests (SPT and SCPT)	**T1,R1**		
8	1	Core cutter method, its significances and applications	**T1,R1**		
9	1	Selection of foundation based on soil condition	**T1,R1**		
Total	**9 Hrs**				
UNIT-II (SHALLOW FOUNDATION)					
10	1	Introduction to shallow foundation, Location and depth of foundation	**T1,R1**		
11	1	Bearing capacity of shallow foundation on homogeneous deposits	**T1,R1**		
12	1	Terzaghi’s formula and BIS formula	**T1,R1**		
13	1	factors affecting bearing capacity	**T1,R1**		
14	1	Bearing Capacity from insitu tests (SPT, SCPT and plate load)	**T1,R1**		
15	1	Bearing Capacity – Example problems	**T1,R1**		
16	1	Settlement, Components of settlement	**T1,R1**		
17	1	Methods of minimizing settlement, differential settlement	**T1,R1**		
18	1	Subsoil stabilization - codal provisions	**T1,R1**		
Total	**9 Hrs**				

UNIT-III (FOOTINGS AND RAFT)					
19	1	Types of foundation	**T1,R1**		
20	1	Raft foundation	**T1,R1**		
21	1	Deep foundations	**T1,R1**		
22	1	Dewatering system	**T1,R1**		
23	1	Contact pressure distribution below footings & raft	**T1,R1**		
24	1	Isolated and combined footings, Types and Proportioning	**T1,R1**		
25	1	Mat foundation	**T1,R1**		
26	1	Types of Mat foundation, Uses - Proportioning	**T1,R1**		
27	1	Floating foundation	**T1,R1**		
Total	**9 Hrs**				
UNIT-IV (PILES)					
28	1	Types of piles and their function	**T1,R1**		
29	1	Factors influencing the selection of pile	**T1,R1**		
30	1	Load Carrying capacity of single pile in granular soil	**T1,R1**		
31	1	Load Carrying capacity of single pile in cohesive soil	**T1,R1**		
32	1	Static formula	**T1,R1**		
33	1	dynamic formulae (Engineering News and Hiley's	**T1,R1**		
34	1	Negative skin friction	**T1,R1**		
35	1	Settlement of pile groups	**T1,R1**		
36	1	Under reamed piles	**T1,R1**		
Total	**9 Hrs**				
UNIT-V (RETAINING WALLS)					
37	1	Plastic equilibrium in soils	**T1,R2**		
38	1	Types of Retaining Wall	**T1,R2**		
39	1	Active and Passive states	**T1,R2**		
40	1	Rankine's theory – cohesionless and cohesive soil	**T1,R2**		
41	1	Coloumb's wedge theory	**T1,R2**		
42	1	Graphical methods (Rebhann and Culmann)			
43	1	Pressure on the wall due to line load	**T1,R2**		
44	1	Stability of Retaining walls	**T1 ,R2**		
45	1	Introduction to Geo textiles and applications	**T1,R2**		
Total	**9 Hrs**				
46	1	Discussion on previous year ESE			

SUPPORTING MATERIALS

Sl.No	Title of Book	Author of Book	Publisher	Year of Publishing
1	Soil Mechanics and Foundations	Punmia B.C	Laximi Publications Pvt. Ltd., New Delhi	2012
2	Basic and Applied Soil Mechanics	Gopal Ranjan and Rao A.S.R.	Wile Eastern Ltd., New Delhi, India	2012
3	Analysis and Design of Structures – Limit state Design	Swami saran	Oxford IBH Publishing Co-Pvt. Ltd., New Delhi	2012
4	Foundation Engineering Standard	Varghese P C	Publishers Distributors New Delhi	2005
5	Soil Mechanics and Foundations Engineering	Arora K.R	Published by A.K Jain, New Delhi	2012

STAFF INCHARGE

(**Ms.A.Nisha Devi**)

HOD (Department of Civil Engineering)

DEAN (FOE)

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LECTURE PLAN

Number of credits	**: 3**
Contact hours	**: 4 hours per week**
Lecturer	**: Ms.A. Nisha Devi**
Semester	**: V– (2017-2018)**
Course Type	**: Core**

UNIT I

SITE INVESTIGATION AND SELECTION OF FOUNDATION

PART A

1.Explain Representative and Non-Representative Samples. (May/June 2009)

(April/May 2015) (Nov/Dec 2011)

Representative samples – Natural moisture content and the proportion of the mineral constituents are preserved even though the structure is disturbed. Non-Representative samples – In addition to alteration in the original soil structure, soils from the other layers gets mixed up or the mineral constituents gets altered.

2.What is mean by dilatancy? (Nov/Dec 2015)

Silty fine sands and fine sands below the water table develop pore pressure which is not easily dissipated. The pore pressure increases the resistance of the soil and hence the Penetration number (N). Terzaghi and peck recommend the following correction when the observed N value exceeds 15. The corrected Penetration Number,

$N_c = 15 + ½ [N_R – 15]$

Where, N_c – corrected value

N_R – Recorded Valu

If $N_R \leq 15$, then

$N_c = N_R$

3.Write the uses of bore log report.

(Nov/Dec 2012) (Nov/Dec 2015)

1. Used to record the change of layers depth
2. Used to record the water level
3. Used to record the water quality in deeper level

4.What is the objective of site investigation? (May/June 2013)

The objective of site investigation is to provide reliable, specific and detailed information about the soil and ground water conditions of the site, economic design and execution of the engineering works.

5.What is site reconnaissance? (May/June 2013) (May/June 2011)

Site reconnaissance is defined as the inspection of the site and study of topographical features, the soil and ground water conditions and in deciding the future programme of exploration.

6.What is significant depth? (Nov/Dec 2009) (Nov/Dec 2014)

Exploration in general should be carried out to a depth up to which the increase in pressure due to structural loading is likely to cause perceptible settlement or shear failure. Such a depth is known as significant depth.

7.How is the depth of exploration decided? (Nov/Dec2010)(April/May 2015)

(May/June 2014)

The depth of exploration required, depends on the type of proposed structure, its total eight, the size, shape and disposition of the loaded areas, soil profile and the physical properties of the soil that constitutes each individual stratum.

8.List the field tests used in subsurface investigations. (Nov/Dec 2013)

1. Standard Penetration Test.
2. Static Cone Penetration test
3. Dynamic Cone Penetration test

The outside clearance will help in reducing the friction wh le the sampler is being driven and when it is being withdrawn after the collection of the sample.

9.What is detailed Exploration? (Nov/Dec 2009), (Nov/Dec 2012)

A detailed Exploration is meant to furnish information about soil properties such as Shear strength, Compressibility, Density index and Permeability. Detailed Exploration is followed by the general exploration in case of large engineering works, heavy loads, and complex and costly foundations are involved.

10.What are the factors affecting quality of a sample? (Nov/Dec 2010)

The following are the factors affecting quality of the sample.

1. Cutting edge
2. Inside clearance
3. Outside clearance
4. Inside wall friction
5. Non-return valves

11.What are the various methods of site investigation? (Nov/Dec 2010)

1. Open Excavation
2. Borings
3. Sub- Surface soundings
4. Geophysical method

12.The internal diameter of a sampler is40mm and the external diameter is 42mm. Will you consider the sample obtained from the sampler as disturbed or undisturbed? (April/May 2011) Given data: D1 = 40mm,

D2 = 42mm

Area Ratio , Ar= Do2 – Di2 x 100%

Di2

= 422 -402 x 100%

402

=10.25% < 15%

The sample is undisturbed one

13. How to Select the foundation based on soil condition? (Nov/Dec 2015)

i. Adequate depth

ii. Bearing capacity

iii. Settlement

iv. Quantity

v. Adequate strength

vi. Adverse soil change

vii. Seismic forces

14.What are the guidelines in terms of inside and outside clearance for obtaining undisturbed sample? (May/June 2012)

An undisturbed sample is that in which the natural structure and properties remain preserved. The inside clearance should lie between 1 to 3% and the outside clearance 0 to 2%. The walls of the sampler should be smooth and should be kept properly oiled.

15.What is meant by inside and outside clearance? What is its use?

Inside clearance (Ci) ,

Ci = D3 – D1 x 100%

D1

D1 - Inner diameter of cutting edge D3 - Inner diameter of sample tube

Outside clearance (Co)

Co = D2 – D4 x 100%

D4

D4 - Outer diameter of cutting edge D2 - Outer diameter of sample tube

Outside clearance should not be much greater than inside clearance. It maybe small (or) 1-2%

(Nov/Dec 2013)

Uses:

The inside clearance is given to reduce the friction between the tube, by allowing for the elastic expansion of the soil.

Uses:

The outside clearance will help in reducing the friction while the sampler is being driven and when it is being withdrawn after the collection of the sample

PART-B

1. Describe the various methods of drilling bore holes for sub surface

investigations. (April/May 2011) (Nov/Dec 2014) (May/June 2016)

- When the depth of exploration is large, borings are used for exploration.

- A vertical bore hole is drilled in the ground to get the information about the subsoil strata samples are taken from the bore hole and tested in the laboratory.

- The bore hole may be used for conducting in-situ tests and for locating the water table.

Depending upon the type of soil and the purpose of boring, the following methods are used for drilling the holds.

1. Auger Boring
2. Auger and shell boring
3. Wash Boring
4. Rotary Drilling
5. Percussion drilling
6. Core Boring

1. Auger Boring:

1. Augers are used in cohesive and other soft soils above water table.
2. Hands augers are used for depth up to 6m.
3. Mechanically operated augers are used for greater depths and they can also be used in gravelly soils.
4. Samples recovered from the soil brought up by augers are badly disturbed nature of soil sample; it becomes difficult to locate the exact changes in the soil strata.

5. It can be operated manually or mechanically. Mechanical augers are driven by power. These are used for making holes in hard strata to a great depth.

Even mechanical ugers become inconvenient for depth greater than 12m and other methods of boring are used.

6. The hand augers used in boring are about 15to 20cm in diameter. It is attached to the lower end of the pipe of about 18mm diameter.

7. When the auger is filled with soil, it is taken out. If the hole is already driven, another type of auger known as post hole auger is used for taking soil samples.

Limitation or Disadvantages:

x Sandy soil below water table, a casing is normally required. For such soils, the method of auger boring becomes slow and expensive.

x It cannot be used when there are large cobbles, boulders or other obstructions which prevent drilling of the hole.

x Auger boring is fairly satisfactory for highways, railways, airfield exploration at shallow depth. The sub-surface explorations are done quite rapidly and economically by auger boring.

2. Augers and shell Boring:

1. Cylindrical augers and shell with cutting edge on teeth at the lower end can be used for making deep borings.

2. Hand operated rings are used for depth up to 2.m and the mechanical ring up to 50m.

3. This Augers are suitable for soft to stiff clays, shells for very stiff and hard clays and shells or sand pumps for sandy soils.

4. Small boulders, thin soft strata or rock or cemented gravel can be broken by chisel bits attached to drill rods. The hole usually requires a casing.

3. Wash Boring:

1. In wash boring, the hole is drilled by first driving a casing about 2 to 3m long and then inserted into a hollow drill rod with a chisel shaped chopping bit at its lo er end. Water is pumped down the hollow drill rod, which is known as wash pipe.

2. Water emerges as a strong jet through a small opening of the chopping bit. The hole is advanced by a combination of chopping action and the jetting action as the drilling bit and the accompanying water jet disintegrates the soil.

3. The water and chopped soil particles rise upward through the annular space between the drill rod and the casing. The return water also known as wash water which is collected in a tub through a T-shaped pipe fixed at the top of the casing.

4. The hole is further advanced by alternately raising and dropping the chopping by a winch. The swivel joint provided at the top of the drill rod facilitates the

turning and twisting of the rod. The process is conti ued v n below the costing till the hole begins to cave in. At that stage the bottom of the casing can be extended by providing additional pieces at the top.

5. However in stable, cohesive soils the casing is required only in the top portion. Sometimes instead of casing, special drilling fluids made of suspensio or emulsion of fat clays or bentonite combined with some special additives are used for supporting walls of the hole.

6. The change in strata is provided by the reaction of the chopping bit as the hole is advanced. It is also indicated by a change in color of the wash water. The wash boring is mainly used for advancing a hole in the ground. Once the hole has been drilled, a sampler is inserted to obtain soil samples for testing in the laboratory.

Limitation or Disadvantages:

x The equipment used in

ash boring is relatively light and inexpensive. The main disadvantage of the method is that it is slow in stiff soils and coarse grained soils. It c nnot be

used efficiently in hard oils, rocks and the soil containing boulders.

x The method is not suitable for taki g good quality undisturbed samples above

ground water table, as the wash water enters the strata below the bottom of the hole and causes an increase i its water co tent.

4. Percussion Drilling:

1. The percussion drilling method is used for making holes n rocks, boulders and other hard strata.

2. The main advantage of the percussion drilling method is that it can be used for all types of materials. It is particularly useful for drilling holes in is glacial tills containing boulders.

3. In this method a heavy chisel is alternately lifted and dropped in a vertical hole. The material gets pulverized. If the point where chisel strikes is above the water table, water is added to the hole. The water forms slurry with the pulverized material which is removed by a sand pipe.

4. Percussion drilling may require a casing. It is also used for drilling tube wells.

Limitation or Disadvantages:

x One of the major disadvantages is that the material at the bottom of the hole is disturbed by heavy blows of the chisel.

x It is not possible to get good quality undisturbed samples. This method is generally more expensive.

5. Rotary Drilling:

1. Rotary boring or drilling is a very fast method of advancing hole in the both rocks and soils.

2. Rotary drilling can be used in clay, sand and rocks.

3. Bore holes of diameter 50mm to 200mm can be easily made by this method.

4. A drill bit, fixed to the lower end of the drill rods, is rotated by a suitable chunk and is always kept in firm contact with the bottom of the hole.

5. A drilling mud, usually a water solution of bentonite with or without other admixtures is continuously forced down the hollow drill rods.

6. The mud entering upwards brings the cuttings to the surface. This method is also known as 'MUD ROTARY DRILLING 'and the hole usually requires no casing.

6. Core Drilling:

1. The core drilling method is used for drilling holes and for obtaining rock cores.

2. In this method a core barrel fitted with a drilling bit is fixed to a hollow drilling rod. As the drilling rod is rotated, the bit advances and cuts an annular hole an intact hole

3. The core is then removed from its bottom and is retained by a core –lifter and brought to the ground urface.

4. The core drilling may be done using either a diamond studded bit or cutting edge consists of chilled shot. The diamond driller is superior to the other type of drilling, but it is costlier

5. Water is pumped continuously into the drill ng rod to keep the drilling bit cool and to carry the disintegrated materials to the ground surface.

2.Explain the (i) Seismic refraction method and (ii) Electrical resist vity method of soil exploration. (May/June 2009), (Nov/Dec 2015)

(OR)

Explain in detail the geophysical methods of soil explorations with neat sketch.

(Nov/Dec 2012),(Nov/Dec 2013), (May /June 2013)

(i) SESMIC REFRACTION METHOD

x This method is based on the fact that seismic waves have different velocities in different types of soils and besides the wave refract when they cross boundaries between different types of soils.

x In this method an artificial impulse are produced either by detonation of explosive or mechanical blow with a heavy hammer at ground surface or at the shadow depth within a hole.

These shocks generate three types of waves:

x Longitudinal or compressive wave or primary (p) wave

x Transverse or shear waves or secondary (s) waves

x Surface waves

It is primarily the velocity of longitudinal or the compression waves which is utilized in this method. The equation on the p-waves (Vc) and s-waves (Vs) is given as

These aves are classified as direct, reflected and refracted waves.

x The direct waves travel in approximately straight line from the source of impulse.

x The reflected and refracted wave undergoes a change in direction when they encounter a boundary eparating media of different seismic velocities.

x This method is more suited to the shallow explorations for civil engineering purpose.

x The time required for the impulse to travel from the shot point to various points on the ground surface is determined by m ans of geophones which transform the vibrations into electrical curre ts and t ansmit them to a recording unit or oscillograph, with a timing mechanism.

Assumptions

The various assumptions involved are

x All the soil layers are horizontal

x The layers are sufficiently thick to produce a response

x Each layer is homogeneous and isotropic

x Velocity should increase with depth following the Snell's law as given

i1 is the angle of incidence

Velocity in two different mediums

Procedure

x The detectors are generally placed at varying distance from the shot point but along the straight line.

x The arrival time of the first impulse at each geophone is utilized.

x If the successfully deeper strata transmit the waves with increasingly greater velocities the path travelled by the first impulse will be similar to those.

x Those recorded by the nearest recorders pass entirely through the overburden, whereas those first reaching the after detectors travel downward through the lower velocity material, horizontally within the higher velocity stratum and return to the surface.

x (A T1 and A T2) as the function of the distances between the geophones and the shot points (L1 and L2).

x A curve obtained which indicates the wave velocity in each stratum and which may be used to determine the depths to the boundaries between the strat .

Applications

x Depth and characterization of the bed rock surfaces.

x Buried channel location.

x Depth of the water table.

x Depth and continuity of stratigraphy interfaces.

x Mapping of faults and other structural features.

