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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – I - SITE INVESTIGATION, SELECTION OF FOUNDATION AND BEARING CAPACITY – SCIA7007

UNIT I

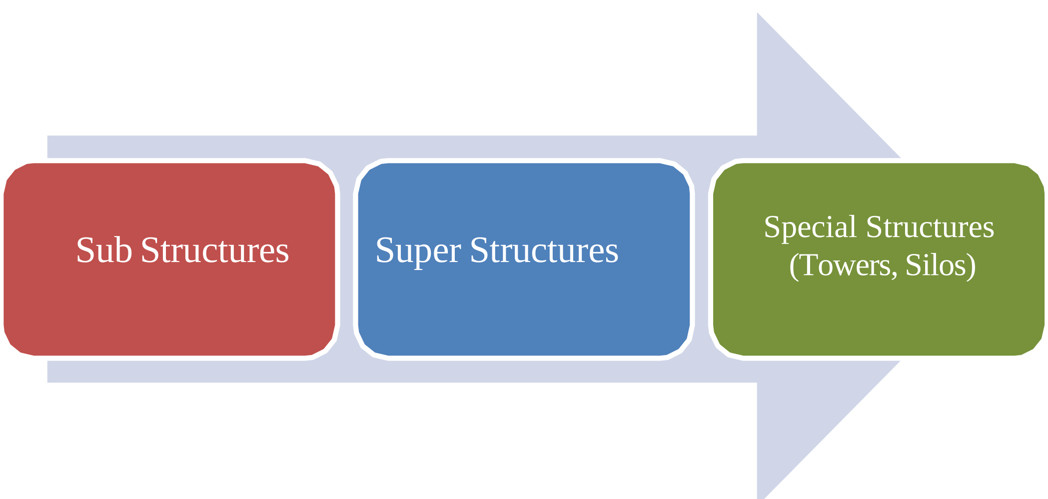
Site Investigation, Selection of Foundation and Bearing Capacity

Foundation Engineering:

The art of selecting, designing, and constructing the elements that transfer the weight (Weight may also include horizontal loads in addition to vertical loads) of a structure to the underlying soil or rock. A foundation is interfacing element between the superstructure and the underlying soil or rock. The loads transmitted by the foundation to the underling soil must not cause soil shear failure or damaging settlement of the superstructure.

DEFINITION OF STRUCTURES

- *Assemblage of Load Bearing Elements such as Beam, Column, etc*



The term "foundation engineering" is used to include the design of foundations for buildings and other structures and also for such non foundation problems as designs of retaining walls, bulkheads, cofferdams, tunnels, and earth dams, as well as the design of natural slopes, dewatering of soils, and stabilization of soils mechanically and chemically.

The geotechnical engineer is responsible for all geotechnical requirements of all types ofstructures.

For any construction project, the geotechnical engineer‟s responsibilities include:

- developing a soil exploration plan;
- preparing the Preliminary Geotechnical Report (PGR) to assist in the selection of foundation type and to perform a preliminary seismic analysis/evaluation;
- identifying the proposed boring locations and anticipated foundation type;
- Assisting the Construction engineers by preparing pile driving criteria, reviewing pileinstallation plans and determining acceptance of as-built piles.
- Also assisting bridge designer in determining pile production lengths based on field loadtests.

PROPERTIES OF FOUNDATION

- Strength: Load bearing capacities: Crystalline rocks (very strong - 12,000), sedimentaryrocks (intermediate - 6,000) and other types of soils (relatively lower - 2,000 to 3,000)
- Stable under loads (creep, shrinkage and swelling)
- Drainage characteristics: Porosity and permeability
- Soil property estimation: Subsurface exploration (test pits - less than 8 ft in depth; borings - greater than 8 ft) - Estimate level of water table - Testing of soil sample in
  laboratory for various properties: Particle size distribution, Liquid limit, Plastic limit, Water content, Permeability, Shrinkage/ swelling, Shear/compressive strength, Consolidation (creep and settlement)

CONSTRUCTION OF FOUNDATIONS

- Some amount of excavation required for every building - Top soil consisting of organic matter is removed - Below the region of soil erosion (by water and wind) & below the level of permafrost - To the required depth at which the bearing capacity necessary for the building is met - A variety of machines used for excavation - The sides of excavation too be protected from caving in by benching, sheeting (soldier beams and lagging, sheet piles, slurry walls, etc.) or bracing (cross-slot, rakers or tiebacks) - De-watering using well-points & sumps, and watertight barriers - Mixing the soil by rotating paddles

- Bulldozers, Shovel dozers, Back hoes ,Bucket loaders, Scrapers, Trenching machines Power shovels, Tractor-mounted rippers, Pneumatic hammers, Drop balls, Hydraulicsplitters and Blasting.

Purpose of Foundation:

All engineering structures are provided with foundations at the base to fulfill the following objectives and purposes;

i. To distribute the load of the structure over a large bearing area so as to bring intensity ofloading within the safe bearing capacity of the soil lying underneath.
ii. To load the bearing surface at a uniform rate so as to prevent unequal settlement.
iii. To prevent the lateral movement of the supporting material.
iv. To secure a level and firm bed for building operations.
v. To increase the stability of the structure as a whole.

Factors Affecting the Depth of Foundation

For economic consideration, the depth at which the foundation is located (i.e. **Df**) is kept as small as possible. Typically it is in the range of 0.5 to 1.5 m below the ground surface for buildings that do not have a basement. For those building having a basement, **Df** can be 3.5 m or more. In determining **Df,** the soil profile is carefully studied and the influence of the following factors accounted for.

1. Presence of Loose Fill

Often one encounters pockets of loose fill of recently dumped soil or construction waste at or near the ground surface. Foundations should be placed below such loose fills.

2. Depth of Water Table

Wherein possible, shallow foundations are not placed below the ground water level to avoid expensive de-watering costs during foundation construction.

3. Lateral Variability

Usually all foundations are placed at the same Df. there are soil profiles that calls for a different Dffor different footings. For example, consider a profile in which rock exist at shallow depth and is gently sloping in some direction. In an attempt to take advantage of the high allowable soil pressure associated with placing the foundation on rock, we will have a different Df for each footings so that each can be placed on the rock.

4. Zones of Volume Change

In cold regions where temperature changes cause soil near the ground surface to go through cycles of freezing and thawing with consequent changes in soil volume, foundation are placed below the zone so affected. Similarly in swelling soils, there is a zone that undergoes volume change due to wetting and drying cycles. Df is selected such that it is more than the thickness of this zone.

5. Scour

When shallow foundations are designed to be placed below the river bed for river crossing structures, one must recognize that the elevation of the river bed changes on account of scour that occurs when the water flows at high velocity such as during floods.

Soil Exploration

The knowledge of subsoil conditions at a site is a prerequisite for safe and economical design of substructure elements. The field and laboratory studies carried out for obtaining the necessary information about the surface and subsurface features of the proposed area including the position of the ground water table, are termed as soil exploration or site investigation.

The primary objectives of soil exploration are

- Determination of the nature of the deposits of soil.
- Determination of the depth and thickness of the various soil strata and their extent in thehorizontal direction.
- The location of ground water table (GWT).
- Obtaining soil and rock samples from the various strata.
- The determination of the engineering properties of the soil and rock strata that affect theperformance of the structure.
- Determination of the in-situ properties by performing field tests.

Scope of Soil Investigation

The scope of a soils investigation depends on the type, size, and importance of the structure, the client, the engineer's familiarity with the soils at the site, and local building codes. Structures that are sensitive to settlement such as machine foundations and high-use buildings usually require a thorough soils investigation compared to a foundation for a house. A client may wish to take a greater risk than normal to save money and set limits on the type and extent of the site investigation. If the geotechnical engineer is familiar with a site, he/she may undertake a very simple soils investigation to confirm his/her experience. Some local building codes have provisions that set out the extent of a site investigation. It is mandatory that a visit be made to the proposed site. In the early stages of a project, the available information is often inadequate to allow a detailed plan to be made.

A site investigation must be developed in phases.

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

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

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

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

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

Steps Involved in Sub Soil Investigation

- Site Investigation
- Soil Exploration (Boring Holes)
- Collection of Soil Samples
- Conducting the in-situ tests
- Study of Ground water conditions and Collection of water sample forchemical analysis
- Geophysical Exploration (If required)
- Preparation of Drawings, Charts, etc
- Analysis of Data Collected
- Preparation of Report

Exploration Methods

1) Trial pits or test pits
2) Boring
3) probes (in situ test) and geophysical methods

Specific recommendations are made by Indian standards regarding the type, extent and details of subsurface explorations and the number, depth and spacing of boreholes for the following civil engineering works. Following is the list of various codes specified for the said purpose: Foundations of Multi-storeyed Buildings (IS: 1892, 1979)

Earth and rockfill Dams (IS: 6955, 1973) Power House Sites (IS: 10060, 1981)

Canals and Cross Drainage Works (IS: 11385,
1985) Ports and Harbours (IS: 4651 – Part 1,
1974)

The methods available for soil exploration may be classified as follows

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

The direct method of soil exploration usually consists of sinking a borehole at a predetermined location to the required depth by a method suitable for the site and to obtain fairly intact samples of soils from every stratum encountered or at suitably selected depths. The samples obtained are utilized to get necessary information about the soil characteristics by means of laboratory tests.

During recent years, indirect methods of soil exploration have also been used for civil engineering structures. These methods include various sounding and geophysical methods. In sounding methods, the variation in penetration resistance of sample or cone is utilized to interpret some of the physical properties of the strata. In geophysical methods, the change in subsoil strata are identified by measuring certain physical characteristics, e.g. electrical conductance, wave velocity of subsurface deposits. In addition to these methods, projectiles, probes, and aerial photographs are also useful in interpreting the soil characteristics.

Trial pits or test pits

Applicable to all types of soils

Provide for visual examination in their natural condition

Disturbed and undisturbed soil samples can be conveniently obtained at differentdepths

Depth of investigation: limited to 3 to 3.5 m.

Advantages

i) Cost effective

ii) Provide detailed information of stratigraphy

iii) Large quantities of disturbed soils are available for testing

iv) Large blocks of undisturbed samples can be carved out from the pits

v) Field tests can be conducted at the bottom of the pits

Disadvantages

i) Depth limited to about 6m

ii) Deep pits uneconomical

iii) Excavation below groundwater and into rock difficult and costlyiv)Too many pits may scar site and require backfill soils.

Limitations

i) Undisturbed sampling is difficult ii) Collapse in granular soils or below ground water table

Semi Direct Methods –Boring

Boring: Making or drilling bore holes into the ground with a view to obtaining soil or rocksamples from specified or known depths is called „boring"

The common methods of advancing bore holes are: Auger boring, Wash boring, rotary boringand Percussion boring.

Exploratory borings

Boring is carried out in the relatively soft and uncemented ground (engineering „soil") which is normally found close to ground surface. The techniques used vary widely across the world.

Location, spacing and depth of borings

It depends on:

i) Type of structure

ii) Size of the structure

iii) Weight coming from the General guidelines for location and depth of bore holes

Boreholes aregenerally located at

The building corners

The centre of the site

Where heavily loaded columns or machinery pads are proposed.

At least one boring should be taken to a deeper stratum, probably up to the bedrock if practicable other borings may be taken at least to significant stress level.

Spacing of Bore Holes – Codal Recommendations

According to IS 1892 (1979) Code of practice for subsurface investigation:

- For a small building one bore hole or test pit at the centre can give necessary data
- For a building covering not more than 4000 sq.m, one bore hole or test pit at each corner and one at centre is adequate.
- For a large project, the number will depend on its geological features and variation of strata. Generally a grid of 50 m spacing should be used with a combination of bore holes and sounding tests.

Depth of Investigation

The depth of investigation depends on the size and type of proposed structureSequence of proposed strata.

The depths of boreholes should cover the zone of soil that will be affected by the structural loads.There is no fixed rule to follow. In most cases, the depths of boreholes are governed by experience based on the geological character of the ground, the importance of the structure, the structural loads, and the availability of equipment

Guidelines for depth of investigation:

1. At least one boring should be taken to deeper stratum, probably up to the bedrock ifpracticable.
2. Borings should penetrate at least 3 m into rock.

3. Other borings may be taken at least to significant stress level.

4. In compressible soils such as clays, the borings should penetrate at least between I and 3 times the width of the proposed foundation or until the stress increment due to the heaviest foundation load is less than 10%, whichever is greater.

5. In very stiff clays, borings should penetrate 5-7 m to prove that the thickness of the stratum is adequate.

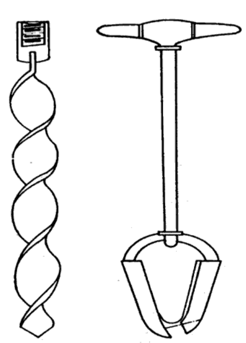
6. Borings must penetrate below any fills or very soft deposits below the proposed structure.

7. The minimum depth of boreholes should be 6 m unless bedrock or very dense material is encountered.

Significant depth The investigation shall be carried out to the point at which the vertical stress due to proposed structure is equal to or less than 10% of original effective stress at the point before the structure is constructed – significant depth

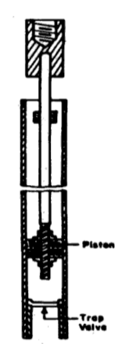
Methods of borings i) Auger boring – preferred for shallow depths , low ground water table ii) Wash boring: high water table, deeper soil deposit iii) Rotary drilling: high quality boring, also for rock drilling iv) Percussion drilling: fast drilling, not taking samples, gravel

Auger boring:-Augers are used in cohesive and other soft soils above water table. They may either be operated manually or mechanically. Hands augers are used up to a depth up to 6 m. mechanically operated augers are used for greater depths and they can also be used in gravelly soils. Augers are of two types: (a) spiral auger and (b) post-hole auger.



(a) HELICAL AUGER (b) POST-HOLE AUGER.

Hand Augers



Sand pump

Samples recovered from the soil brought up by the augers are badly disturbed and are useful for identification purposes only. Auger boring is fairly satisfactory bore explorations at shallow depths and for exploratory borrow pits.

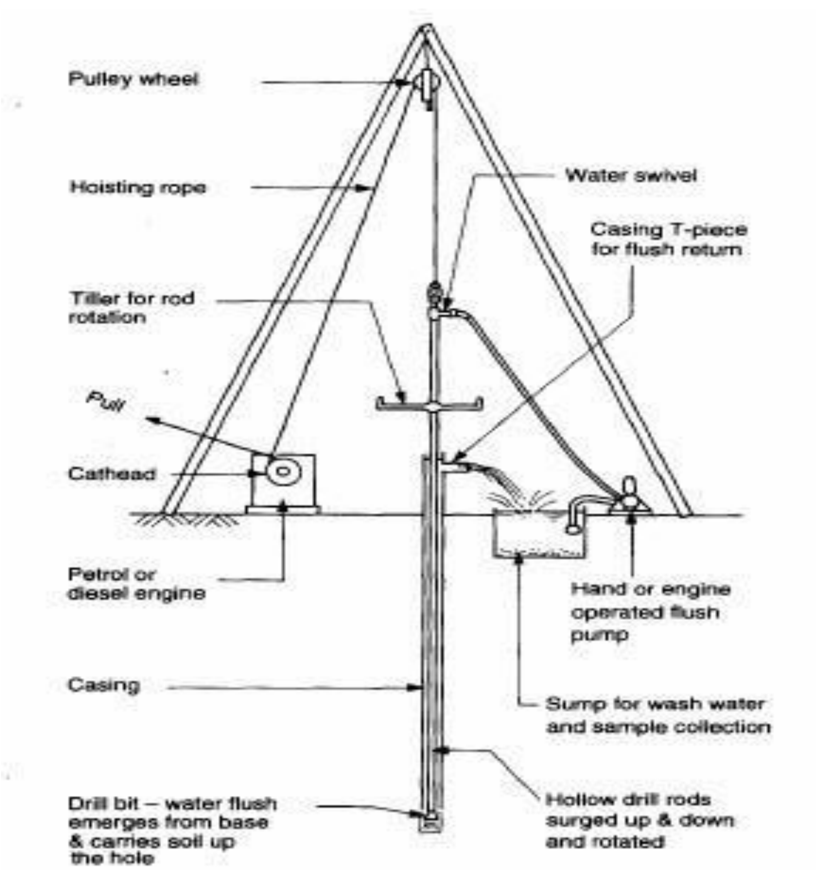
Auger and shell boring:- cylindrical augers and shells with cutting edge or teeth at Iower end can be used for making deep borings. Hand operated rigs are used for depths up to 25 m and mechanized rigs up to 50 m. Augers are suitable for soft to stiff clays, shells for very stiff and hard clays, and shells or sand pumps for sandy soils. 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.

Wash boring:-Wash boring is a fast and simple method for advancing holes in all types of soils. Boulders and rock cannot be penetrated by this method. The method consists of first driving a casing through which a hollow drilled rod with a sharp chisel or chopping bit at the lower end is inserted. Water is forced under pressure through the drill rod which is alternativety raised and dropped, and also rotated. The resulting chopping and jetting action of the bit and water disintegrates the soil. The cuttings are forced up to the ground surface in the form of soil-water
slurry through the annular space between the drill rod and the casing. The change in soil stratification could be guessed from the rate of progress and colour of wash water. The samples recovered from the wash water are almost valueless for interpreting the correct geo-technical properties of soil.

Percussion drilling:-In this method, soil and rock formations are broken by repeated blows of heavy chiesel or bit suspended by a cable or drill rod. Water is added to the hole during boring, ifnot already present and the slurry of pulverised material is bailed out at intervals. The method is suitable for advancing a hole in all types of soils, boulders and rock. The formations, however, get disturbed by the impact.

Rotary boring:- Rotary boring or rotary drilling is a very fast method of advancing hole in both rocks and soils. A drill bit, fixed to the lower end of the drill rods, is rotated by a suitable chuck, and is always kept in firm contact with the bottom of the hole. A drilling mud, usually a water solution of bentonite, with or without other admixtures, is continuously forced down to the hollow drill rods. The mud returning upwards brings the cuttings to the surface. The method is also known as mud rotary drilling and the hole usually requires no casing.

Rotary core barrels, provided with commercial diamond-studded bits or a steel bit with shots, are also used for rotary drilling and simultaneously obtaining the rock cores or samples. The method is them also known as core boring or core drilling. Water 15 circulated down drill rods during boring.



Wash boring

Soil Samples and samplers.

Types of Samples

Samples of soil taken out of natural deposits for testing may be classified as:

- Disturbed sample
- Undisturbed sample

A disturbed sample is that in which the natural structure of the soil gets modified partly or fully during sampling and an undisturbed sample is that in which the natural structure and other physical properties remain preserved.

Disturbed but representative samples can generally be used for v Grain-size analysis v Determination of liquid and plastic limits, Specific gravity of soil solids, Organic content determination and Soil classification Undisturbed samples must be used for -- Consolidation test, Hydraulic conductivity test and Shear strength test There is an increasing variety of plant, sampling methods and tools, with particular advantages in cost, quality of sampling, speed of operation, use in conditions of limited access or headroom, etc., and the choice of rig is affected by the likely soil conditions to be encountered.

Spacing of Borings

Type of project	**Spacing (m)**
Multistory buildings	10 – 30
One-story industrial plants	20 – 60
Highways	250 – 500
Residential subdivision	250 – 500
Dams and dikes	40 – 80

Soil Sampling

Need for sampling: -Sampling is carried out in order that soil and rock description, and laboratory testing can be carried out.

Laboratory tests typically consist of:

i). Index tests (for example, specific gravity, water content)

ii). Classification of tests (for example, Atterberg limit tests on clays); and

iii) Tests to determine compressibility, and permeability.

Factors to be considered while sampling soil

i) Samples should be representative of the ground from which they are taken.

ii) They should be large enough t and fissuring and fracturing.

iii) They should be taken in such a way that they have not lost fractions of the situ soil(for compressibility disturbance as possible.

Type of soil samples

Non-Representative samples:-Non-Representative soil samples are those in which neither thein-situ soil structure, moisture content nor the soil particles are preserved.

• They are not representative

• They cannot be used for any tests as the soil particles either gets mixed up or some particlesmay be lost.

• e.g., Samples that are obtained through wash boring or percussion drilling.

Disturbed soil samples:- Disturbed soil samples are those in which the in-situ soil structure and moisture content are lost, but the soil particles are intact.

• They are representative

• They can be used for grain size analysis, liquid and plastic limit, specific gravity, compaction tests, moisture content, organic content determination and soil classification test performed in thelab

• e.g., obtained through cuttings while auguring, grab, split spoon (SPT), etc.

Undisturbed soil samples:-Undisturbed soil samples are those in which the in-situ soilstructure and moisture content are preserved.

• They are representative and also intact

• These are used for consolidation, permeability or shear strengths test (Engineering properties) •More complex jobs or where clay exist

• In sand is very difficult to obtain undisturbed sample

• Obtained by using Shelby tube (thin wall), piston sampler, surface (box), vacuum, freezing,etc., Design Features affecting the sample disturbance

- Area ratio
- Inside Clearance
- Outside Clearance
- Recovery Ratio
- Inside wall friction
- Design of non-return value
- Method of applying force
- sizes of sampling tubes

Inside clearance ratio $C_i = \frac{D_s - D_c}{D_c} \times 100\ \%$

The soil is under great stress as it enters the sampler and has a tendency to laterally expand. The inside clearance should be large enough to allow a part of lateral expansion to take place, but it should not be so large that it permits excessive deformations and causes disturbances of the sample.

For good sampling process, the inside clearance ratio should be within 0.5 to 3 %.

For sands silts and clays, the ratio should be 0.5 % and for stiff and hard clays (below water table), it should be 1.5 %.

For stiff expansive type of clays, it should be 3.0 %.

Area ratio $A_r = \frac{D^2 - D_c^2}{D_c^2} \times 100\%$

Outside clearance ratio $C_o = \frac{D_w - D_r}{D_r} \times 100\%$

For good sampling process, the ratio should be within 0-2 %. Minimum inside diameter = = 75mm. The length (L) should be at least equal to (the intended length + 100mm) for residual soils.

The tube should be uniform and should not have any protrusions or irregularities. The inside ofthe tube should be clean and smooth.

Area ratio

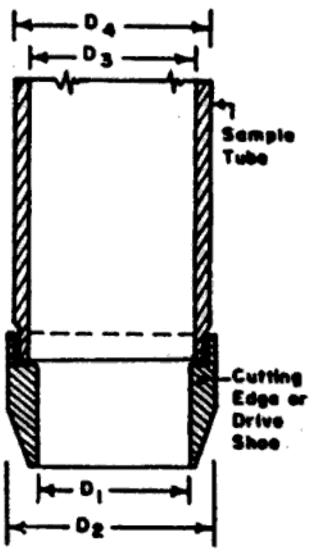
$$= \frac{D_2^2 - D_1^2}{D_1} \times 100$$

Inside clearance

$$= \frac{D_3 - D_1}{D_1} \times 100$$

Outside clearance

$$= \frac{D_2 - D_4}{D_4} \times 100$$



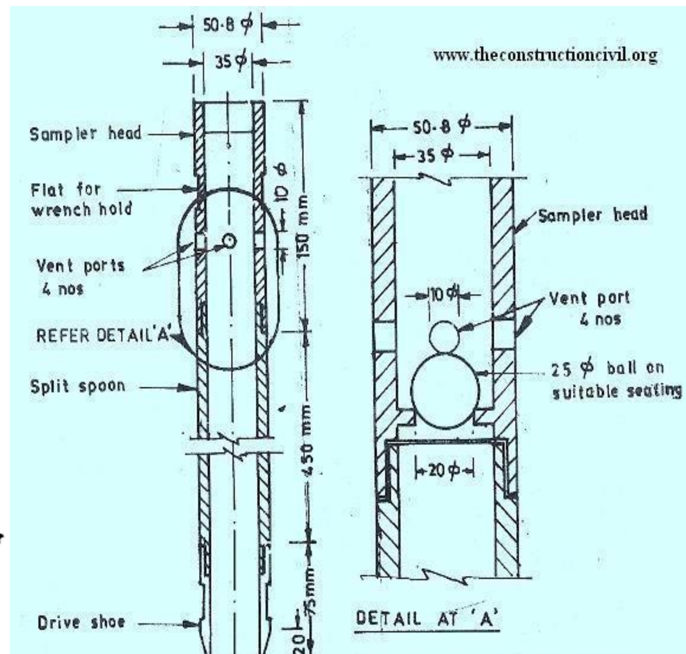


Fig. Standard Split Spoon Sampler

Penetration Tests

These tests involve the measurement of the resistance to penetration of a sampling spoon, a cone or other shaped tools under dynamic or static loadings. The resistance is empirically correlated with some of the engineering properties of soil as density index, consistency, bearing capacity, etc., These tests are useful for general exploration of erratic soil profiles, for finding depth to bedrock or hard stratum, and to have an approximate indication of the strength and other properties of soils, particularly the cohesionless soils, from which it is difficult to obtain undisturbed samples. The two commonly used tests are the standard penetration test and the cone penetration test.

