



Objectives of soil exploration:

Following are the objectives of site investigation or subsurface exploration.

1. Determination of the nature of the deposits of soil.
2. Determination of the depth and thickness of various soil strata and their extent in horizontal direction.
3. Location of ground water and fluctuations in ground water level.
4. Obtaining soil and rock samples from the various strata.
5. Determination of engineering properties of soil and rock strata that affects the performance of the structure.
6. Determination of in-situ properties by performing field tests.
7. To know about the order of occurrence of soil and rock strata.
8. To know about the location of the groundwater table level and its variations.
9. To select a suitable type of foundation.
10. To estimate the probable and maximum differential settlements.
11. To find the bearing capacity of the soil.
12. To predict the lateral earth pressure against retaining walls and abutments.
13. To select suitable soil improvement techniques.
14. To select suitable construction equipment.
15. To forecast problems occurring in foundations and their solutions.

Purpose of soil exploration :

- (i) To determine the basic properties of soil which affect the design and safety of structure i.e., compressibility, strength and hydrological conditions.
- (ii) To determine the extent and properties of the material to be used for construction.
- (iii) To determine the condition of groundwater.
- (iv) To analyze the causes of failure in existing works.

The nature and extent of soil exploration depends upon the ultimate use to which



transmit heavy load on the soil, the aim of soil exploration is to provide data which will help in the selection of proper types of foundation, its location and design of foundations.

Planning of Subsurface Investigation:

To obtain the most useful information at minimum cost and effort, proper planning of subsurface investigation program is essential.

For planning of the program, the soil engineer-in-charge of the program should include the following steps:

- (i) Completely familiar with the kind of information required from the investigation.
- (ii) Knowledge of type, size and importance of the project.
- (iii) Preparation of layout plan of the project,
- (iv) Preparation of borehole layout plan which includes the number and spacing of boreholes, depth and frequency of sampling.
- (v) Selection of proper drilling and sampling equipment.
- (vi) Selection of personnel to supervise the field investigation.
- (vii) Marking on the layout plan any additional types of soil investigation.
- (viii) Preparation of guidelines for laboratory testing of collected samples.

Stage of Subsoil Investigation:

Different stages of sub-soil investigation of a major civil Engineering project are mentioned below:

- (i) Reconnaissance study:
 - (a) Geological data
 - (b) Serial photographs
 - (c) Pedological data
- (ii) Preliminary site exploration
- (iii) Detailed investigation:
 - (a) Boring



- (c) Testing
 - (i) Lab test
 - (ii) Field test
- (d) Aerial photographs
- (e) Geophysical methods
- (iv) Performance study
 - (a) Further testing
 - (b) Instrumentation
 - (c) Performance evaluation
- (v) Preparation of Report of Sub-Soil Exploration

(i) Reconnaissance Study:

Site reconnaissance is the first stage of site investigation. In this stage, visual inspection of the site is done and information about topographical and geological features of the site are collected. It involves the preliminary feasibility study that is undertaken before any detailed planning is done. The main objective of this phase of exploration is to obtain rough idea about the soil type in the area. This study is aimed to get a rough soil profile and representative sampling of the major soil strata and groundwater condition which will be helpful in deciding the future programme of explorations. This study is to be done at minimum cost and no large scale exploratory work is usually undertaken at this stage.

The general observations made in site reconnaissance are as follows :

1. Presence of drainage ditches and dumping yards etc.
2. Location of groundwater table by observing well in that site.
3. Presence of springs, swamps, etc.
4. High flood level marks on the bridges, high rise buildings, etc. are observed.
5. Presence of vegetation and nature of the soil.
6. Past records of landslides, floods, shrinkage cracks, etc. of that region.
7. Study of aerial photographs of the site, blueprints of present buildings, geological maps, etc.
8. Observation of deep cuts to know about the stratification of soils.



(ii) Preliminary site exploration:

Preliminary site exploration is carried out for small projects, light structures, highways, airfields, etc. The main objective of preliminary exploration is to obtain an approximate picture of sub-soil conditions at low cost. It is also called general site exploration.

The soil sample is collected from experimental borings and shallow test pits and simple laboratory tests such as moisture content test, density, unconfined compressive strength test, etc. are conducted. Simple field tests such as penetration methods, sounding methods, geophysical methods are performed to get the relative density of soils, strength properties, etc.

The data collected about subsoil should be sufficient enough to design and build light structures. Following are some of the general information obtained through primary site exploration.

1. Approximates values of soil's compressive strength.
2. Position of the groundwater table.
3. Depth and extent of soil strata.
4. Soil composition.
5. Depth of hard stratum from ground level.
6. Engineering properties of soil (disturbed sample)

(iii) Detailed Soil Investigation:

In detailed soil investigation, boring, sampling and testing is done to obtain the engineering properties of soil.

Detailed exploration is preferred for complex projects, major engineering works, heavy structures like dams, bridges, high rise buildings, etc. A huge amount of capital is required for a detailed site exploration hence, it is not recommended for minor engineering works where the budget is limited. For such type of works, data collected through preliminary site exploration is enough.

In this stage, numerous field tests such as in-situ vane shear test, plate load test, etc. and laboratory tests such as permeability tests, compressive strength test on undisturbed soil samples are conducted to get exact values of soil properties.



Trial pits can be used for all types of soils. It is the cheapest way of site exploration and do not require any specialized equipment. In this method a pit is manually excavated and soil is inspected in the natural condition. Both disturbed and undisturbed sample can be conveniently taken. Trial pits are suitable for exploration of shallow depth only.

(v) Preparation of Report of Sub-Soil Exploration

After performing preliminary or detailed site exploration methods a report should be prepared. A sub-soil investigation or exploration report generally has the following sections :

1. Introduction
2. Scope of site investigation
3. Description of the proposed structure, purpose of site investigation
4. Site reconnaissance details
5. Site exploration details such as number, location and depth of boreholes, sampling details etc.
6. Methods performed in site exploration and their results.
7. Laboratory tests performed and their results.
8. Details of Groundwater table level and position.
9. Recommended improvement methods if needed.
10. Recommended types of foundations, structural details, etc.
11. Conclusion.