Advantages

x Complete picture of stratification of layer up to 10 m depth.

x Refraction observations generally employ fewer source and receiver location and thus relatively cheap to acquire.

x Little processing is done on refraction observations with the exception of trace scaling or filtering to help in the process of picking the arrival times of the initial ground motion.

x Because such a small portion of the recorded ground motion is used developing models and interpretations is no more difficult than our previous efforts with other geophysical surveys.

x Provides seismic velocity information for estimating material properties.

x Provides greater vertical resolution than electrical, magnetic or gravity methods.

x Data acquisition requires very limited intrusive activity is non- destructive.

Disadvantages

x Blind zone effect: If v2< v1, then wave refracts more towards normal then the

thickness of the strata is neglected.

x Error also introduced due to some dissipat o of the velocity as longer the path of travel, geophone receives the erroneous r adings.

x Error lies in all assumptions.

(ii) ELECTRICAL RESISTIVITY METHOD

Electrical resistivity method is based on the difference in the electrical conductivity or electrical resistivity of different soils. Resistivity is defined as the resistance in ohms between opposite phases of a unit cube of a material.

ρ resistivity in ohm-cm R is resistance in ohms

A is the cross sectional area (cm2)

L is the length of the conduction (cm)

x In this method the electrodes and driven approximately 20 cms in to the ground and a dc or a very low frequency ac current of known magnitude is passed between the outer electrodes thereby producing within the soil an electrical field and the boundary conditions.

x The electrical potential at point C is Vc and at the point D is Vd which is measured by means of the inner electrodes respectively.

Where ρ is resistivity

I is current

r1, r2, r3 and r4 are the distances between the various electrodes Potential difference between C

If r1=r4= (r2/2)= (r3/2)Then the resistivity is given as

x Thus the apparent resistivity of the soil to the depth approximately equal to the spacing r1 of the electrode can be computed.

x The resistivity unit is often so designed that the apparent resistivity can be read directly on the potentiometer.

x In resistivity mapping or transverse profiling the electrodes are moved from place to place without changing their spacing and the apparent resistivity and any anomalies within a depth a depth equal to the spacing of the electrodes can thereby be determined for a number of points.

x In resistivity sounding or depth profiling the center point of the set up is stationary whereas the spacing of the electrode is varied.

x A detailed evaluation of the results of the resistivity sounding is rather complicated, but preliminary indications of the subsurface conditions may be obtained by plotting the apparent resistivity as a function of electrode spacing.

x When the electrode spacing reaches a value equal to the depth to a deposit with a resistivity materially different from that of overlying strata, the resultant diagram will generally show a more or less pronounced break in the strata depth beyond A2 .

Where a is the spacing between the electrodes.

x The Schlumberger array is used for profiling and sounding. In sounding configuration the current electrodes separated by AB are symmetric about the potential electrodes MN.

x The current electrodes are then expanded and the resistivity is given as

Applications

x Characterize subsurface hydrogeology.

- Determine depth to bedrock /over burden thickness.
- Determine depth to ground water.
- Map stratigraphy.
- Map clay aquitards.
- Map salt water intrusion.
- Map vertical extent of certain types of soil and ground water contamination.

Resistivity profiling

- Map faults.
- Map lateral extent of conductive contaminant process.
- Locate voids.
- Map heavy metals soil contamination.
- Delineate disposal areas.
- Map paleochannels.
- Explore for sand nd gravels.
- Map archaeologic l sites.

Advantages of this method are

- It is very rapid and economical method.
- It is good up to 30 m depth.
- The instrumentation of this method is very s mple.
- It is a non destructive method.

Disadvantages of this method are

- It can only detect absolutely different strata like rock and water.

- It provides no information about the sample.
- Cultural problems cause interference.

x Data acquisition can be slow compared to other geophysical methods, although that difference is disappearing with the very latest techniques.

3. Briefly explain with neat sketch Standard Penetration Test and the

correction to be applied to find ‘N’ value. (May/June 2016), (May/June 2014), (Nov/Dec 2011), (May /June 2012), (Nov/Dec 2013)

Standard Penetration Test (SPT Test)

1. The standard Penetration Test is the most commonly used in –site test, especially for cohesion less soils which cannot be easily sampled.

2. The test is extremely useful for determining the relative density and the angle to determine the UCC strength of the cohesive soil.

3. The standard penetration test is conducted in a bore hole using a standard split spoon sampler, when the bore hole has been drilled to the desired depth, the drilling tools are removed and the sampler is lowered to the bottom of the hole.

4. The sampler is driven into the soil by a drop hammer of 63.5kg mass falling through a height of 750mm at the rate of 30blows per minutes.

5. The number of hammer blows required to drive 150mm of the sample is counted.

6. The sampler is further driven by 150mm and the number of blows recorded.

7. Likewise the sampler is once again further driven by 150mm and the number of blows recorded. The number of blows recorded for the first 150mm is disregarded.

8. The plumber of blows recorded for the last two 150mm intervals are added to give the standard Penetration Number (N).

9. In other words, ‘N’ i equal to the number of blows required for 300mm of penetration beyond a seating drive of 150mm.

10. If the number of blows for 150mm drive exceeds 50, it is taken as refusal and the test is discontinued. The standard Penetration number is corrected for decadency correction and our burden correction.

(a) Dilatancy Correction.

x Silty fine sands and fine sands below the water table develop pore pressure which is not easily dissipated.

x The pore pressure increases the resistance of the soil and hence the Penetration number

x Terzaghi and peck recommend the following correction when the observed N value exceeds 15. The corrected Penetration Number,

$N_c = 15 + ½ [N_R – 15]$

Where, N_c – corrected value

N_R – Recorded Value

If $N_R \leq 15$, then $N_c = N_R$

(b) Over burden Pressure Correction:

x In granular soils, the overburden pressure affects the penetration resistance.

x Generally, the soil with high confining pressure gives higher penetration number.

x As the confining pressure in cohesion soil increases with depth, the penetration number for the soils at shallow depths is under estimated and that at greater depths is over estimated for uniformity, the N values obtained from field tests under different effective overburden pressure are corrected to a standard effective overburden pressure.

For dry or moist clean sand, (Gibbs and Holtz)

$N_c = N_R$ X 350

$\overline{V}0+70$

N_c - corrected value

N_R - Recorded Value

$\overline{V}0$ - effective over burden pressure

It is applicable for $\overline{V} \leq$ 280kN/ m2. Usually the overburden correct on is applied first and then dilatancy correction is applied first and then dilatancy correction is applied.

The correction given by Bazara & peck is

N =4NR if $V\overline{0} <$ 71.8 kN/m2

1+ 0.0418$V\overline{0}$

N= 4NR if $V\overline{0} >$ 71.8 kN/m2

3.25+ 0.0104V0̅

N=NR if 2

V0̅= 71.8 kN/m

Correction of N with engineering properties:

x The value of standard Penetration number N depends upon the relative density of the cohesionless soil and the unconfined compressive strength of the cohesive soil.

x If the soil is compact or stiff, the penetration number is high.

x The angle of sheaing resistance (ϕ) of the cohesionless soil depends upon the number N.

x In general, greater the N-value greater the ϕ value.

x The consistency &UCC strength of cohesive soils can be approximately determined from SPT, N-value.

4.Explain in detail the various types of samplers with sketches. (May/June 2016) (May/June 2013) (May/June 2011)

OPEN DRIVER SAMPLER:

x Most commonly used for disturbed samples.

x Driving shoe made up of tool steel about 75mm long. Steel tube of 450mm long.

x The coupling head provided with check valve and 4 venting port of 10mm diameter.

x After borehole sampler attached to drilling rod and lowered into the hole.

x Drop hammer is used for forcing the sampler.

x When the sample is taken out removing the shoe and coupling transported to lab.

x Spring core catches is used for taking sand below ground water level and spring closes when lifted up and forms a dome.

x Water level slightly above the piezometric level at the bottom of the hole to avoid quick sand condition.

SPT - Soil Testing

SPLIT SPOON SAMPLER:

The most commonly used sampler for obtaining disturbed sample of soil is the standard split spoon sampler. It consists mainly of three parts

i) Driving shoe, made of tool steel, about 75mm long

ii) Steel tube abut 450mm long, split lon itud al in two halves and

iii) Coupling at the top of the tube about 150mm long.

x The inside diameter of the split tube is 38mm and the outs de diameter is 50mm.

x The coupling head may be provided with a check valve and 4 ventin ports of 10mm diameter to improve sample recovery.

x This sampler is also used in conducting standard penetration test.

x After the bore hole has been made, the sampler is attached to the drilling rod and lowered into the hole.

x The sample is collected by jacking or forcing the sampler into the soil by repeated blows of a drop hammer.

x The sampler is then withdrawn.

x The split tube is separated after removing the shoe and the coupling and the sampler is taken out. It is then paced in a container, sealed and transported to the laboratory.

x If the soil encountered in the bore hole is fine sand and it lies below the water table, the sample is recovery becomes difficult.

x For such soil, a spring core catcher device is used to aid recovery.

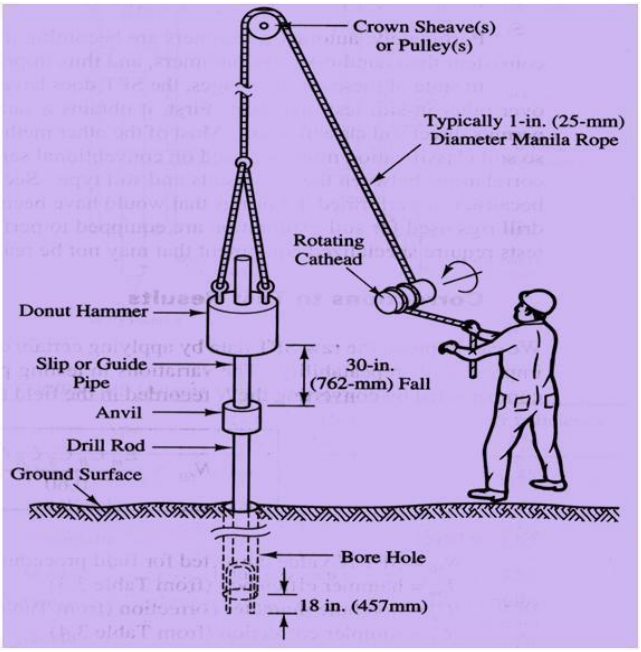
x As the sampler is lifted springs close and form a dome and retain the sample.

x While taking samples, care should be taken to ensure that the water level in the hole is maintained slightly higher than the piezometric level at the bottom of the hole.

x It is necessary to prevent quick sand conditions.

x The split tube may be provided with a thin metal or plastic tube liner to protect the sample and to hold it together.

x After the sample has been collected, the liner and the sample it contains are removed from the tube and ends are sealed.



Standard Penetration Test (SPT):

STATIONARY PISTON SAMPLER:

x Stationary piston s mpler consists of a sampler with a piston attached to a long piston rod extending up to the ground surface through the drill rods.

x The lower end of the sampler is kept closed with piston while the sampler is lowered through the bore hole.

x When the desired elevation reached, the piston rod is clamped; thereby keeping the piston stationary and the sampler tub is advanced further into the soil.

x The sampler is then lifted and the piston rod clamped in positio .

x The piston prevents the entry of water and soil into the tube, when it is bei g lowered and also helps to retain the sample during the process of lifting the tube.

x The sampler is therefore very much being suited for sampling in soft soils and saturated sands.

ROTARY SAMPLERS:

x Rotary samplers are core barrel type with an outer tube provided with cutting teeth and a removable thin liner inside. It is used for sampling in stiff cohesive soils.

SCRAPER BUCKET SAMPLER:

x If a sandy deposit contains pebbles it is not possible to obtain samples by standard split spoon sampler by standard split spoon sampler or split spoon fitted with a spring core catcher.

x The pebbles come in between the springs and prevent their closure.

x For such deposits, a scraper bucket sampler can be used.

x A scraper bucket sampler can also be used for obtaining the samples of cohesion less soils below the water table.

SHELBY TUBES AND THIN WALL D SAMPLERS:

x Shelby tubes are thin wall tube samplers made of seamless steel.

x The outside diameter of the tube may be between 40 to 125mm.

x The area ratio is less than 15% and the inside clearance s between 0.5 to 3%.

x The length of the tube is 5 to 10 times the diameter for sandy soils and 10 to 15 times the diameter for clayey soils.

x The diameter generally varies between 40 and 125mm and thickness varies from 1.25 to 3.15mm.

x The sampler tube is attached to the drilling rod and lowered to the bottom of the bore hole.

x It is then pushed into the soil.

x Care should be taken to push the tube into the soil by a continuous rapid motion without impact or twisting.

x The tube should be pushed to the length provided for the sample.

UNIT II

SHALLOW FOUNDATION

Objectives

In this section you will learn the following

- Introduction
- Basic definitions
- Presumptive bearing capacity

Bearing capacity : It is the load carrying capacity of the soil.

Basic definitions

Ultimate bearing capacity or Gross bearing capacity (q_u): It is the least gross pressure which will cause shear failure of the supporting soil immediately below the footing.

Net ultimate bearing capacity (q_{un}**):** It is the net pressure that can be applied to the footing by external loads that will just initiate failure in the underlying soil. It is equal to ultimate bearing capacity minus the stress due to the weight of the footing and any soil or surcharge directly above it. Assuming the density of the footing (concrete) and soil (γ) are close enough to be considered equal, then

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

where,

D_f is the depth of the footing, Ref. fig. 4.7

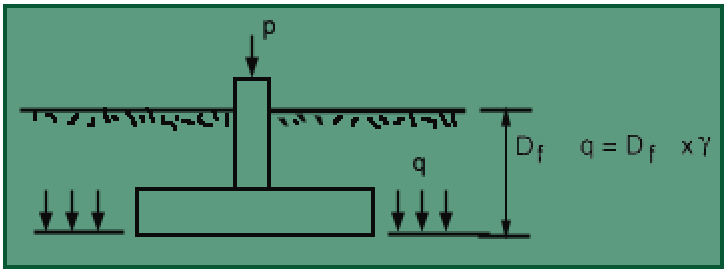
Safe bearing capacity: It is the bearing capacity after applying the factor of safety (FS). These are of two types,

- Safe net bearing capacity (q_{ns}) : It is the net soil pressure which can be safety applied to the soil considering only.

Safe gross bearing capacity (q_s): It is the maximum gross pressure which the soil can carry safely without shear failure. It is given by,

$$q_s = q_{ns} + \gamma D_f$$

Allowable Bearing Pressure: It is the maximum soil pressure without any shear failure or settlement failure.



Bearing capacity of footing

Presumptive bearing capacity : Building codes of various organizations in different countries gives the allowable bearing capacity that can be used for proportioning footings. These are "Presumptive bearing capacity values based on experience with other structures already built. As presumptive values are based only on visual classification of surface soils, they are not reliable. These values don't consider important factors affecting the bearing capacity such as the shape, width, depth of footing, location of water table, strength and compressibility of the soil. Generally these values are conservative and can be used for preliminary design or even for final design of small unimportant structure. IS1904-1978 recommends that the safe bearing capacity should be calculated on the basis of the soil test data. But, in absence of such data, the values of safe bearing capacity can be taken equal to the presumptive bearing capacity values given in table 4.1, for different types of soils and rocks. It is further recommended that for non-cohesive soils, the values should be reduced by 50% if the water table is above or near base of footing.

Presumptive bearing capacity values as per IS1904-1978.

Type of soil/rock	**Safe/allowable bearing capacity (KN/ m^2)**
Rock	3240
Soft rock	440
Coarse sand	440
Medium sand	245
Fine sand	440
Soft shell / stiff clay	100
Soft clay	100
Very soft caly	50

Presumptive analysis

This is based on experiments and experiences.

For different types of soils, IS1904 (1978) has recommends the following bearing capacity values.

Bearing Capacity Based on Presumptive Analysis

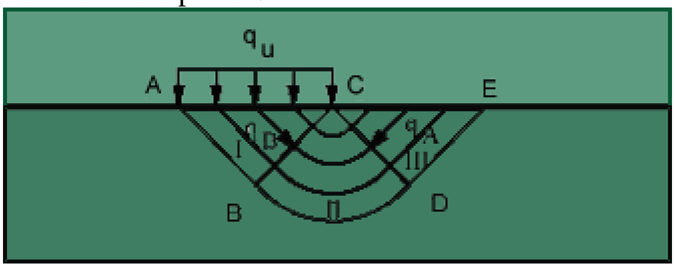
Types	Safe /allowable bearing capacity(kN/m^2)
Rocks	3240
Soft rocks	440
Coarse sand	440
Medium sand	245
Fine sand	100
Soft shale/stiff clay	440
Soft clay	100
Very soft clay	50

Analytical methods

- The different analytical approaches developed by various investigators are briefly discussed in this section.

Prandtl's Analysis

Prandtl (1920) has shown that if the continuous smooth footing rests on the surface of a weightless soil possessing cohesion and friction, the loaded soil fails as shown in figure by plastic flow along the composite surface. The analysis is based on the assumption that a strip footing placed on the ground surface sinks vertically downwards into the soil at failure like a punch.



Prandtl's Analysis

Prandtl analysed the problem of the penetration of a punch into a weightless material. The punch was assumed rigid with a frictionless base. Three failure zones were considered.