Standard Penetration Test

- Test is performed in a clean hole,55 OD to 150 IDmm in diameter
- A thick all split tube sampler,50.8 mm and 35 mm is driven into the undistributed soil atthe bottom of the hole
- A 65kgdrive weight with 75cm free fall is used to drive the sampler
- The sampler is first driven through 15cm as a seating drive
- It is further driven through 3cm
- The number of blows required to drive the sampler 30cm beyond the seating drive istermed as penetration resistance N.
- In very fine silty saturated sand an apparent increase in resistance occurs
- For overburden pressure on the value of N(Terzaghi and Peck) $N_o = 15 + \frac{1}{2}(N-15)$
- For air-dry or moist sand, $N_o = N \frac{50}{1.42\sigma' + 10}$

Where,Ne= corrected value for overburden effectN= actual value of blows

σ'=effective overburden pressure in t/m^2 (not to exceed 28.2 t/m^2)Meyerhoff$\emptyset$= 25 + 0.15 I_D fines greater than 5 %

Φ= 20 + 0.15 I_D fines less than 5 %

Dutch Cone Test

- Test is used for getting a continuous record of the resistance of soil by penetratingsteadily under static pressure
- A cone with a base of 10cm^2 and an angle of 60 °at the vertex
- The cone is carried at the lower end of a steel driving rod
- To know the cone resistance, the cone alone is first forced down upto 8cm and resistanceis recorded
- The steel tube is then pushed down upto the cone, and both together are furher penetrated upto 20cm
- Cone test is useful in determining the bearing capacity of pits in cohesionless soils, particularly fine sands

- The cone resistance(kg/cm^2) is approximately equal to 10 times the penetration resistanceN

Soil investigation is required for the following purposes –

- To know the allowable bearing capacity of foundation for proposed building.
- To know the depth and type of foundation for the proposed building.
- To know the allowable passive resistance for the foundation of proposed building.
- To know the type, grading and nature of soil.
- To know the ground water level.

Methods of soil investigation

The common methods of soil investigation are –

- Inspection
- Test pits
- Probing, and
- Boring.

Inspection: In some places you don't have to investigate much. You'll get enough data to design the foundation of the proposed building by just inspecting the plot. This method of soil investigation includes know the geological condition of the plot, getting data about neighbor buildings, their foundation type and depth, etc.

Test pits: This is done to collect soil samples for detail analysis. In this method several pits are dug by hand or excavator. The depth of pit is below 5 feet so that one can have visual inspection.Several samples are collected from the pit of both disturbed and undisturbed soil.

Probing: In this method a 25 mm or 40 mm diameter steel bar is driven into the ground till solid soil strata is found. It is normally driven by hammer. The penetration and withdrawal of the steel rod is closely observed to know the nature of soil layer.

Boring: In this method several bore holes are made for the purpose of collecting soil sample from below the ground. Then the collected sample is analyzed for preparing the soil report.

Typical steps of soil investigation

Soil investigation involves following steps -

- Details planning for the sequence of operations
- Collecting the samples of soil from the plot.
- Determining the soil characteristics by conducting field tests.
- Study the condition of ground water level.
- Collecting ground water sample for chemical analysis.
- Soil exploration.
- Testing all collected samples in the laboratory.
- Analysis the test results.
- Preparing report.

A recommended procedure is as follows:

- Collect data, categorize it and rough out a preliminary draft.
- Edit the draft and seek methods of visual presentation and tabulation.
- Polish re-draft and check for improvements in presentation check for typing errors andappearance.

Factors affecting quality of report.

There are other factors which can affect the quality of the investigation, recommendations and the engineering judgment. Among those which may affect some engineers are:

(1) Uncritical acceptance of well-presented opinion, results of sophisticated (but not necessarilyrelevant) tests and over- and unqualified respect for some specialists.

(2) Allowing site difficulties to dictate the investigation in an attempt to keep the investigationsimple and cheap.

(3) Lack of recognition that piling and other foundation techniques can be used to economicadvantage even on good sites.

(4) Lack of recognition that some fills, possibly upgraded by ground improvement techniques,can provide adequate and economic bearing strata.

(5) Lack of appreciation that advances in structural design can accommodate relatively highsettlements.

(6) Under-estimation of the importance of the designer, at least, visiting the site during theinvestigation or dismissal of trial pits as unscientific or out-dated.

Sequence of report

Foundation reports follow the normal sequence of items of engineering reports in having a title, contents list, and synopsis, and introduction, body of the report, conclusions and recommendations. Lengthy descriptions of tests and similar matters are best dealt with in appendices and the test results tabulated in the body of the report. The client tends to read the synopsis and recommendations; the main and sub-contractors concentrate on the body of the report and the design office on its conclusions and recommendations.

Site description

This, as far as possible, should be given on small-scale plans showing site location, access and surrounding area. The proposed position of the buildings and access roads should be shown. The site plan should also show the general layout and surface features, note presence of existing buildings, old foundations and previous usage, services, vegetation, surface water, any subsidence or unstable slopes, etc.

Written description of the site exposure (for wind speed regulations) should be given together with records of any flooding, erosion and other geographical and hydrographic information.

Geological maps and sections should, when they are necessary, be provided, noting mines, shafts, quarries, swallow holes and other geological features affecting design and construction.

Photographs taken on the site, preferably color ones, can be very helpful and should be supplemented by aerial photographs if considered necessary.

The ground investigation

(1) Background study and location of holes. This should give a full account of the desk-top study, examination of old records, information from local authorities, public utilities and the like, and the field survey. It should detail the position and depth of trial pits and boreholes, equipment used and in situ testing and information.

(2) Boreholes, trial pits and soil profiles. This section will be mainly a visual presentation of the logs and profiles together with colour photographs of the trial pits.Where possible, written information should be given in note form on the soil profiles.

(3) Soil tests. This should list the site and laboratory tests drawing attention to any unusual, unexpected or special results. The results of the tests should be tabulated, for ease of reference, and diagrams of such information as particle size distribution, pressure–void ratio curves and Mohr‟s circles should be given.

Results

This must give details of ground conditions, previous use of site, present conditions, groundwaterand drainage pattern

The tests must give adequate information to determine the soil‟s bearing capacity, settlement characteristics, behavior during and after foundation construction and, where necessary, its chemical make-up and condition

Recommendations This is both comment on the facts and also opinions based on experience; the difference should be made clear. Since the discussion is usually a major part of the report it should be broken down into sections for ease of reference and readability. The final section should give firm recommendations on the foundation type or types to be adopted

Objectives of Site Investigation

- To access the general suitability of the site.
- To achieve safe and economical design of foundations and temporary works.
- To know the nature of each stratum and engineering properties of the soil and rock, which may affect the design and mode of construction of proposed structure and foundation.
- To foresee and provide against difficulties that may arise during construction due to ground and other local conditions.
- To find out the sources of construction material and selection of sites for disposal of water or surplus material.
- To investigate the occurrence or causes of all natural and man made changes in conditions and the results arising from such changes.
- To ensure the safety of surrounding existing structures.
- To design for the failed structures or remedial measures for the structures deemed to beunsafe.
- To locate the ground water level and possible corrosive effect of soil and water onfoundation material.

Methods of site exploration

The various types of site investigation are:

Open excavation, Boring, Subsurface Sounding and Geophysical Methods

Geophysical exploration

Geophysical exploration may be used with advantage to locate boundaries between different elements of the subsoil as these procedures are based on the fact that the gravitational, magnetic, electrical, radioactive or elastic properties of the different elements of the subsoil may be different. Differences in the gravitational, magnetic and radioactive properties of deposits near the surface of the earth are seldom large enough to permit the use of these properties in exploration work for civil engineering projects. However, the resistivity method based on the electrical properties and the seismic refraction methods based on the elastic properties of the deposits have been used widely in large civil engineering projects. Different methods of geophysical explorations

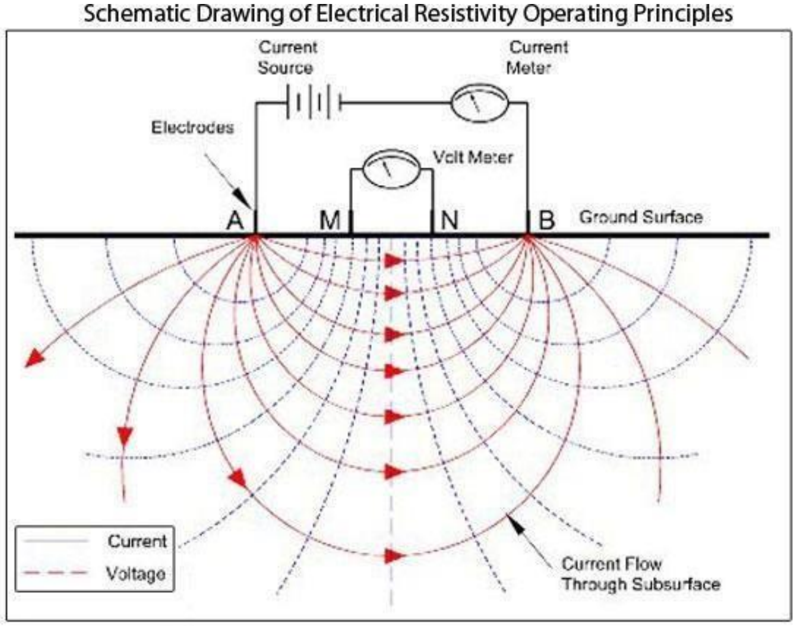
Electrical resistivity method

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

$$\rho = \frac{\mathbf{RA}}{\mathbf{L}}$$

ρ is resistivity in ohm-cm, R is resistance in ohms, A is the cross sectional area (cm 2), L islength of the conductor (cm).

Schematic Drawing of Electrical Resistivity Operating Principles



Applications of resistivity soundings are:

Characterize subsurface hydrogeology, Determine depth to bedrock/overburden thickness, Determine depth to groundwater, Map stratigraphy, clay aquitards, salt-water intrusion and vertical extent of certain types of soil and groundwater contamination .Estimate landfill thickness

Resistivity profiling is used to:

Map faults, Map lateral extent of conductive contaminant plumes, Locate voids, Map heavy metals soil contamination ,Delineate disposal areas ,Map paleochannels, Explore for sand and gravel ,Map archaeological sites

Seismic Method

Seismic refraction is a geophysical method used for investigating subsurface ground conditions utilizing surface-sourced seismic waves. The methods depend on the fact that seismic waves have differing velocities in different types of soil (or rock): in addition, the waves are refracted when they cross the boundary between different types (or conditions) of soil or rock. The methods enable the general soil types and the approximate depth to strata boundaries, or to bedrock, to be determined.

Operation

Pulses of low frequency seismic energy are emitted by a seismic source such as a hammer-plate, weight drop or buffalo gun. The type of source is dependent on local ground conditions and required depth penetration. Explosives are best for deeper applications but are constrained by environmental regulations.

The seismic waves propagate downward through the ground until they are reflected or refracted off subsurface layers. Refracted waves are detected by arrays of 24 or 48 geophones spaced at regular intervals of 1 - 10 metres, depending on the desired depth penetration of the survey. Sources are positioned at each end of the geophone array to produce forward and reverse wave arrivals along the array. Additional sources may be used at intermediate or off-line positions for full coverage at all geophone positions.

A **geophone** is a device that converts ground movement (velocity) into voltage, which may be recorded at a recording station. The deviation of this measured voltage from the base line is called the seismic response and is analyzed for structure of the earth.

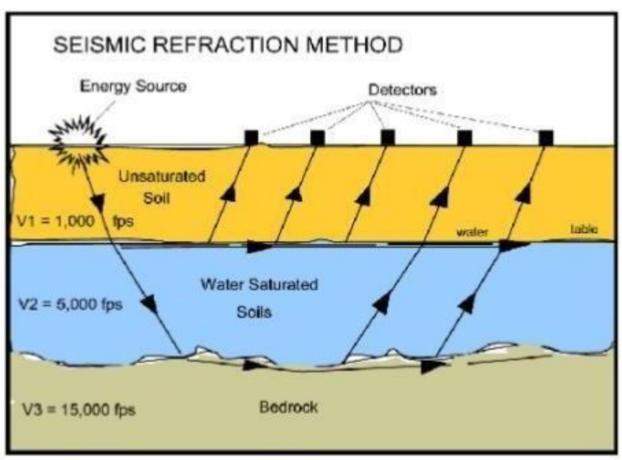
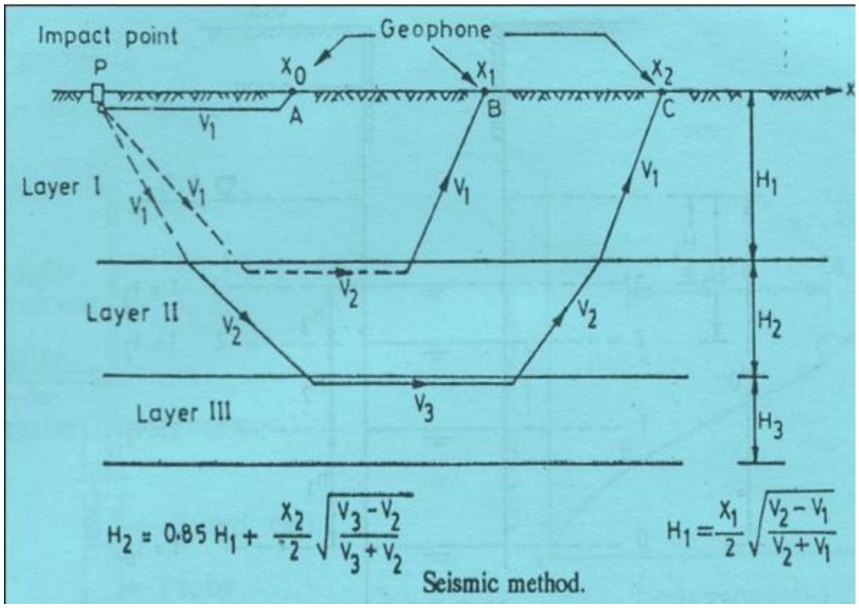
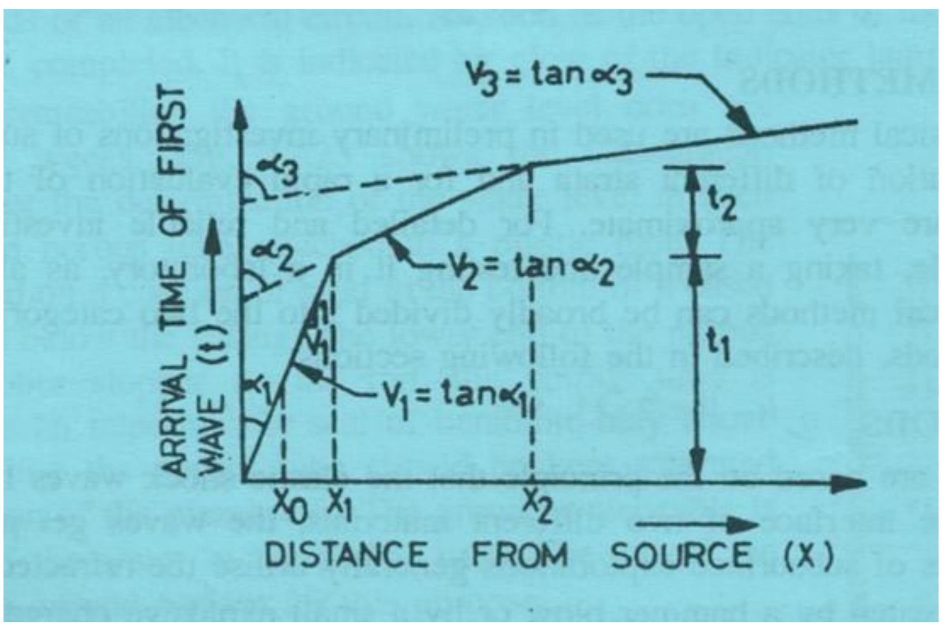


Fig. seismic refraction method





APPLICATIONS

- Measures Bedrock Depth & Overburden Thickness
- Determines Rip ability Parameters
- Investigates Pipeline Routes
- Locates Geological Structures

- Evaluates Sand & Gravel Deposits
- Defines Ancient Landfill Sites

Boring Log

During soil exploration all suitable details are recorded and presented in a boring log. Additionalinformation consisting mainly of lab and field test result is added to complete the boring log.

Details of Boring Log

The ground conditions discovered in each borehole are summarised in the form of a bore log. The method of investigation and details of the equipment used should be stated on each log. The location, ground level and diameter of the hole should be specified. The names of the client and the project should be mentioned.

Other Details of Boring Log

- The soil profile with elevations of different strata.
- Ground water level.
- Termination level of the bore hole.
- The depth at which samples were taken or at which in-situ tests were performed.
- The type of soil samples.
- N-values at the measured elevation.
- The results of important laboratory tests

Soil Exploration Report

At the end of the soil exploration program, the soil and rock samples, collected from the field aresubjected to visual observation and laboratory tests. Then, a soil exploration report is prepared for use by the planning and design office. Any soil exploration report should contain thefollowing information:

1. Scope of investigation
2. General description of the proposed structure for which the exploration has been conducted

3. Geological conditions of the site
4. Drainage facilities at the site
5. Details of boring
6. Description of subsoil conditions as determined from the soil and rock samples collected
7. Ground water table as observed from the boreholes
8. Details of foundation recommendations and alternatives
9. Any anticipated construction problems
10. Limitations of the investigation

The following graphic presentations also need to be attached to the soil exploration report:

1. Site location map
2. Location of borings with respect to the proposed structure
3. Boring logs
4. Laboratory test results
5. Other special presentations

The boring log is the graphic representation of the details gathered from each bore hole.

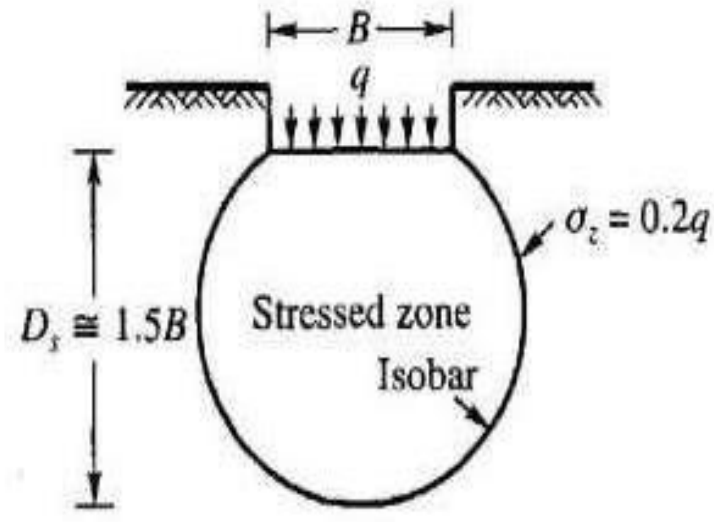
Depth of Exploration

• Depth of soil exploration should up to to which the increase in pressure due to structural loading will have no damaging effect (such as settlement & shear failure) on the structure. This depth is termed as significant depth.

Factors affecting Significant Depth

• Type of structure
• Weight of structure
• Dimension of structure
• Disposition of the loaded area
• Soil profile and layer properties

Significant Depth



Bearing Capacity Failure / Shear Failure

• Failure in which movement caused by shearing stresses in a soil or rock mass.

• Bearing Capacity Failure is a foundation Failure that occurs when shear stresses in soil exceedsthe shear strength of soil.

Types of Shear Failure

1. General Shear Failure:

• It occurs in dense or hard state soils with relative density greater than 70%.

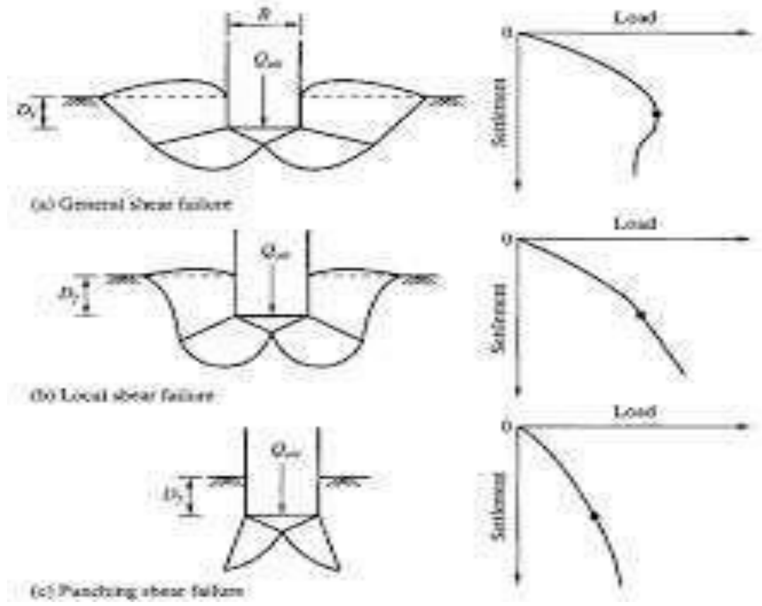
• It involves total rupture of the underlying soil.

• Continuous Shear failure of soil from below the footing to the ground surface.

• Distinct load at which the foundation fails.

• This failure ruptures and pushes up the soil on both sides of the footing.

• In practical, soil often pushes up on only one side of the footing with subsequent tilting of thestructure.



Local Shear Failure:

• It occurs for soils that are in medium dense state with relative density between 35 % to 70 %.

• It involves rupture of the soil only immediately below the footing.

• Bulging of soil appears on both sides of the footing but the bulging is not as significant as ingeneral shear.

• It is considered as a transitional phase between General Shear and Punching Shear.

Punching shear failure:

• It occurs in loose soils with relative density less than 35%.

• It involves compression of soil directly below the footing as well as vertical shearing of soilaround the footing perimeter.

• It does not develop the distinct shear surfaces associated with a general shear failure (no bulgingand no tilting)

• Soil outside the loaded area remains relatively unaffected.

Criteria for the design of foundation

• Failure of foundation should not occur due to shear failure

• Differential settlement and Total settlement in a structure should not affect the function of thestructure during its life time

BEARING CAPACITY

- **Ultimate Bearing Capacity (qu)**
  Minimum Gross pressure at the base of foundation at which soil fails in shear.
- **Gross Pressure (q)**
  Total pressure at the base of the footing due to the weight of the superstructure,self-weight of the footing and weight of the earth fill.
- **Net Pressure Intensity (qn)**
  differences in intensities of the gross pressure and original overburden pressure.
- **Net Ultimate Bearing Capacity (qnu)**
  minimum net pressure intensity which causes shear failure of soil.
- **Safe Bearing Capacity (qsafe)**
  Maximum pressure intensity which the soil can carry safely without the risk ofshear failure.
- **Allowable bearing capacity**
  Net loading intensity at which neither the soil fails in shear nor these is excessivesettlement detrimental to the structure.

Terzaghi's Analysis

Based on Terzaghi's bearing capacity theory, column load P is resisted by shear stresses at edges of three zones under the footing and the overburden pressure, q (=γD) above the footing. The first term in the equation is related to cohesion of the soil. The second term is related to the depth of the footing and overburden pressure. The third term is related to the width of the footing and the length of shear stress area. The bearingcapacity factors, Nc, Nq, Nγ, are function of internal friction angle, φ.

Terzaghi's Bearing capacity equations:

Strip footings: $Qu = c\ Nc + \gamma\ D\ Nq + 0.5\ \gamma\ B\ N\gamma$

Square footings: $Qu = 1.3\ c\ Nc + \gamma\ D\ Nq + 0.4\ \gamma$

$B\ N\gamma$ Circular footings: $Qu = 1.3\ c\ Nc + \gamma\ D\ Nq +$

$0.3\ \gamma\ B\ N\gamma$

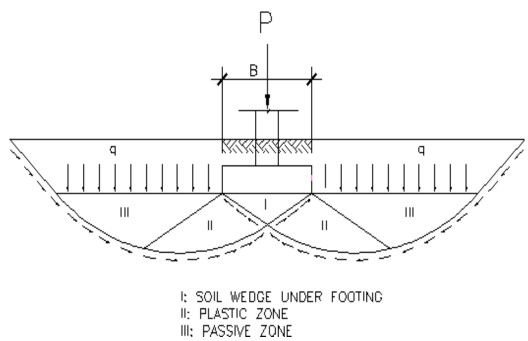


Fig: Zones of plastic equilibrium in Terzaghi analysis

Where: C: Cohesion of soil, γ : unit weight of soil,

D: depth of footing, B: width of footing

Nc, Nq, Nr: Terzaghi's bearing capacity factors depend on soil friction angle,

φ.Nc = cotφ(Nq –1)

Nq = e 2 (3π/4-φ/2)tanφ / [2 cos2(45+φ/2)] Nγ

= (1/2) tanφ(Kpr/cos 2 φ -1)

Kpr = passive pressure coefficient.