1.2 Method of soil exploration: Methods of drilling bore holes for subsurface investigations:

The following are the various methods used in soil exploration:

1. Test Pits or Trial Pits (for Shallow Depths)
2. Auger Boring (for Shallow or Large Depths).
3. Wash Boring.
4. Percussion Drilling.
5. Rotary Boring.

1. Tests Pits:

Trial pits are a simple and economical method of soil exploration to shallow depths. In this method of exploration, a square pit is excavated and soil samples are collected at required depths.

Method of Excavation:

Pits can be excavated manually with crowbars by local labor. Small tractor-mounted mechanical excavators can also be used, if locally available, which makes the exploration faster and economical.

Size of Pits:

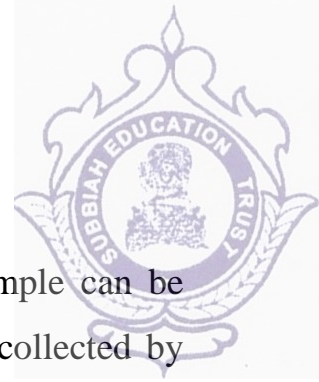
The size of the excavation depends primarily on the space required for efficient excavation and sample collection and on economic limits. Test pits normally are square or circular in plan, of size 1.2-3 m. Test trenches usually are 1-2 m wide and may be extended to any length, as required, to reveal soil conditions along a specific line.

In general, test trenches are relatively shallow, whereas test pits may be deep. It is common to limit even test pit depths to a minimum, as the cost increases with depth. Deeper excavations are justified in countries where labor is inexpensive.

Stabilizing Side Walls:

Excavations to depths of approximately 1.5 m often do not require lateral support to side walls. Shallow test pits can be stabilized more economically by sloping the side walls. Deeper excavations are generally more economical if sheeted.

Excavation for test pits below GWT requires proper dewatering arrangements to ensure stability of the side-walls, to prevent the bottom of the pit from heaving, and to keep the test pit dry to facilitate collection of undisturbed soil sample.



Collection of Soil Samples:

After excavating the test pit up to the required depth, the soil sample can be collected from the bottom of the test pit. An undisturbed soil sample is collected by driving the thin-wall soil sampler into the bottom of the test pit. This is done by placing the sampling tube vertically on the bottom of the test pit, after making the surface level and horizontal.

A flat metal plate is placed on the top of the sampler and the sampler is driven by smooth blows of a hammer on the flat plate up to the required depth. The soil surrounding the sampler outside is removed and then the sampling tube, along with the soil inside, is carefully lifted from the ground.

Any loose soil on both ends of the sampling tube is removed and the surfaces are made level. Molten wax is poured on the surface of the soil in the sampling tube on either ends to prevent evaporation of water from the soil sample. The sampling tube is properly labeled with sample number, depth, test pit number, and project or site number. The sample is then carefully transported to the laboratory for testing.

After collecting the undisturbed soil sample, the loose soil at the bottom of the pit is taken in sufficient quantity and collected into a bag. The bag is properly labeled and transported to the laboratory along with other samples.

Advantages of Test Pits:

Following are the advantages of test pits:

- i. The method is simple and fast.
- ii. Test pits are one of the most economical means of soil exploration to shallow depths.
- iii. Undisturbed samples can be collected with minimum disturbance.
- iv. It is possible to directly observe the soil profile and its variation in the vertical or lateral direction by observing the walls of the test pit.
- v. The presence of any lenses or pockets of weaker material can be readily identified.
- vi. Test pits are particularly valuable in investigating the nature of fill material, where voids, loosely deposited layers, or deleterious material can be readily recognized.



vii. Test pits or trenches are the only reliable means of obtaining adequate information on a filled ground or very variable natural deposits.

Method # 2. Auger Boring:

Auger boring is a cheap and simple means of soil exploration.

Method:

In auger boring, vertical holes are advanced by rotating the cross arm of the auger and pushing the auger into the ground. When the auger is filled with the soil, it is withdrawn. The soil is removed from the auger and examined. The auger is then inserted into the borehole, pushed into the bottom soil by rotation of the cross arm, and the process is repeated.

Helical augers, when used with casing, facilitate collection of undisturbed soil samples by fixing the sampler to the bottom of the drill rod. Field tests such as SPT may also be done by attaching the standard spilt-spoon sampler to the bottom of the drill rod after removing the helical screw.

Types of Augers:

Following are the two types of augers:

- i. Spiral or helical augers.
- ii. Post-hole augers.

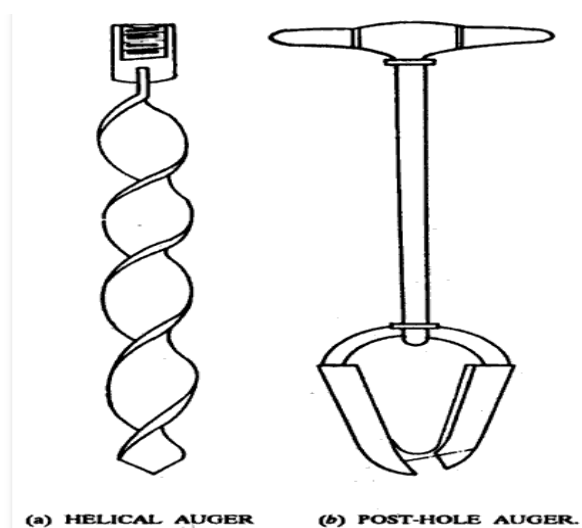


Fig1 Auger Boring

[Fig1 <http://www.abuildersengineer.com/2012/10/boring-methods-siteexploration.html>]

Augers may be operated manually by labor or may be power driven. Hand augers (generally post-hole type) are usually suitable for depths up to 5-7 m in soft-to-firm



i. Auger boring is generally suitable in soils where the walls of the borehole can stand without casing or stabilization with a drilling fluid above the GWT.

ii. Augers are found to be particularly suitable for highway, railway, or airfield projects, where low cost, rapid drilling, and high mobility of the equipment make them ideally suited for such projects.