- Zone I is an
- active failure

zone Zone II is

a radial shear

zone

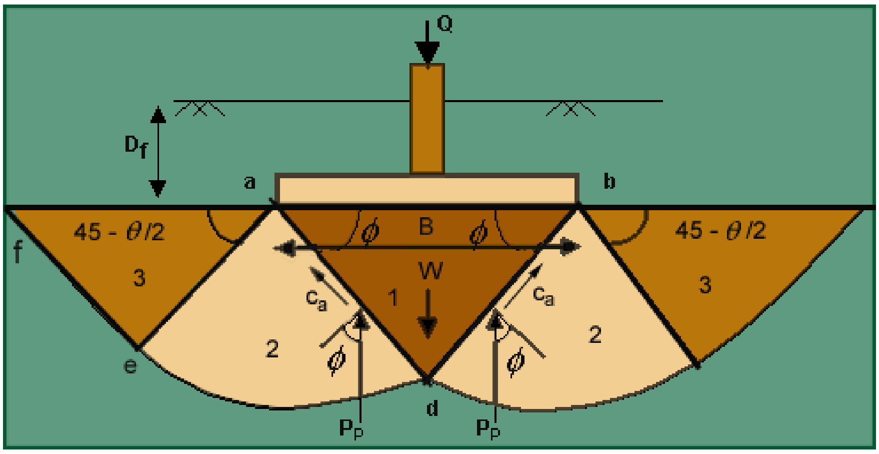
- Zone III is a passive failure zone identical for $\phi = 0$

Zone1 consist of a triangular zone and its boundaries rise at an angle $45 + \phi/2$ with the horizontal two zones on either side represent passive Rankine zones. The boundaries of the passive Rankine zone rise at angle of $45 - \phi/2$ with the horizontal. Zones 2 located between 1 and 3 are the radial shear zones. The bearing capacity is given by (Prandtl 1921) as

$q_d = cN_c$

where c is the cohesion and is the bearing N_c capacity factor given by the expression

$$N_c = \cot\phi [e^{\pi \tan\phi} \tan^2[(45 + \phi/2) - 1]$$



Terzaghi's Bearing Capacity Theory

Terzaghi's bearing capacity factors

ϕ_f	N_c	N_q	N_γ		ϕ_f	N_c	N_q	N_γ
28	17.81	31.61	15.7		0	1.00	5.70	0.0
30	22.46	37.16	19.7		2	1.22	6.30	0.2
32	28.52	44.04	27.9		4	1.49	6.97	0.4
34	36.50	52.64	36.0		6	1.81	7.73	0.6
35	41.44	57.75	42.4		8	2.21	8.60	0.9
36	47.16	63.53	52.0		10	2.69	9.60	1.2
38	61.55	77.50	80.0		12	3.29	10.76	1.7
40	81.27	95.66	100.4		14	4.02	12.11	2.3
42	108.75	119.67	180.0		16	4.92	13.68	3.0
44	147.74	151.95	257.0		18	6.04	15.52	3.9
45	173.29	172.29	297.5		20	7.44	17.69	4.9
46	204.19	196.22	420.0		22	9.19	20.27	5.8
48	207.85	258.29	780.1		24	11.40	23.36	7.8
50	415.15	347.51	1153.2		26	14.21	27.06	11.7

Bearing capacity of square and circular footings

If the soil support of a continuous footing yields due to the imposed loads on the footings, all the soil particles move parallel to the plane which is perpendicular to the centre line of the footing. Therefore the problem of computing the bearing capacity of such footing is a plane strain deformation problem. On the other hand if the soil support of the square and circular footing yields, the soil particles move in radial and not in parallel planes. Terzaghi has proposed certain shape factors to take care of the effect of the shape on the bearing capacity. The equation can be written as,

$$q_u = nN_c S_c + \bar{\gamma} D N_q s_q + \frac{1}{2} \gamma B N \gamma s_\gamma$$

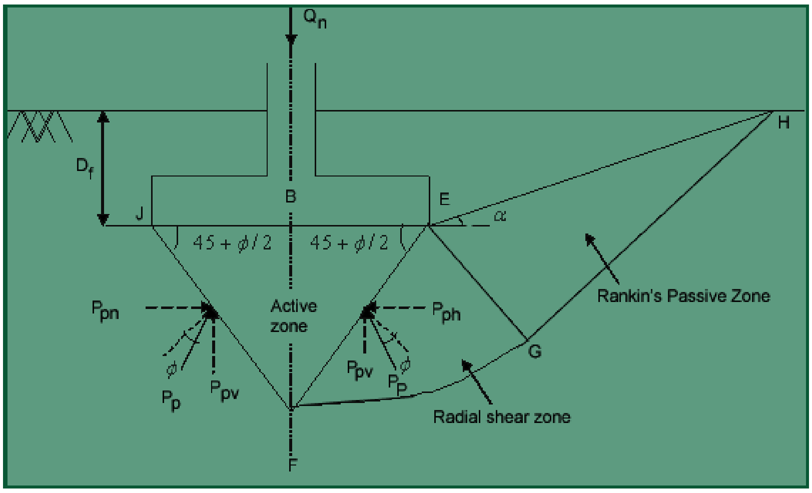
where,

s_q s_c s_γ are the shape factors whose values for the square and circular footings are as follows,

For long footings: $s_c = 1$, $s_q = 1$, $s_\gamma = 1$,

For square footings: $s_c = 1.3$, $s_q = 1$, $s_\gamma = 0.8$,

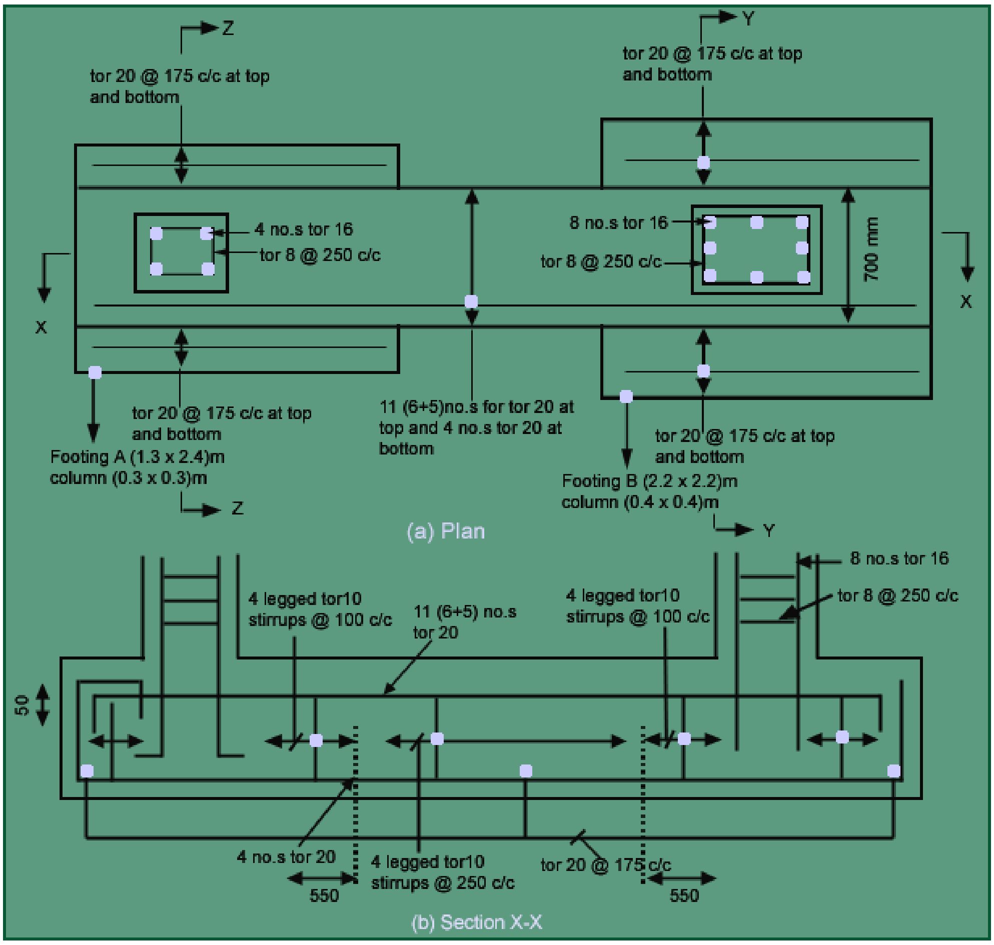
For circular footings: $s_c = 1.3$, $s_q = 1$, $s_\gamma = 0.6$.



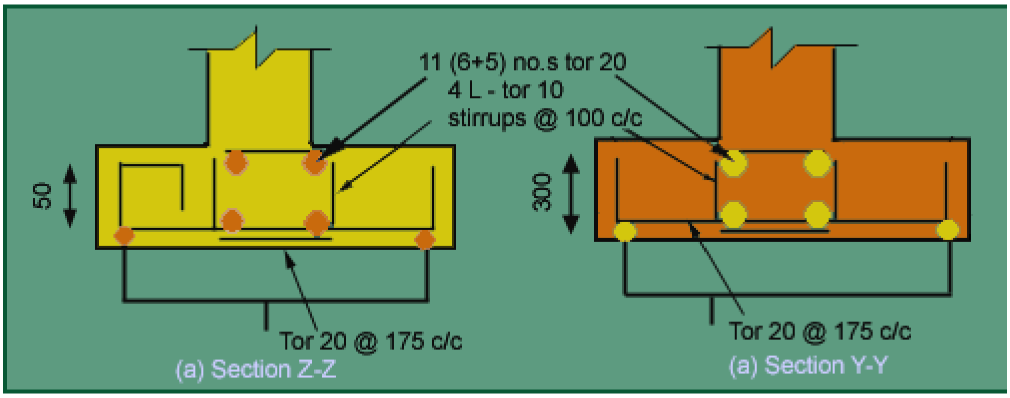
Failure zones considered by Meyerhof

UNIT III

FOOTINGS AND RAFT

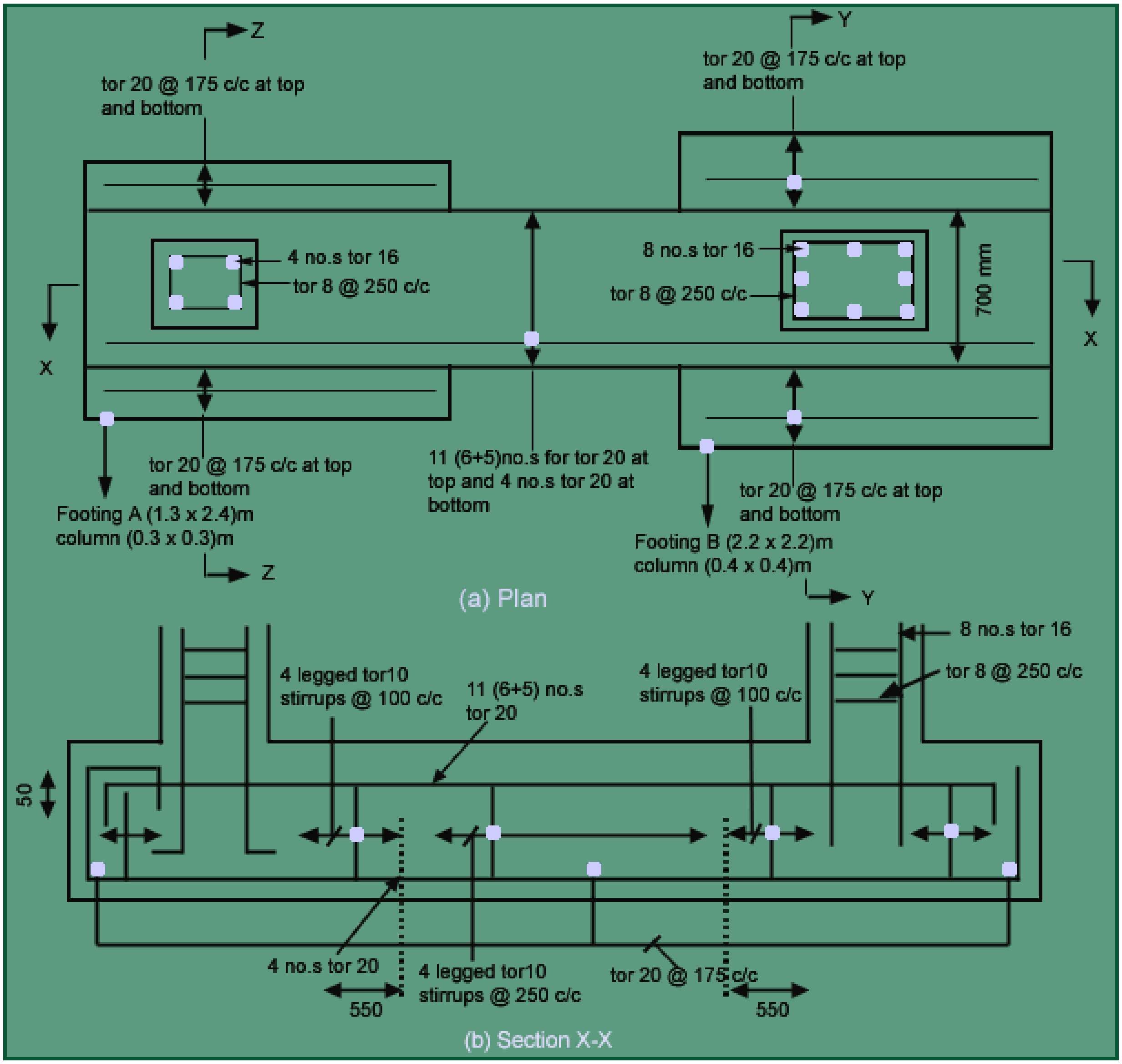


Design of Combined Footing



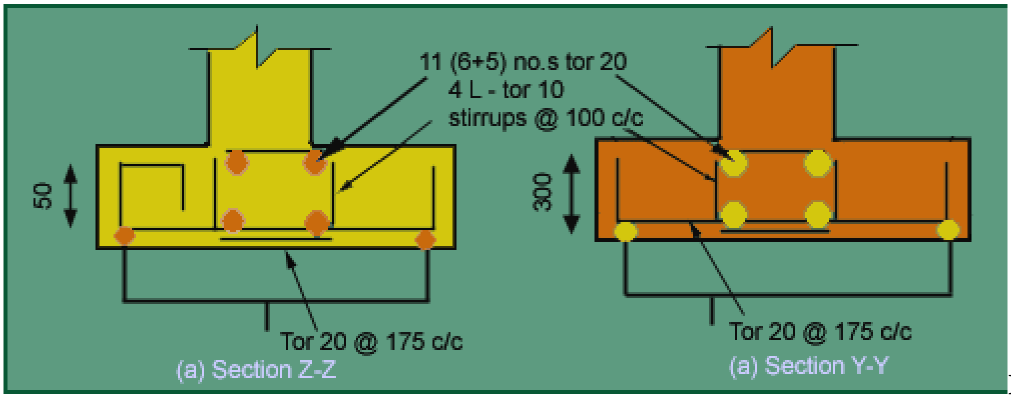
UNIT III

FOOTINGS AND RAFTS



(a) Plan

(b) Section X-X



Design of Combined Footing

P_1 =800kN

P_2 =1000kN

q_a =20 t/m^2,M15, f_y =415kN/m^2

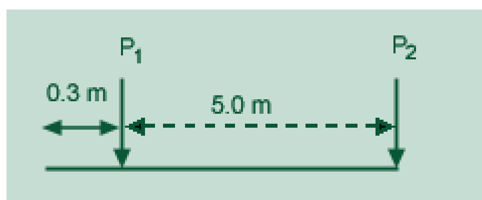


Fig. 4.51 Loading on combined footing

Column size: 400x400mm.

See Fig 4.54 for details of footing. **Column design**

Let pt=0.8%

A_{sc} =.008A; A_c =0.992A

Clause.39.3 of IS 456-2000

A=146763.8mm^2

A_{sc} =1174.11 mm^2, A_c =145589.746mm^2

Provide footing of 400x400size for both columns.

Using 8-16 ϕ as main reinforcement and 8 ϕ @250c/c as lateral tie

Design of Footing

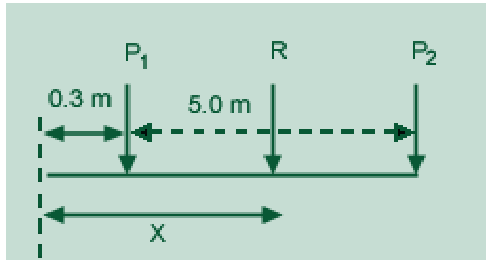


Fig. 4.52 Forces acting on the footing

Resultant of Column Load

R =1800 kN acting 3.08m from the boundary.

Area of the footing :

Taking length L=6m, Depth of footing D_f =0.9m, $\gamma_c = 2.5t/m^2$, $\gamma_s = 1.8t/m^2$

Width of footing, $B = \dfrac{P_1 + P_2}{L[q_a - (\gamma_c - \gamma_s)D_f}$ =1.549m.

Therefore, provide footing of dimension 6m x 1.6m

Soil Pressure q = $\dfrac{180}{6x1.6}$ =18.75 t/m^2< 20 t/m^2 OK.

q_u =28.125 t/m^2

Soil pressure intensity acting along the length =B x q_u =1.6x28.125 =45t/m.

R_B =119.88kN, R_C =150.12kN.