Meyerhof's bearing capacity theory

Meyerhof's general bearing capacity equationsVertical load:

$Qu = c\ Nc\ Sc\ Dc + \gamma\ D\ Nq\ Sq\ Dq + 0.5\ \gamma\ B\ N\gamma\ S\gamma\ D\gamma$

Inclined load:

$Qu = c\ Nc\ Sc\ Dc\ Ic + \gamma\ D\ Nq\ Sq\ Dq\ Iq + 0.5\ \gamma\ B\ N\gamma\ S\gamma\ D\gamma\ I\gamma$

Where: Nc, Nq, Nr: Meyerhof's bearing capacity factors depend on soil friction angle, φ.$Nc = \cot\varphi\ (Nq - 1)$

$Nq = e\pi\tan\varphi \quad \tan2$
$(45+\varphi/2)]\ N\gamma = (Nq-1)$
$\tan(1.4\varphi)$

Sc, Sq, Sγ: shape
factors Dc, Dq, Dγ:
depth factors
Ic, Iq, Iγ: incline load factors

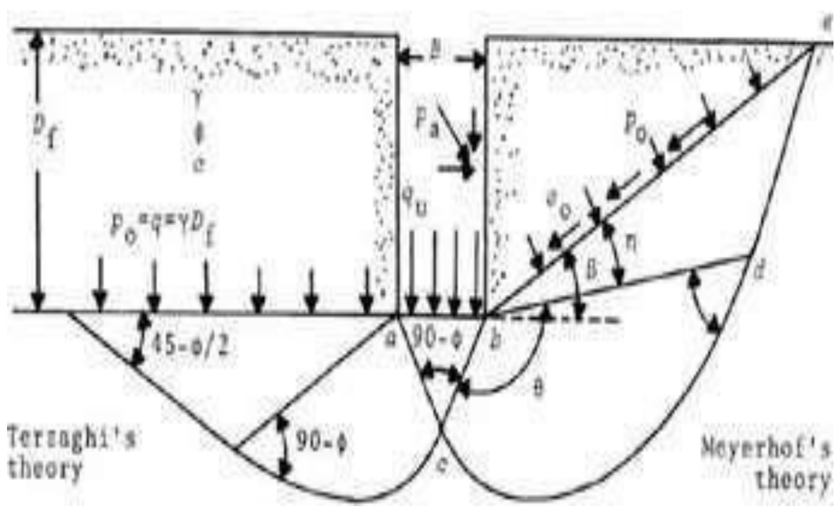


Fig:Comparison between Terzaghi and meyerhof analysis

Any φ,

$Sc=1+0.2Kp(B/L)$

$Dc=1+0.2\sqrt{Kp}\ (B/L)$

$Ic=Iq=(1-\theta/90°)\ 2\ \varphi$

$= 0\ Sq=S\gamma=1$

$Dq=D\gamma=1\ I\gamma=1$

Φ ≥10°

$Sq=S\gamma=1+0.1Kp(B/L)$

$Dq=Dr=1+0.1\sqrt{Kp}\ (D/B)$

$I\gamma=(1-\theta/\varphi)\ 2$

C: Cohesion of soil γ : unit weight of soilD: depth of footing
B, L: width and length of footing
Kpr = tan2 (45+φ/2), passive pressure coefficient. θ = angle of axial load to vertical axis

Design Principles of Shallow Foundation

- Thickness of footing should be sufficient enough to take care of shear force without providingshear reinforcement.
- Reinforcement should be provided for Bending Moment without compression reinforcement
- The footing should be very rigid so that settlement of foundations may be uniform
- Adequate Bond Strength and transfer of load by bearing should be ensured

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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT –II - DESIGN OF SHALLOW FOUNDATIONS– SCIA7007

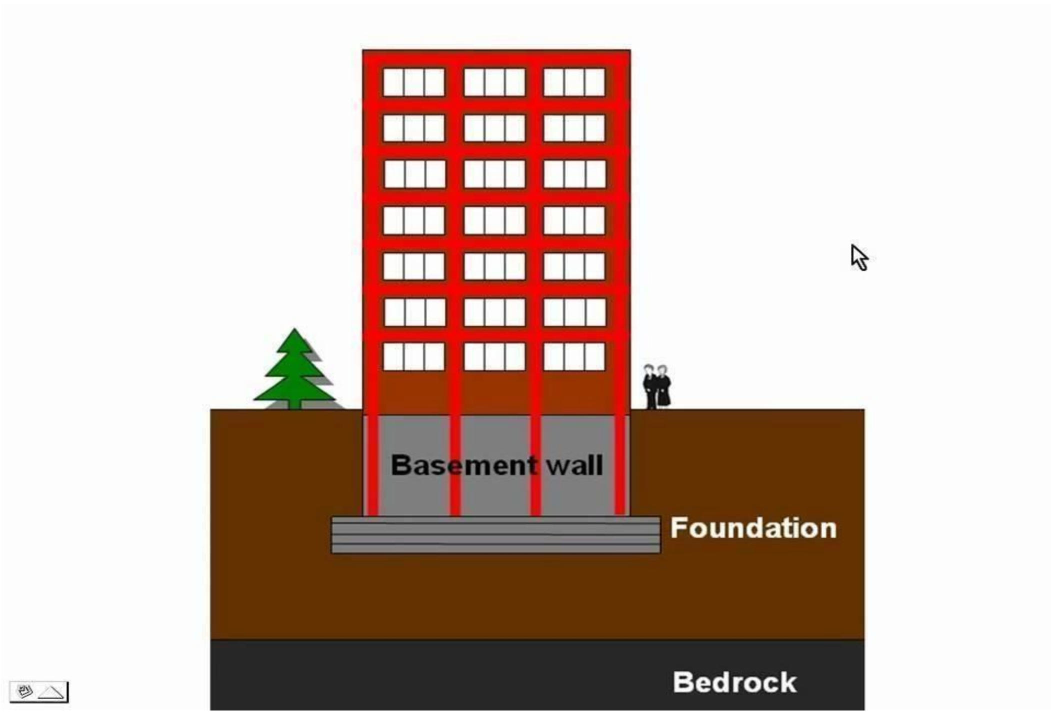
UNIT II

Design of Shallow Foundations

INTRODUCTION:

Footings are structural elements that transmit column or wall loads to the underlying soil below the structure. Footings are designed to transmit these loads to the soil without exceeding its safe bearing capacity, to prevent excessive settlement of the structure to a tolerable limit, to minimize differential settlement, and to prevent sliding and overturning. The settlement depends upon the intensity of the load, type of soil, and foundation level. Where possibility of differential settlement occurs, the different footings should be designed in such a way to settle independently of each other.

Foundation design involves a soil study to establish the most appropriate type of foundation and a structural design to determine footing dimensions and required amount of reinforcement. Because compressive strength of the soil is generally much weaker than that of the concrete, the contact area between the soil and the footing is much larger than that of the columns and walls.



UNIT II

DESIGN OF FOOTINGS

General

Most of the structures built by us are made of reinforced concrete. Here, the part of the structure above ground level is called as the superstructure, where the part of the structure below the ground level is called as the substructure. Footings are located below the ground level and are also referred as foundations. Foundation is that part of the structure which is in direct contact with soil. The R.C. structures consist of various structural components which act together to resist the applied loads and transfer them safely to soil. In general the loads applied on slabs in buildings are transferred to soil through beams, columns and footings. Footings are that part of the structure which are generally located below ground Level. They are also referred as foundations. Footings transfer the vertical loads, Horizontal loads, Moments, and other forces to the soil.

The important purpose of foundation are as follows;

i. To transfer forces from superstructure to firm soil below.
ii. To distribute stresses evenly on foundation soil such that foundation soil neither fails nor experiences excessivesettlement.
iii. To develop an anchor for stability against overturning.
iv. To provide an even surface for smooth construction of superstructure.

Due to the loads and soil pressure, footings develop Bending moments and Shear forces. Calculations are made as per the guidelines suggested in IS 456 2000 to resist the internal forces.

Types of Foundations

Based on the position with respect to ground level, Footings are classified into twotypes;

- Shallow Foundations
- Deep Foundations

Shallow Foundations are provided when adequate SBC is available at

relatively short depth below ground level. Here, the ratio of Df / B < 1, where Df is the depth of footing and B is the width of footing. Deep Foundations are provided when adequate SBC is available at large depth below ground level.
Here the ratio of Df / B >= 1.

Types of Shallow Foundations

The different types of shallow foundations are as follows:

- Isolated Footing
- Combined footing
- Strap Footing
- Strip Footing
- Mat/Raft Foundation
- Wall footing

Some of the popular types of shallow foundations are briefly discussed below.

a) Isolated Column Footing

A spread footing (or isolated or pad) footing is provided to support an individual column. A spread footing is circular, square or rectangular slab of uniform thickness. Sometimes, it is stepped or hunched to spread the load over a large area. These are independent footings which are provided for each column. This type of footing is chosen when

- SBC is generally high
- Columns are far apart
- Loads on footings are less

The isolated footings can have different shapes in plan. Generally it depends on the shape of column cross section.

Some of the popular shapes of footings are;

- Square
- Rectangular
- Circular

The isolated footings essentially consist of bottom slab. These bottom Slabs can be flat, stepped or sloping in nature. The bottom of the slab is reinforced with steel mesh to resist the two internal forces namely bending moment and shear force.

The sketch of a typical isolated footing is shown in Fig. 1.

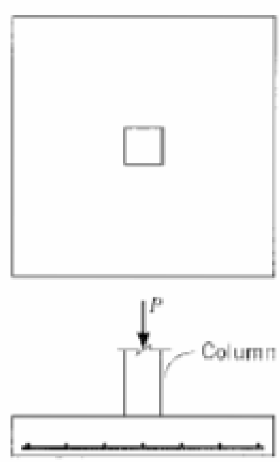


Fig. 1 Plan and section of typical isolated footing

b) <u>Combined Column Footing</u>

A combined footing supports two columns. It is used when the two columns are so close to each other that their individual footings would overlap. A combined footing is also provided when the property line is so close to one column that a spread footing would be eccentrically loaded when kept entirely within the property line. By combining it with that of an interior column, the load is evenly distributed. A combined footing may be rectangular or trapezoidal in plan.

These are common footings which support the loads from 2 or more columns. Combined footings are provided when

- SBC is generally less
- Columns are closely spaced
- Footings are heavily loaded

In the above situations, the area required to provide isolated footings for the columns generally overlap. Hence, it is advantageous to provide single combined footing. In some cases the columns are located on or close to property line. In such cases footings cannot be extended on one side. Here, the footings of exterior and interior columns are connected by the combined footing.

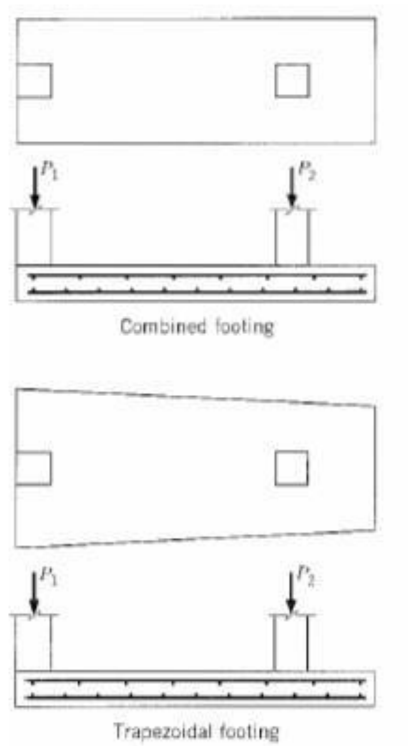


Fig. 2 Plan and section of typicalcombined footing

Combined footings essentially consist of a common slab for the columns it is supporting. These slabs are generally rectangular in plan. Sometimes they can also be trapezoidal in plan (refer Fig. 2). Combined footings can also have a connecting beam and a slab arrangement, which is similar to an inverted T – beam slab.

c) <u>Strap Footing</u>

A strap (or cantilever) footing consists of two isolated footings connected with a structural strap or a lever. The strap connects the two footings such that they behave as one unit. The strap is designed as a rigid beam. An alternate way of providing combined footing located close to property line is the strap footing. In strap footing, independent slabs below columns are provided which are then connected by a strap beam. The strap beam does not remain in contact with the soil and does not transfer any pressure to the soil. Generally it is used to combine the footing of the outer column to the adjacent one so that the footing does not extendin the adjoining property. A typical strap footing is shown in Fig. 3.

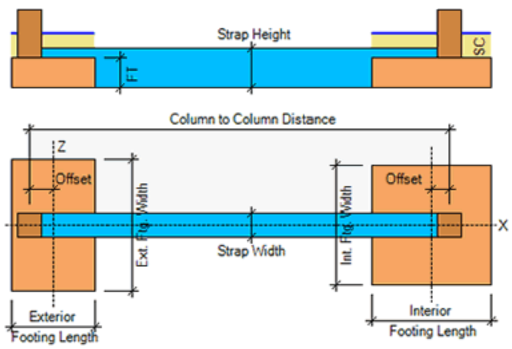


Fig. 3 Plan and section of typical strapfooting

d) Strip Footing

A strip footing is provided for a load-bearing wall. A strip footing is also provided for a row of columns which are so closely spaced that their spread footings overlap or nearly touch each other. Strip footing is a continuous footing provided under columns or walls. A typical strip footing for columns is shown in Fig. 4.

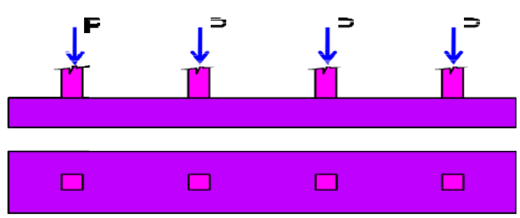


Fig. 4 Plan and section of typical stripfooting

e) Mat Foundation

A mat or raft foundation is a large slab supporting a number of columns and walls under the entire structure or a large part of the structure. A mat is required when the allowable soil pressure is low or where the columns and walls are so close that individual footings would overlap or nearly touch each other.

Mat foundations are useful in reducing the differential settlements on non- homogeneous soils or where there is a large variation in the loads on individual columns. Mat foundation covers the whole plan area of structure.

The detailing is similar to two way reinforced solid floor slabs or flat slabs. It is a combined footing that covers the entire area beneath a structure and supports all the walls and columns. It is normally provided when

- Soil pressure is low
- Loads are veryheavy
- Spread footings cover > 50% area

A typical mat foundation is shown in Fig. 5.

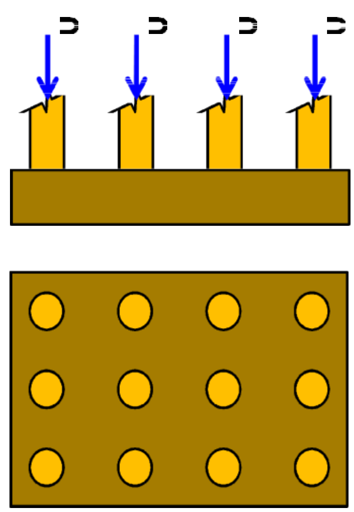


Fig. 5 Plan and section of typical Mat footing

ASSUMPITIONS IN CONVENTIONAL DESIGN OF RAFT FOUNDTAION:

i. In the conventional method of design, the raft is assumed to be infinitely rigid and the pressure distribution is taken as linearly varying.
ii. The assumption is valid when the raft rests on soft clay which is highlycompressible and the eccentricity of the load is small.
iii. In case when the soil is stiff or the eccentricity is large, the method does not giveaccurate results.

Types of Deep Foundations

Deep foundations are provided when adequate SBC is available at large depth belowGL.

There are different types of deep foundations. Some of the common types of deepfoundations are listed below.

- Pile Foundation
- Pier Foundation
- Well Foundation

Bearing Capacity of Soil

The safe bearing capacity of soil is the safe extra load soil can withstand without experiencing shear failure. The Safe Bearing Capacity (SBC) is considered unique at a particular site. But it also depends on the following factors:

- Size of footing
- Shape offooting
- Inclination offooting
- Inclination ofground
- Type of load
- Depth of footingetc.

SBC alone is not sufficient for design. The allowable bearing capacity is taken as thesmaller of the following twocriteria

- Limit states of shear failure criteria (SBC)
- Limit states of settlement criteria

Based on ultimate capacity, i.e., shear failure criteria, the SBC is calculated as

SBC = Total load / Area of footing

Usually the Allowable Bearing Pressure (ABP) varies in the range of 100 kN/m^2 to 400 kN/m^2. The area of the footing should be so arrived that the pressure distribution below the footing should be less than the allowable bearing pressure of the soil. Even for symmetrical Loading, the pressure distribution below the footing may not be uniform. It depends on the Rigidity of footing, Soil type and Conditions of soil. In case of Cohesive Soil and Cohesion less Soil the pressure distribution varies in a nonlinear way. However, while designing the footings a linear variation of pressure distribution from one edge of the footing to the other edge is assumed. Once the pressure distribution is known, the bending moment and shear force can be determined and the footing can be designed to safely resist these forces.

Design of Isolated Column Footing

The objective of design is to determine

- Area of footing
- Thickness of footing
- Reinforcement details of footing (satisfying moment and shear considerations)
- Check for bearing stresses and development length

This is carried out considering the loads of footing, SBC of soil, Grade of concrete and Grade of steel. The method of design is similar to the design of beams and slabs. Since footings are buried, deflection control is not important. However, crack widths should be less than 0.3 mm.

The steps followed in the design of footings are generally iterative. The important steps in the design of footings are;

- Find the area of footing (due to service loads)
- Assume a suitable thickness of footing
- Identify critical sections for flexure and shear
- Find the bending moment and shear forces at these critical sections (due to factoredloads)
- Check the adequacy of the assumed thickness
- Find the reinforcement details
- Check for development length
- Check for bearing stresses

Limit state of collapse is adopted in the design of isolated column footings. The variousdesign steps considered are;

- Design for flexure
- Design for shear (one way shear and two way shear)
- Design forbearing
- Design for development length

The materials used in RC footings are concrete and steel. The minimum grade of concrete to be used for footings is M20, which can be increased when the footings are placed in aggressive environment, or to resist higher stresses.

Cover: The minimum thickness of cover to main reinforcement shall not be less than 50 mm for surfaces in contact with earth face and not less than 40 mm for external exposed face. However, where the concrete is in direct contact with the soil the cover should be 75 mm. In case of raft foundation the cover for reinforcement shall not be less than 75 mm.

Minimum reinforcement and bar diameter: The minimum reinforcement according to slab and beam elements as appropriate should be followed, unless otherwise specified. The diameter of main reinforcing bars shall not be less 10 mm. The gradeof steel used is either Fe 415 or Fe 500.

Specifications for Design of footings as per IS 456 : 2000

The important guidelines given in IS 456 : 2000 for the design of isolated footings are as follows:

General

Footings shall be designed to sustain the applied loads, moments and forces and the induced reactions and to ensure that any settlement which may occur shall be as nearly uniform as possible, and the safe bearing capacity of the soil is not exceeded (see IS 1904).

In sloped or stepped footings the effective cross-section in compression shall be limited by the area above the neutral plane, and the angle of slope or depth and location of steps shall be such that the design requirements are satisfied at every section. Sloped and stepped footings that are designed as a unit shall be constructed to assure action as a unit.

Thickness at the Edge of Footing

In reinforced and plain concrete footings, the thickness at the edge shall be not less than 150 mm for footings on soils, nor less than 300 mm above the tops of piles for footings on piles.

In the case of plain concrete pedestals, the angle between the plane passing through the bottom edge of the pedestal and the corresponding junction edge of the column with pedestal and the horizontal plane (see Fig. 20) shall be governed by the expression:

$$\tan \alpha \nless 0.9 * \sqrt{(100 q_0 / f_{ck}) + 1}$$

where

q_0 = calculated maximum bearing pressure at the base of the pedestal in N/mm^2

f_{ck} = characteristic strength of concrete at 28 days in N/mm^2.

Moments and Forces

In the case of footings on piles, computation for moments and shears may be based on the assumption that the reaction from any pile is concentrated at the centre of the pile.

For the purpose of computing stresses in footings which support a round or octagonal concrete column or pedestal, the face of the column or pedestal shall be taken as the side of a square inscribed within the perimeter of the round or octagonal column or pedestal.

Bending Moment

The bending moment at any section shall be determined by passing through the section a vertical plane which extends completely across the footing, and computing the moment of the forces acting over the entire area of the footing on one side of the said plane.

The greatest bending moment to be used in the design of an isolated concrete footing which supports a column, pedestal or wall, shall be the moment computed in the manner prescribed in 34.2.3.1 at sections located as follows:

a) At the face of the column, pedestal or wall, for footings supporting a concrete column, pedestal or wall;

b) Halfway between the centre-line and the edge of the wall, for footings under masonrywalls; and

c) Halfway between the face of the column or pedestal and the edge of the gussetted base, for footings under gussetted bases.

Shear and Bond

The shear strength of footings is governed by the more severe of the following two conditions:

a) The footing acting essentially as a wide beam, with a potential diagonal crack extending in a plane across the entire width; the critical section for this condition shall be assumed as a vertical section located from the face of the column, pedestal or wall at a distance equal to the effective depth of footing for footings on piles.

b) Two-way action of the footing, with potential diagonal cracking along the surface of truncated cone or pyramid around the concentrated load; in this case, the footing shall be designed for shear in accordance with appropriate provisions specified in 31.6.

In computing the external shear or any section through a footing supported on piles, the entire reaction from any pile of diameter Dp whose centre is located DP/2 or more outside the section shall be assumed as producing shear on the section; the reaction from any pile whose centre is located DP/2 or more inside the section shall be assumed as producing no shear on the section, For intermediate positions of the pile centre, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight line interpolation between full value at DP/2 outside the section and zero value at DP/2 inside the section.

The critical section for checking the development length in a footing shall be assumed at the same planes as those described for bending moment in 34.2.3 and also at all other vertical planes where abrupt changes of section occur. If reinforcement is curtailed, the anchorage requirements shall be checked in accordance with 26.2.3.

Tensile Reinforcement

The total tensile reinforcement at any section shall provide a moment of resistance at least equal to the bending moment on the section calculated in accordance with 34.2.3.

Total tensile reinforcement shall be distributed across the corresponding resisting section as given below:

a) In one-way reinforced footing, the-reinforcement extending in each direction shall be distributed uniformly across the full width of the footing;

b) In two-way reinforced square footing, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing; and

c) In two-way reinforced rectangular footing, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. For reinforcement in the short direction, a central band equal to the width of the footing shall be marked along the length of the footing and portion of the reinforcement determined in accordance with the equation given below shall be uniformly distributed across the central band:

$$\frac{\text{Reinforcement in central band width}}{\text{Total reinforcement in short direction}} = \frac{2}{\beta + 1}$$

where β is the ratio of the long side to the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portions ofthe footing.

ß+ 1

Transfer of Load at the Base of Column

The compressive stress in concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting Pedestal or footing. The bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value equal to

$$\frac{\sqrt{A_1}}{\sqrt{A_2}}$$

but not greater than 2, where *A1* = supporting area for bearing of footing, which in sloped or stepped footing may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal; and *A2* = loaded area at the column base.

Where the permissible bearing stress on the concrete in the supporting or supported member would be exceeded, reinforcement shall be provided for developing the excess force, eitherby extending the longitudinal bars into the supporting member, or by dowels (see 34.4.3).

Where transfer of force is accomplished by, reinforcement, the development length of the reinforcement shall be sufficient to transfer the compression or tension to the supportingmember in accordance with 26.2.

Extended longitudinal reinforcement or dowels of at least 0.5 percent of the cross-sectional area of the supported column or pedestal and a minimum of four bars shall be provided. Where dowels are used, their diameter shall no exceed the diameter of the column bars by more than 3 mm.

Column bars of diameters larger than 36 mm, in compression only can be dowelled at the footings with bars of smaller size of the necessary area. The dowel shall extend into the column, a distance equal to the development length of the column bar and into the footing, a distance equal to the development length of the dowel.

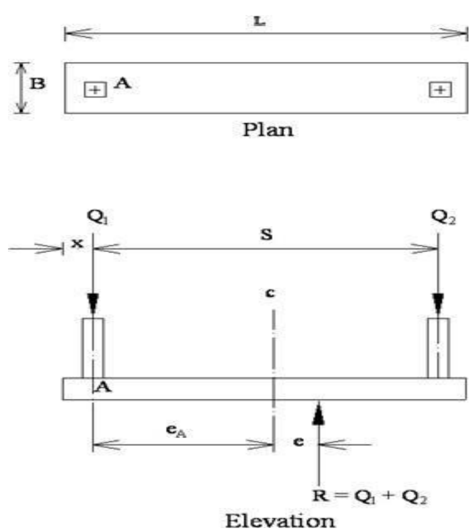
Nominal Reinforcement

Minimum reinforcement and spacing shall be as per the requirements of solid slab. The nominal reinforcement for concrete sections of thickness greater than 1 m shall be 360 mm^2 per metre length in each direction on each face. This provision does not supersede the requirement of minimum tensile reinforcement based on the depth of the section.