Disadvantages:

Following are the disadvantages of auger boring:

i. The main disadvantage with auger boring is that the samples are highly disturbed and mixed.

ii. Work may be held up and auger boring is not suitable if large cobbles, boulders, or other obstructions are present at any depth.

iii. It is generally difficult, if not impossible, to locate the exact changes in the soil strata.

Method # 3. Wash Boring:

Wash boring is one of the most commonly used economical method for advancing boreholes in medium soft-to-firm clays and dense sands for soil exploration.

Procedure:

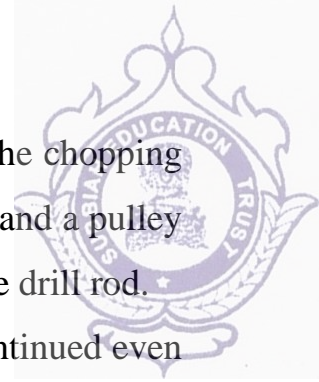
The wash boring method of soil exploration is done in the following steps:

i. A casing is first driven into the ground to a depth of 1.5-3 m. A hollow drill rod, with a chisel-shaped chopping bit at its bottom, is inserted inside the casing.

ii. Water is pumped down into the drill rod that emerges as a strong jet through the small openings of the bit at the bottom of the drill rod.

iii. The jet disintegrates the soil in the borehole and carries the broken fragments upward through the annular space between the casing and the drill rod.

iv. This return water, carrying soil fragments, known as cuttings, is collected in a sump tank through a T-shaped pipe fixed at the top of the casing.



v. The hole is further advanced by alternately raising and dropping the chopping bit by a winch. The drill rod is supported through a swivel joint, wire rope, and a pulley by a triangular or equivalent frame. The swivel joint facilitates turning of the drill rod.

vi. The process of raising, dropping, and turning of the drill rod is continued even below the bottom of the casing until the borehole begins to collapse.

vii. At this stage, the casing is further driven into the borehole and extended at the top by providing additional pieces.

viii. Soil samples can be collected by attaching soil samplers to the bottom of the drill rod, after removing the chopping bit. The soil sampler is pushed into the bottom of the borehole vertically after cleaning and then withdrawn. The undisturbed soil sample is brought along with the soil sampler.

ix. Bentonite slurry (5% bentonite mixed in water as solution) may be generally used instead of water as the drilling fluid, which stabilizes the walls of the borehole.

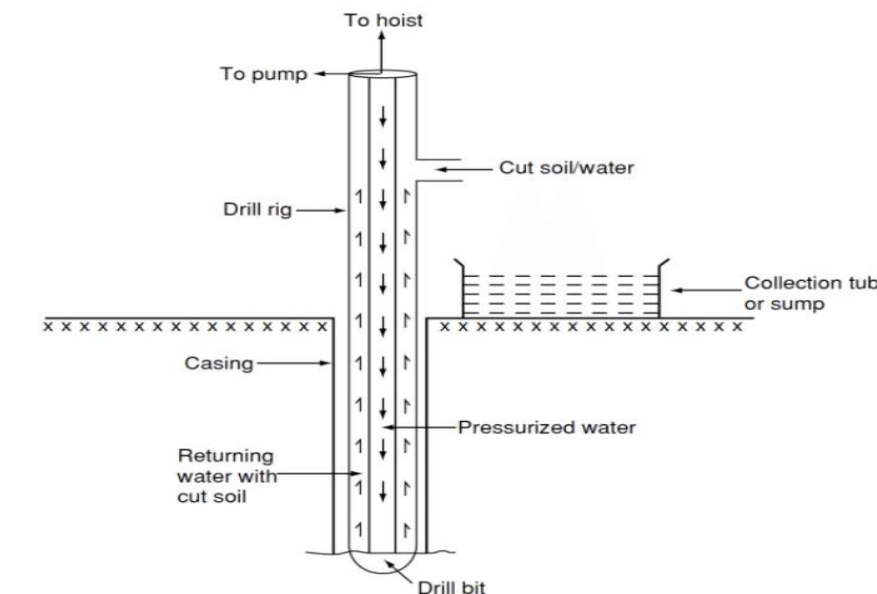


Fig 2 Wash Boring

[Fig2 https://www.researchgate.net/figure/Schematic-diagram-of-a-wash-boring-drill-rig-after-Gunaratne-2006_fig5_323256947]

Changes in the soil profile are indicated by:

- i. The rate of progress of drilling.
- ii. Color of the drilling fluid.
- iii. Examination of the soil cuttings.

Applicability:



The soil cuttings are not representative of the soil in situ, due to breakdown of particles, loss of fines during transport to ground level, and segregation in the sump tank. However, the wash boring method today is primarily useful as a means of advancing a borehole in the interval between the collections of soil samples at different levels in the borehole.

Advantages:

Following are the advantages of wash boring method of soil exploration:

- i. Wash boring method is one of the simplest and fastest methods of soil exploration in soft- to medium-stiff cohesive soils and in sand or gravels without boulders.
- ii. The equipment used for this method is light and inexpensive.
- iii. It can also be adopted in inaccessible locations, such as on water, in swamps, or in between buildings.
- iv. Undisturbed soil sampling or field testing, such as SPT or vane shear test, can be readily done in this method.

Disadvantages:

Some of the disadvantages of wash boring are:

- i. The method is slow in the stiffer and coarse-grained soils and is not efficient in materials such as hard or cemented soils, rock, and soils that contain boulders.
- ii. The method is not suitable for collecting undisturbed soil samples above GWT since the drilling fluid enters the soil mass and may increase its water content.

Method # 4. Percussion Drilling:

The percussion drilling method of advancing boreholes is of common use in drilling water wells; it is also known as cable tool drilling or churn drilling.