Thickness of Footing i. Wide beam shear:

Maximum shear force is on footing C,SF=115.02KN

$\tau_c \times B \times d = qu[2.556 - 0.2 - d]$

$\tau_c = 0.32N/mm^2$ for percentage reinforcement P_t =0.2%

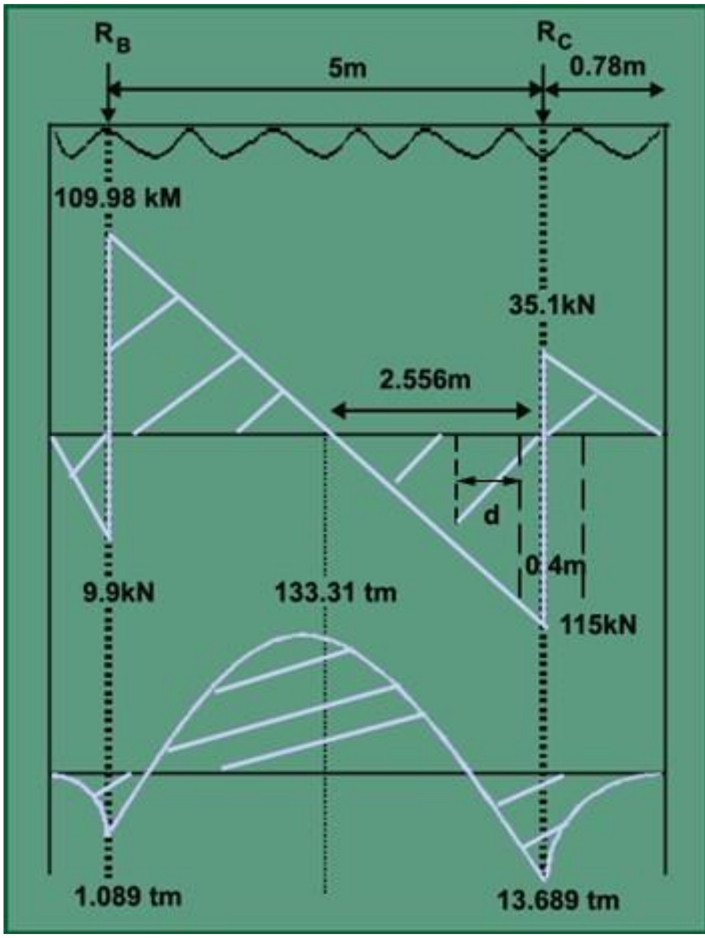
0.32 x d x 1.6=45 [2.556-0.2-d]

d=1.1m

$\tau_c = 0.5N/mm^2$ for percentage reinforcement P_t =0.6%

0.6 x d x 1.6=45 [2.556-0.2-d]

d=0.847m.D=900mm.OK.

ii.Two way Shear 

Thickness of Footing i. Wide beam shear:

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$\tau_c = 0.32N/mm^2$ for percentage reinforcement P_t =0.2%

0.32 x d x 1.6=45 [2.556-0.2-d]

d=1.1m

$\tau_c = 0.5N/mm^2$ for percentage reinforcement P_t =0.6%

0.6 x d x 1.6=45 [2.556-0.2-d]

d=0.847m.D=900mm.OK.

ii.Two way Shear $\beta_c = 1, k_s = 1.5 \Rightarrow 1,$
$\tau_c' = k_s \tau_c = 96.8t/m^2$

Column B

$4(0.4+d)d \times 96.8 = 150 - 28.125(0.4+d)^2$
d=0.415m.

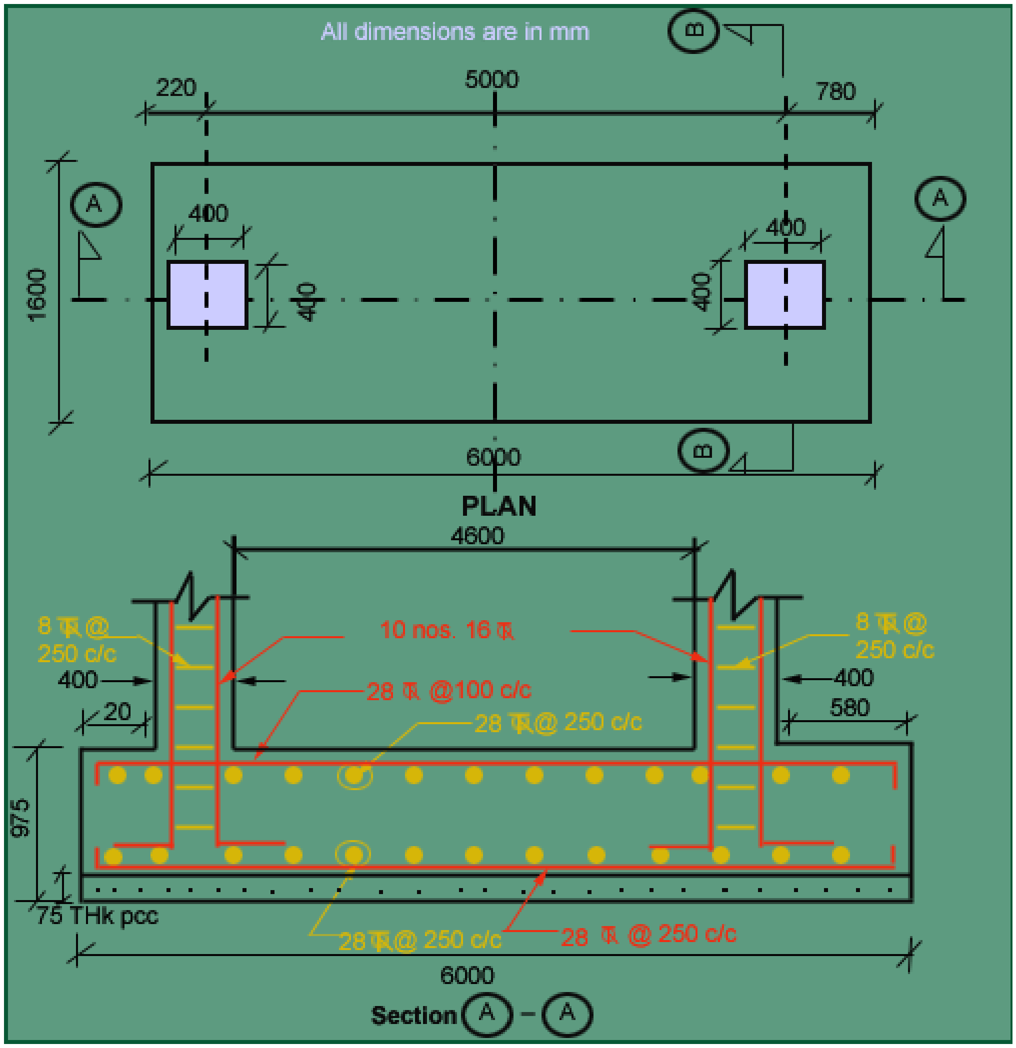
Column A

2d[(0.4+d)+(0.42+d/2)] x 96.8=120-28.125[(0.4+d)(0.42+d/2)]
d=0.3906m
d_{reqd} =0.85mm
$D_{provided}$ =900mm, d_{reqd} =850mm.OK.

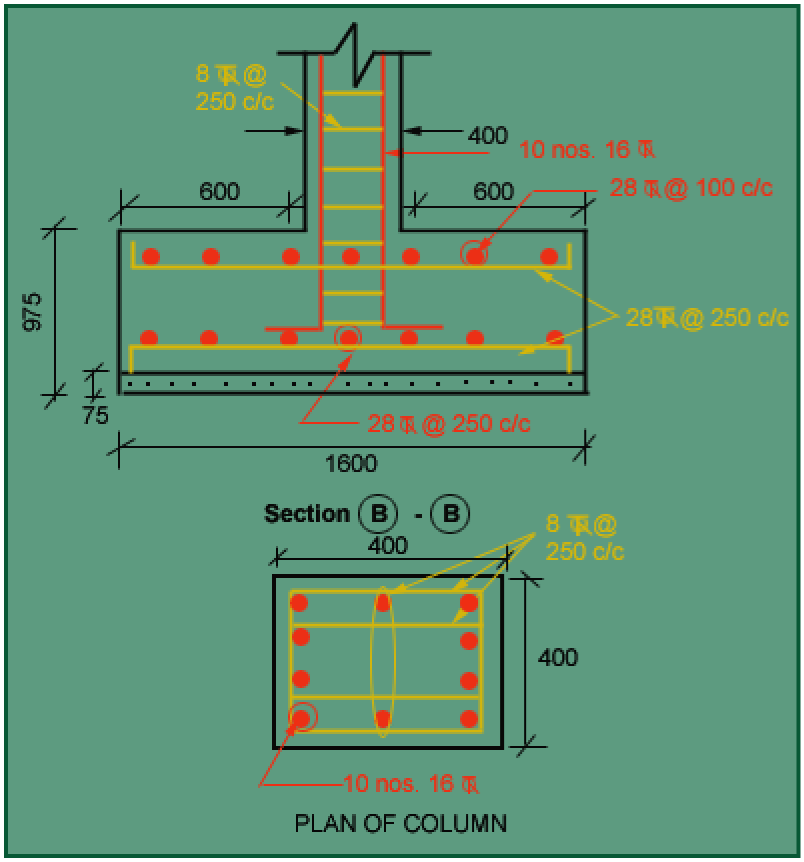
Flexural reinforcement

Along Length Direction
$\frac{M_u}{bd^2} = \frac{133.31 \times 10^4}{1.6 \times 850^2}$ =1.15N/mm^2
Table 1of SP16
P_t =0.354%
P_t provided=0.6%
A_{st} required=5100 mm^2/mm
Provide 28 ϕ@120mmc/c at top and bottom of the footing
Along width direction

$$M_u = \frac{28.125 \times 1.6^2}{2} = 36tm$$



$$M_u = 0.073 N / mm^2$$



Raft Footing Design the raft footing for the given loads on the columns and spacing between the columns as shown below.

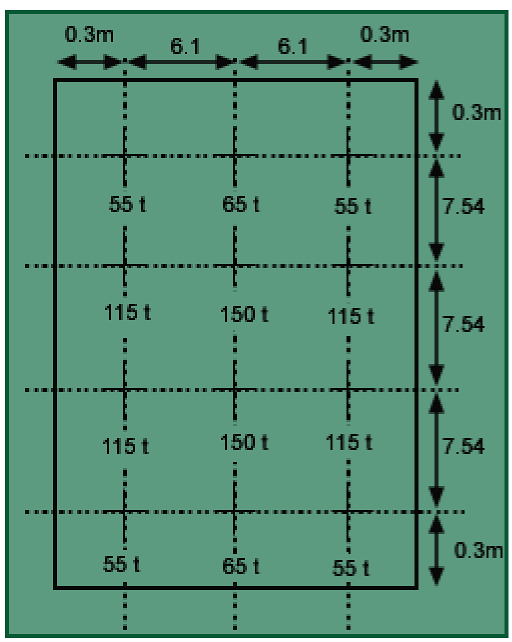


Fig 4.57 column locations and intensity of loads acting on the raft

a) Column sizes

Take size of the columns are as: 300*450 mm for load of less than 115 ton

450*450 mm for a load of greater than 115 ton

Thickness of raft

$$q_{us} = \frac{1110}{12.8 \times 23.22} * 1.5 = 5.607\, t/m^2$$

Two way shear

The shear should be checked for every column, but in this case because of symmetry property checking for 115 t, 150 t, and 55 t is enough.

For 150 t column

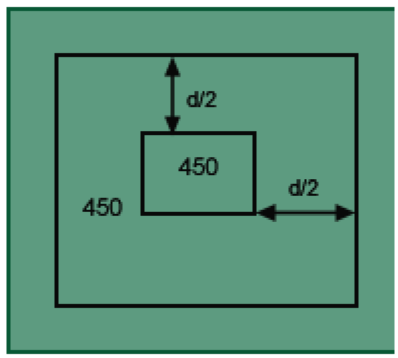


Fig 4.58 section for two way shear for 150 t column

IS: 456-1978, β_c =450/450=1.0

K_s =(0.5+ β_c)=1.0=1.0

Therefore K_s =1.0

$$\tau_c = 0.25\sqrt{f_{ck}} = 96.8\, t/m^2$$

$$\tau_c' = k_s \tau_c = 96.8\, t/m^2$$

$$\tau_c' = 96.8\, t/m^2$$

4(0.45+d)*d*96.8=150*1.5-5.607(0.45+d)2

Therefore d=0.562 m

For 115 t column

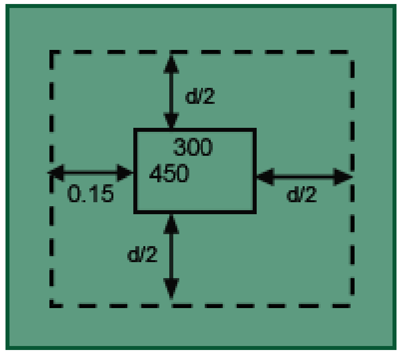


Fig 4.59 section for two way shear for 115 t column

2(0.45+d+0.15+0.3+d/2) d*96.8=115*1.5-5.607(0.45+d)(0.3+0.15+0.5d)

Therefore d=0.519 m

For 55 t column

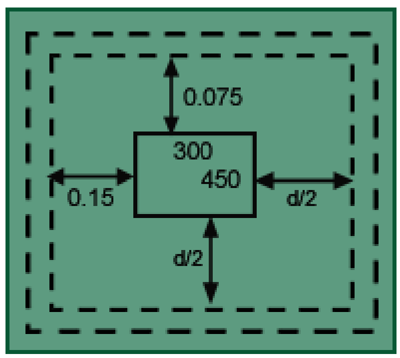


Fig 4.60 section for two way shear for 55 t column

2(0.45+0.075+0.5d+0.15+0.3+0.5d) d*96.8=55*1.5-5.607(0.45+0.5d+0.075)(0.3+0.5d+0.15)

Therefore d=0.32 m

The guiding thickness is 0.562m and code says that the minimum thickness should not be less than 1.0m.

let provide a overall depth of 1.1m=D

d_{prov}=1100-75-20/2=1015mm.

To calculate k & λ-Stiffness factors

There are two criterions for checking the rigidity of the footing:
Plate size used is 300*300 mm.
For clays: μ_s =0.5,

$$k = B\frac{E_s}{1-\mu^2{}_s}$$

Take k=0.7 and B=30 cm

Es=15.75 kg/cm^2=1.575 N/mm^2

$$K = \frac{EI}{E_s b^3 a}, \qquad where \;\; I = \frac{ad^3}{12}$$

$$E = 5000\sqrt{f_{ck}}$$

$$= 5000\sqrt{15} = 19364.92\, N/mm^2$$

b=23.2*103 mm, a=12.8*103 mm 4 , d=1015 mm

$$K = \frac{19364.92 \times 1015^3 \times 12.8*10^3}{12 \times 1.575 \times (23.2*10^3)^3 \times 12.8*10^3}$$

=0.085<0.5

Therefore it is acting as a flexible footing.

$$\lambda = \left(\frac{kB}{4E_c I}\right)^{\frac{1}{4}} = \frac{0.7 \times 12.8*10^2 \times 12}{4 \times 19364.92 \times 12.8*10^2 \times 101.5^3}$$

=0.00179*10^{-3}

1.75/ λ =975.184=9.75m

If column spacing is less than 1.75/ λ, then the footing is said to be rigid.

Therefore the given footing is rigid.

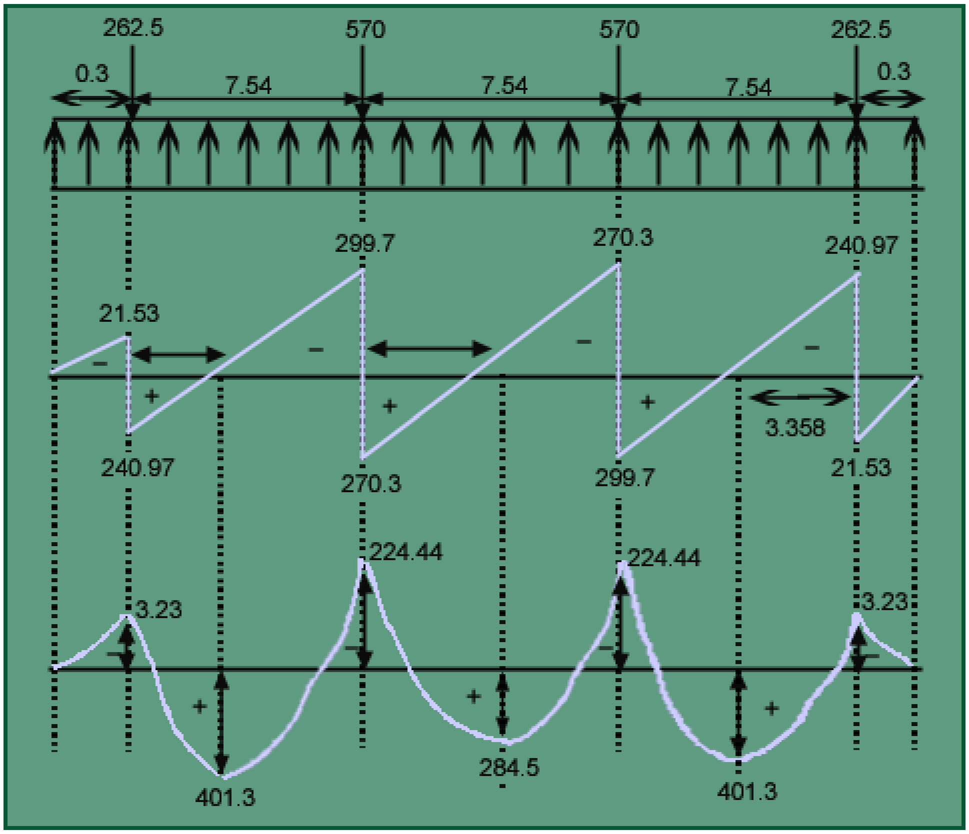
One criterion showing the footing is flexible and another showing that the given footing is rigid. Both are contradicting each other, so design the footing for both criterions.