Important Design Concepts

Rectangular Combined Footings

Space restrictions or architectural features may require one footing to carry at least two column loads as shown in given Fig. Conventional design and analysis are based on the assumption thatthe footing is infinitely stiff and that stress distribution on the base of the footing is planar.



Design steps:

1. AssumeQ1, Q2 and S are known; therefore $\Sigma Q = Q1 + Q2$
2. Find the base area of the footings, A= Q/ qna ,whereqna is the allowable soil pressure.
3. Locate the line of action of the resultant of the column loads measured from one of the column ,centre of gravity of the load x = (Q2 x c/c distance) / Q: the location of the resultant force $\Sigma Q$with respect to any pointmay be obtained by taking moments about that point.
4. For uniform stress distribution, the required length L of the footing is: $L = 2(\bar{x} + b1)$
5. The required width B of the footing is: B =Area /L = (Q1+ Q2) / (q all x L)
6. Actual allowable soil pressure qo= Q/Ao Where

   Ao= B x L

Design of Combined Footings by Rigid Method (Conventional Method)

The rigid method of design of combined footings assumes that

1. The footing or mat is infinitely rigid; hence, the deflection of the footing or mat does not influence the pressure distribution,

2. The soil pressure is distributed in a straight line or a plane surface such that the centroid of the soil pressure coincides with the line of action of the resultant force of all the loads acting on the foundation.

Design of Combined Footings

Two or more columns in a row joined together by a stiff continuous footing form a combined footing as shown in Fig. 3.3(a). The procedure of design for a combined footing is as follows:

1. Determine the total column loads ∑Q = Q1 + Q2 + Q3 ... and location of the line of action of the resultant ∑Q. If any column is subjected to bending moment, the effect of the moment

should be taken into account.

2. Determine the pressure distribution q per lineal length of footing.
3. Determine the width B, of the footing.
4. Draw the shear diagram along the length of the footing. By definition, the shear atany section along the beam is equal to the summation of all vertical forces to the left or right of the section. For example, the shear at a section immediately to the left of Q1 is equal to the area abed, and immediately to the right of Q1 is equal to (abcd - Q1) as shown in Fig. (a).
5. Draw the moment diagram along the length of the footing. By definition the bending moment at any section is equal to the summation of moment due to all the forces and reaction to the left (or right) of the section. It is also equal to the area under the shear diagram to the left (or right) of the section.
6. Design the footing as a continuous beam to resist the shear and moment.
7. Design the footing for transverse bending in the same manner as for spread footings.

It should be noted here that the end column along the property line may be connected to the interior column by a rectangular or trapezoidal footing. In such a case no strap is required and both the columns together will be a combined footing as shown in Fig. b. It is necessary that the center of area of the footing must coincide with the center of loading for the pressure to remain uniform.

Design of Mat Foundation by Rigid Method

In the conventional rigid method the mat is assumed to be infinitely rigid and the bearing pressure against the bottom of the mat follows a planar distribution where the centroid of the bearing pressure coincides with the line of action of the resultant force of all loads acting on the mat.

The procedure of design is as follows:

Step 1: The column loads of all the columns coming from the superstructure are calculated as perstandard practice. The loads include live and dead loads.

Step 2.Centre of gravity of the mat foundation:

(1)Taking moment about X and Y direction, we can calculate X and Y .The line of action of the resultant of all the loads is found. However, the weight of the mat is not included inthe structural design of the mat because every point of the mat is supported by the soil under it, causing no flexural stresses.

(2) $\mathbf{ex} = \frac{Lx}{2} - X$ **and** $\mathbf{ey} = \frac{Ly}{2} - Y$

Step 3. Calculate the soil pressure at desired locations by the use of

$$q = \frac{Q}{A} - \frac{Qex}{Iyy}X - \frac{Qey}{Ixx}y$$

Where $Q = \Sigma Q$ = total load on the mat

A = total area of the mat

x, y = coordinates of any given point on the mat with respect to the x and y axes passing throughthe centroid of the area of the mat

ex ,y = eccentricities of the resultant force

Ix ,Iy= moments of inertia of the mat with respect to the x and y axes respectively. To calculate all the corner column loads, we have to know the coordinates from the origin.

Step 4. The mat is treated as a whole in each of two perpendicular directions. Thus the total shear force acting on any section cutting across the entire mat is equal to the arithmetic sum of all forces and reactions (bearing pressure) to the left (or right) of the section. The total bending moment acting on such a section is equal to the sum of all the moments to the left (or right) of the section.

Step 5: The pressure intensity on the strip = $\frac{\textbf{Qave}}{\textbf{StripArea}}$

Step 6: Factor of safety = $\frac{\textbf{Qave}}{\textbf{Q1+Q2+Q3}}$

$$e_x' = e_x \frac{I_{xy}}{I_x} e_y \quad e_y' = e_y \frac{I_{xy}}{I_y} e_x \quad I_x' = I_x - \frac{I^2_{xy}}{I_y} \quad I_y' = I_y - \frac{I^2_{xy}}{I_x}$$

e_x And e_y = eccentricities in x and y direction of the load from the centroid.

I_x And I_y = moment of inertia of the area of the raft respectively about the x and y axis throughthe centroid

$$I_{xy} = \int xydA$$

For the whole area about x and y axis through the centroid.

Numerical problems

1.A raft foundation 10m wide and 12m long is to be constructed in a clayey soil having a shear strength of 12KN/m^2 .Unit weight of soil is 16Kn/m3.If the ground surface carries a surcharge of 20KN/m2.Calculate the maximum depth of foundation to ensure a factor of safety of 1.2 against base failure. Nc for clay is 5.7.

Solution:

Bearing capacity of soil for rectangular footing in cohesive soil is given by

$$q_f = cN_c(1+0.3\frac{B}{L}) + \sigma = cN_c(1+0.3\frac{B}{L}) + (\gamma D + q)$$

$$q_f = 12 \times 5.7\ (1 + 0.3 \times \frac{10}{12}) + 16\,D + 20 = 105.5 + 16\,D$$

Base failure will occur when qf is equal to zero.

$D = -(\frac{105.5}{16}) = -\,6.59$ (minus sign indicates that it is excavation)

Therefore Critical depth = 6.59 m and Safe depth = $\frac{6.59}{1.2} = 5.49$m

2. A Trapezoidal footing is to be produced to support two square columns of 30cm and 50cm sides respectively. Columns are 6meters apart and the safe bearing capacity of the soil is 400KN/m^2.The bigger column carries 5000KN and the smaller 3000KN.Design a suitable size of the footing so that it does not extend beyond the faces of the columns.

Answer:

Trapezoidal footing of size

a=1m and b=4.9mL=4.889m

3. A footing 3m x 1.5m in plan transmits a pressure of 160KN/m2 on a cohesive soil having E= 8x 104 kN/m^2 and μ = 0.48.Determine the immediate settlement at the centre, assuming the footing to be (a) flexible, and (b) Rigid.

Solution:

L/B = 3/1.5 =2 .From table I_W= 1.52 for flexible footing and 1.22 for rigid footing.

(a) $Si = 160 \times 1.5 \frac{((1)-(0.48)2\ )}{8x10\ \ 4} \times 1.52 = 3.51$ mm

(b) $Si = 160 \times 1.5 \frac{((1)-(0.48)2\ )}{8x10\ 4} \times 1.22 = 2.82$mm

Floating Foundation

General Consideration A floating foundation for a building can be defined as a foundation in which the weight of the building is approximately equal to the full weight which includes water of the soil removed from the site of the building. With reference to Fig, this principle of flotation may be explained.

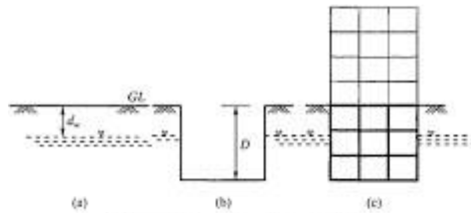
Fig. (a) shows a horizontal ground surface with water table at a depth dw below the ground surface. Fig. (b) shows an excavation made in the ground to a depth D where, D > dw and Fig.

Fig. (c) shows a structure built in the excavation and completely filling it.

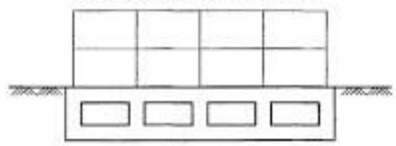
If the weight of the building is equal to the weight of the soil and water removed from the excavation, then it is clear that the total vertical pressure in the soil below depth D in Fig. (c) is the same as in Fig. (a) before excavation. Since there is no change in the water level, the neutral pressure and the effective pressure remain unchanged. If we could move from Fig. (a) to Fig. (c) without coming across the intermediate case of (b), the building in Fig. (c) would not settle at all, since an increase in effective vertical pressure cause settlements.

Principle of a floating foundation:

An exact balance of weight removed against weight imposed. The obtained result is zerosettlement of the building



Balance of stresses in foundation excavation



(d) Rigid raft foundation

Problems to be considered in the Design of a Floating Foundation

The following problems are to be considered during the design and construction stage of a floatingfoundation.

1. **Excavation**

The excavation for the foundation has to be done with care. The sides of the excavation should suitably be supported by sheet piling, soldier piles and timber lagging or some other standard method.

2. **Dewatering**

Dewatering will be necessary when excavation has to be taken below the water table level. Care has to be taken to see that the adjoining structures are not affected due to the lowering ofthe water table.

3. **Critical depth**

In Type 2 foundations the shear strength of the soil is low and there is a theoretical limit to thedepth to which an excavation can be made. Terzaghi (1943) has proposed the following equation for computing the critical depth D

4. **Bottom heave**

Excavation for foundations reduces the pressure in the soil below the founding depth which results in the heaving of the bottom of the excavation. Any heave which occurs will be reversed and appear as settlement during the construction of the foundation and the building. Though heaving of the bottom of the excavation cannot be avoided it can be minimized to a certain extent.

There are three possible causes of heave:

1. Elastic movement of the soil as the existing overburden pressure is removed.
2. A gradual swelling of soil due to the intake of water if there is some delay in placing thefoundation on the excavated bottom of the foundation.
3. Plastic inward movement of the surrounding soil.

The last movement of the soil can be avoided by providing proper lateral support to the excavated sides of the trench. Heaving can be minimized by phasing out excavation in narrowtrenches and placing the foundation soon after excavation.

It can be minimized by lowering the water table during the excavation process. Friction piles can also be used to minimize the heave. The piles are driven either before excavation commences or when the excavation is at half depth and the pile tops are pushed down to belowfoundation level.

As excavation proceeds, the soil starts to expand but this movement is resisted by the upper part of the piles which go into tension. The heave is prevented or very much reduced. It is onlya practical and pragmatic approach that would lead to a safe and sound settlement free floating

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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT –III - PILE FOUNDATION– SCIA7007

UNIT III

PILE FOUNDATION

A **deep foundation** is a type of foundation which transfers building loads to the earth farther down from the surface than a shallow foundation does, to a subsurface layer or a range of depths.

A **pile** is a vertical structural element of a deep foundation, driven deep into the ground at the building site

Need for pile foundation

- very large design loads,
- a poor soil at shallow depth,
- site constraints (like PROPERTY LINE)

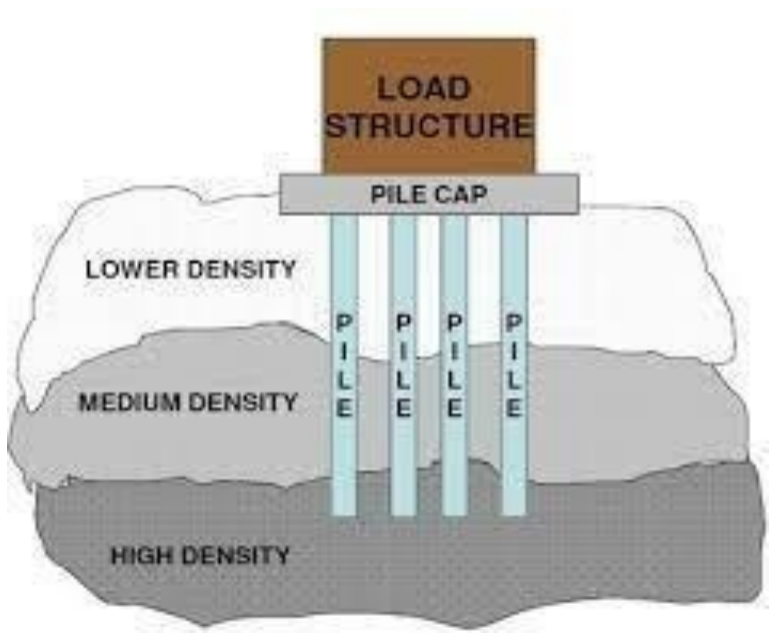


Fig.1.Pile foundation

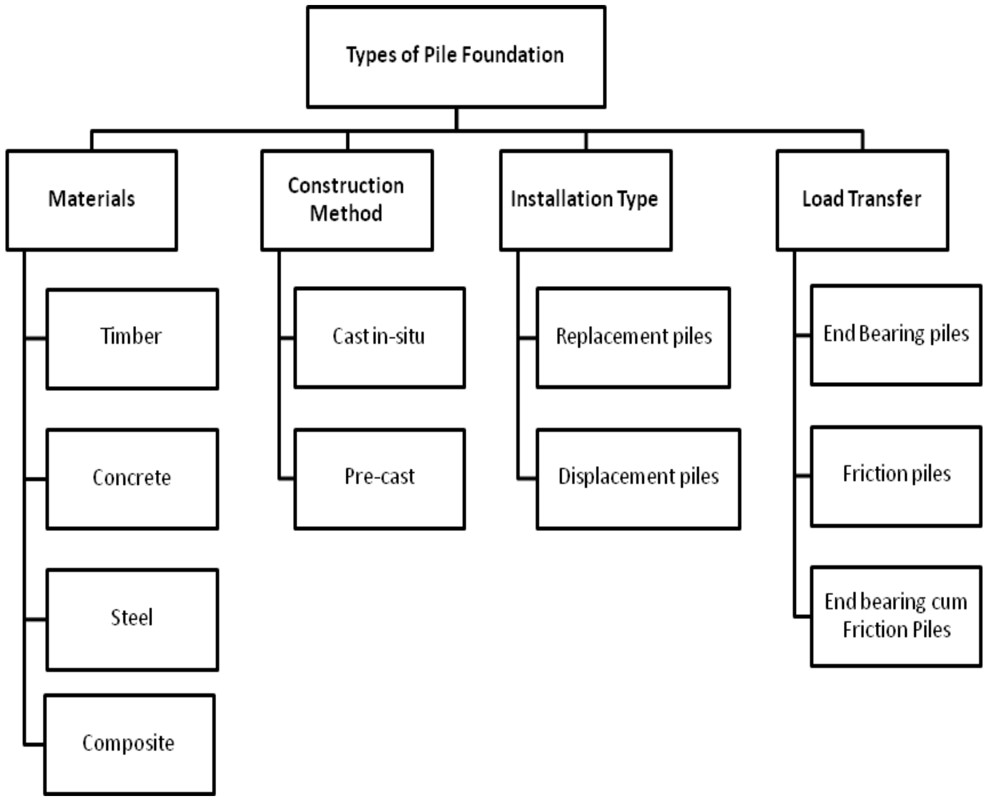


Fig.2 Classification of piles

Types of Piles based on Materials

i. Timber piles

- Timber piles are made of-tree trunks driven with small end as a point
- Maximum length: 35 m; optimum length: 9 - 20m
- Max load for usual conditions: 450 kN; optimum load range = 80 - 240 kN

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.

Advantages:

Comparatively low initial cost, permanently submerged piles are resistant to decay, easy to handle, best suited for friction piles in granular material.

ii. Steel piles

- Maximum length practically unlimited, optimum length: 12-50m
- Load for usual conditions = maximum allowable stress x cross-sectional area
- The members are usually rolled HP shapes/pipe piles. Wide flange beams & I beams proportioned to withstand the hard driving stress to which the pile may be subjected. In HP pile the flange thickness = web thickness, piles are either welded or seamless steel pipes, which may be driven either open ended or closed end. Closed end piles are usually filled with concrete after driving.
- Open end piles may be filled but this is not often necessary., dm

Advantages of steel piles:

Easy to splice, high capacity, small displacement, able to penetrate through light obstructions, best suited for end bearing on rock, reduce allowable capacity for corrosive locations or provide corrosion protection.

Disadvantages:

- Vulnerable to corrosion.
- HP section may be damaged/deflected by major obstruction

iii. Concrete Piles

- Concrete piles may be precast, prestressed, cast in place, or of composite construction

- Precast concrete piles may be made using ordinary reinforcement or they may be prestressed.
- Precast piles using ordinary reinforcement are designed to resist bending stresses duringpicking up & transport to the site & bending moments from lateral loads and to
  provide sufficient resistance to vertical loads and any tension forces developed duringdriving.
- Prestressed piles are formed by tensioning high strength steel prestress cables, and casting the concrete about the cable. When the concrete hardens, the prestress cables are cut, with the tension force in the cables now producing compressive stress in the concretepile. It is common to higher-strength concrete (35 to 55 MPa) in prestressed piles because of the large initial compressive stresses from prestressing. Prestressing the piles, tend to counteract any tension stresses during either handling or driving.

- Max length: 10 - 15 m for precast, 20 - 30 m for prestressed
- Optimum length 10 - 12 m for precast. 18 - 25mprestressed
- Loads for usual conditions 900 for precast. 8500 kN for prestressed
- Optimum load range: 350 - 3500 kN

Advantages:

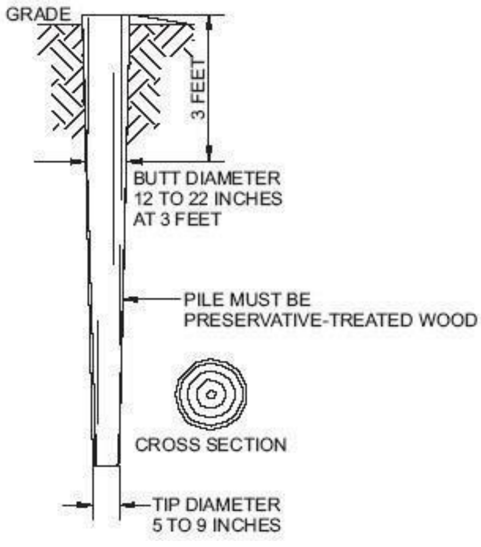
1. High load capacities, corrosion resistance can be attained, hard driving possible
2. Cylinder piles in particular are suited for bending resistance.
3. Cast in place concrete piles are formed by drilling a hole in the ground & filling it with concrete. The hole may be drilled or formed by driving a shellor casing into the ground.

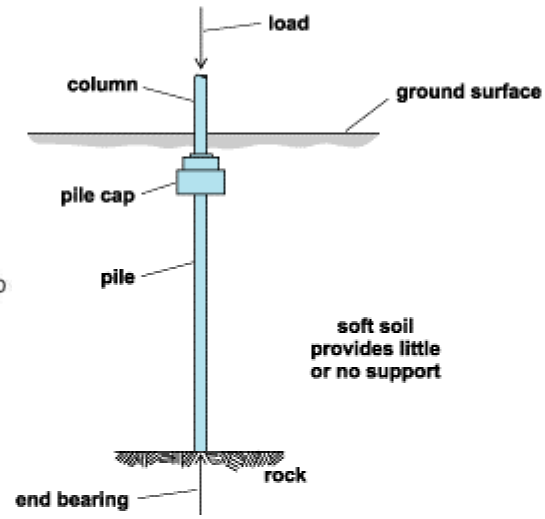
Disadvantages:

1. Concrete piles are considered permanent, however certain soils (usually organic) contain materials that may form acids that can damage the concrete.
2. Salt water may also adversely react with the concrete unless special precautions are takenwhen the mix proportions are designed. Additionally, concrete piles used for marine structures may undergo abrasion from wave action and floating debris in the water.
3. Difficult to handle unless prestressed, high initial cost, considerable displacement,prestressed piles are difficult to splice.
4. Alternate freezing thawing can cause concrete damage in any exposed situation.

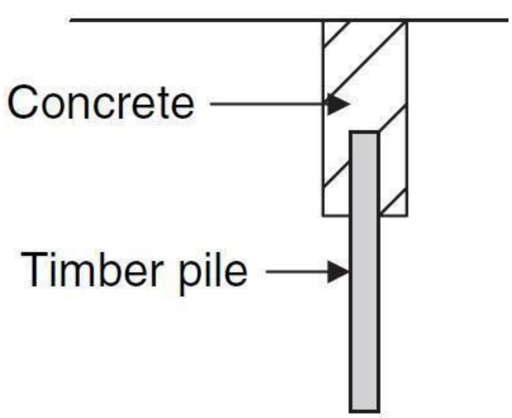
iv. Composite piles

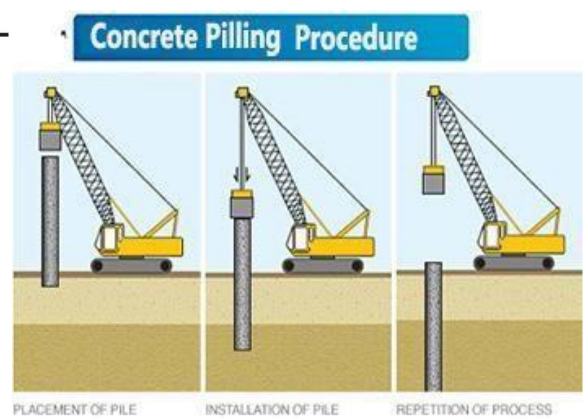
In general, a composite pile is made up of two or more sections of different materials or different pile types. The upper portion could be eased cast-in-place concrete combined with a lower portion of timber, steel H or concrete filled steel pipe pile. These piles have limited application and arc employed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





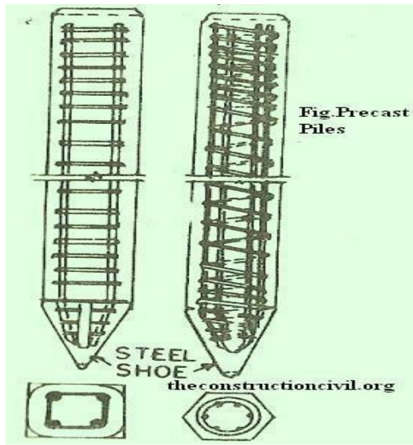
Timber pile **Steel pile**





Composite pile Concrete pile

2. Types of Piles based on construction methods



i. Precast Concrete Piles

May be defined as a reinforced concrete pile which is moulded in circular, square, rectangular or octagonal form. The **precast concrete piles** are cast and cured in a casting yard and then transported to the site for driving. In case space is available, pile can also be cast and cured near the site of works. They are driven in a similar manner as timber piles with the help of pile drivers. The diameter of the pile normally varies 1mm 35 cm to *65* cm and their length varies from *45* in to 30 m.

ii. Cast-in-situ piles

Are those piles which are cast in position inside the ground. Since the **cast-in-situ piles** is not subjected to handling or driving stresses, it is not necessary to reinforce the pile in ordinary cases or in places where the pile is completely submerged in the soil. Reinforcements are necessary to be provided in a **cast-in-situ piles**, when the pile acts as a column and is subjected to lateral forces. **Cast-in-situ piles** can be divided into two types. In one the metallic shell of the pile is permanently left in place inside the ground along with the core while in the other type the outer shell is withdrawn.