Method of Drilling Borehole:

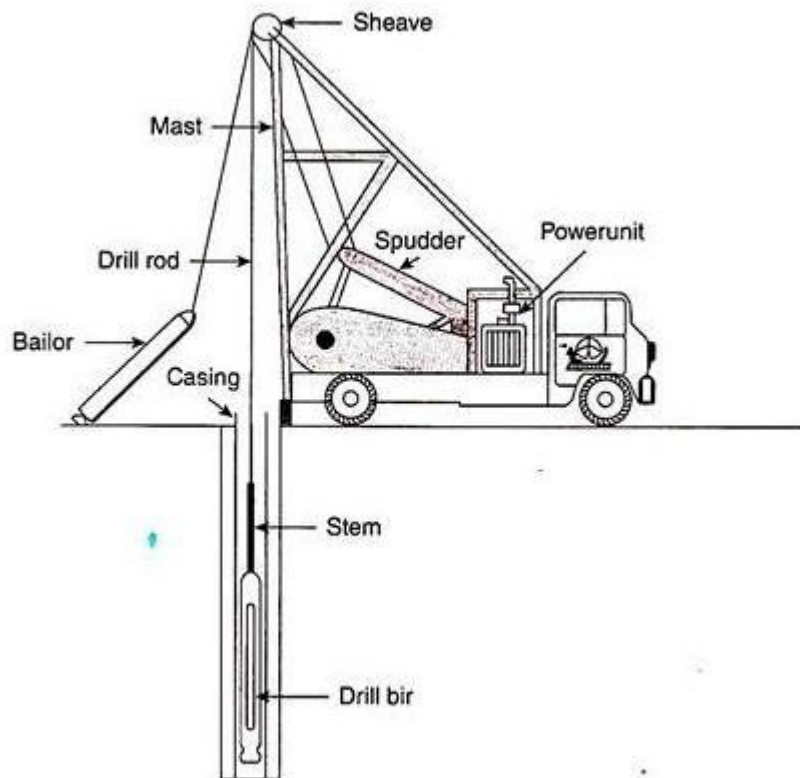


Fig 3 Percussion Boring

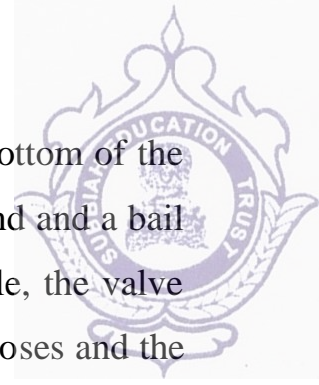
[Figure3 <https://www.soilmanagementindia.com/soil-exploration/top-5-methods-used-in-soil-exploration-soil-engineering/13877>]

Figure 3 shows the schematic diagram of percussion drilling equipment.

Advance of the borehole in this method is made by the following procedure:

- i. Alternately raising and dropping a combination of heavy drilling tools to break down the material at the bottom and to form slurry of the material.
- ii. The combination of drilling tools consists of a drill rod with a chisel-shaped chopping bit at its lower end. The chopping bit has beveled edges for cutting the material at the bottom of the borehole.
- iii. Periodically removing the slurry using bailers.
- iv. The amount of water introduced into the hole in this method is kept to the minimum required to form the slurry. In soft soils and cohesionless material below GWT, no water is generally used in this method.
- v. Changes in the soil profile are indicated by the rate of progress of drilling, color of the slurry, and examination of the contents of the slurry.

Removal of Soil Slurry:



Bailers or sand pumps are used for removal of the slurry from the bottom of the borehole. A bailer consists of a pipe having a one-way valve at its lower end and a bail at its upper end. When the bailer is pushed into the bottom of the borehole, the valve opens and the slurry enters the bailer. When the bailer is lifted, the valve closes and the slurry is retained in the bailer.

The process is repeated several times to collect the entire slurry into the bailer. The bailer is then lifted to ground level and tipped upside down to remove the slurry from the bailer. There are two types of bailers.

They are:

- i. Flat-valve bailer.
- ii. Dart-valve bailer.

The flat-valve bailer requires more time for emptying than the dart-valve bailer. A sand pump is costlier than a bailer. The dart-valve bailer is, therefore, more commonly used than the other.

Advantages:

Main advantages of using percussion drilling are as follows:

- i. The main advantage of percussion drilling is that it can be used in all types of soil or rock and particularly useful for soils containing boulders and rock.
- ii. It may also be useful to probe cavities and weakness in rock, by observing changes in the drill rate.

Disadvantages:

The disadvantages of percussion drilling are as follows:

- i. The main disadvantage of percussion drilling is that the blows of the chisel disturb the soil at the bottom of the borehole heavily.
- ii. It is also not economical for boreholes of diameter less than 100 mm.
- iii. It is difficult to trace thin layers or slight changes in soil strata when this method is used.

Method 5. Rotary Boring or Drilling:

Rotary boring or drilling is a very fast method of advancing holes in rocks and soils.

Method of Drilling Borehole:

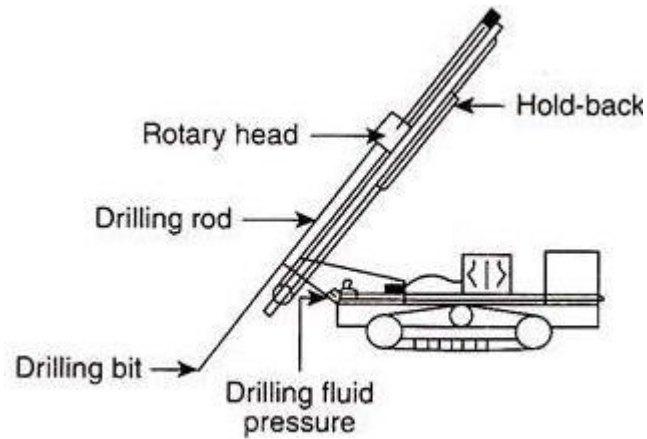


Fig 4 Rotary boring

[Figure4 https://www.soilmanagementindia.com/wp-content/uploads/2018/02/clip_image004-40.jpg]

Figure 4 shows the schematic diagram of rotary boring equipment. In this method, the borehole is advanced by rotating a hollow drill rod, which has a cutting bit at its lower end. The drill rod is rotated by a drill head, provided at the top of the drill rod. The drill head consists of a rotary drive mechanism and an arrangement for applying downward pressure.