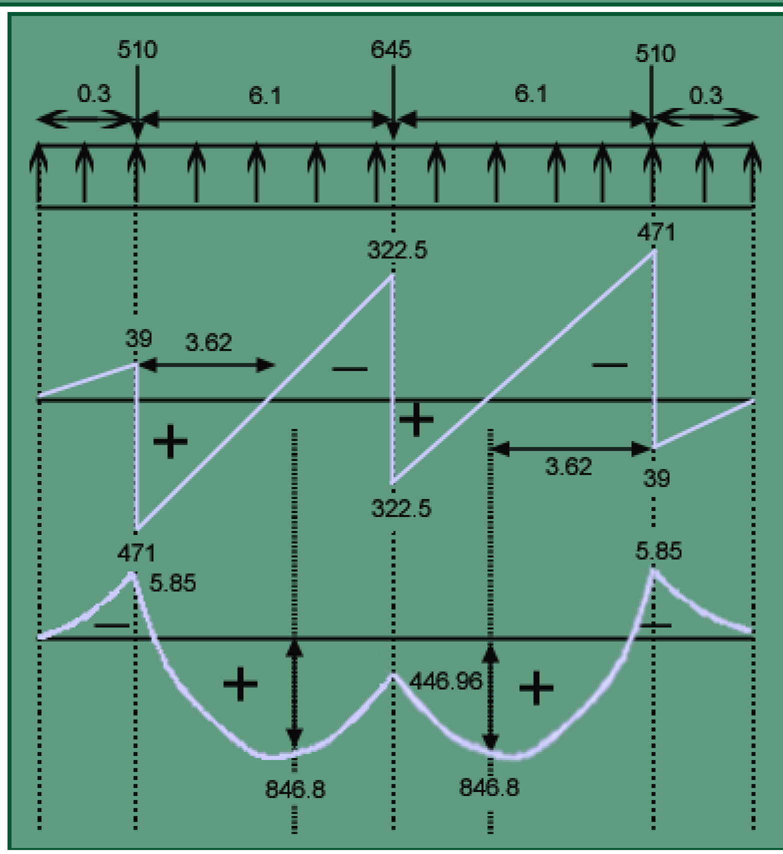
$$q_{act} = \frac{1100}{12.8*23.2} = 3.738\,t/m^2$$

q_{us} =5.607 t/m^2

$$q = \frac{2*510+645}{6.1+6.1+0.6} = 130.08\,t/m^2$$

NOTES 3


262.5
570
570
262.5
0.3
7.54
7.54
7.54
0.3
299.7
270.3
240.97
21.53
240.97
270.3
299.7
21.53
3.358
224.44
224.44
3.23
3.23
401.3
284.5
401.3


510
645
510
0.3
6.1
6.1
0.3
322.5
471
39
3.62
3.62
39
322.5
471
5.85
5.85
446.96
846.8
846.8

Reinforcement in width direction

$$\frac{M_u}{b\,d^2{}_x} = \frac{846.8\times10^7}{23.2\times10^3 * 1015^2} = 0.354$$

From SP-16 graphs

P_t =0.102%, but minimum is 0.12%.

A_{stx} =(0.12*1000*1015)/100=1218 mm^2

Provide 20 mm diameter bars @250 c/c along shorter direction in bottom.

❖ **Reinforcement in length direction**

$$\frac{M_u}{b\,d^2{}_y} = \frac{401.3\times10^7}{12.8\times10^3 * (1015-20)^2} = 0.316$$

Provide 20 mm diameter bars @250 c/c in longer direction.

Clause 33.3.1

$$A_{stx_{centralband}} = \frac{2}{\frac{23.2}{12.8}+1}(1218 * 23.2)\,mm^2$$

Provide 20 mm diameter bars @ 200 c/c in central band and 20 mm diameter bars @300 c/c at other parts along shorter direction at bottom.

Shear (wide beam shear criterion)

In width direction

$$\frac{V_u}{b\,d_x} = \frac{471\times10^4}{23.2\times10^3 * 1015}$$ 0.2 N/mm^2 < τ_v

Pt_{prov} =0.123%,

τ_c =0.27 N/mm 2 > τ_v (from table 61 of SP – 16 by extrapolation)

Therefore no shear reinforcement is required.

$$\frac{V_u}{b\,d_y} = \frac{229.7\times10^4}{12.8\times10^3 *(1013-20)}$$ =0.235 N/mm^2 < τ_c (0.27 N/mm^2)

Therefore no shear reinforcement is required.

Along the width direction

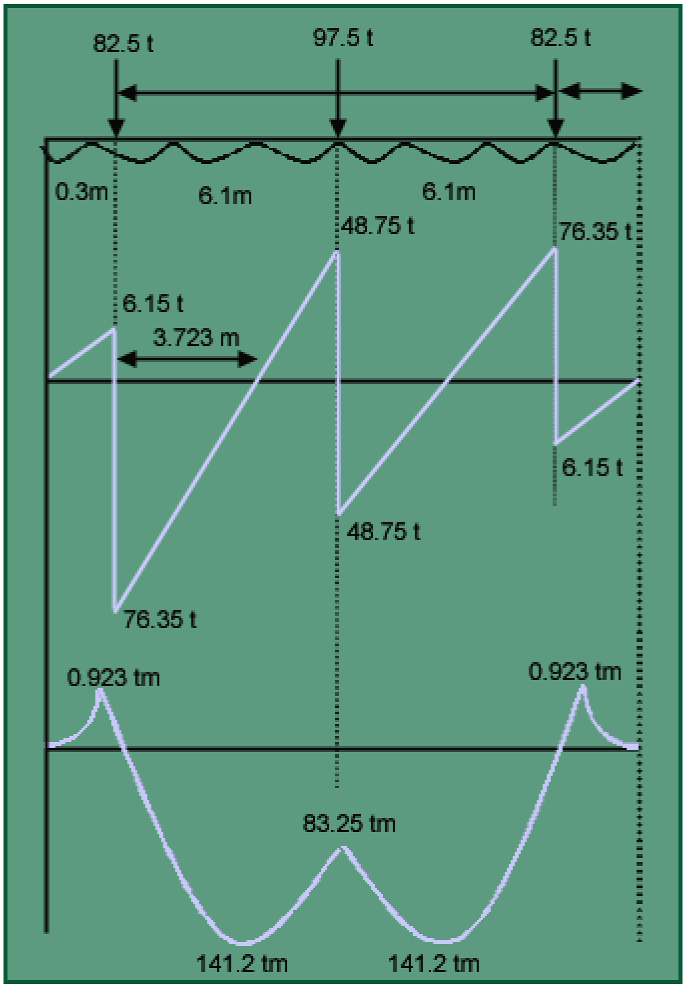


Fig. 4.63 Shear Force and Bending Moment Diagrams of strips 1 and 4

In width direction: Strip1/4:-

M_u =141.2tm

$$\frac{M_u}{bd^2} = \frac{141.2\times10^7}{4.067\times10^3\times(1015)^2}$$ =0.337N/mm^2

Strip2/3

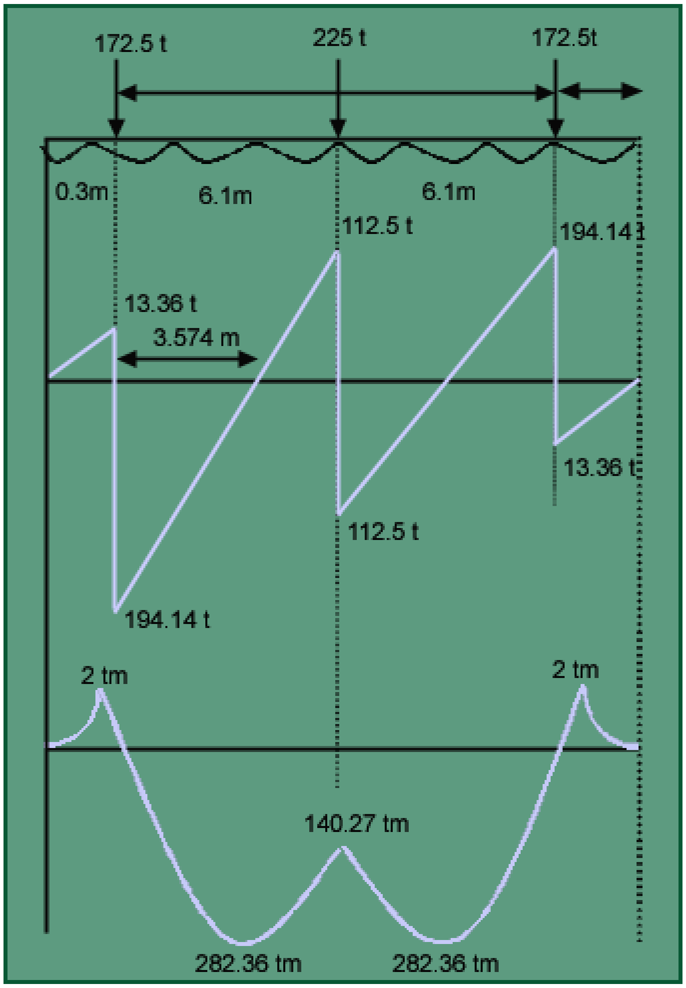


Fig. 4.64 Shear Force and Bending Moment Diagrams of strips 2 and 3

Strip 2/3

M_u =282.36tm

$\frac{M_u}{bd^2} = \frac{282.36 \times 10^7}{7.533 \times 10^3 \times (1015)^2}$ =0.364N/mm^2

Minimum P_t =0.12%has to be provided.

Provide 20 ϕ@200c/c in centre band and 20 ϕ@300c/c at other parts along the shorter direction.

1. **Shear check**

Along width direction:-

For strip1/4:

V_u=76.35t

$\frac{V_u}{bd_x}$ = $\frac{76.35\times10^4}{4.067\times10^3\times1015}$ =0.185N/mm^2< τ_c , OK.

For strip 2/3:

V_u=159.14 t

$\frac{V_u}{bd_x}$ = $\frac{159.14\times10^4}{7.533\times10^3\times1015}$ =0.208N/mm^2< τ_c, OK.

Hence no shear reinforcement is required.

Development Length

$L_d = \frac{\phi\sigma_S}{4\tau bd}$ = $\frac{20\times0.87\times415}{4\times1\times1.6}$ =1128.3mm

At the ends, length of bar provided=150mm.

Extra length to be provided=1128.3-150-8x20=818.3mm.

Provide a Development length of 850mm

3. **Transfer of load at the base of the column:-**

For end column;

A_1 =2650X2725=7.22125x106mm^2

A_2 =300x450=135000mm^2

$\sqrt{\frac{A1}{A2}}$ = 7.31 But not greater than 2.0

q_{prem} = $\sqrt{\frac{A1}{A2}}\times0.45f_{ck}$ =13.5N/mm^2

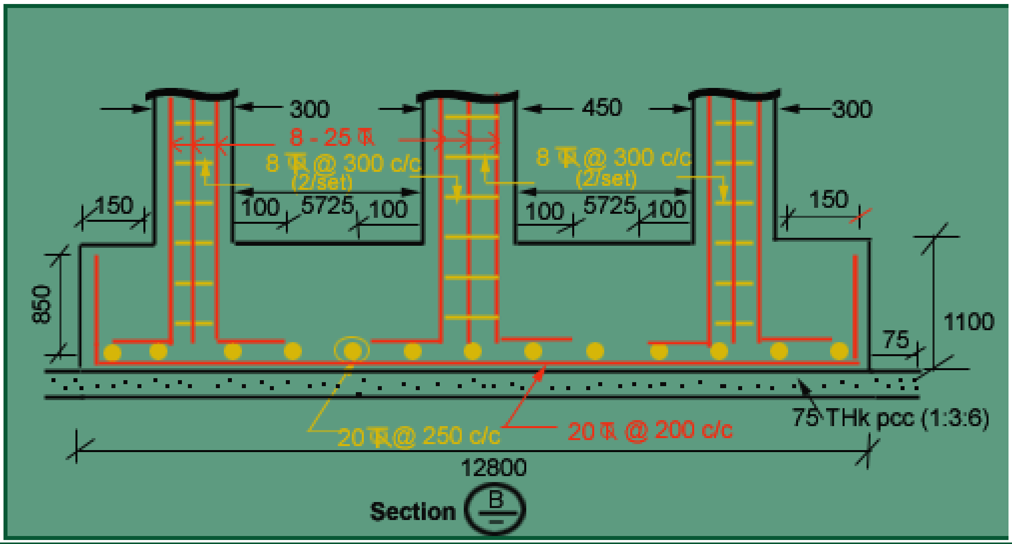
q_{acting} = $\frac{55\times10^4}{300\times450}$ =4.07N/mm^2< q_{prem} .OK.

For 150t columns

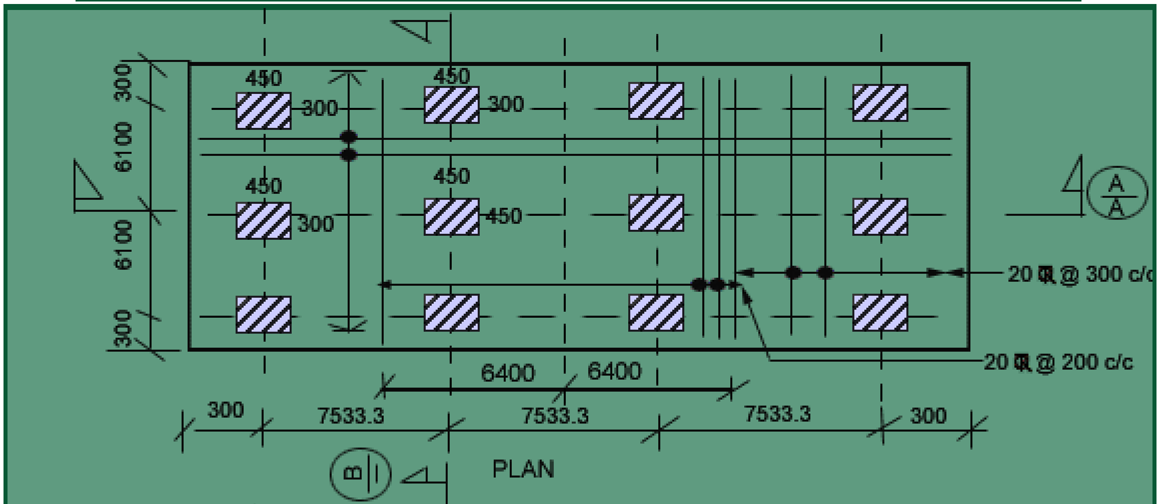
q_{acting} = $\frac{150\times10^4}{450^2}$ =7.41N/mm^2<q_{prem} .OK.

For 115t columns

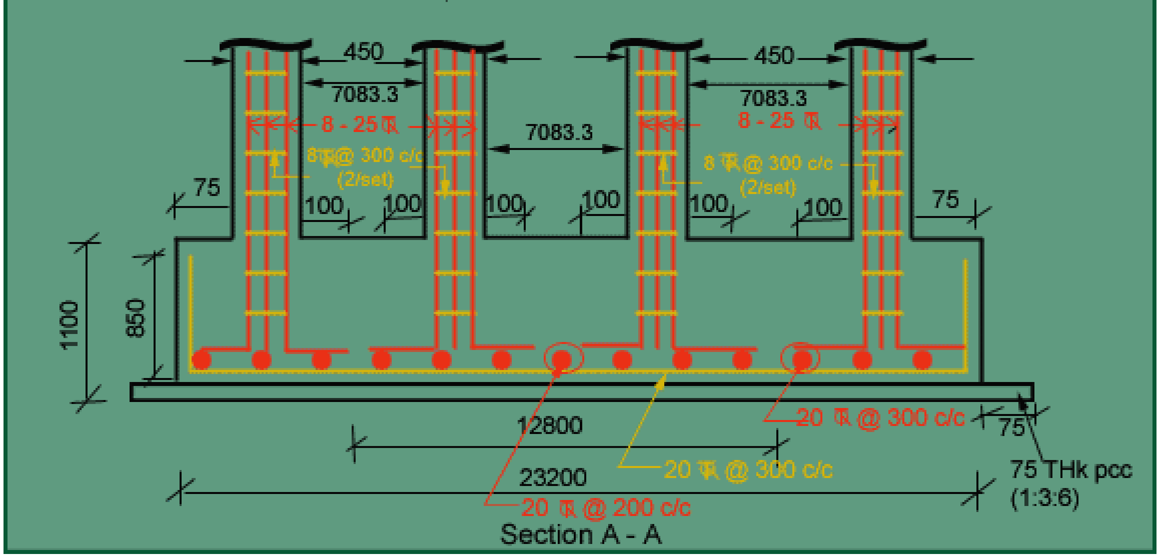
$\sqrt{\frac{A1}{A2}} = 2$, $q_{acting} = \frac{115 \times 10^4}{300 \times 450}$ =8.52N/mm^2< q_{prem}.OK.