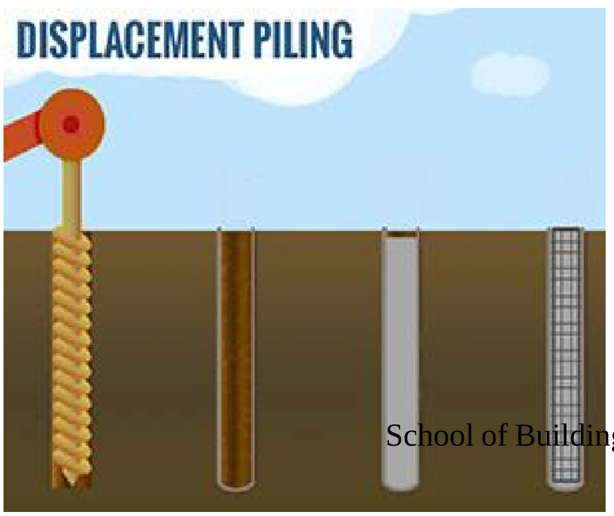
3. Types of Piles based on installation type Piling techniques

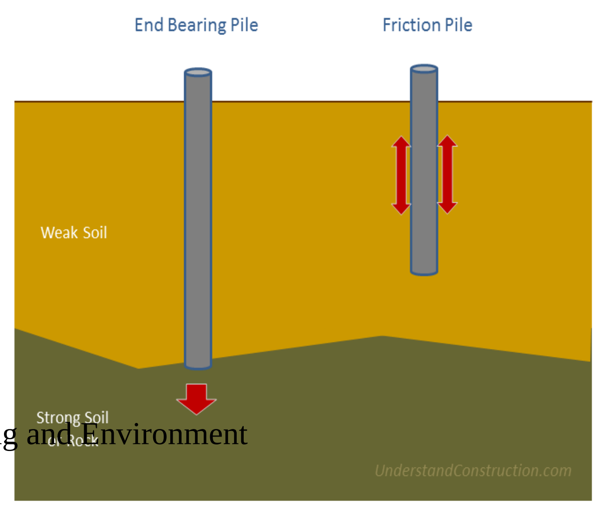
can be split into 2 categories;

1. **Displacement** piling technique
2. **Replacement**. piling technique

In simple terms, during the **displacement piling method**, piles are driven into the ground pushing the ground out of the way, as you would see in sheet piling. Displacement piling is good for e.g. contaminated sites where it costs a lot to take the spoil away.

Using the **replacement piling method,** muck is dug out and replaced with the pile. We can use far bigger piles using replacement piling.





4. Types of Piles based on load transfer

End bearing piles

In end bearing piles, the bottom end of the pile rests on a layer of especially strong soil or rock.The load of the building is transferred through the pile onto the strong layer

Friction piles

Friction piles work on a different principle .the pile transfer load of the building to the soil acrossthe full height of the pile by friction.

5. Classification based on method of installation

(i) Bored piles

Bored piles are constructed in pre-bored holes either using a casing or by circulating stabilizing agent like betonies slurry. The borehole is then filled with concrete after placing the reinforcement. The advantage of board pile is that there is no damage due to handling and driving which is common in driven piles.

Board piles are of following types:

Small diameter piles-up to 600 mm diameter; large diameter piles-diameter greater than 600mm; under reamed piles.

(ii) Driven piles

Driven piles may be of concrete, steel or timber. These piles are driven into the soil by the impact of hammer. Boring is not required for this type of piles. When a pile is driven into granular soils it densities the soil and increases strength of soil. But when a pile is driven in saturated clay, the soil instead of being compacted gets remolded with reduction in strength.

(iii) Driven and cast-in-situ piles

It is a type of driven pile. They are constructed by driving a steel casing in to the ground. The hole is then filled with concrete by placing the reinforcement and the casing is gradually lifted.

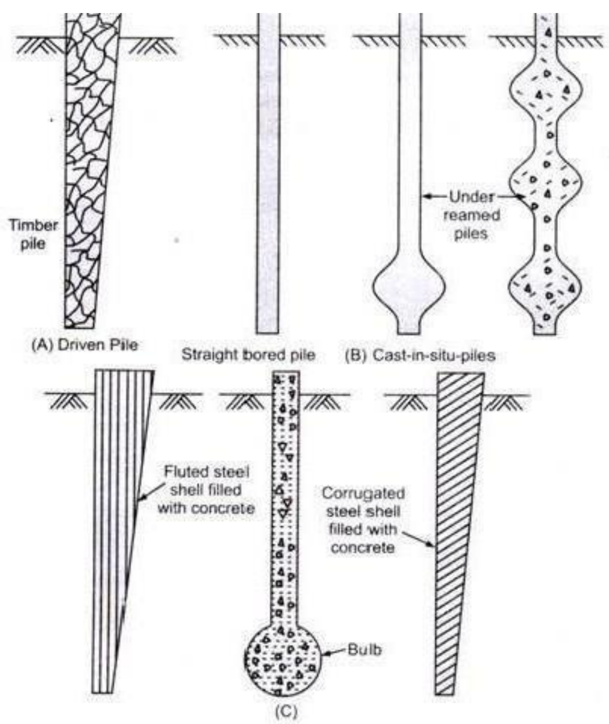
(c) Classification based on the function Pilesare of following types based on its use

(i) End bearing piles

The piles which transfer its load to a hard and relatively incompressible stratum like rock or dense sand are called end bearing piles. These piles derive its bearing capacity from end bearingat the pile tip.

(ii) Friction piles

The piles which do not rest on hard stratum but derives its carrying capacity from skim frictionor adhesion between the pile surface and surrounding soil are called friction piles.



Driven and cast in situ piles

(iii) Tension pile

Tension piles are also called uplift piles. These piles are used to anchor down the structures subjected to uplift due to hydrostatic pressure.

(iv) Compaction piles

These piles are used to compact loose granular soil to increase its bearing capacity. Compaction piles do not carry load and hence they can be of weaker material. Sand piles can be used as compaction piles.

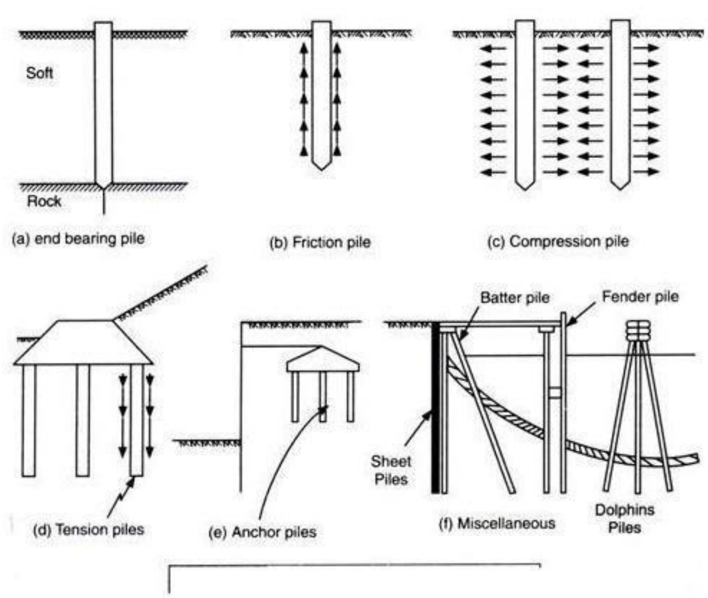
(v) Anchor piles

These piles are used to provide anchorage against horizontal pull from sheet piling.

(vi) Fender piles and dolphins

Fender piles and dolphins are used to protect water front structure from impact of any floatingobject or ship.

Piles based on uses



Selection of pile foundation

Selection of pile types and length depends on following conditions:

Soil conditions

Loads from structuresNature of loads

Number of piles to be usedCost of construction

If hard soil is available at deeper levels of earth, then there is a need of some source that can transfer the load of the structures on the deep hard soil strata. This source can be said to be as the deep foundation. Pile foundation is a type of foundation in which pile is usually used as the source to transfer the load to deep soil levels. **Piles are long and slender members that transfer the load to hard soil ignoring the soil of low bearing capacity. Transfer of load depends on capacity of pile**. There is a need that pile should be strong enough to transfer the whole load coming on it to underlying hard strata. For this purpose, pile design is usually given much consideration. Depending on the load, typeof material is usually selected for the piles.

Factors affecting the selection of piles.

Length of pile in relation to load and soil conditionBehavior of structure

Availability of material in locality of construction

Type of loading

Ease of maintenance

Availability of funds Factors causing damage

Cost of piles

Load carrying capacity of pile

The **Ultimate load carrying capacity** of a pile is defined as the maximum load which can be carried by a pile and at which the pile continues to sink without further increase in load.

The **allowable load** is the load which the pile can carry safely which can be determined fromthe ultimate load carrying capacity divided by suitable factor of safety

Following are general methods available to establish load capacity:

1. Static Analysis
2. Dynamic Analysis
3. Pile Load Testing
4. Correlation with field tests (SPT, CPT etc)(Penetration tests)

Dynamic formulae are used for driven piles. Static formulae are used ***both*** for bored and driven piles. Load testing is the most reliable method to determine the load capacity of the pile in the field.

They should be performed on all piling projects. However, they are considerably more expensive than the other methods used to determine pile capacity, and economic considerations sometimes preclude their use on projects.

Field tests like SPT, CPT are also used to correlate to load carrying capacity particularly for cohesion less soils.

1. **Static method**

Based on the assumption that the ultimate bearing capacity Q_{up} of a pile is the sum of the total ultimate skin friction R_f and the total ultimate point or end bearing resistance R_p

$Qup = R_f + R_p$ or

$Qup = A_s . r_f + A_p . r_p$

A_s	=	Surface area of pile upon which the skin friction acts.
A_p	=	Area of cross section of pile on which bearing resistance acts.
r_f	=	Average skin friction
r_p	=	Unit point or toe resistance.

A factor of safety of 2.5 or 3 may be adopted for finding the allowable load.For cohesive soil,

rf	=	Average skin friction along the length of the pile
	=	m c'
r_p		Point resistance
	=	C_pN_c
	=	
	=	9 Cp

Qup = m c' As +9 CpAp

Where **m** = Adhesion coefficient

Cp	=	Average undrained cohesion of soil @ pile tip
C'	=	Average undrained cohesion along of the pile

c'= cp= qu/2

2. Dynamic formulae

Engineering News Formula

As per the Engineering News formula the allowable load of driven pile is given by:

$$Qa = Wh/(F(S+C))$$

Where,	Qa	=	Allowable load in kg
	W	=	Weight of hammer
	h	=	Height of fall in cms
	S	=	Final settlement per blow known as set

(c = 2.5 cm for drop hammer and 0.25 cm for single acting and double acting hammer)F = Factor of safety (Usually taken as 6)

(1) Drop hammer $\mathbf{Q_a} = \frac{WH}{6(S+2.5)}$

(2) Single acting stream hammer $\mathbf{Q_a} = \frac{WH}{6(S+0.25)}$

(3) Double acting hammer

$$Qa = \frac{(W+ap)}{6(S+0.25)}$$

a = Effective area of piston in square cm.

p = mean effective stream pressure (kg/cm^2)

Hiley's Formula (IS: 2911 part-I) 1964

The relation suggested by Hiley for ultimate bearing capacity of the pile is:

$$Qu = \frac{\eta h W H \eta b}{(S+C/2)}$$

Where, c = Sum of elastic compression (c = cp + cc + cq)

cp = Compression of pile

cc = Compression of pile head

cq = Compression of ground

W = Weight of hammer

H = Height of fall of hammer

s = penetration or Set in cm /blow

ηh = Efficiency of hammer [65% for DAH and 100% for DH]

ηb = Efficiency of hammer blow (ratio of energy after impact to striking energy of ram)

W > eP, $\eta_b = (W + e^2 P)/((W + P))$

W< eP , $\eta_b = \left(\frac{W+e^2 P}{W+P}\right) - \left[\frac{W-e P}{W+P}\right]^2$

P=Weight of pile

E= Coefficient of restitution Hammer coefficient is given in Table 1

Table 1 Values of Hammer Coefficient K

Hammer	K
Drop Hammer, Winch operated	0.8
Drop Hammer, Trigger release	1.0
Single Acting Hammer	1.9
BSP Double Acting Hammer	1.0
McKiernan-Terry Diesel Hammers	1.0

PILE LOAD TEST

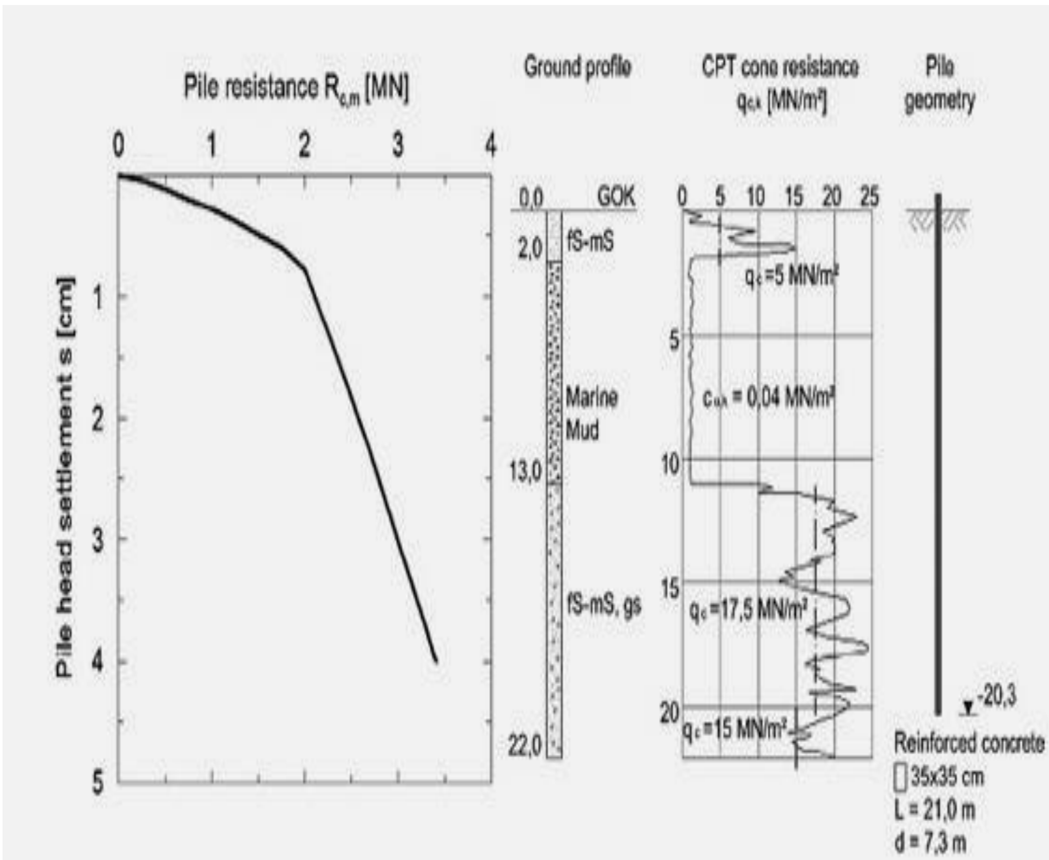
To determine settlement under working load To determine ultimate bearing capacity

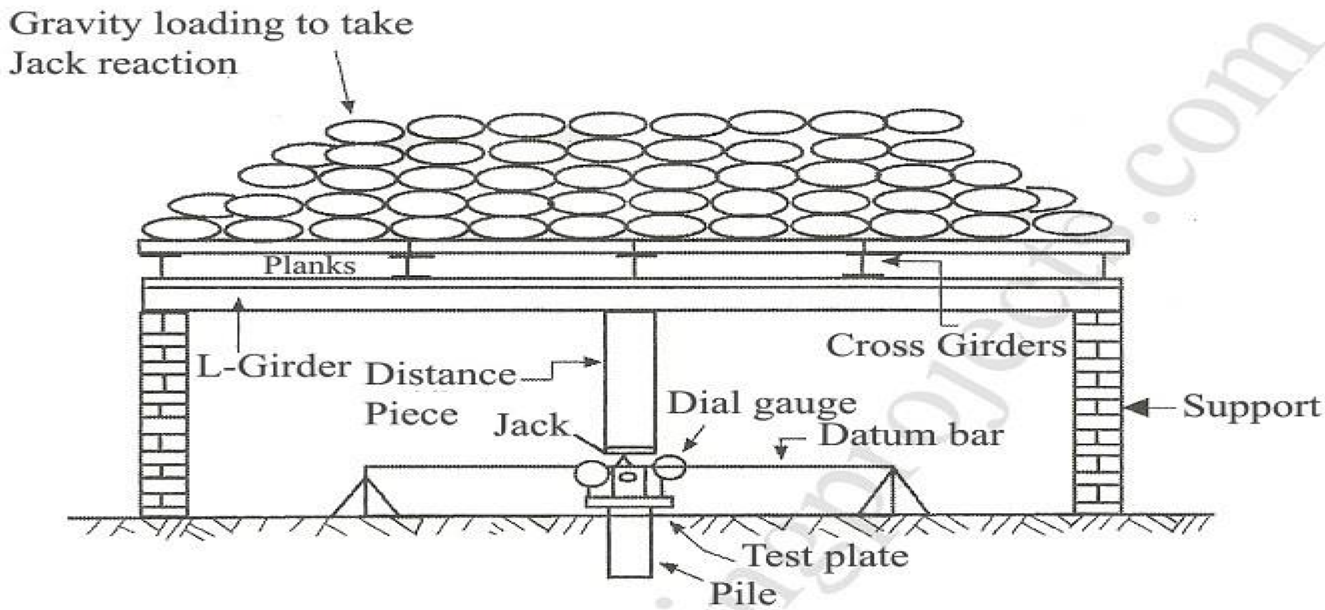
To ascertain as a proof of acceptability

The test can be initial or routine test

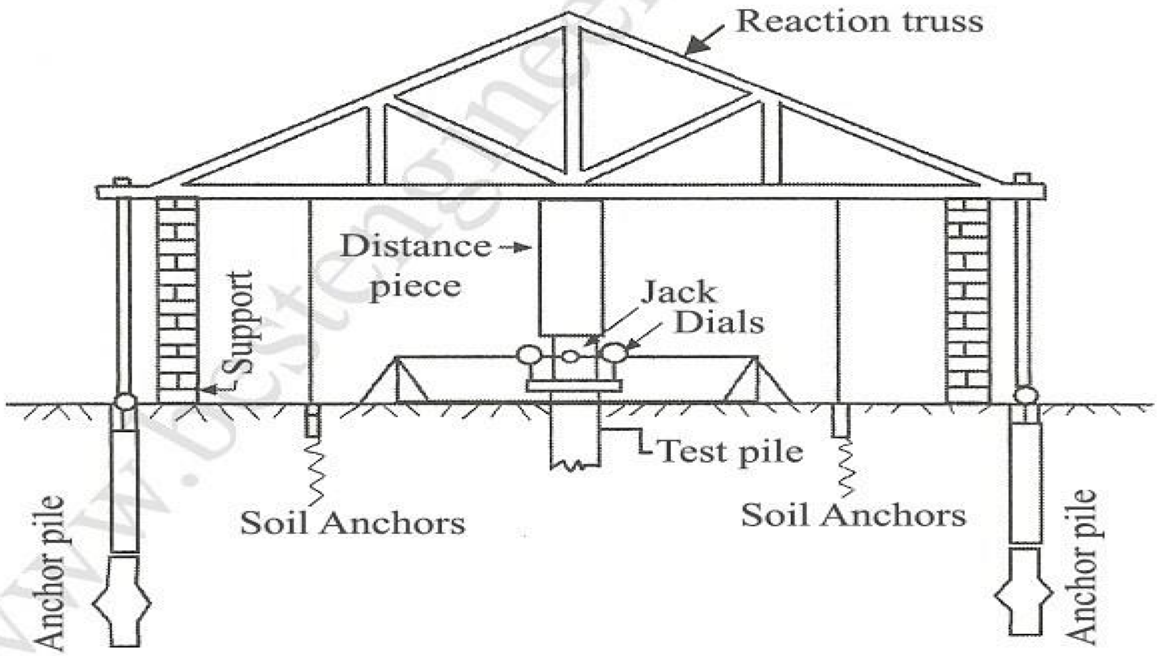
- The load is applied in increments of 20% of the estimated safe load. Hence the failureload is reached in 8-10 increments.
- Settlement is recorded for each Settlement is recorded for each increment until the rate ofsettlement is less than 0.1 mm/hr.
- The ultimate load is said to have reached when the final settlement is more than 10% ofthe diameter is more than 10% of the diameter of pile or the settlement keeps on increasing at constant load. 45

- After reaching ultimate load the after reaching ultimate load, the load is released in decrements of 1/6th of the total load and recovery is measured until full recovery is measured until full rebounds is established and next unload is done.
- After final unload the settlement is measured for 24 hrs to estimate full elastic recovery.
- Load settlement curve depends on the type of pile





(a) Jack loading reaction by loaded platform



(b) Jack loading reaction by anchors

Distance of anchor piles from test pile – The distance cannot be less than 1.5 m. It should not be less than 4 times the diameter of test pile for straight pile and not less than 2 times the diameter of the bell for belled pile.

Load Application – The load is applied in the pile in the following sequence.

- Load applied in increment at the rate of 25 % of working load till working load is reached
- For each load increment maintain the load constant till settlement is 0.1 mm for 5 min asper IS Code, 0.1 mm for 20 min as per BS Code
- Go for next loading
- When working load is reached hold the load for 24 hr and unload
- Reload from working load to higher loads
- Hold load constant till settlement is 0.1 mm for 5 min as per IS Code, 0.1 mm for 20 minas per BS Code
- Repeat the process for subsequent load increments
- Go either up to 5/2 times the working load for initial or routine test or to a settlementequal to 10 % of pile diameter for straight piles and 7.5 % of base diameter for belledpile

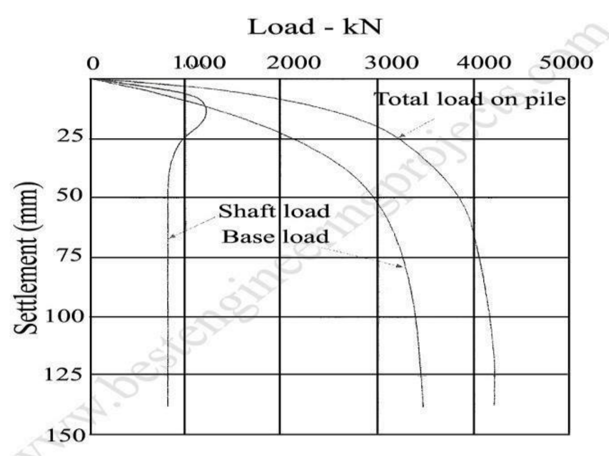
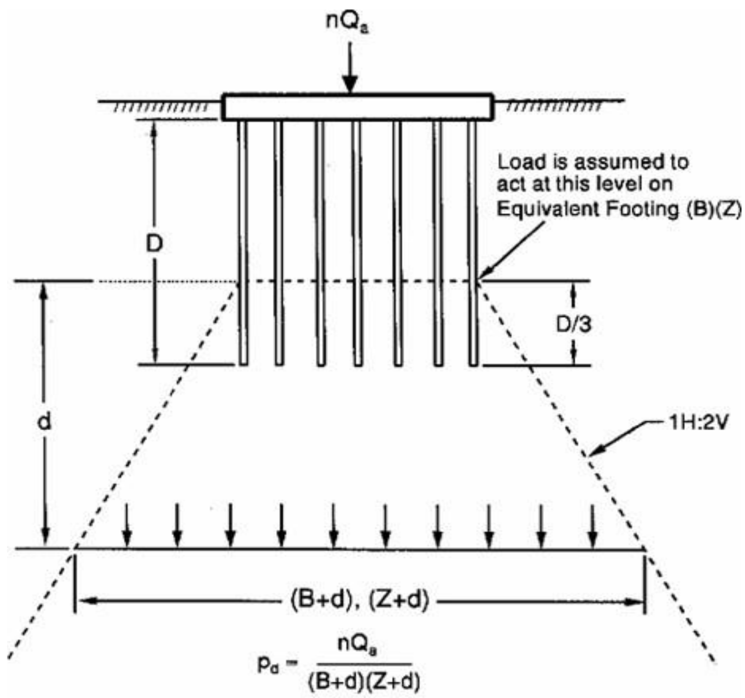


Fig 2 Load Taken by Shaft and Base

Pile group

- When several piles are clustered, it is reasonable to expect that the soil pressures produced from either side friction or point bearing will overlap.
- The super-imposed pressure intensity will depend on both the pile load and spacing, and if sufficiently large the soil will fail in shear or the settlement will be excessive.
- The stress intensity from overlapping stressed zones will obviously decrease with increased pile spacing s; however, large spacing's are often impractical in a pile cap is cast over the pile group for the column base and/or to spread the load to the several pilesin the group.



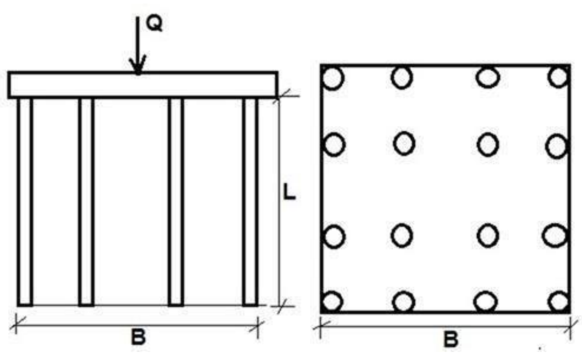
Note: Pile Group has Plan Dimension of B and Z

EFFICIENCY OF PILE GROUP

Capacity of pile group is the sum of the individual capacities of piles, but it is influenced by thespacing between the piles.