As the drill rod is rotated, the cutting bit shears off chips of the material penetrated. A drilling mud, usually a water solution of bentonite with or without other admixtures, is continuously forced down the hollow drill rod.

The drilling fluid serves the following three functions:

- i. It carries the cuttings of the material penetrated from the bottom of the borehole to the ground surface, through the annular space between the drill rod and the walls of the borehole.
- ii. It also cools the cutting bit.
- iii. It supports the walls of the borehole in uncased boreholes. Casing is usually not required except near ground surface.

Sample Collection:

When sampling is required, the drill rod is raised and the cutting bit is replaced by a soil sampler. When the rotary boring is done through rock and when it is required to collect rock core samples, a coring bit is used at the bottom of the drill rod, instead of the cutting bit.



The coring bit cuts an annular hole around an intact core that enters the barrel and is recovered. Thus, the core barrel is used primarily in bedrock, which is usually cored continuously up to the required depth.

Types of Rotary Boring:

There are following two methods of rotary boring:

- i. Straight rotary method.
- ii. Reverse rotary method.

In the reverse rotary method, the drilling fluid is supplied through the annular space between the drill rod and walls of the borehole, while the soil cuttings are collected through the central hollow portion of the drill rod. The reverse rotary method is best suited to holes 30 cm and larger in diameter.

Advantages:

Rotary boring is superior to all other methods of soil exploration.

Following are the advantages:

- i. It is more rapid, in general, than the other methods of boring.
- ii. It also causes less disturbance to the soil during sampling. Due to this reason, its applications are increasing day by day.

Disadvantages:

Following are the disadvantages of rotary boring:

- i. The equipment is bulky and expensive.
- ii. The method is not suitable for inaccessible locations.
- iii. If the soil contains large gravel, it will rotate beneath the drill bits and cannot be easily broken. Thus, a nest of gravel will continually remain at the bottom of the borehole, preventing or delaying the progress of advancing the borehole.



type:

(i) Disturbed sample:

Disturbed sample is a sample in which soil structure is significantly or completely disturbed and the moisture content may also differ from in-situ value. The particle size distribution of in-situ soil is preserved. These samples are required for identification and classification tests.

(ii) Undisturbed sample:

Undisturbed sample is a sample which retains as closely as practicable, the true in-situ structure and moisture content of soil. These samples are required for shear strength, permeability and consolidation tests.

(iii) Representative Sample:

The soil sample which contains the same particle size distribution as in the in situ stratum, but the natural structure of sample gets partly or entirely disturbed and modified, is called a representative or disturbed soil sample.

Such disturbed soil samples can be used for

- i. Identification of soil types
- ii. Determining Atterberg limits, specific gravity, organic and carbonate content.
- iii. Compaction tests etc.

(iv) Non-representative Soil Sample:

These are mixtures of soil from different soil strata. These samples are obtained by auger boring or sedimentation of wash boring. Such samples may help in determining the depth at which major changes in soil profile occur.

Sampling from Trial Pits:

Block samples are obtained from trial pits. Block samples are hand cut samples and are obtained from clay soil. A block sample is carefully trimmed and a wooden box is kept around the protruding sample. The sample is then cut at the bottom with Knife and turned upside down with the wooden box. The sample is then covered with lid and is sealed with wax or grease.



2. Inside-wall friction.
3. Non-return valve.
4. Recovery ratio.

1. Cutting Edge:

Cutting edge is the beveled and sharp edge at the bottom of the soil sampler. It may be an integral part of the soil sampler or a separate cutting bit may be screwed to the bottom of the sampler. It mainly facilitates driving of the soil sampler through the soil.

A typical cutting edge shown in Fig. 1 should have the following design features:

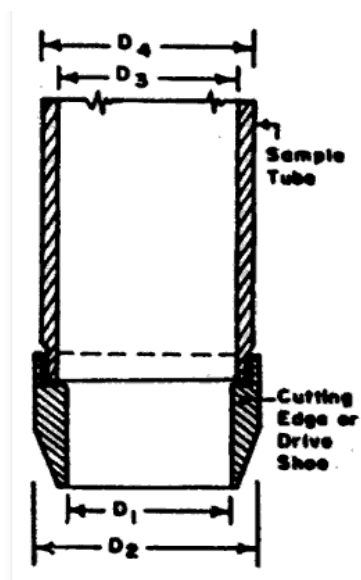


Fig 1 cutting edge

[Fig 1 <http://www.abuildersengineer.com/2012/10/soil-samples-and-samplers-foundations.html>]

(a) Inside Clearance

$$C_i = (D_s - D_e / D_e) \times 100 \text{ or}$$

$$C_i = (D_3 - D_1 / D_1) \times 100$$

$$= 1-3\%$$

The inner diameter of the cutting shoe should be kept slightly smaller than that of the sampling tube. This helps for elastic expansion of the soil as it enters the sampling tube and reduces frictional drag on the sample from the wall of the tube.



(c) Area ratio

$$A_r = (D_w^2 - D_e^2 / D_e^2) \times 100 \%$$

$$A_r = (D_2^2 - D_1^2 / D_1) \times 100 \%$$

This represents the amount of soil that is displaced when a sampler is forced into the ground. The area ratio should be kept as low as possible.

For stiff formation, $ar > 20\%$

For soft soil, $ar = 10\%$ or less

where

D_s = Inside diameter of sampling tube

D_t = Outside diameter of sampling tube

D_e = Inside diameter of cutting shoe

D_w = Outside diameter of cutting shoe

2. Inside-Wall Friction:

The inside-wall friction should be minimized for minimizing the disturbance of the sample by the following actions:

- i. Provide suitable inside clearance.
- ii. Provide a smooth finish to the sampling tube.
- iii. Oil the inside surface of the sampling tube properly.

3. Non-Return Valve:

The valve should have a large orifice to allow air and water to escape quickly and easily when driving the sampler.