300
450
300
8 - 25 ₵
8 ₵ @ 300 c/c
(2/set)
8 ₵ @ 300 c/c
(2/set)
150
100
5725
100
100
5725
100
150
850
1100
75
75 THk pcc (1:3:6)
20 ₵ @ 250 c/c
20 ₵ @ 200 c/c
12800
Section B



300
6100
6100
300
450
300
450
300
450
300
450
450
A
A
20 ₵ @ 300 c/c
20 ₵ @ 200 c/c
6400
6400
300
7533.3
7533.3
7533.3
300
B
PLAN



450
450
7083.3
7083.3
7083.3
8 - 25 ₵
8 - 25 ₵
8 ₵ @ 300 c/c
(2/set)
8 ₵ @ 300 c/c
(2/set)
75
100
100
100
100
100
100
75
1100
850
12800
20 ₵ @ 300 c/c
75
23200
20 ₵ @ 300 c/c
20 ₵ @ 200 c/c
75 THk pcc
(1:3:6)
Section A - A

UNIT IV

PILES

Foundations provide support to the structure, transfers the loads from the structure to the soil. But the layer at which the foundation transfers the load shall have an adequate bearing capacity and suitable settlement characteristics. There are several types of foundation depending on various considerations such as-

- Total load from the superstructure.
- Soil conditions.
- Water level.
- Noise and vibrations sensitivity.
- Available resources.
- Time-frame of the project.
- Cost.

Broadly speaking, foundations can be classified as shallow foundations and deep foundations. Shallow footings are usually used when the bearing capacity of the surface soil is adequate to carry the loads imposed by a structure. On the other hand, deep foundations are usually used when the bearing capacity of the surface soil is not sufficient to carry the loads imposed by a structure. So, the loads have to be transferred to a deeper level where the soil layer has a higher bearing capacity.

Pile foundation, a kind of deep foundation, is actually a slender column or long cylinder made of materials such as concrete or steel which are used to support the structure and transfer the load at desired depth either by end bearing or skin friction.

Pile foundations are deep foundations. They are formed by long, slender, columnar elements typically made from steel or reinforced concrete, or sometimes timber. A foundation is described as 'piled' when its depth is more than three times its breadth.

When to Use Pile Foundation

Following are the situations when using a pile foundation system can be

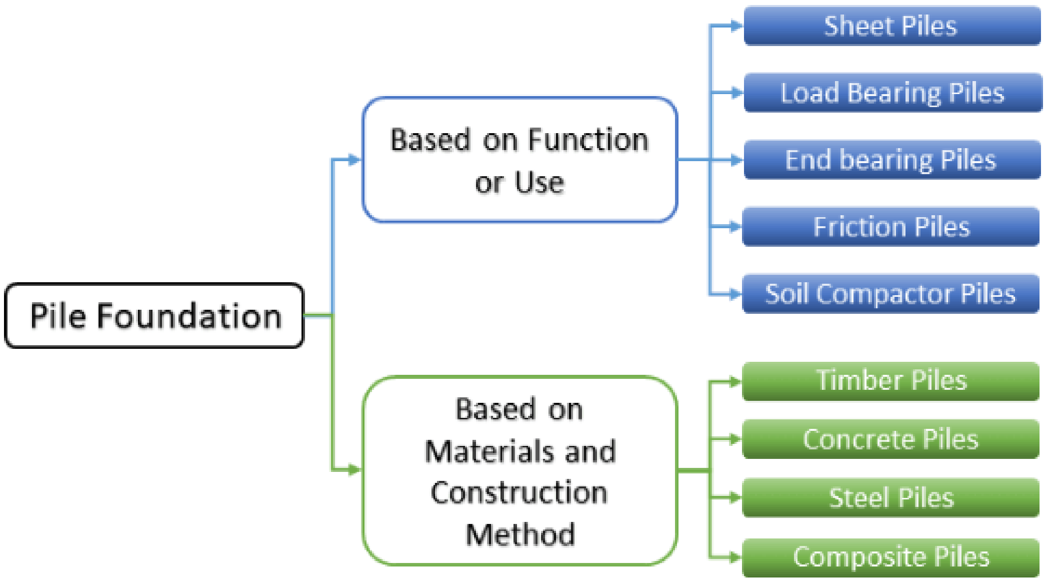
- When the groundwater table is high.
- Heavy and un-uniform loads from superstructure are imposed.
- Other types of foundations are costlier or not feasible.
- When the soil at shallow depth is compressible.
- When there is the possibility of scouring, due to its location near the river bed or seashore, etc.
- When there is a canal or deep drainage systems near the structure.
- When soil excavation is not possible up to the desired depth due to poor soil condition.
- When it becomes impossible to keep the foundation trenches dry by pumping or by any other measure due to heavy inflow of seepage.

Types of Pile Foundation

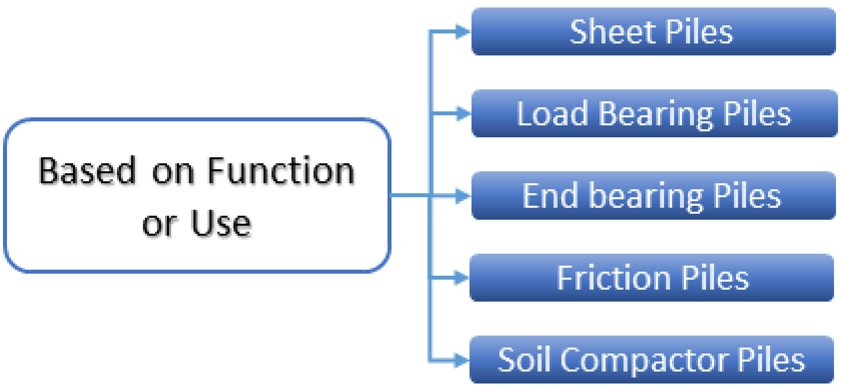
Pile foundations can be classified based on function, materials and installation process, etc. Followings are the types of pile foundation used in construction:

A. Based on Function or Use
    1. Sheet Piles
    2. Load Bearing Piles
    3. End bearing Piles
    4. Friction Piles
    5. Soil Compactor Piles
B. Based on Materials and Construction Method
    1. Timber Piles
    2. Concrete Piles
    3. Steel Piles
    4. Composite Piles

The following diagram is representing pile foundation types discussed above.



Classification of Pile Foundation Based on Function or Use



Sheet Piles

This type of pile is mostly used to provide lateral support. Usually, they resist lateral pressure from loose soil, the flow of water, etc. They are usually used for cofferdams, trench sheeting, shore protection, etc. They are not used for providing vertical support to the structure. They are usually used to serve the following purpose-

- Construction of retaining walls.
- Protection from river bank erosion.
- Retain the loose soil around foundation trenches.
- For isolation of foundation from adjacent soils.
- For confinement of soil and thus increase the bearing capacity of the soil.

Load Bearing Piles

This type of pile foundation is mainly used to transfer the vertical loads from the structure to the soil. These foundations transmit loads through the soil with poor supporting property onto a layer which is capable of bearing the load. Depending on the mechanism of load transfer from pile to the soil, load-bearing piles can be further classified as flowed.

End Bearing Piles

In this type of pile, the loads pass through the lower tip of the pile. The bottom end of the pile rests on a strong layer of soil or rock. Usually, the pile rests at a transition layer of a weak and strong slayer. As a result, the pile acts as a column and safely transfers the load to the strong layer.

The total capacity of end bearing pile can be calculated by multiplying the area of the tip of the pile and the bearing capacity of at that particular depth of soil at which the pile rests. Considering a reasonable factor of safety, the diameter of the pile is calculated.

Friction Pile

Friction pile transfers the load from the structure to the soil by the frictional force between the surface of the pile and the soil surrounding the pile such as stiff clay, sandy soil, etc. Friction can be developed for the entire length of the pile or a definite length of the pile, depending on the strata of the soil. In friction pile, generally, the entire surface of the pile works to transfer the loads from the structure to the soil.

The surface area of the pile multiplied by the safe friction force developed per unit area determines the capacity of the pile.

While designing skin friction pile, the skin friction to be developed at a pile surface should be sincerely evaluated and a reasonable factor of safety should be considered. Besides this one can increase the pile diameter, depth, number of piles and make pile surface rough to increase the capacity of friction pile.

Soil Compactor Piles

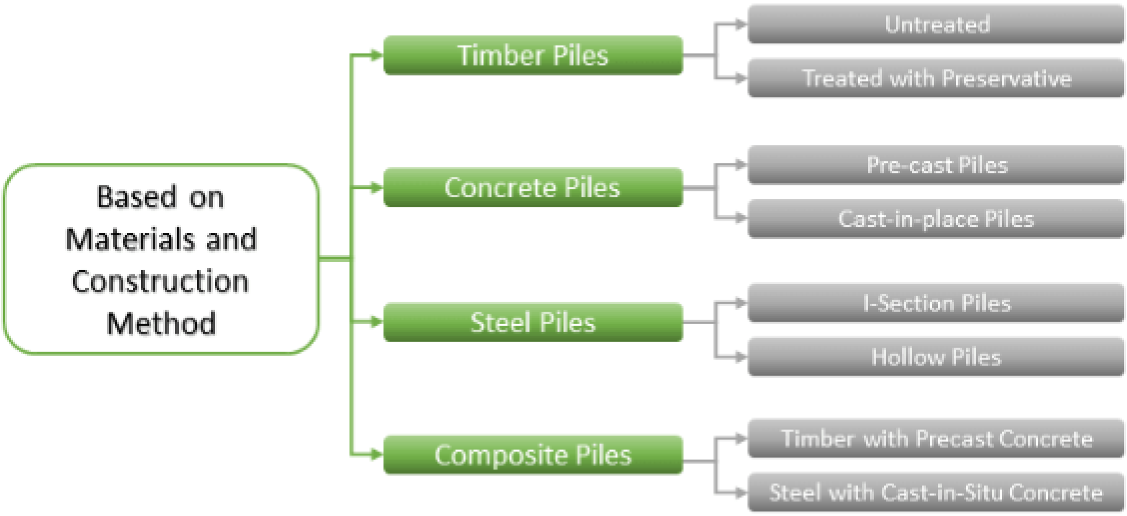
Sometimes piles are driven at placed closed intervals to increase the bearing capacity of soil by compacting.

Classification of Piles Based on Materials and Construction Method

Primarily piles can be classified into two parts. Displacement piles and Non-displacement or Replacement piles. Piles which causes the soil to be displaced vertically and radially as they are driven to the ground is known as Displacement piles. In case of Replacement piles, the ground is bored and the soil is removed and then the resulting hole is either filled with concrete or a pre-cast concrete pile is inserted. On the basis of materials of pile construction and their installation process load-bearing piles can be classified as follows:

1. Timber Piles
    i. Untreated
    ii. Treated with Preservative
2. Concrete Piles
    i. Pre-cast Piles
    ii. Cast-in-place Piles

3. Steel Piles
    i. I-Section Piles
    ii. Hollow Piles
4. Composite Piles



Timber Piles

Timber piles are placed under the water level. They last for approximately about 30 years. They can be rectangular or circular in shape. Their diameter or size can vary from 12 to 16 inches. The length of the pile is usually 20 times of the top width.

They are usually designed for 15 to 20 tons. Additional strength can be obtained by bolting fish plates to the side of the piles.

Advantages of Timber Piles-

- Timber piles of regular size are available.
- Economical.
- Easy to install.
- Low possibility of damage.
- Timber piles can be cut off at any desired length after they are installed.
- If necessary, timber piles can be easily pulled out.

Disadvantages of Timber Piles-

- Piles of longer lengths are not always available.
- It is difficult to obtain straight piles if the length is short.
- It is difficult to drive the pile if the soil strata are very hard.
- Spicing of timber pile is difficult.
- Timber or wooden piles are not suitable to be used as end-bearing piles.
- For durability of timber piles, special measures have to be taken. For example- wooden piles are often treated with preservative.

Concrete Piles

Pre-cast Concrete Pile

The precast concrete pile is cast in pile bed in the horizontal form if they are rectangular in shape. Usually, circular piles are cast in vertical forms. Precast piles are usually reinforced with steel to prevent breakage during its mobilization from casting bed to the location of the foundation. After the piles are cast, curing has to be performed as per specification. Generally curing period for pre-cast piles is 21 to 28 days.

Advantages of Pre-cast Piles

- Provides high resistance to chemical and biological cracks.
- They are usually of high strength.
- To facilitate driving, a pipe may be installed along the center of the pile.
- If the piles are cast and ready to be driven before the installation phase is due, it can increase the pace of work.
- The confinement of the reinforcement can be ensured.
- Quality of the pile can be controlled.
- f any fault is identified, it can be replaced before driving.
- Pre-cast piles can be driven under the water.
- The piles can be loaded immediately after it is driven up to the required length.

Disadvantages of Pre-cast Piles

- Once the length of the pile is decided, it is difficult to increase or decrease the length of the pile afterward.
- They are difficult to mobilize.
- Needs heavy and expensive equipment to drive.
- As they are not available for readymade purchase, it can cause a delay in the project.
- There is a possibility of breakage or damage during handling and driving od piles.

Cast-in-Palace Concrete Piles

This type of pile is constructed by boring of soil up to the desired depth and then, depositing freshly mixed concrete in that place and letting it cure there. This type of pile is constructed either by driving a metallic shell to the ground and filling it with concrete and leave the shell with the concrete or the shell is pulled out while concrete is poured.

Advantages of Cast-in-Place Concrete Piles

- The shells are light weighted, so they are easy to handle.
- Length of piles can be varied easily.
- The shells may be assembled at sight.
- No excess enforcement is required only to prevent damage from handling.
- No possibility of breaking during installation.
- Additional piles can be provided easily if required.

Disadvantages of Cast-in-Place Concrete Piles

- Installation requires careful supervision and quality control.
- Needs sufficient place on site for storage of the materials used for construction.
- It is difficult to construct cast in situ piles where the underground water flow is heavy.
- Bottom of the pile may not be symmetrical.
- If the pile is un-reinforced and uncased, the pile can fail in tension if there acts and uplifting force.

Steel Piles

Steel piles may be of I-section or hollow pipe. They are filled with concrete. The size may vary from 10 inches to 24 inches in diameter and thickness is usually ¾ inches. Because of the small sectional area, the piles are easy to drive. They are mostly used as end-bearing piles.

Advantages of Steel Piles

- They are easy to install.
- They can reach a greater depth comparing to any other type of pile.
- Can penetrate through the hard layer of soil due to the less cross-sectional area.
- It is easy to splice steel piles
- Can carry heavy loads.

Disadvantage of Steel Piles

- Prone to corrosion.
- Has a possibility of deviating while driving.
- Comparatively expensive.

UNIT V

RETAINING WALLS

Introduction Retaining walls must be designed for lateral earth pressure. Different types of retaining walls are used to retain soil in different places.

Three main types of retaining walls:

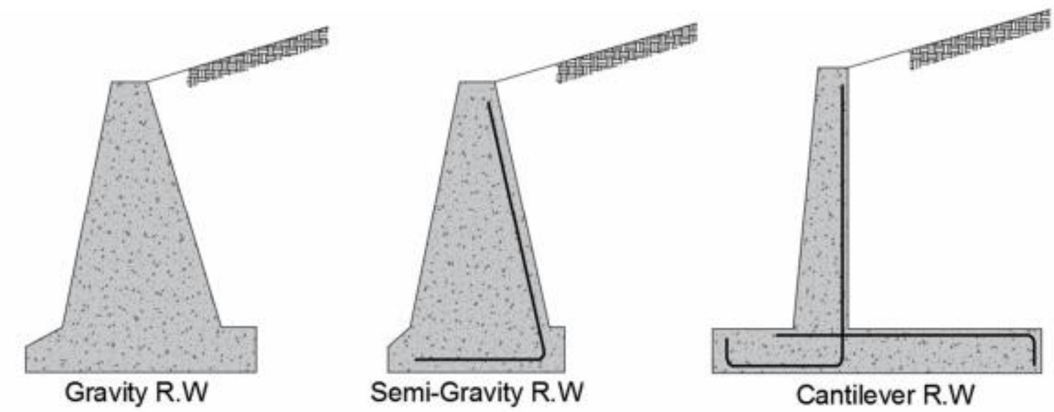
1. Gravity retaining wall (depends on its weight for resisting lateral earth force because it have a large weigh)

2. Semi-Gravity retaining wall (reduce the dimensions of the gravity retaining wall by using some reinforcement).

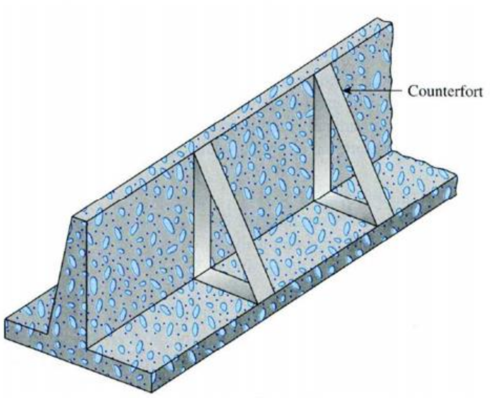
3. Cantilever retaining wall (reinforced concrete wall with small dimensions and it is the most economical type and the most common)

Note: Structural design of cantilever retaining wall is depend on separating each part of wall and design it as a cantilever, so it's called cantilever R.W.