Piles are driven generally in groups in regular pattern to support the structural loads. The structural load is applied to the pile cap that distributes the load to individual piles. If piles are spaced sufficient distance apart, then the capacity of pile group is the sum of the individual capacities of piles. However, if the spacing between piles is too close, the zones of stress around the pile will overlap and the ultimate load of the group is less than the sum of the individual pile capacities specially in the case of friction piles, where the efficiency of pile group is much less.

Group action of piles is evaluated by considering the piles to fail as a unit around the perimeter of the group. Both end bearing and friction piles are considered in evaluating the group capacity. End bearing pile is evaluated by considering the area enclosed by the perimeter of piles as the area of footing located at a depth corresponding to the elevation of pile tips. The friction component of pile support is evaluated by considering the friction that can be mobilized around the perimeter of the pile group over the length of the piles as shown in figure below:



Efficiency of pile group

1. When closely spaced piles are grouped together it is reasonable to expect that the soil asresistance will overlap.
2. The bearing capacity of pile group may or may not be the sum of the bearing capacity ofindividual piles constituting the group.

3. Theory and tests have shown the total bearing capacity Qug of a group of friction pilesparticularly in clay may be less than the product of the friction bearing value Qup of individual pile multiplied by the number of piles in a group.
4. There is no reduction in the case of end bearing piles.
5. For combined end bearing and friction piles only the load carrying capacity of thefrictional portion is reduced.
6. A method of estimating the bearing capacity of a pile group of friction piles is to multiply the quantity **nQup** by a reduction factor called the efficiency of pile group.

$$Qug = n.Qup.\ \eta g$$

Qug	=	load carried by group of friction piles
Qup	=	load carried by each friction piles
n	=	number of piles
ηg	=	efficiency of pile group

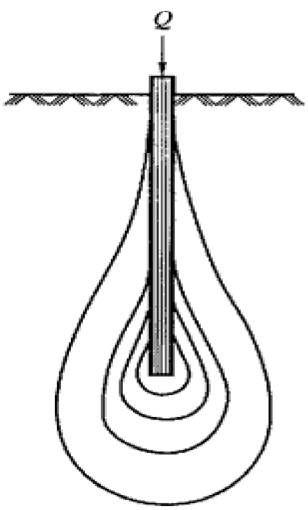
The efficiency of the pile group depends upon the following factors

- Characteristics of pileSpacing of pile
- Total number of piles
- No of formulae are available for finding the efficiency of pile.

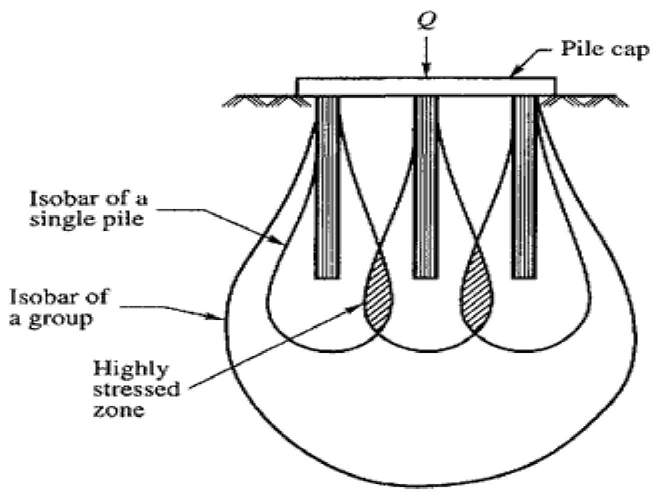
Pile Spacing

The spacing of piles depends upon the method of installing the piles and the type of soil. The piles can be driven piles or cast-in-situ piles. When the piles are driven there will be greater overlapping of stresses due to the displacement of soil. If the displacement of soil compacts the soil in between the piles as in the case of loose sandy soils, the piles may be placed at closer intervals.

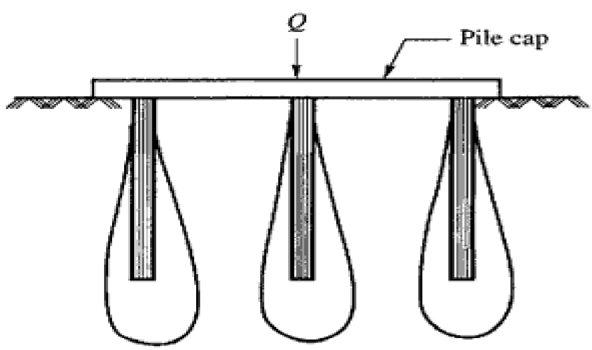
When piles are placed in a group, there is a possibility the pressure isobars of adjacent piles will overlap each other as shown in Fig. b. The soil is highly stressed in the zones of overlapping of pressures. With sufficient overlap, either the soil will fail or the pile group will settle excessively since the combined pressure bulb extends to a considerable depth below the base of the piles. It is possible to avoid overlap by installing the piles further apart as shown in Fig. c. large spacing are not recommended sometimes, since this would result in a larger pile cap which would increase the cost of the foundation.



(a) Single pile



(b) Group of piles closely spaced

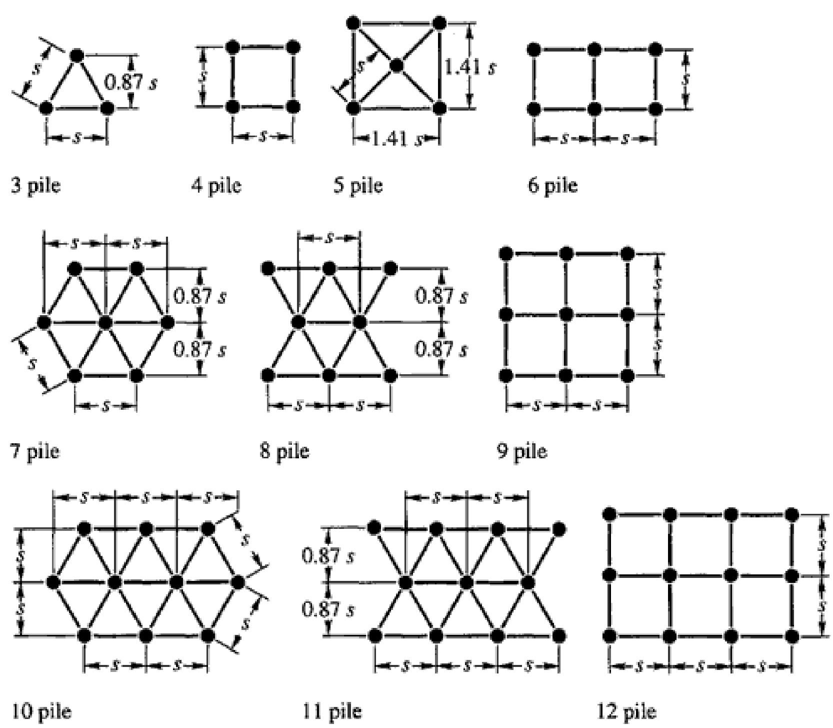


(c) Group of piles with piles far apart

The spacing of piles in a group depends upon many factors such as

1. Overlapping of stresses of adjacent piles,
2. Cost of foundation,
3. Efficiency of the pile group.

The minimum allowable spacing of piles is usually stipulated in building codes. The spacing for straight uniform diameter piles may vary from 2 to 6 times the diameter of the shaft. For friction piles, the minimum spacing recommended is 3d where d is the diameter of the pile. For end bearing piles passing through relatively compressible strata, the spacing of piles shall not be less than 2.5d.
For end bearing piles passing through compressible strata and resting in stiff clay, the spacing may be increased to 3.5d. For compaction piles, the spacing may be Id. Typical arrangements of piles in groups are shown in **Fig.**



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Negative Skin Friction

Pile installed through compressive soils can experience “down drag” forces or negative resistance along the shaft, which results from downward movement (settlement) of adjacent soil. Negative resistance results primarily from consolidation of soft deposits caused by dewatering or fill placement.

Negative skin friction (NSF) is in fact a downward friction imposed on foundation piles as a result of subsoil settlement. It needs only few millimeters of relative displacement between the settling subsoil and the pile shaft surface, which is not uncommon to have relative displacement at the pile-soil interface more than these values in normal subsoil settlement problem, to fully mobilize the shaft resistance in either upward or downward directions.

There are five probable, but not limited to, reasons of existence of NSF, namely,

a. Self-weight of unconsolidated recent fill,

b. Surcharge-induced consolidation settlement,

c. Consolidation settlement after dissipation of excess pore pressure induced by pile driving,

d. Lowering of groundwater level,

e. Collapse settlements due to wetting of unsaturated fill, and

f. Crushing of crushable subsoil under sustained loading, causing subsoil settlement For individual piles the magnitude of negative friction Qnf may be taken as follows

For cohesive soils Qnf = p.c.Lf

For granular soils Qnf = ½ Lf^2 p.r.K.f

P = perimeter of the pile

r = unit weight of

soil C = cohesion

K = earth pressure

F = coefficient of friction

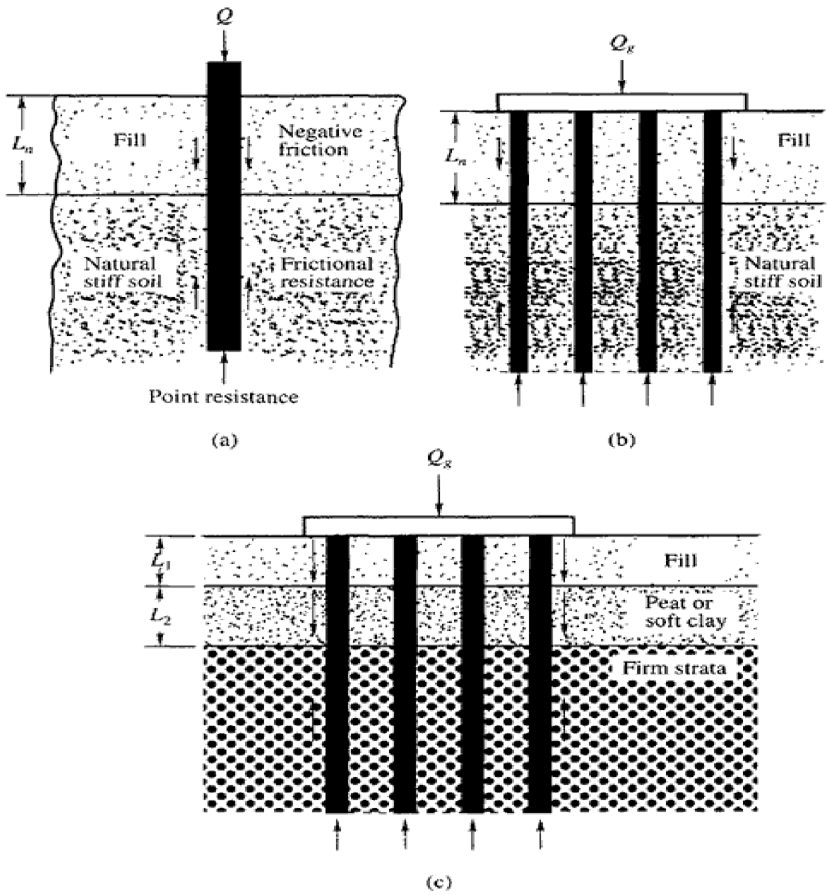
When the fill starts consolidating under its own overburden pressure, it develops a drag on the surface of the pile. This drag on the surface of the pile is called '*negative friction*'. Negative friction may develop if the fill material is loose cohesion less soil. Negative friction can also occur when fill is placed over peat or a soft clay stratum as shown in **Fig.C**.

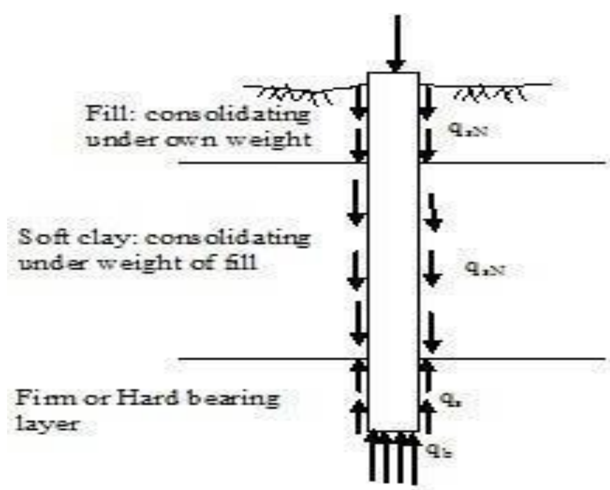
The superimposed loading on such compressible stratum causes heavy settlement of the fill with consequent drag on piles.

Negative friction may develop by lowering the ground water which increases the effective stress causing consolidation of the soil with resultant settlement and friction forces being developed on the pile.

Negative friction must be allowed when considering the factor of safety on the ultimate carrying capacity of a pile. The factor of safety, Fs, where negative friction is likely to occur may be written as

$$F_s = \frac{\text{Ultimate carrying capacity of a single pile or group of piles}}{\text{Working load + Negative skin friction load}}$$


Q
L_n
Fill
Negative friction
Natural stiff soil
Frictional resistance
Point resistance
(a)
Q_g
L_n
Fill
Natural stiff soil
(b)
Q_g
L_1
L_2
Fill
Peat or soft clay
Firm strata
(c)


Fill: consolidating under own weight
Soft clay: consolidating under weight of fill
Firm or Hard bearing layer

Problem 1

A group of 9 piles arranged in a square pattern with diameter and length of each pile as 25cm and 10m respectively, is used as a foundation in soft clay deposit. The unconfined compressivestrength of clay as 120kN/m^2 and the pile spacing as 100cm c/c. Find the load capacity of the group. Assume the bearing capacity factor as (Nc) 9 and adhesion factor (m) =0.75. Factor ofsafety of 3.5 may be taken.

Solution

Formula used: Qup = Ap r p + As rf

(1) Pile acting individually,

C	=	120/2	=	60 kN/m^2
Ap	=	$\pi d^2/4$	=	0.049 m^2
rp	=	c Nc	=	60 X 9
As	=	πdl		
rf	=	m c		
Qup	=	380 kN		

Load capacity of 9 piles = 9 x 380 = 3419 kN

(2) Pile acting on a group,

B	=	2s +d	=	2.25m
Ap	=	B^2	=	(2.25X 2.25) m^2
rp	=	9 c	=	60 X 9
As	=	4B l	=	4 x 2.25 x 10
rf	=	c	=	60 kN/m^2
Qug	=	8133kN		

Minimum of Qup and Qug is Qn min= 3419 kN

$$Qa = \frac{Qu\ min}{F} = 3419 / 23.5 = 1367\ kN$$

---

(2). A group of 16 piles of 600mm diameter is arranged in a square pattern with the c/c spacing of 1.2m. The piles are 10m long and are embedded in soft clay with cohesion of 30kN/m2. Bearing resistance may be neglected for the piles. Adhesion factor is 0.6. Determine the Ultimateload capacity of the pile group.

Solution:

Neglecting the bearing resistance, **Qup= As rf**

(1) Pile acting individually,

Qun = n Qup = n(As rf)

As = πdl = πX 0.6 X 10 = 18.84 m^2

rf = m .C = 0.6 X 30 = 18 kN/m^2

Qun = 16(18.84 X 18) = 5428.7 kN

2. Pile acting on a group,

B = 2 x 1.2 + 0.6 = 4.2 m.

Qug = Asg rf

Asg = 4 X B L = 4X 4.2 X 10 = 16.8 m^2

rf = c = 30 kN/m^2

Qug = Asg rf

= 16.8 X 30 = 5040 kN

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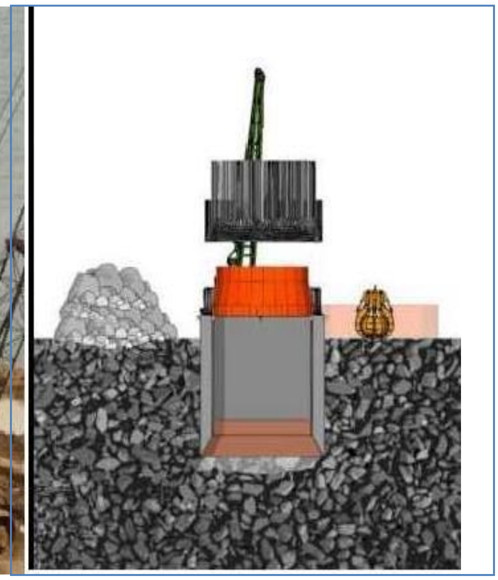
SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT –IV - WELL AND CAISSON FOUNDATIONS– SCIA7007

UNIT IV

WELL FOUNDATIONS



HISTORY

- They had their origin in India.
- It have been used for 100 of years as a deep foundation for important buildingsand structures.
- Well foundations are used since Mughal period (TAJ MAHAL is the bestexample).
- They were used for the first time for irrigation structures at Ganga canal atRoorkee. (middle of 19th century)
- In towers of Howrah bridge, wells of size 24.8x53.5m were used and sinkingdepth was 31.4m below gnd level.(largest in India)

- Many other such examples are Rajendra Pul, Mahanadi bridge etc.
- In spite of excellent development of technology on well foundations there are still some areas where engineers face difficulty while sinking of wells.

Benefits of Well Foundation

- Provides massive and solid foundation.
- Possible to sink well through boulders, logs of wood found at depth.
- Large sectionmodulus with minimum cross sectional area is advantageous.
- The strata through which well passes is known exactly.
- Well raising and steining is done in steps so foundation level can bevaried.
- Economical to provide it for unstable soil mass.

Well foundation is a type of deep foundation which is generally providedbelow the water level for bridges. Cassions or wells have been in use for foundations of bridges and other structures since Roman and Mughal periods.

The term **'cassion'** is derived from the French word caisse which means box or chest. Hence cassion means a box like structure, round or rectangular, which is sunkfrom the surface of either land or water to some desired depth.

TYPES OF CAISSON

The cassions are of three types:

(i) **Box cassion:**

It is open at the top and closed at the bottom and is made of timber, reinforced concrete or steel. This type of cassion is used where bearingstratum is available at shallow depth.

(ii) **Open cassion (wells):**

Open cassion is a box opened both at top and bottom. It is made up to either timber, concrete or steel. The open cassion is called well. Well foundation is the most common type of deep foundation used for bridges in India.

(iii) **Pneumatic cassions** has its lower end designed as a working chamber in which compressed air is forced to prevent the entry of water and thus excavation can be done in dry conditions.

Shapes of WellsThe common types of well shapes are:

- Single circular
- Twin circular
- Dumb well
- Double-D
- Twin hexagonal
- Twin octagonal
- Rectangular

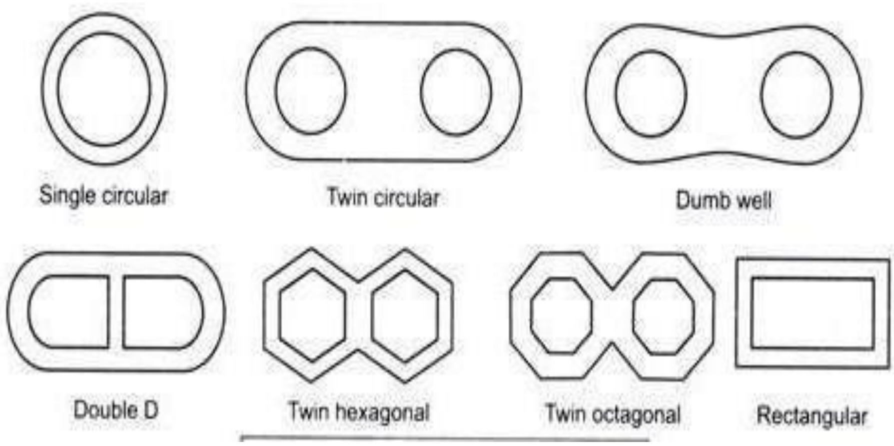


Fig: Common types of well shapes

- The choice of a particular shape of well depends upon the size of the pier, the care and cost of sinking, the considerations of tilt and shift during sinking and the vertical and horizontal forces to which well is subjected.
- A circular well has the minimum perimeter of a given dredge area. Since the perimeter is equidistant at the points from the centre of dredge hole, the sinking is more uniform than the other shapes. However, the circular well is that in the direction parallel to the span of bridge, the diameter of the well is much more than required to accommodate minimum size of pier and hence circular well obstruct water way much incomparison to other shapes.

WELL FOUNDATION

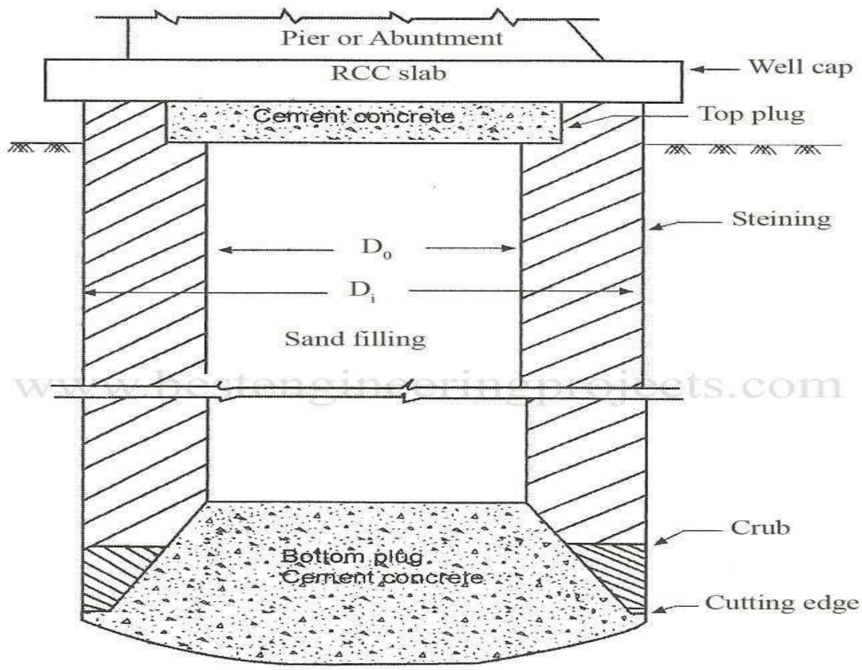


Fig 1 Parts of a Well Foundation

- **Well Cap –** The well cap is a RCC slab of sufficient strength to transmit the forces from pier to the body of well. It is generally kept at low water level. The dimension of the well cap should be sufficient to accommodate the pier. The recommended minimum thickness is 0.75 m.
- **Steining –** It is the wall of well & is built over a wedge shaped portion called Bwell curb. The steining is designed such that it can be sunk under it own weight. The thickness should be sufficient so as to overcome skin friction developed during sinking by its own weight.
- The minimum reinforcement in the well staining should be 5 to 6 kg/m of which 75 % is to be provided as vertical and 25 % as lateral ties or hoop rings. The minimum thickness is 0.45 m or 1/ 8 of the external dia. of well for brick masonry and 1/10 for wells with cement concrete. The thickness is increased by 12 cm per each 3 m depth after sinking the 3 m of steining for brick well

and 15 cm for each 6 m depth after sinking the first 6 m of cement concrete well.

- The thickness can be computed as:

$$t = D/2[\ 1 - \sqrt{(1-4q_s/\text{Ï•}D)}\ ]$$

- Where D is the diameter of well, qs is the unit skin friction, It is the unit weight of concrete or Brick masonry. The thickness of the steining is afunction of the size of the well.
- **Well Curb** – The well curb supports the steining. The curb should be slightly projected from the steining to reduce the skin friction during sinking of well. It is made of RCC with steel cutting edge. The minimum reinforcement to be provided is 72 kg per cubic meter. The inner portion of the steining should have a slope of 2:1 (V:H).
- **Cutting Edge –** The cutting edge is either projected below the curb as a sharpedge or can also have flat bottom. The projected edge is likely to be damaged in strata of gravels and boulders. In such soils the flat bottom cutting edge is provided.
- **Bottom Plug –** The bottom plug is made bowled shape in order to have an arch action. The bottom plug transmits load to soil below. When sunk to its final depth bottom part is concreted to seal the bottom completely. The thickness varies from ½ to full inside diameter of the well so as to be able to resist uplift forces. The concreting should be done in one continuous operation. When wells contain more than one dredge hole all should be plugged to the same height. If the well is to rest on rock, it should be anchoredproperly by taking it 25 cm to 30 cm deep into rock The bottom plug should be of rich concrete (1:2:4) with extra 10 % of cement.

The thickness of the bottom plug is given by:

$t^2 = 1.18R(q/f_c)$, for circular well and

$t^2 = [3qb^2/\{4f_c(1 + 1.61\text{Î±})\}]$, for rectangularwell

Where q is the bearing pressure at base ofwell,

R is the radius of the arc b is the width of well,

f_c is the flexural strength of concrete seal and,

Î± = Width/Length.