4. Recovery Ratio:

It is defined as the ratio of length of the sample within the sampling tube to the depth of penetration of the sampler during sampling. Thus, recovery ratio (R_r) is expressed by



- i. Thin-wall samplers.
- ii. Thick-wall samplers.

Thin-wall samplers are the samplers in which the wall thickness of the sampling tube is less than 2.5% of the diameter. In other words, thin-wall samplers are those for which the area ratio is less than or equal to 10%. Samplers for which the area ratio is more than 10% are known as thick-wall samplers.

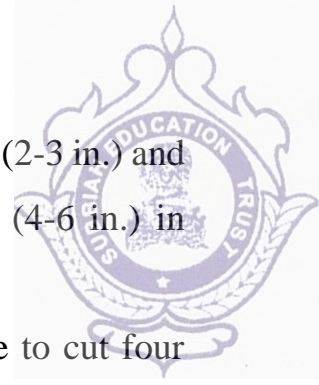
Later studies have modified the definition of thin-wall samplers to include the effect of cutting edge on sample disturbance. Accordingly, thin-wall samplers may be defined as those with an area ratio less than 20% when the sampler has a suitably designed cutting edge. Thick-wall samplers are those with an area ratio more than 20%. Based on the sampler design and use, soil samplers are classified into the following types:

1. Open-tube sampler.
2. Standard split-spoon sampler.
3. Stationary piston sampler.
4. Rotary sampler.
5. Scraper bucket sampler.

1. Open-Tube Sampler:

The open-tube samplers are the simplest type of samplers for collection of undisturbed samples. They are thin-wall tube samplers made of seamless steel and are also known as thin-wall Shelby tube samplers. The bottom of the tube is sharpened and beveled, which acts as a cutting edge. The upper portion of the sampler has threads on the inside surface to enable the sampler to be attached to the bottom of the drill rod.

Size of Samplers:



The most commonly used tube samplers are 5.08-7.62 cm in diameter (2-3 in.) and 60.96-76.2 cm (2-2.5 ft) in length. However, the use of tubes 10-15 cm (4-6 in.) in diameter has increased within the past two decades.

The use of large-diameter tubes has the advantage that it is possible to cut four small-diameter triaxial specimens (3.8 cm, i.e., 1.5 in. diameter) from the same level of the sampler. This is extremely important in residual soils, where the engineering properties vary considerably with depth.

The U.S. Army Corps of Engineers recommends the use of 12.5cm (5 in.) diameter tubes in cohesive soils and shallow deposits of cohesionless soils. In the deeper cohesionless deposits, the use of 7.5-cm diameter tubes is recommended, since the penetration resistance of the larger tubes exceeds the pushing capacity of the commonly used drilling rigs. The area ratio of the sampler is below 15% and the inside clearance is between 0.5% and 3%.

Method of Sampling:

The procedure for collection of soil samples from a bore hole using open-tube sampler is as follows:

- i. The sampler is attached to the bottom of the drill rod and is lowered to the bottom of the bore, where the undisturbed sample is to be collected.
- ii. It is then driven into the soil, with fast and smooth strokes of careful hammering. Erratic pressure, if applied during hammering, becomes a source of sample disturbance.
- iii. After driving the sampler up to the required length (equal to the sampler length minus provision for waxing), the sampler is rotated twice completely to shear off the sample from its intact bottom and is then withdrawn.
- iv. The tube is removed from the sampler head and its ends are sealed with molten wax before transportation.
- v. The sampler head is provided with vents at top to permit water to escape when sampling under water.
- vi. It is also provided with a check valve to help retain the sample when withdrawing the sampler.

Suitability:



Shelby tube samplers are commonly used for collection of samples in fine-grained soft soils. The presence of obstructions to the sampler during driving, in the form of gravel or stones, cramps the beveled bottom edge and causes disturbance to the sample collected. The Shelby tube samplers are, therefore, unsuitable for hard or dense gravelly soils.

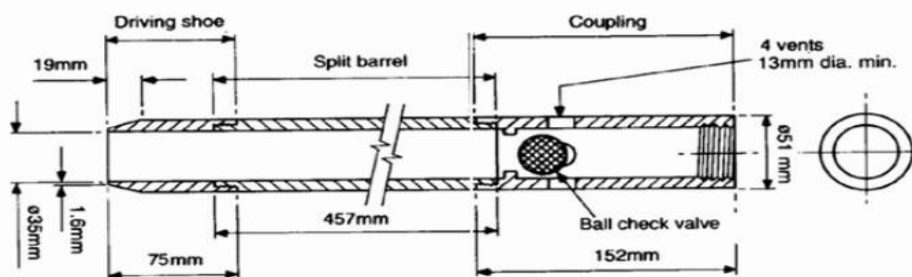
2. Standard Split-Spoon or Split-Barrel Sampler:

It is the most commonly used sampler for obtaining undisturbed soil samples. It is also known as split-barrel and split-tube sampler. A split-spoon sampler is also used to conduct SPT in the borehole. When the SPT is conducted, the soil sample simultaneously enters the sampler by the end of the test, which is then withdrawn and taken to the laboratory.

Features of the Sampler:

The standard split-spoon sampler consists of the following [Fig. 2(a)]:

- i. A driving shoe made of 7.5-cm-long tool steel at the bottom.
- ii. A 45-cm-long steel tube, split into two halves longitudinally, as shown in Fig. 2(b)
- iv. A coupling or sampler head at the top of the tube, about 15 cm long.