The following figure shows theses different types of retaining walls:

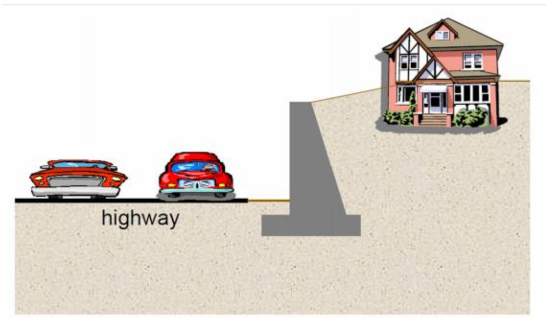


There are another type of retaining wall called "counterfort RW" and is a special type of cantilever RW used when the height of RW became larger than 6m, the moment applied on the wall will be large so we use spaced counterforts every a specified distance to reduce the moment RW.



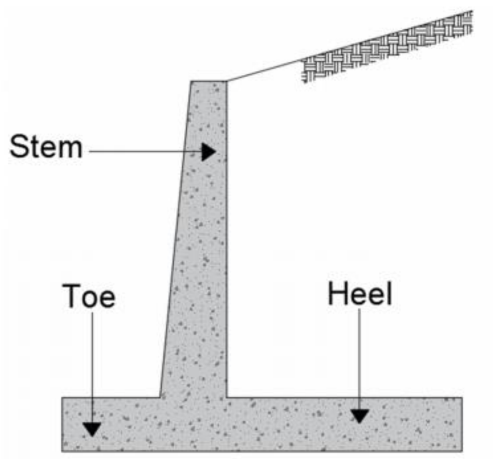
Where we use Retaining Walls:

Retaining walls are used in many places, such as retaining a soil of high elevation (if we want to construct a building in lowest elevation) or retaining a soil to save a highways from soil collapse and for several applications. The following figure explain the function of retaining walls:



Elements of Retaining Walls:

Each retaining wall divided into three parts; stem, heel, and toe as shown for the following cantilever footing (as example):



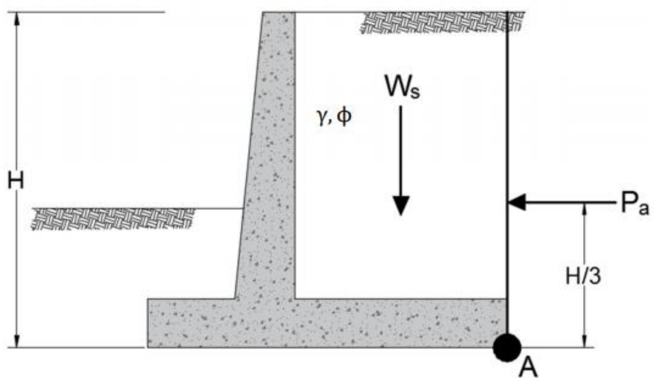
Application of Lateral Earth Pressure Theories to Design:

Rankine Theory:

Rankine theory was modified to be suitable for designing a retaining walls. This modification is drawing a vertical line from the lowest-right corner till intersection with the line of backfill, and then considering the force of soil acting on this vertical line. The soil between the wall and vertical line is not considered in the value of Pa, so we take this soil in consideration as a vertical weight applied on the stem of the retaining wall as will explained later.

The following are all cases of Rankine theory in designing a retaining wall:

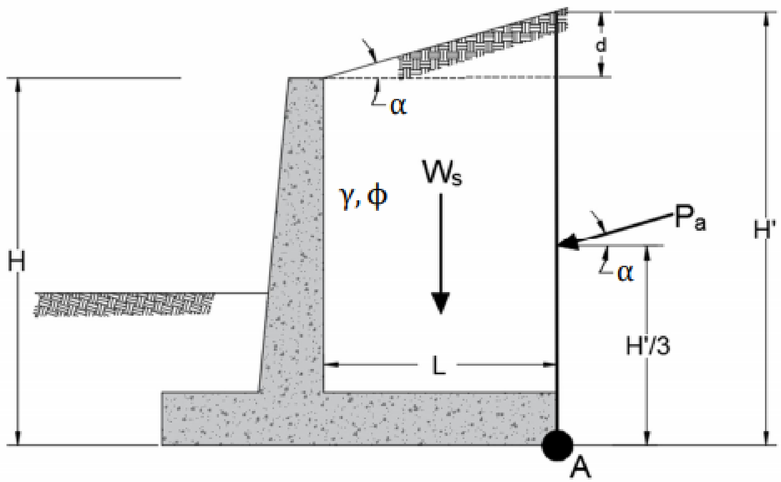
1. The wall is vertical and backfill is horizontal:



Here the active force P_a is horizontal and can be calculated as following:

$$P_a = \frac{1}{2}\gamma H^2 K_a \qquad , \qquad K_a = \tan^2\left(45 - \frac{\phi}{2}\right)$$

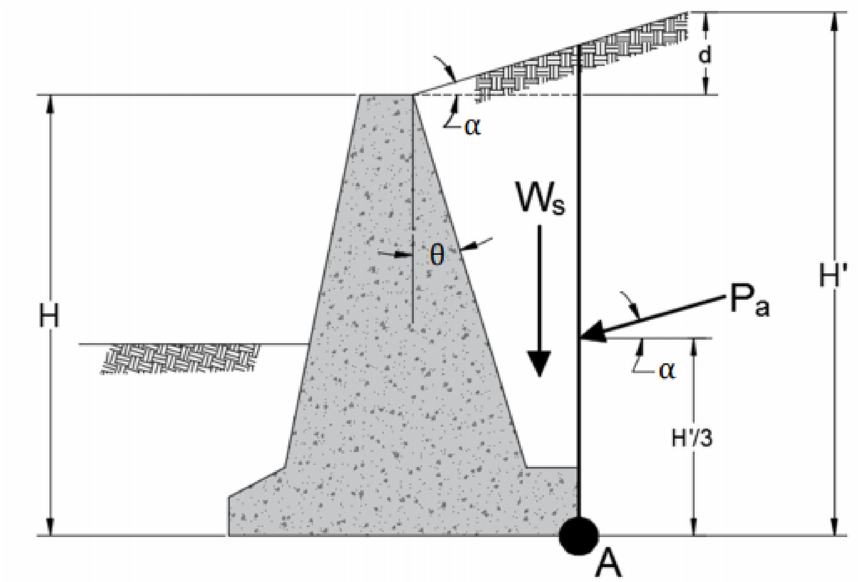
2. The wall is vertical and the backfill is inclined with horizontal by angle (α):



Here the active force P_a is inclined with angle (α) and can be calculated as following:

$$P_a = \frac{1}{2}\gamma H'^2 K_a$$

3. The wall is inclined by angle (θ) with vertical and the backfill is inclined with horizontal by angle (α):

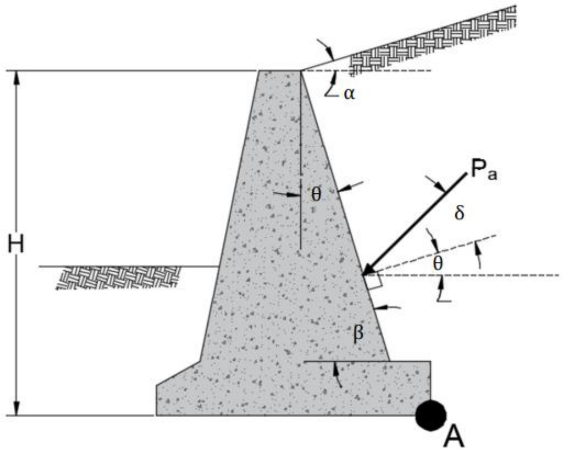


Note that the force P_a is inclined with angle (α) and not depend on the inclination of the wall because the force applied on the vertical line and can be calculated as following:

$$P_a = \frac{1}{2}\gamma H'^2 K_a$$

Coulomb's Theory:

Coulomb's theory will remains unchanged (without any modifications) in this chapter. The force P_a is applied directly on the wall, so whole soil retained by the wall will be considered in P_a and thereby the weight of soil will not apply on the heel of the wall.



$$P_a = \frac{1}{2}\gamma H^2 K_a$$

Stability of Retaining Wall

A retaining wall may be fail in any of the following:

1. It may overturn about its toe.

2. It may slide along its base.

3. It may fail due to the loss of bearing capacity of the soil supporting the base.

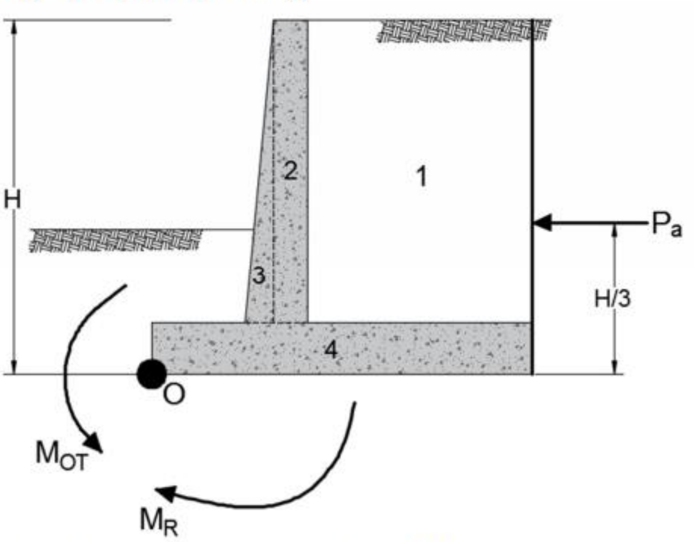
4. It may undergo deep-seated shear failure.

5. It may go through excessive settlement.

We will discuss the stability of retaining wall for the first three types of failure (overturning, sliding and bearing capacity failures).

We will use rankine theory to discusses the stability of these types of failures. Coulomb’s theory will be the same with only difference mentioned above (active force applied directly on the wall).

Stability for Overturning



The **horizontal** component of active force will causes overturning on retaining wall about point O by moment called "overturning moment"

$$M_{OT} = P_{a,h} \times \frac{H}{3}$$

The **horizontal** component of active force will causes overturning on retaining wall about point O by moment called "overturning moment"

$$M_{OT} = P_{a,h} \times \frac{H}{3}$$

This overturning moment will resisted by all vertical forces applied on the base of retaining wall:

1. Vertical component of active force $P_{a,v}$(if exist).
2. Weight of all soil above the heel of the retaining wall.
3. Weight of each element of retaining wall.
4. Passive force (we neglect it in this check for more safety).

Now, to calculate the moment from these all forces (resisting moment) we prepare the following table:

$\text{Force} = \text{Volume} \times \text{unit weight}$ but, we take a strip of 1 mlength

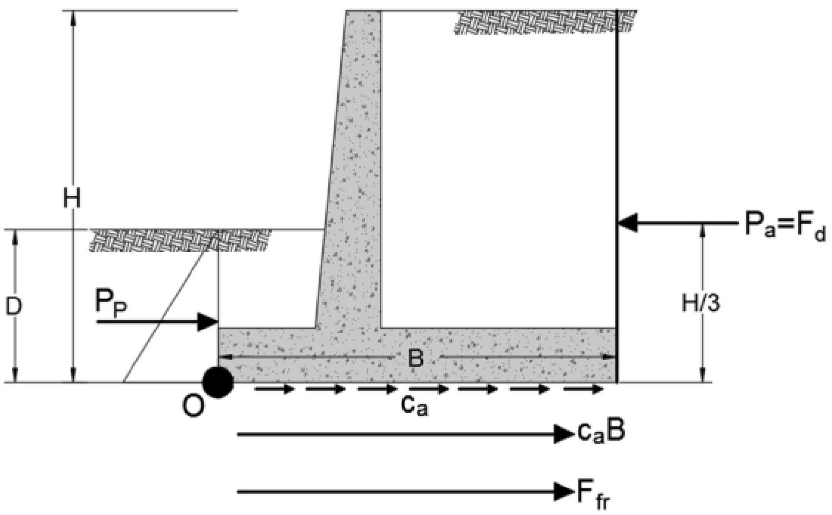
$\rightarrow \text{Force} = \text{Area} \times \text{unit weight}$

Section	Area	Weight/unit length of the wall	Moment arm measured from O	Moment about O
1	A_1	$W_1 = A_2 \times \gamma_1$	X_1	M_1
2	A_2	$W_2 = A_2 \times \gamma_c$	X_2	M_2
3	A_3	$W_3 = A_2 \times \gamma_c$	X_3	M_3
4	A_4	$W_4 = A_2 \times \gamma_c$	X_4	M_4
		$P_{a,v}$(if exist).	B	M_V
$\sum$		$\sum V$		$\sum M = M_R$

γ_1 = unit weight of the soil above the heel of RW

$$FS_{OT} = \frac{M_R}{M_{OT}} \geq 2$$

Stability for Sliding along the Base



Also, the horizontal component of active force may causes movement of the wall in horizontal direction (i.e. causes sliding for the wall), this force is called driving force $F_d = P_{a,h}$.

This driving force will be resisted by the following forces:

1. Adhesion between the soil (under the base) and the base of retaining wall:

c_a = adhesion along the base of RW (KN/m)

$C_a = c_a \times B$ = adhesion force under the base of RW (KN)

c_a can be calculated from the following relation:

$c_a = K_2 c_2$ $\qquad$ c_2 = cohesion of soil under the base

So adhesion force is:

$C_a = K_2 c_2 B$

2. Friction force due to the friction between the soil and the base of RW:

Always friction force is calculated from the following relation:

$F_{fr} = \mu_s N$

Here N is the sum of vertical forces calculated in the table of the first check (overturning)

$\rightarrow N = \sum V$ (including the vertical component of active force)

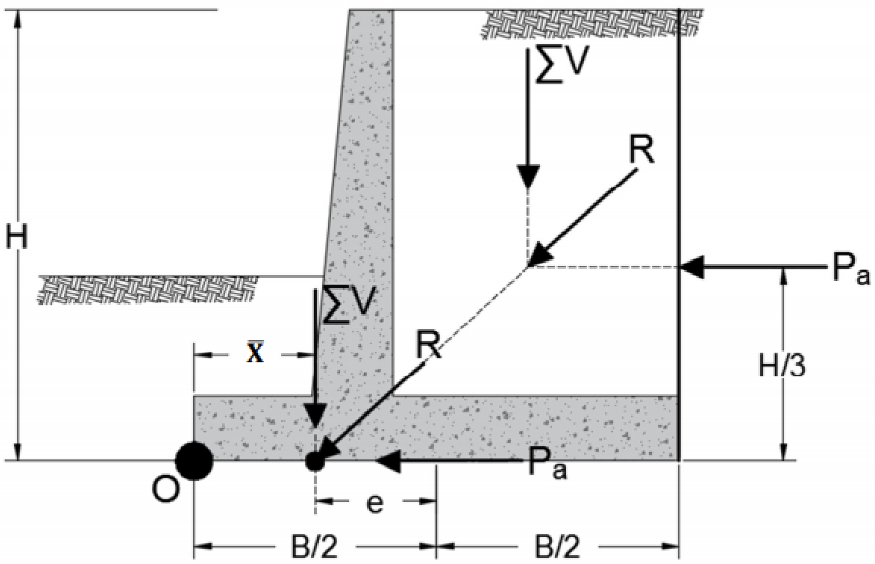
μ_s = coefficient of friction (related to the friction between soil and base)

$\mu_s = \tan(\delta_2)$ $\qquad$ $\delta_2 = K_1 \phi_2 \rightarrow\rightarrow \mu_s = \tan(K_1 \phi_2)$

ϕ_2 = friction angle of the soil under the base.

$$\rightarrow F_{fr} = \sum V \times \tan(K_1 \phi_2)$$

Check Stability for Bearing Capacity Failure



As we see, the resultant force (R) is not applied on the center of the base of retaining wall, so there is an eccentricity between the location of resultant force and the center of the base, this eccentricity **may be calculated as**

following:

From the figure above, take summation moments about point O:

$$M_O = \sum V \times \bar{X}$$

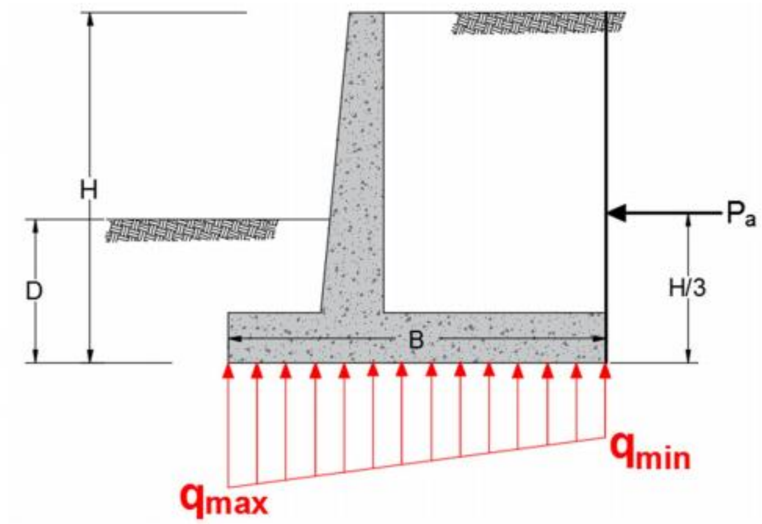
From the first check (overturning) we calculate the overturning moment and resisting moment about point O, so the difference between these two moments gives the net moment at O.

$$M_O = M_R - M_{OT}$$

$$\rightarrow M_R - M_{OT} = \sum V \times \bar{X} \rightarrow\rightarrow \bar{X} = \frac{M_R - M_{OT}}{\sum V}$$

$e = \frac{B}{2} - \bar{X} = \checkmark$ (see the above figure).