- **Sand Filling –** After concreting the bottom plug the sand is filled above the bottom plug and below top plug. Sand filling provide stability of well, reduce tensile stress produced by bending moment and distributes the load of super structure on to the bottom plug. Sand filling relieves load to steining to someextent.

- **Top Plug –** This is a plug at the top of the well below the well cap. This helps transferring the load through the granular material into the staining.

Forces Acting on a Well Foundation

In addition to the self-weight and buoyancy, it carries the dead load of superstructure, bearing and piers and subjected to the following horizontal forces:

- Braking effort of the moving vehicle.
- Force due to the resistance of bearings against movement due to temperature variations.
- Force of water current
- Seismic forces
- Wind force
- Earth pressure

Construction Procedure:-

- Layout
- Fabrication of cutting edge
- Well curb
- Construction of steining
- Island construction
- Well Sinking
- Plugging
- Sand filling
- Casting of well cap

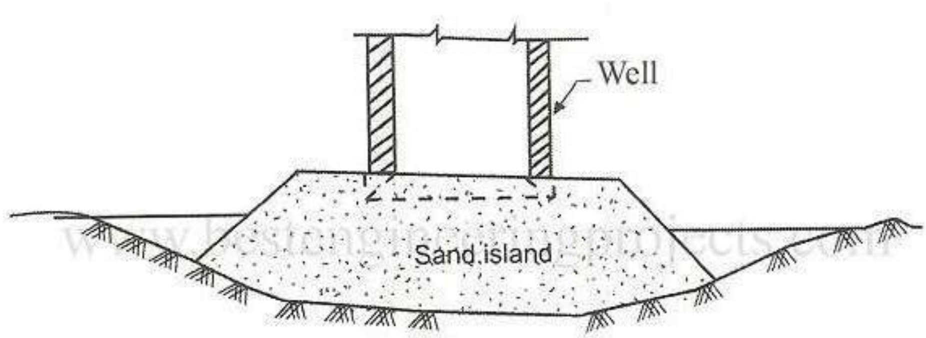
Sinking Operations

- Concrete the steining
- Dredge inside the well
- Sink the well in stages
- Sinking is done by uniform excavation of material
- Use of water jetting and explosives may be done
- Normally dewatering should not be done
- Tilts must be rectified wherever necessary

Sinking Operations

- **Laying of Curbs –** In dry ground excavate up to 50 cm in river bed and place the cutting edge at the required position. If the curb is to be laid under water and depth of water is greater than 5 m, prepare Sand Island and lay the curb. If depthof water exceeds 5 m built curb in dry ground and float it to the site. A typical sand island is shown in Figure.

Typical Sand Island



- **Construction of Well Steining**

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

Precautions

- When two wells sunk near each other, they should be sunk alternately
- Least possible area must be disturbed in vicinity
- In sinking of dumb bell shaped well, excavation must be donesimultaneously
- Dredged material must not be accumulated near well
- In sinking of two wells through sand, timber logs are provided betweensteining
- Care must be taken when cutting edge approaches junction of strata

DIFFICULTIES IN SINKING

Sinking Well Through Clay Strata

- It is one of the tough situations to face as well becomes stationary.
- Tilting occurs due to horizontal force by water.
- The well becomes vulnerable to tilt if a step is provided on outside face of the well steining to reduce
- It may lead to a very expensive and time-consuming affair for attempting tomake well straight and vertical.

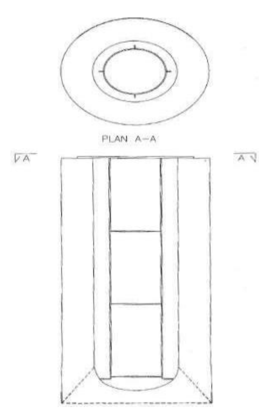
Measures Adopted

- Remove soil in contact with the outside surface of the well by grabbingto a certain depth.
- Continue grabbing much below the cutting edge level of the well.
- Dewatering well results into increasing effective weight.
- Flushing with jet of water on the outside face of well.

CASE STUDY

- Outside projection of well curb was 75 mm.
- Pipes were not kept in well steining to inject water on the outside surface of well.
- Stiff Clay was observed below the curb.
- It was overcome by cutting stiff clay layer by a jet of waterthrough a pipe using a high pressure pump.
- This pipe was supported on a circular frame.
- Actually 4 vertical pipes were fixed at 4 quadrants.
- Thus this method was found to be quite effective, safe and efficient method.

Elevation of Welland Pipelines



Sinking In Bouldery Strata

- Bouldery strata is treated in 3 ways-

✓ If they are lying loose,than with grabbing.

✓ If cemented but not so firmly, than underwater blasting.

✓ If cemented very firmly, than Pneumatic sinking.

- Soil investigation is required to decide method to be adopted.

- Three dimensional final element analysis is to be done for eccentric blastingforce.

- The steining of well must be designed in vertical direction as well in thetransverse direction providing appropriate design reinforcement.

- The wells of Varanasi Bridge of 13 m dia,2.5 m thick steining andup to 67 m deep is the best example of it.

Formation of Heaves

- When a well passes through soft strata over a considerable depth, the upward resistance acting on the outside surface of the well is less than the weight of the well.

    - Thus the well sinks down and quite often a heap is formed inside the dredge hole.

- Formation of heave at the designed foundation level, creates problem oflaying bottom plug.

- Bottom plug laid in the dredge hole, does not serve its purpose.

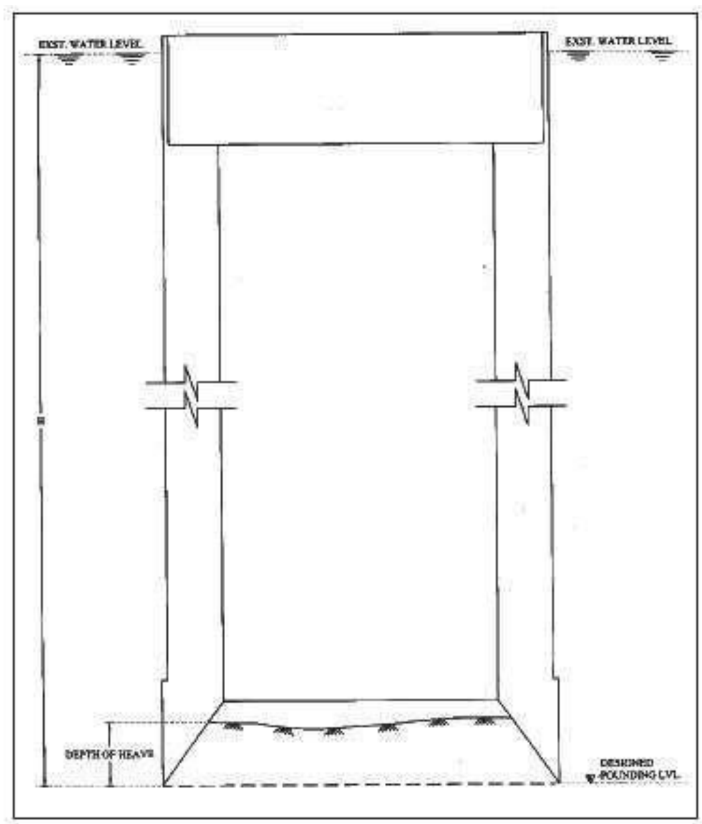


Fig: Formation of heaves

Solution for Heave formation

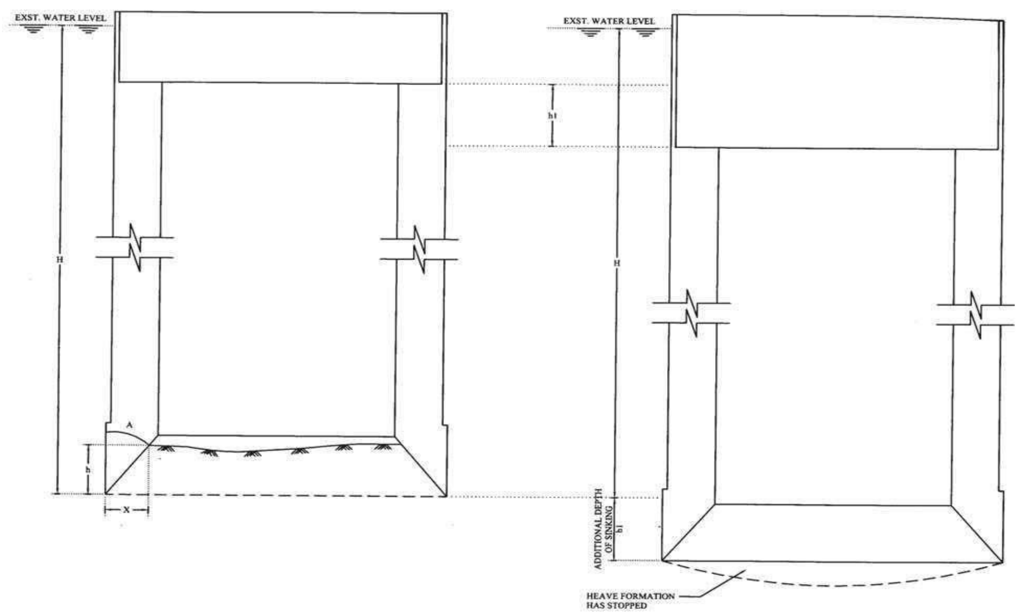


Fig: Showing additional sinking without any further concretesteining

- Achieving the condition that weight of well will be resisted throughoutside frictional forceonly.
- No further concreting of steining well should be done and sinking iscontinued.
- It will result into constant weight of well, and the frictional force outsidewill increase veryfast.

- When the heave develops, the equilibrium of the well takes place as perfollowing equation:

$$W = F + qa$$

- F = frictional force in the vertical direction along the outside surface ofthe well.
- q = bearing pressure of soil.
- a = area of the well supported over the heave.

General Measures for Ease of Sinking

- Appropriate choice of cutting edge and adoption of proper detailing.
- The "Angle iron" cutting edge works well when the well passes through alluvialsoil strata without any hard obstruction.
- A "V type" cutting edge is more appropriate in meeting various obstructivesituation provided correct detailing is adopted.
- The inclined plate should be stopped about 25 mm above the bottom tip of verticalplate.
- Adequate no. of Bore logs must be taken in the location of each well.
- Presence of very large boulder covering a part of the well at some depth in thebridge over Brahmaputra at Jogighopa.
- Similar type of problems including sudden change of bed profile is encountered invarious rivers in India.

Causes of Tilt and Shift

- Non uniform bearing capacity
- Obstruction on oneside of the well
- Sand blowing in wells during sinking. It will cause sudden

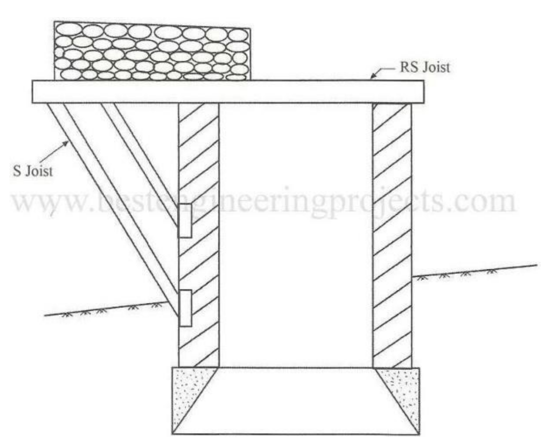
sinking of well

- Method of sinking : Material should be removed from all sides equally otherwise the well may experience tilt
- Sudden sinking due to blasting may also cause tilting of well
- Irregular casting of steining will cause less friction on one side leads tocances of tilting of well

Rectification of Tilt

- Eccentric grabbing
- Eccentric loading
- Water jetting
- Arresting the cutting edge
- Pulling the well
- Strutting the well
- Pushing the well by jacks

ECCENTRIC LOADING



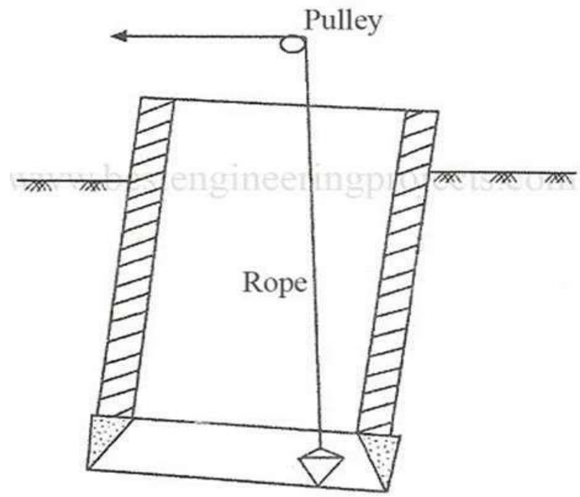


Fig (a) Dredging the Well

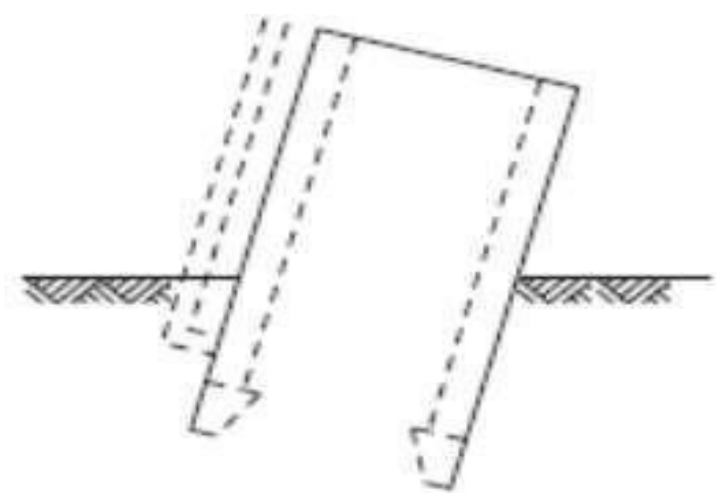
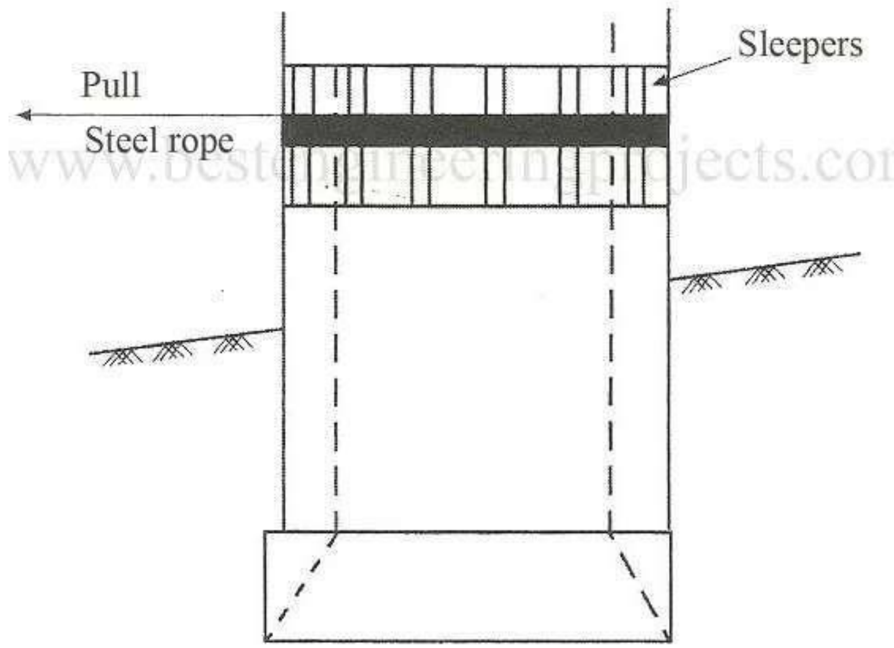
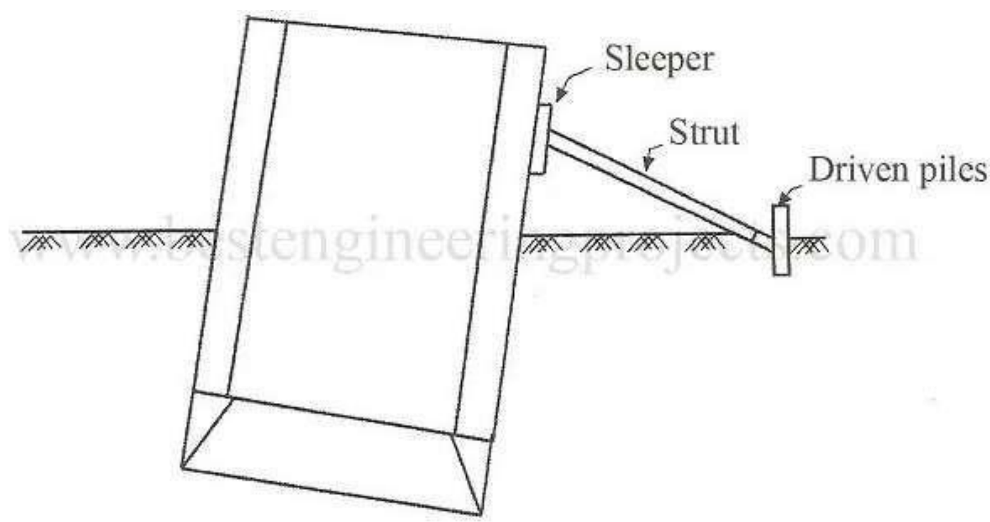


Fig: b) Water Jetting

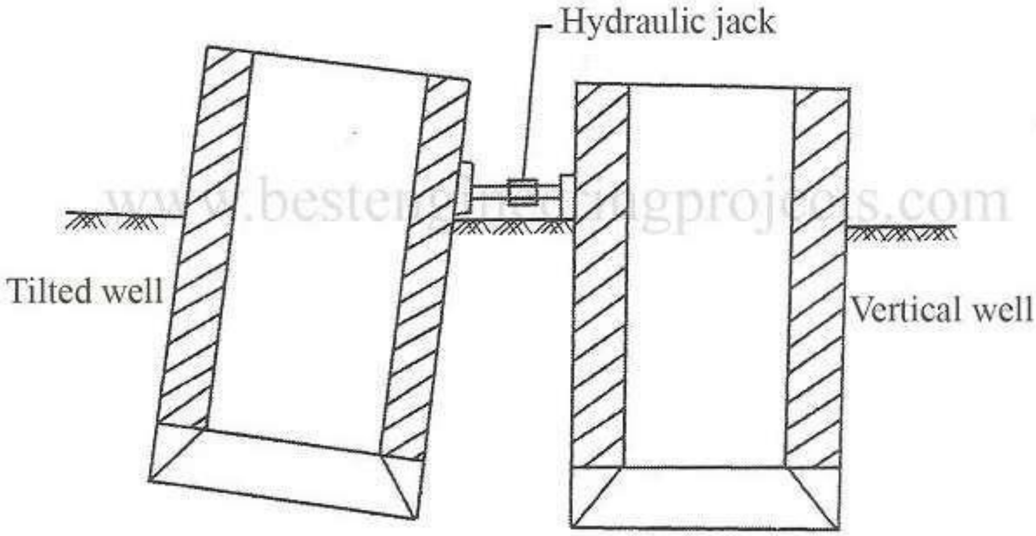
PULLING THE WELL



STRUTTING THE WELL



PUSHING THE WELL



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- Manual on *"Design and Construction of Well and Pile Foundation"*.
- IRC-21.
- IRC-78

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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT –V - TOWER FOUNDATION– SCIA7007

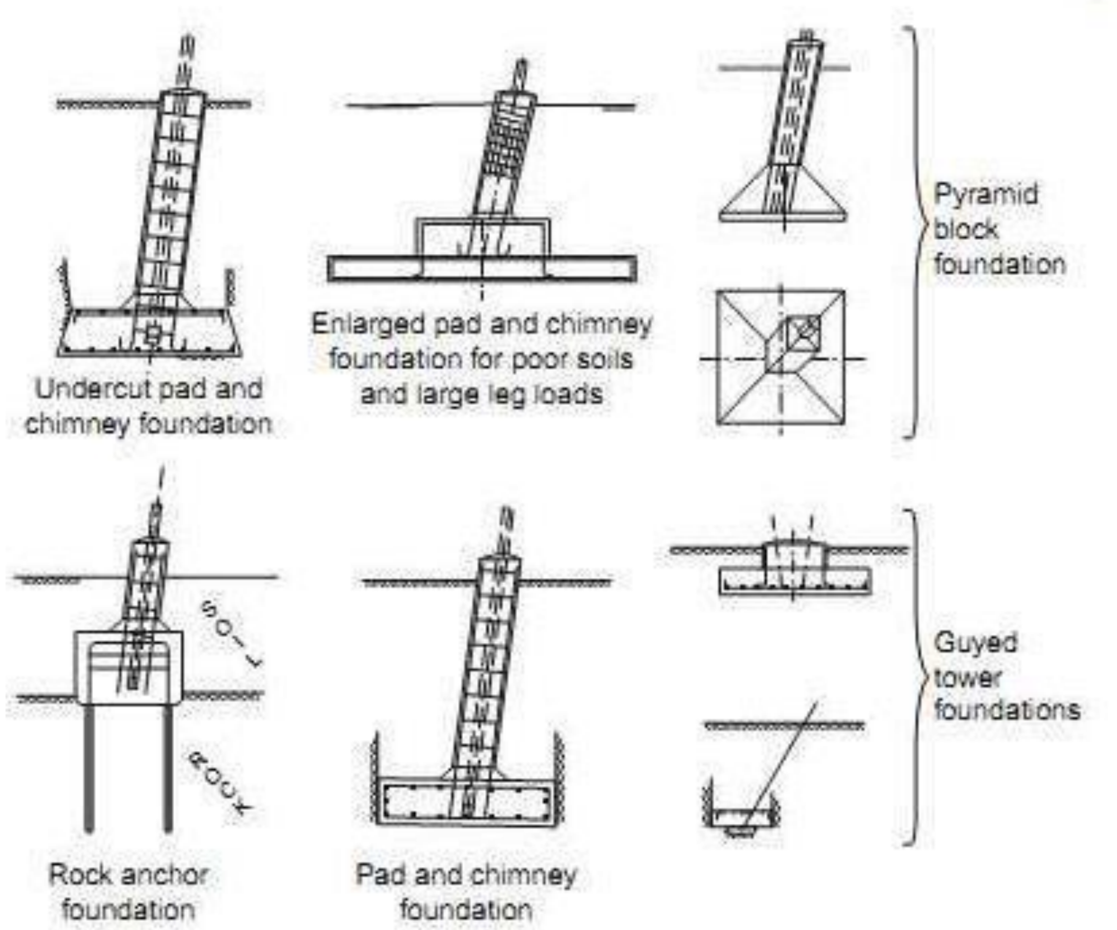
UNIT V

DESIGN OF FOUNDATION OF TRANSMISSION TOWERS

Foundation of any structure plays an important role in safety and satisfactory performance of the structure as it transmits mechanical loads of the electrical transmission system to earth. A transmission structure without having a sound and safe foundation, it cannot perform the functions for which it has been designed. The foundations in various types of soils have to be designed to suit the soil conditions of particular type.

In addition to foundations of normal towers, there are situations where considering techno- economical aspect for special towers required or river crossing which may be located either on the bank of the river or in the mind stream or both, pile foundation may be provided.