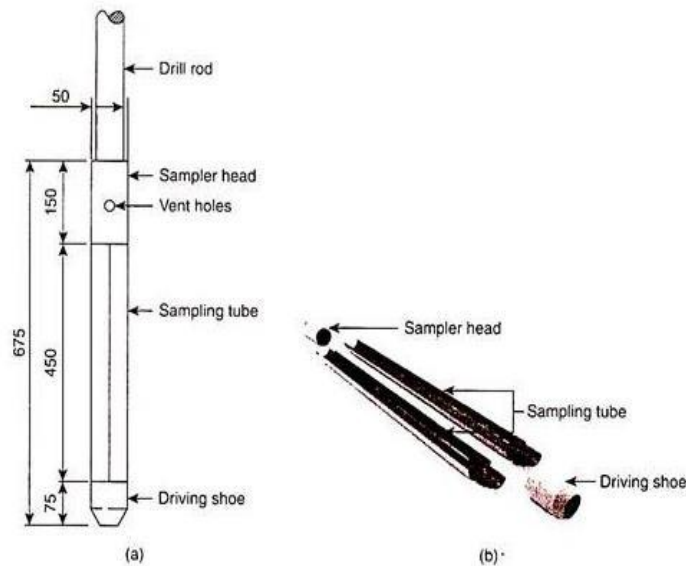


Fig 2 Standard split spoon sampler a) schematic diagram b) typical sampler

[Fig 2 https://www.researchgate.net/figure/Standard-Penetration-Test-Split-Spoon-Sampler-after-ASTM-D1586_fig3_312192755]

The inside diameter of the steel tube is 3.8 cm (1.5 in.) and the outside diameter is 5.08 cm (2 in.). The coupling head is provided with a check valve and four venting ports of 1-cm diameter to improve sample recovery. In some split-spoon samplers, a separate sampling tube of 3.8-cm internal diameter is provided.

Method of Sampling:

Following is the procedure for the collection of soil samples using standard split-spoon sampler:

- i. After the borehole has been made, the sampler is attached to the bottom of the drill rod and lowered to the bottom of the hole.
- ii. The sampler is forced into the soil by repeated blows of a drop hammer.
- iii. The sampler is driven to the required length, taking care not to overstress the sample, and is then withdrawn.
- iv. The split tube is separated after removing the shoe and the coupling, and the sampling tube with sample is taken out.
- v. The sampling tube containing sample is then placed in a container, sealed, and transported to the laboratory.



vi. In the samplers with separate a 3.8-cm-diameter tube inside the split barrel, the tube is taken out, waxed on both ends, and transported to the laboratory.

vii. If the soil encountered in the borehole is fine sand below the water table, the sample recovery becomes difficult. For such soils, a spring core catcher device is used to aid recovery. As the sampler is lifted, the springs close and form a dome to retain the sample.

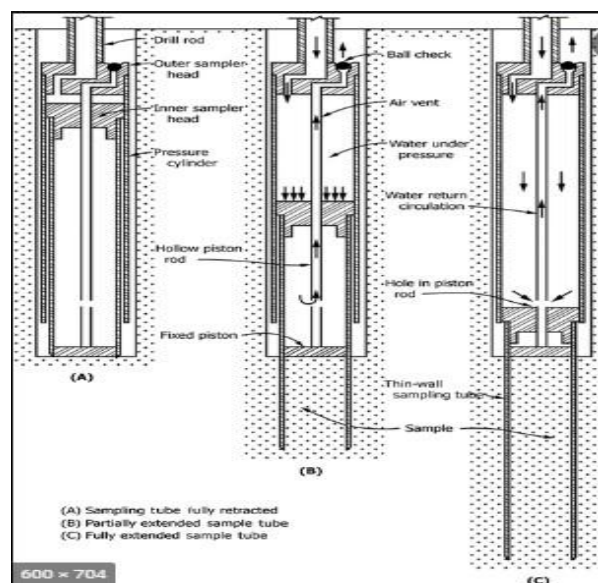
viii. While taking samples, the water level in the borehole should be always maintained higher than the GWT to prevent quick conditions.

3. Stationary Piston Sampler:

A piston sampler consists of two parts – (a) a sampler cylinder and (b) a piston system. The piston rod is 30 cm (12 in.) in diameter at the bottom end and fits easily inside the hollow drill rod.

Method of Sampling:

Figure 3 illustrates the operation of stationary position sampler. For obtaining a sample, the bottom of the piston is brought flush with the under cutting edge of the sampler and the sampler is lowered into the borehole. When the sampler reaches the bottom of the hole, the piston rod is held fixed relative to the ground surface and the thin-wall tube is pushed into the soil by hydraulic pressure or mechanical jacking. The sampler is never driven. The sampling tube is pushed to the required depth and then the drill rod along with the whole sampler is withdrawn.



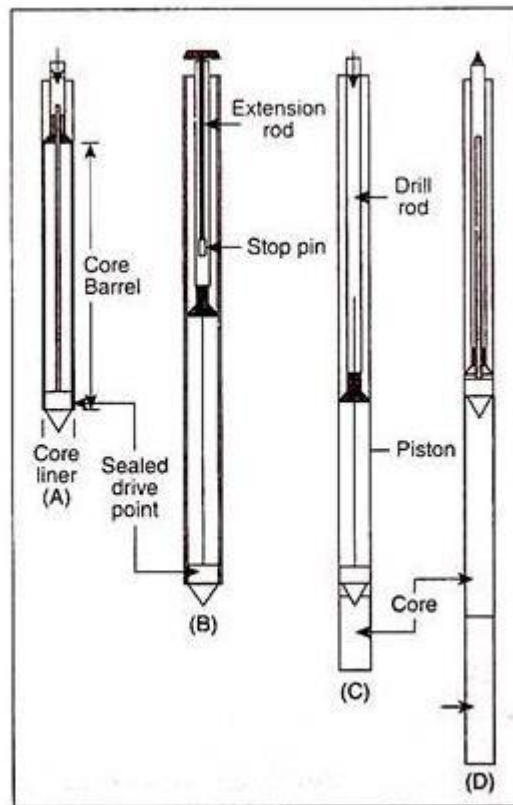


Fig 3 stationary piston sampler

[Fig 3 <https://link.springer.com/article/10.1007/s41062-017-0086-3>]

Use of the Piston:

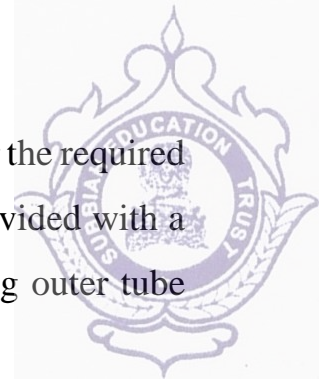
The piston prevents water and dirt from entering the tube during the lowering operation. When the sampling tube is pushed into the soil, keeping the piston fixed in position, a negative pressure is developed above the sample, which holds back the sample during withdrawal.