Since there exist eccentricity, the pressure under the base of retaining wall is not uniform (there exist maximum and minimum values for pressure).



We calculate q_{max} and q_{min} as stated in chapter 3:

Eccentricity in B-direction and retaining wall can be considered strip footing

If $e < \frac{B}{6}$

$$q_{max} = \frac{\sum V}{B \times 1}\left(1 + \frac{6e}{B}\right)$$

$$q_{min} = \frac{\sum V}{B \times 1}\left(1 - \frac{6e}{B}\right)$$

If $e > \frac{B}{6}$

$$q_{max,new} = \frac{4 \sum V}{3 \times 1 \times (B - 2e)}$$

Now, we must check for q_{max}:

$q_{max} \leq q_{all} \rightarrow q_{max} = q_{all}$ (at critical case)

$$FS_{B.C} = \frac{q_u}{q_{max}} \geq 3$$

Calculation of q_u:

q_u is calculated using Meyerhof equation as following:

$$q_u = cN_c F_{cs} F_{cd} F_{ci} + qN_q F_{qs} F_{qd} F_{qi} + 0.5B\gamma N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

Where

c = Cohesion of soil under the base

q = Effective stress at the level of the base of retaining wall.

$q = \gamma_2 \times D_f$

D_f here is the depth of soil above the toe = D (above figure)

$\rightarrow q = \gamma_2 \times D$

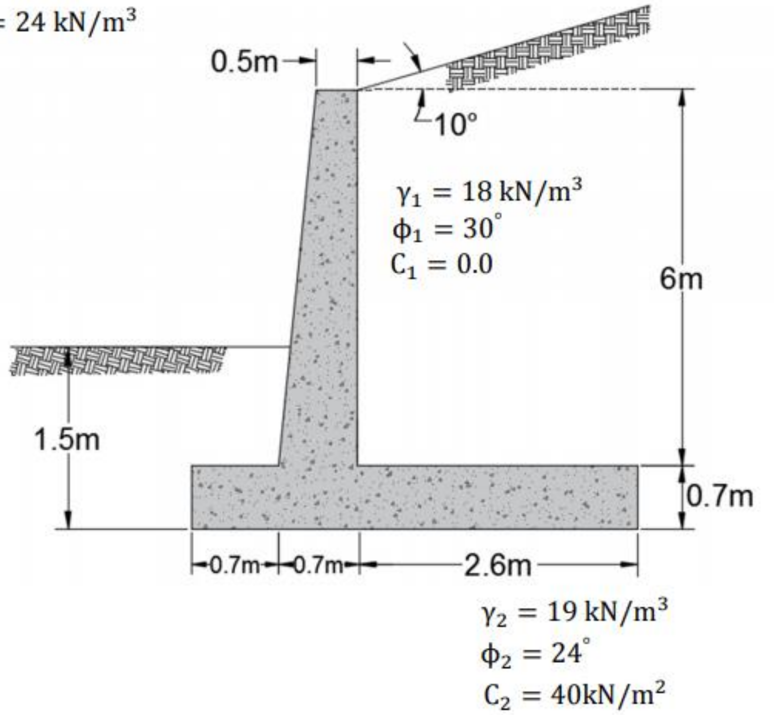
γ = unit weight of the soil under the base of the RW.

Problems:

1.

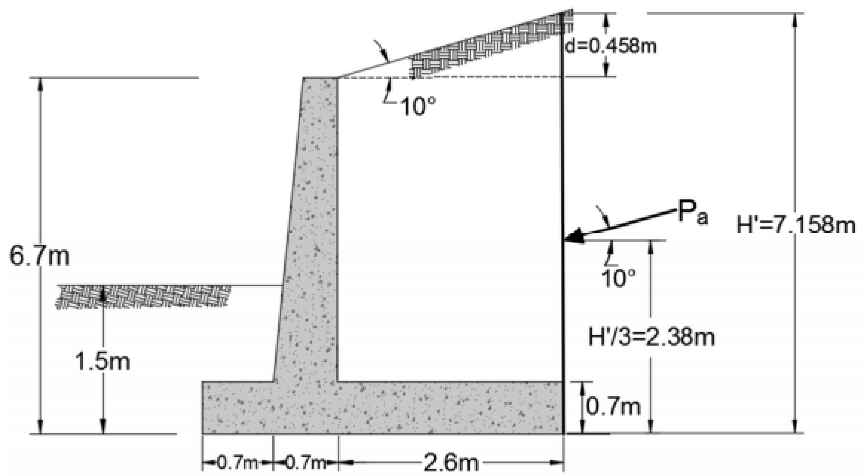
The cross section of the cantilever retaining wall shown below. Calculate the factor of safety with respect to overturning, sliding, and bearing capacity.

$\gamma_c = 24 \text{ kN/m}^3$



Since it is not specified a method for solving the problem, directly we use Rankine theory.

Now draw a vertical line starts from the right-down corner till reaching the backfill line and then calculate active force (P_a):



$$\tan 10 = \frac{d}{2.6} \rightarrow d = 2.6 \times \tan 10 = 0.458\text{m}$$

$$H' = 6.7 + d = 6.7 + 0.458 = 7.158\text{m}$$

$$\tan 10 = \frac{d}{2.6} \rightarrow d = 2.6 \times \tan 10 = 0.458\text{m}$$
$$H' = 6.7 + d = 6.7 + 0.458 = 7.158\text{m}$$

Now we calculate P_a:

$$P_a = \frac{1}{2} \times \gamma_1 \times H'^2 \times K_a$$

Since the backfill is inclined and the wall is vertical, K_a is calculated from

$$K_a = 0.3495$$
$$\rightarrow P_a = \frac{1}{2} \times 18 \times 7.158^2 \times 0.3495 = 161.2 \text{ kN}$$

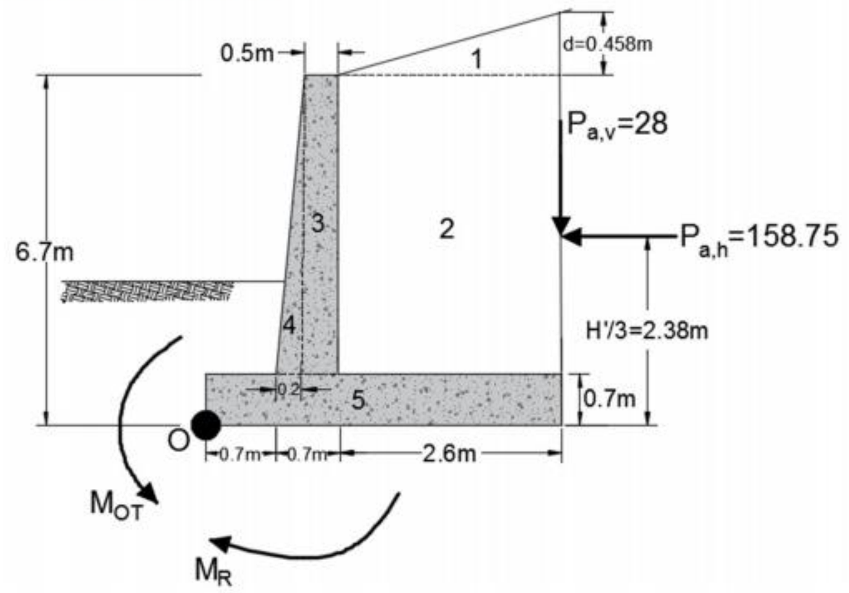
Location of P_a:

Location$= \frac{H'}{3} = \frac{7.158}{3} = 2.38$

The force P_a is inclined with angle $\alpha = 10$ with horizontal:

$P_{a,h} = 161.2 \cos(10) = 158.75$, $P_{a,v} = 161.2 \sin(10) = 28$

Check for Overturning:



$M_{OT} = 158.75 \times 2.38 = 337.8$ KN.m

Now to calculate M_R we divided the soil and the concrete into rectangles and triangles to find the area easily (as shown above) **and to find the arm from the center of each area to point O** as prepared in the following table:

Section	Area	Weight/unit length of the wall	Moment arm measured from O	Moment about O
1	0.595	$0.595 \times 18 = 10.71$	$4 - \frac{2.6}{3} = 3.13$	33.52
2	15.6	$15.6 \times 18 = 280.8$	$1.4 + 1.3 = 2.7$	758.16
3	3	$3 \times 24 = 72$	$1.4 - 0.25 = 1.15$	82.8
4	0.6	$0.6 \times 24 = 14.4$	$0.9 - \frac{0.2}{3} = 0.833$	12
5	2.8	$2.8 \times 24 = 67.2$	$\frac{4}{2} = 2$	134.4
		$P_{a,v} = 28$	B=4	112
$\sum$		$\sum V = 470.11$		$M_R = 1132.88$

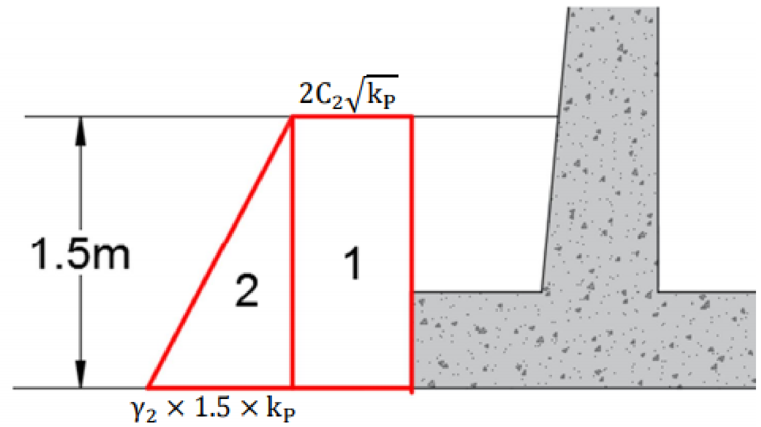
Note that we neglect passive force because it is not obligatory.

$FS_{OT} = \frac{M_R}{M_{OT}} = \frac{1132.88}{377.8} = 2.99 > 2 \rightarrow$ **OK ✓**.

Check for Sliding:

$FS_S = \frac{F_R}{F_d} \geq 2$ (if we consider P_P in F_R)

It is preferable to consider passive force in this check.
Applying rankine theory on the soil in the left (draw vertical line till reaching the soil surface).



k_P is calculated for the soil using rankine theory without considering any iniclination of the wall, because it is calculated for the soil below the bas

$$k_P = \tan^2\left(45 + \frac{\phi_2}{2}\right) = \tan^2\left(45 + \frac{20}{2}\right) = 2.04$$

$$P_1 = (\text{rectangle area}) = \left(2 \times 40 \times \sqrt{2.04}\right) \times 1.5 = 171.4\ \text{kN}$$

$$P_2 = (\text{triangle area}) = \frac{1}{2} \times (19 \times 1.5 \times 2.04) \times 1.5 = 43.6\ \text{kN}$$

$$P_P = P_1 + P_2 = 171.4 + 43.6 = 215\ \text{kN}$$

$$F_d = P_{a,h} = 158.75\ \text{Kn}$$

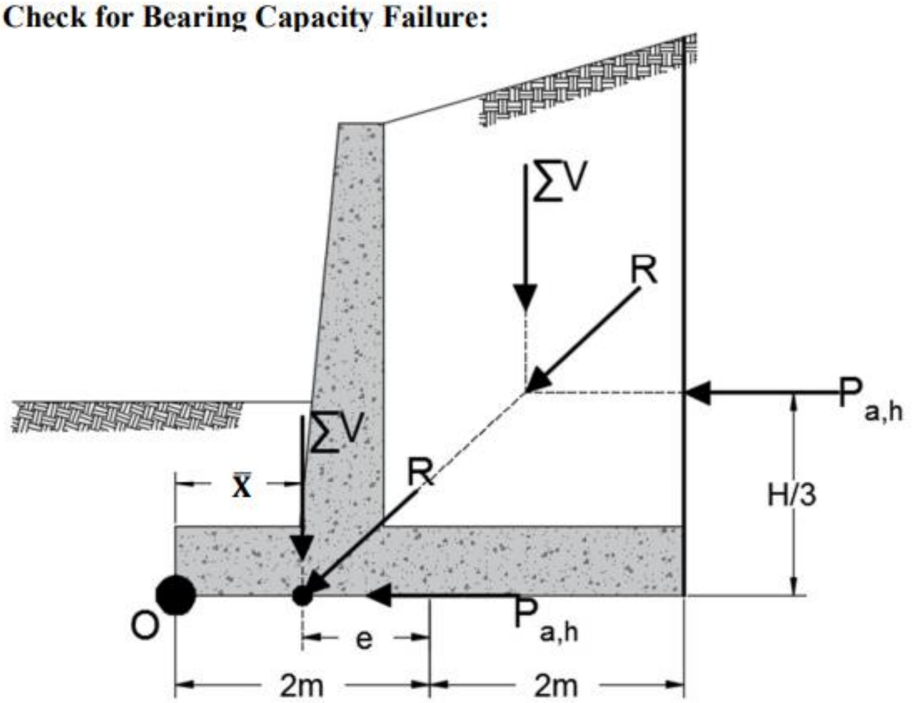
$$F_R = \sum V \times \tan(K_1\phi_2) + K_2 c_2 B + P_P$$

Take $K_2 = K_2 = 2/3$ $\quad \sum V = 470.11$ (from table of first check)

$$F_R = 470.11 \times \tan\left(\frac{2}{3} \times 20\right) + \frac{2}{3} \times 40 \times 4 + 215 = 433.1\ \text{kN}$$

$$\rightarrow FS_S = \frac{433.1}{158.75} = 2.72 > 2 \rightarrow \textbf{OK} ✓.$$

Check for Bearing Capacity Failure:



As stated previously, $\overline{X}$ can be calculated as following:

$$\overline{X} = \frac{M_R - M_{OT}}{\sum V} = \frac{1132.88 - 377.8}{470.11} = 1.6 \text{ m}$$

$$e = \frac{B}{2} - \overline{X} = 2 - 1.6 = 0.4\text{m}$$

$$\frac{B}{6} = \frac{4}{6} = 0.667 \rightarrow e = 0.4 < \frac{B}{6} \rightarrow\rightarrow\rightarrow$$

$$q_{max} = \frac{\sum V}{B \times 1}\left(1 + \frac{6e}{B}\right) = \frac{470.11}{4 \times 1}\left(1 + \frac{6 \times 0.4}{4}\right) = 188.04 \text{ kN/m}^2$$

$$q_{min} = \frac{\sum V}{B \times 1}\left(1 - \frac{6e}{B}\right) = \frac{470.11}{4 \times 1}\left(1 - \frac{6 \times 0.4}{4}\right) = 47 \text{ kN/m}^2$$

Calculation of q_u (for the soil below the bas):

$$q_u = cN_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + 0.5B\gamma N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

$c = 40 \quad , \quad q = 1.5 \times 19 = 28.5 \quad , \quad \gamma = 19$

$B = B' = B - 2e = 4 - 2(0.4) = 3.2\text{m}$

Shape factors = 1 (RW can be considered strip footing).

For $\phi = 20 \rightarrow N_c = 14.83, N_q = 6.4, N_\gamma = 5.39$ (from table 3.3)

Depth factors: (We use B not B')

$$\frac{D}{B} = \frac{1.5}{4} = 0.375 < 1 \text{ and } \phi = 20 > 0.0 \rightarrow\rightarrow$$

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

$$= 1 + 2\tan 20\,(1 - \sin 20)^2\,(0.375) = 1.12$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi} = 1.12 - \frac{1 - 1.12}{14.83 \times \tan 20} = 1.14$$

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

Inclination Factors:

$$\beta = \Psi = \tan^{-1}\left(\frac{P_{a,h}}{\sum V}\right) = \tan^{-1}\left(\frac{158.75}{470.11}\right) = 18.6 \text{ (with vertical)}$$

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta^\circ}{90}\right)^2 \left(1 - \frac{18.6}{90}\right)^2 = 0.63$$

$$F_{\gamma i} = \left(1 - \frac{\beta^\circ}{\phi^\circ}\right) = \left(1 - \frac{18.6}{20}\right) = 0.07$$

$$\rightarrow q_u = 40 \times 14.83 \times 1.14 \times 0.63 + 28.5 \times 6.4 \times 1.12 \times 0.63$$
$$+ 0.5 \times 3.2 \times 19 \times 5.39 \times 1 \times 0.07$$

$$\rightarrow q_u = 566.2 \text{ kN/m}^2$$

$$FS_{B.C} = \frac{q_u}{q_{max}} = \frac{566.2}{188.04} = 3.01 > 3 \text{ (slightly satisfied)} \textbf{OK} \checkmark.$$