TYPES OF FOUNDATION OF TRANSMISSION TOWERS



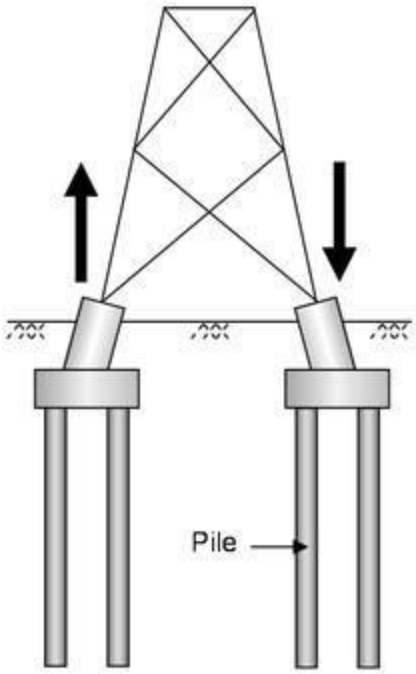
Types of loads on Foundation

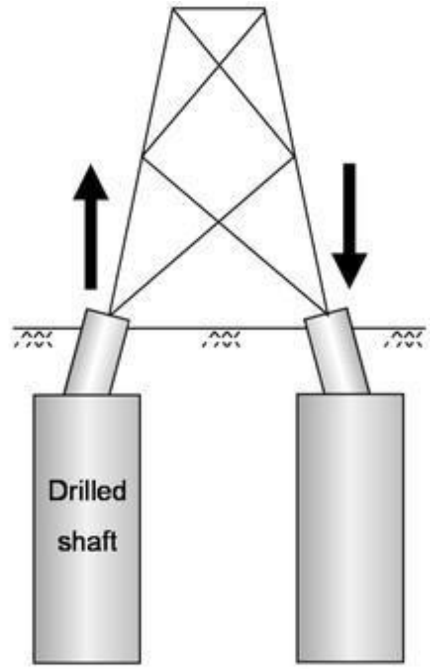
The foundation of towers normally subjected to three types of forcesThese are

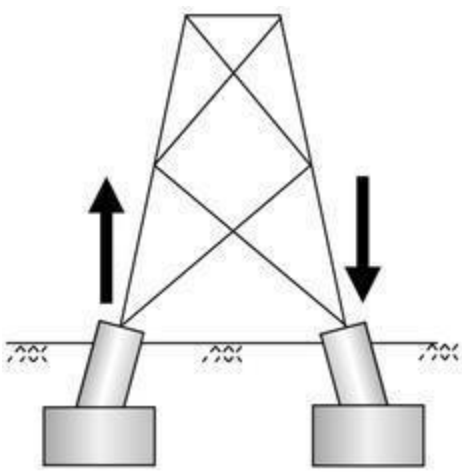
a) The Compression or downward thrust

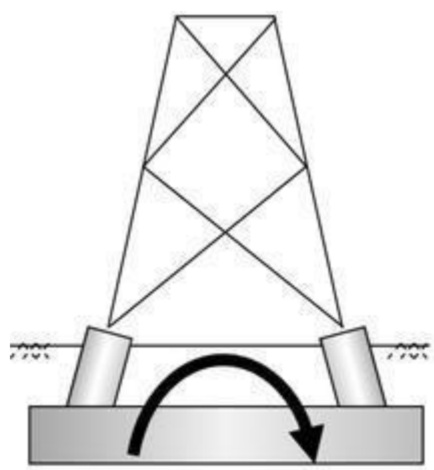
b) The Tension or uplift

c) The lateral forces of side thrust in both transverse and longitudinal directions


Pile


Drilled
shaft





The magnitude or limit loads for foundations should be taken 10% higher than these for the corresponding towers.

The base slab of the foundation shall be designed for additional moments developing dueto eccentricity of the loads.

The additional weight of concrete in the footing below ground level over the earth weightand the full weight of concrete above ground level in the footing and embedded steel parts also be taken into account; adding to the down-thrust.

Soil parameters

For designing the foundations, following parameters are required.

- Limit bearing capacity of soil.
- Density of soil.
- Angle of earth frustum.

The above values are available from soil test report.

Guidelines for the Design of Foundation of Transmission Towers in Different Soils

1. All foundation shall be of RCC. The design and construction of RCC structures shall be carried out as per IS:456 and minimum grade of concrete shall be M-20.
2. Limit state method of design shall be adopted.
3. Cold twisted deformed bars as per IS:1786 or TMT bars shall be used as reinforcement.
4. Foundations shall be designed for the critical loading combination of the steel structure and orequipment and/or superstructure.
5. If required protection to the foundation, shall be provided to take care of any special requirements for aggressive alkaline soil, black cotton soil or any soil which is detrimental/harmful to the concrete foundations.
6. All structures shall be checked for sliding and overturning stability during both construction and operating conditions for various combination of loads.

7. For checking against overturning, weight of soil vertically above footing shall be taken and inverted frustum of pyramid of earth on foundation should not be considered.
8. Base slab of any underground enclosure shall also be designed for maximum ground watertable. Minimum factor of safety of 1.5 against bouncy shall be ensured.
9. The tower and equipment foundations shall be checked for a factor of safety of 2.2 for normal condition and 1.65 for short circuit condition against sliding, overturning and pullout.

Design procedure for foundation

The design of any foundation consists of following two parts.

1. Stability analysis

Stability analysis aims at removing the possibility of failure of foundation by tilting, overturning, uprooting and sliding due to load intensity imposed on soil by foundation being in excess of the ultimate capacity of the soil. The most important aspect of the foundation design is the necessary check for the stability of foundation under various loads imposed on it by the tower, which it supports.

The foundation should remain stable under all the possible combinations of loading, to which it is likely to be subjected under the most stringent conditions.

The stability of foundations should be checked for the following aspects.

Check for bearing capacity

The total downward load at the base of footing consists of compression per leg derived from the tower design, buoyant weight of concrete below ground level and weight of concrete above ground level.

While calculating over weight of concrete for checking bearing capacity of soil, the position of water table should be considered at critical location i.e., which would give maximum over weight of concrete. In case of foundation with chimney battered along the slope of leg, the center line of chimney may not coincide with the center of gravity of base slabs/pyramid/block. Under such situation, axial load in the chimney can be resolved into vertical and horizontal components at the top of the base slabs/pyramid/block. The additional moments due to the above horizontal loads should be considered while checking the bearing capacity of soil.

Further even in cases where full horizontal shear is balanced try the passive pressure of soil, the horizontal shears would caused moment at the base of footing as the line of action of side thrusts (horizontal shears) and resultant of passive pressure of soil are not in the same line. It may be noted that passive pressure of soil is reactive forces from heat soil for balancing the external horizontal forces and as much mobilized passive pressure in soil adjoining the footing cannot be more than the external horizontal shear.

Thus the maximum soil pressure below the base of the foundation (toe pressure) will depend up on the vertical thrust (compression load) on the footing and the moments at the base level due to the horizontal shears and other eccentric loadings.

Check for uplift resistance

In the case of spread foundations, the resistance to uplift is considered to be provide by the buoyant weight of the foundation and the weight of the soil volume contained in the inverted frustum of cone on the base of the footing with slides making an angle equal to the angle of earth frustum applicable for a particular type of the soil.

Up = Ws + Wf

Where Ws is the weight of the soil in the frustum of cone Wf is the buoyant weight /overload of the foundation.
Depending up on the type of foundation i.e., whether dry or wet or partially submerged or fully submerged, the weights Ws & Wf should be calculated taking into account the location of ground water table.

Under-cut type of foundation offers greater resistance to uplift than an identical footing without under-cut. This is for the simple reason that the angle of earth frustum originates from the toe of the under-cut and there is perfect bond between concrete and the soil surrounding it and there is no need to depend on the behavior of back filled earth. Substantial additional uplift resistance is developed due to use of under-cut type of foundation. However, to reflect advantage of additional uplift resistance in the design the density of soil for under-cut foundation has been increased as given in Table of Annexure.

In cases where frustum of earth pyramid of two adjoining legs overlap, the earth frustum is assumed truncated by a vertical plane passing through the center line of the tower base.

Check for side thrust

In towers with inclined stub angles and having diagonal bracing at the lowest panel point, the netshearing force of the footing is equal to the horizontal component of the force in hte diagonal bracing whereas in towers with vertical footings, the total horizontal load on the tower is divided equally between the numbers of legs. The shear force causes bending stresses ink the unsupported length of the stub angles as well as in the chimney and tends to overturn the foundation.

When acted upon by a lateral load, the chimney will act as a cantilever beam free at the top and fixed at the base and supported by the soil along its height. Analysis of such foundations and design of the chimney for bending moments combined with down thrust uplift is very important. Stability of a footing under a lateral load depends on the amount of passive pressure mobilized inthe adjoining soil as well as the structural strength of the footing in transmitting the load to the soil.

Check for over-turning

Stability of the foundation against overturning under the combined action of uplift and horizontalshears may be checked by the following criteria

i The foundation over-turns at the toe

ii The weight of the footing acts at the center of the base and

iii Mainly that part of the earth cone which stands over the heel causes the stabilizing moment. However, for design purposes this may be taken equal to the half of the cone of earth acting on the base. It is assumed to act through the tip of the heel.

For stability of foundation against overturning, factor of safety shall notbe less than 1.5 (DL + LL + WL) (IS: 1904-1986)

Check for sliding

In the foundation of towers, the horizontal shear is comparatively small and possibility of sliding is generally negligible. However, resistance to sliding is evaluated assuming that passive earth pressure conditions are developed on vertical projections above the toe of foundations. The friction between bottom of the footing and soil also resist the sliding of footing and can be considered in the stability of foundation against sliding. The coefficient of friction between concrete and soil can be considered between 0.2 and 0.3. However, the frictional force is directly proportional to vertical downward load and as such may not exist under uplift condition. For cohesive soil the following formula can be applied for calculating the passive pressure to resist sliding.

$Pp = 2C \tan + \gamma h \tan2 \theta$Where C = Cohesion

$\theta = 45o + 1/2$ of angle of earth frustum

H = height of foundation

γ = unit weight of soil

For stability of foundation against sliding.

Factor of safety shall not be less than 1.5(DL + LL + WL) (IS: 1904-1986)

Check for Uprooting of Stub

Over Load Factor of 10% (Ten percent) shall be considered i.e. Over Load Factor = 1.10 for normal suspension towers and 1.15 for angle tower including Dead end/anchor tower. For special towers Over Load Factor shall be 1.20.

Check for Down Thrust

The following load combinations shall be resisted by the bearing strength of the soil:

1. The down thrust loads combined with an additional weight of concrete above earth areassumed to be acting on the total area of the bottom of the footing.
2. The moment due to side thrust forces at the bottom of the footing.

The structural design of the base slab shall be developed for the above load combination. In case of toe (τ) pressure calculation due to above load combination allowable bearing pressure to be increased by 25%.

Steps involved in the Construction of Tower Foundation

1. Excavation
2.PCC
3. Stub setting
4. Template alignment
5. Concreteing
6. Curing
7. De shuttering and template removal
8. Back filling

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6. Barken, “Dynamics of Bases and Foundations”, McGraw Hill Company, 1962.

SCIA 7007	**DESIGN OF SUB STRUCTURES**		
Q.No	**Question**	**Course Outcome**	**Level**
	UNIT I –SITE INVESTIGATION, SELCTION OF FOUNDATION AND BEARING CAPACITY		
	PART A		
1	Can you suggest the methods of exploration available for taking disturbed and undisturbed samples of soil?	CO1	L3
2	How do you fix the significant depth of exploration	CO1	L 3
3	Could you outline the general considerations in the choice of the foundation type?	CO1	L 3
4	Show the equation for ultimate bearing capacity for a strip foundation corresponding to general shear failure.	CO1	L 2
5	Identify the major factors that affect bearing capacity of a footing.	CO1	L 3
6	Interpret the types of failure associated with different soils.	CO1	L 4
7	Estimate the bearing capacity of square footing 2.5 m x 2.5 m located at a depth of 2m in sand with angle of internal friction 25° and bulk density 18.5 kN/m^3 Assume factor of safety=3 N_q=12.5, N_r = 9.5.	CO1	L 5
8	A square footing 2.1 m x 2.1 m is built in a homogeneous bed of sand of unit weight 20 kN/m^3 and having an angle of shearing resistance of 36°. The depth of the base of footing is 1.5m below the ground surface. Determine the safe load that can be carried by a footing with a factor of safety of 3 against complete shear failure. Use Terzhagi's analysis.	CO1	L 5
9	Discuss the importance of bore log in soil investigation?	CO1	L 6
10	Briefly discuss disturbed and undisturbed sampling.	CO1	L 6
11	How could you discuss the different types of bearing capacity failures with their significant characteristics?	CO1	L 6
	UNIT I - PART B		
1	What is the required depth of soil exploration to be done for different civil engineering works? What is the number and spacing of bore holes/ open pits for different works?	CO1	L 1
2	Illustrate how the bearing capacity is determined by plate load test by explaining the method of performing it. Also indicate its limitations.	CO1	L 2
3	Experiment with the standard penetration test for subsurface investigation. What are the various corrections applied to standard penetration number.	CO1	L 3
4	List out and Discuss the factors to be considered in design of foundations.	CO1	L 4
5	Derive the Terzaghi's theory of ultimate bearing capacity and the equations for the net ultimate bearing capacity and safe bearing capacities for strip footing.	CO1	L 5

6	A square column foundation has to carry a gross allowable total mass of 15290 kg. The depth of foundation is 0.7m. The load is inclined at an angle of 20 degrees to the vertical. Estimate the width of foundation assuming a factor of safety of 3. Take N_q = 18.4 and N_γ = 22.4. Use BIS recommended procedure. Use the following soil parameters: c' = 0; Ø = 30°; γ = 18 kN/m^3.	CO1	L 6
7	A square footing 3.5m x3.5m is built in a homogenous bed of sand carries a gross pressure of 4000 KN/m^{2}unit weight 20kN/m^3 and having an angle of shearing resistance of 30º. The depth of the base of the footing is 1.5m below the ground surface .Calculate the factor of safety to the following cases a)water table is 5 m below the G.L b)water table is 1.2 m below the G.L . Use Terzhaghi's analysis. (Nq=22, Nγ=20)	CO1	L 6
8	Explain all types of boring method with neat sketch.	CO 1	L5
9	How do you elaborate the complete detailing of standard penetration test?	CO 1	L5
10	How would you adopt geophysical methods of site investigation in your study?	CO 1	L6
11	Explain about the bore log and also the features of the investigation report.	CO 1	L5
12	How do you elaborate the semi direct methods of exploration and its limitations?	CO 1	L5
13	What inference can you make by conducting electrical resistivity method and Seismic refraction method of exploration?	CO 1	L4
14	Elaborate the construction procedure of sub surface sounding method of exploration	CO 1	L4
	UNIT II – DESIGN OF SHALLOW FOUNDATION		
	PART A		
1	What are the important considerations in the structural design of raft foundation?	CO2	L2
2	How do you locate the foundation in a sloping ground as per the codal provisions?	CO2	L 2
3	Show the critical section for one way shear and two way shear for isolated squares footing for a column.	CO2	L 2
4	Summarize the IS code recommendations for the calculation of the ultimate bearing capacity of shallow footings.	CO2	L 2
5	Model the critical sections for bending moment in continuous footing under masonry wall and RC wall.	CO2	L 3

6	Solve and Determine the plan dimensions for a combined footing for two interior columns A and B, carry axial loads 1000 kN and 1200 kN and spaced 4m centre to centre. Column A is 400 mm x 400 mm in size and column B is 450 mm in diameter. The safe bearing capacity of soil is 120 kN/m^2. The property line is at a distance of 0.5 m left to the centre of column A.	CO2	L 3
7	Examine the concept of raft foundation.	CO2	L 4
8	Discuss the general principle of design of Mat or Raft footing	CO2	L 5
9	What are the inferences that you arrived in the pressure distribution of rigid footing?	CO2	L 5
10	Discuss the design principles involved in buoyancy raft.	CO2	L 6
	UNIT II – PART B		
1	Find the plan dimensions of a rectangular combined footing to support two columns 250mm x 250 mm and 300 mm x 300m carrying loads of 300kN and 450 kN respectively. The columns are spaced at 4mc/c. the first column is on the boundary line. The allowable bearing capacity of the soil is 150 kPa. If the second column is also on the boundary line, Select the plan dimensions of another combined footing.	CO2	L 6
2	Design a combined footing connecting two columns 400 mm x 400 mm and 300 mm x 300 mm in section carrying loads of 750 kN and 300 kN respectively and spaced at 4.5 m c/c. The allowable soil pressure may be taken as 130 kN/m^2.	CO2	L 6
3	A building consists of 12 columns 400 mm x 400 mm arranged in three rows of four each. The distance between the columns is 4.5 m each. The load carried by corner column is 500kN each and exterior columns carry 600 kN and interior column carry 800 kN each. The allowable soil pressure may be taken as 60 kN/m^2. Using the conventional method, perform the structural design.	CO2	L 6
4	A square column of size 500mm x 500mm carries an axial load of 1000kN including its weight. The safe bearing capacity of the soil is 150 kN/m^2. Design the square footing for the column using limit state method. Use M20 grade and Fe415 steel.	CO2	L 6
5	Design a R.C square footing on sand with a safe bearing capacity of 275 kN/m^2 to carry a dead load of 800 kN and live load of 400 kN from a 40 x 40 cm column.	CO2	L 6
6	Design a strip footing for a 30cm brick wall to carry a service load of 156 kN/m. Assume safe bearing capacity of soil as 250 kN/m^2.	CO2	L 6
7	Design a combined footing for two columns 500mm x 500 mm and 400 mm x 400 mm in sections carrying loads of 800 kN and 400 kN respectively spaced at 5m c/c. Take allowable soil pressure as 130 kN/m^2 and coefficient of subgrade reaction of 7.5 x 10^4 kN/m^2.	CO2	L 6

8	A concrete strip footing rectangular in section is located at ground level and extends 1.2m below the ground level. It carries uniformly distributed load of 15000 kg/m. The soil profile consists of homogeneous clay of 6m thick overlying rock. The clay properties are as under: Saturated bulk unit weight = 1750 kg/m3; Shear strength (undrained)= 8500 kg/m^2;Compressiblity = $1x10^{-4} m^2/100kg$. Determine i) Width of footing for factor of safety = F. ii) Ultimate consolidation settlement for F=2. Assume bulk unit weight of concrete = 2500 kg/m^3. Neglect the spread of load beneath the footing and any side cohesion on the foundation.	CO2	L6
	UNIT III – PILE FOUNDATION		
	PART A		
1	Can you state the dynamic formulas to calculate the pile load carrying capacity?	CO3	L 1
2	What are the different tests that can be carried out on piles?	CO3	L 2
3	How could you classify the piles based on functions, mode of transfer of load and method of installation with neat sketches?	CO3	L 3
4	Describe briefly any two non-destructive tests which can be conducted on piles.	CO3	L 2
5	Identify the situations suitable for providing pile foundation. How are piles classified? Describe a method for determining the safe capacity of single pile.	CO3	L 3
6	Choose a method to determine the uplift capacity of piles.	CO3	L 3
7	How could you explain the principals involved in the design of pile cap?	CO3	L 4
8	Justify how the negative skin friction develops in pile foundations.	CO3	L 5
9	A group of 9 piles arranged in a square pattern with diameter of each pile as 30 cm and length as 10m is used as a foundation in soft clay deposit. Taking the unconfined compressive strength of clay as 120 kN/m^2 and pile spacing as 100cm centre to centre, estimate the load capacity of the group. Assume bearing capacity factor N_C = 9 and adhesion factor = 0.75. A factor of safety of 2.5 may be taken.	CO3	L 6
10	Discuss how the pile derives its load capacity.	CO3	L 6
	UNIT III - PART B		
1	What are the various methods for estimating the load carrying capacity of piles? Elaborate any one method.	CO3	L 5
2	How could you conduct different load tests on piles? Explain the test carried out for carrying capacity of vertical loads.	CO3	L 5
3	A pile group consists of 9 piles of 30 cm diameter and 10 m length driven in clay. Unconfined compressive strength of soil is 200 kN/m^2 and in situ density of soil is 20 kN/m^3. Determine the safe load for the pile group.a = 0.6 and F.S. = 3.	CO3	L 5

4	Determine the safe load carrying capacity of 4 x 4 pile group made of 450mm dia concrete piles at 1500mm spacing. The sub soil is made of clay c = 18kN//m^3 and may exhibit negative skin friction for a depth of 3m from ground surface. The soil layer below this is stiff clay with unconfined compressive strength of 120 kN/m^2 and extends to the full length of pile.	CO3	L 5
5	A column 550 mm square has to carry a factored load of 2600 kN to be supported on 4 piles each of 450 mm diameter and spaced at 1350 mm centres. Design a suitable pile cap assuming M25 concrete and Fe415 steel. Show the detailing of reinforcement in the pile cap.	CO3	L 6
6	Design a pile system to carry a load of 2100 kN in a clay soil. Use the following data: Diameter of pile: 250mm; spacing of pile = 500 mm; Pile length:8m; Unconfined compressive strength of soil = 100 kN/m^2; Adhesion factor : 0.9	CO3	L 6
7	A concrete pile 300 mm in diameter is 10 m long and is driven by a double acting hammer (rated energy = 18.62 kJ and total mass = 2200 kg). The driving was done with a short dolly and cushion 2.5m. The average penetration recorded in the last 5 blows was 3mm/blow. Estimate the safe pile load if the factor of safety is 3. The hammer efficiency may be taken as 90% and the coefficient of restitution between pile head and hammer is 0.5.	CO3	L 6
8	A raft foundation is supported by pile group consisting of 15 piles arranged in 3 rows. The diameter and length of each pile are 300mm and 15m respectively. The spacing between he piles is 1.2m. Foundation soil having soft clay layer C= 3.2t/m^2; γ= 1.9t/m^3, Determine the capacity of pile group.	CO3	L 6
	UNIT IV – WELL AND CAISSON FOUNDATION		
	PART A		
1	What are the advantages of well foundation?	CO4	L 1
2	Classify the caissons based on the method of construction?	CO4	L 2
3	Model the components of well foundation with neat diagram.	CO4	L 3
4	List and Explain the various forces acting on well foundation.	CO4	L 4
5	Analyse how the grip length of a well foundation is determined?	CO4	L 4
6	Describe the steps involved in sinking of well.	CO4	L 5
7	Assess the problems associated with the sinking of caissons	CO4	L 5
8	Elaborate the procedure for checking the ultimate failure condition of well foundations.	CO4	L 6
	UNIT IV - PART B		
1	Write short note on bearing capacity of well foundation. Describe how the allowable bearing pressure for sands can be calculated for safety against shear failure.	CO4	L 1
2	Explain the construction of a caisson foundation with a suitable sketch.	CO4	L 2

3	A bridge pier is supported on two round caissons that are to the rest on hard un-weathered marl at a depth of 35m below the river bed. The caissons are to carry 30,000kN of load. The skin friction of the material above the marl layer may be taken as $15kN/m^2$. Estimate the diameter of the caissons and thickness of plug. Use M20 grade of concrete.	CO4	L 5
4	An open caisson 19 m deep has external and internal diameters of 8m and 6m respectively. If the water seal is 2m below the top of the well and the depth of base below the scour level is 5m, determine the minimum thickness of the seal that will enable complete dewatering of the caisson. Take allowable concrete flexural stress, $\sigma_c = 2000\ kN/m^2$ and allowable perimeter shear of $650\ kN/m^2$	CO4	L 5
5	Check for the stability of a caisson whose particulars are given below, using elastic theory approach. Total downward load acting at the base of the caisson = 17718 kN. Total lateral load applied at scour level = 2161kN. Total external moment at base of well = 39718 kNm. Diameter of well B = 10m, length of well = 0.9B. Angle of friction at caisson surface = 17.5°. Angle of shearing resistance of soil is 30°. Depth of foundation below scour level = 5.8m. Assume m = 1. Safe bearing capacity of soil for concentric load = $450\ kN/m^2$.	CO4	L 6
6	Check the lateral stability of the well as per the procedure laid down by IRC:45(1972).The following data refers to a well foundation for single line railway bridge: a)Net downward load on well including self-weight = 1400 t b)Horizontal force at scour level= 200 t c) Moment at scour level = 4150 t-m d)Depth of well below scour level = 15m e)Saturated unit weight of sand = $2\ t/m^3$ f)Angle of shearing resistance of subsoil = 35° g)Angle of wall friction = 20° h) External diameter of well = 8.5m i)Internal diameter of well = 5.5m j)Allowable bearing pressure = $55\ t/m^2$	CO4	L 6
7	A circular well of 4.5m external diameter and 0.75m steining thickness is embedded up to a depth of 12m in a uniform sand deposit. The angle of shearing resistance of sand and submerged unit weight are 30° and 1 t/m^3., respectively. The well is subjected to a resultant horizontal force of 50 t and a total moment of 400 t-m at the scour level. Assuming the well to be a light well, compute the allowable total equivalent resistive force due to earth pressure. A factor of safety of 2 may be adopted for soil resistance. Determine the magnitude and point of maximum bending moment in the well steining. What will be the change in computed value for a heavy well when the well is assumed to rotate about the base?	CO4	L 6
	UNIT V – TOWER FOUNDATION		
	PART A		

1	How do you state the general design criteria to be followed for the satisfactory performance of a tower foundation?	CO5	L 2
2	What do you understand by the various types of foundation used for chimney tower?	CO5	L2
3	Explain stability against overturning in tower foundation.	CO5	L 2
4	How do you check the stability of the foundation for transmission towers?	CO5	L 3
5	Discuss the forces acting on foundation of tower.	CO5	L 5
	UNIT V - PART B		
1	How do you elaborate the different types of foundations used for steel towers? What are the forces acting on them?	CO5	L 4
2	Describe how steel towers can be provided with anchorage in rocks. Draw neat sketch showing the foundation for a tower with rock anchors.	CO5	L 5
3	How is the safety of a tower foundation checked against overturning, uplift and lateral thrust? Explain.	CO5	L 5
4	Elaborate with neat sketch the following types of foundation for tower :(a) Bored under reamed pile foundation (b) Grillage foundation.	CO5	L 6