Suitability:

The stationary piston sampler is used for sampling soft-to-stiff cohesive soils. The quality of samples obtained is excellent and the probability of obtaining a satisfactory undisturbed sample is high. The use of piston enables a high recovery ratio for the soil sample, but at the same time, prevents recovery ratios higher than 100%, preventing entry of excess soil into the sampler.

4. Rotary Sampler:

The rotary samplers are double-tube samplers with a removable thin-wall tube, known as liner, inside an outer tube provided with a cutting bit. The rotary sampler has an outside diameter of 6.35-19.7 cm (3.5-7.75 in.) and a length of 61 cm (24 in. or 2 ft).



The outer tube with the cutting bit is rotated and pushed down into the soil for the required length and the sample enters the liner. The inner tube, that is, the liner, provided with a smooth cutting shoe, remains stationary and the sample cut by the rotating outer tube slides into the liner. The sample is thus received in the liner.

Rotary samples can be used for collection of undisturbed samples in stiff-to-hard clays, silts, and sands with some cementation and also in soft rock. The sampler is, however, unsuitable for gravelly soils and loose cohesionless soils.

5. Scraper Bucket Sampler:

Sampling with a standard split-spoon sampler becomes difficult if the soil contains pebbles. Even if the sampler is fitted with a spring core catcher, the pebbles interfere with the springs and obstruct their closure. A scraper bucket sampler may be helpful for the collection of undisturbed samples in such soil deposits. However, it is possible to collect only disturbed samples using a scraper bucket sampler. The sampler is also useful to obtain samples of cohesionless soil below GWT.

The scraper bucket sampler contains a vertical slit at its upper portion and a driving point at its bottom. As the sampler is rotated, the scrapings of the soil enter into the sampler cylinder through the vertical slit. When the sampler is filled with the scrapings, it is lifted up and collected into a separate container. Although the sample is thoroughly disturbed, it still represents the soil at exact depth from where it is collected. **Preservation of Samples:**

On withdrawal of sampler from the boreholes, the sampling tubes are removed and are sealed on both ends by paraffin's wax or petroleum jelly. The seal thickness should not be less than about 25 mm.

The sampling tubes are then labelled with the following information's:

- (i) Name of project
- (ii) Number of boring
- (iii) Depth of sampling
- (iv) Date of sampling

While at site the sampling tubes are protected from direct sunshine, shock etc. The sampling tubes are taken to the laboratory as soon as possible and are kept in a humid room to preserve the natural water content of the samples.



1.5 Penetration Test:

Standard Penetration Test (SPT):

Standard penetration test (SPT) is the most commonly used in situ test for sub-surface investigation. In SPT a split spoon sampler is made to penetrate 15 cm by light blows of a 65 kgs drop hammer on the top of the drill rod. The drill rod is connected to the top of the split spoon sampler.

After initial penetration of 15 cm of the sampler, the drop hammer is allowed to fall from a height of 75 cm and number of blows required for 30 cms penetration of sampler is recorded. This number of blows is called N-value or penetration number. In this method the driving energy is supplied by the fall of the drop weight. Hence it is essentially a dynamic sounding method.

Detailed procedure of SPT is as follows:

Apparatus required:

(i) Split spoon sampler:

It has an outside diameter of 50 mm, inside diameter of 35 mm and minimum open length (cutting edge to air vent) of 600 mm. The coupling head has four 10 mm (minimum diameter) vent ports or a ball check valve.

(ii) Drive assembly:

It consists of a tripod as hoisting equipment-one of the leg is provided with ladder, a drive mass (hammer) of 65 kgs, a guide to ensure a 75 cm free fall of the drivemass and an anvil (attached to the guide) for transmitting the blow to the sampler rod.

In general practice four methods of releasing the hammer are used:

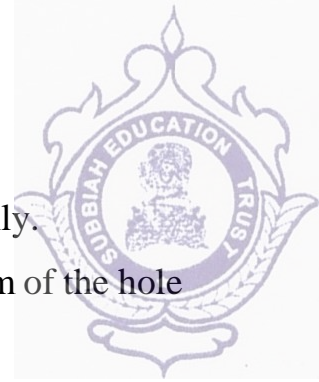
- (a) Normal lifting and releasing of the rope passing through a pulley.
- (b) A trip hammer, such as the Pilcon or Dando hammers

(iii) Extension rods:

These rods are used to transmit the driving energy from the anvil to the sampler.

(iv) Drilling equipment:

Drilling equipment should be for making a reasonably clear hole of 60-75 mm diameter so as to ensure that the test is performed in undisturbed soil and not in the fall in material. Casing or drilling mud may have to be used where the boring sides fall in. In general, hand operated auger of 75 mm diameter are used for drilling boreholes.



Procedure:

- (1) A borehole is drilled to the required depth and is cleaned thoroughly.
- (2) The sampler attached to the extension rods is lowered to the bottom of the hole and is allowed to rest under the self weight.
- (3) The drive assembly is then connected to the rod and the sampler is driven with light blows from the drive mass to a seating penetration of 15 cm.
- (4) The sampler is then driven to an additional penetration of 30 cm by blows from 65 kgs drive mass falling from a height of 75 cm. The number of blows required for 30 cm penetration is recorded as standard penetration resistance, N.
- (5) The sampler is then lifted from the hole and opened. The undisturbed sample is removed from the sampler and sealed from both sides.
- (6) The test is performed in each identifiable soil layer or at an interval of 1.5 m whichever is smaller. As per IS:2131, for a foundation of width B, penetration test has to be carried out at an interval of 0.75 m up to a depth of B from the bottom of the footing and at 1.5 m interval for the rest depth up to a depth of 1.5 to 2 B.
- (7) The measured N-value may indicate more than the actual value in some cases and so they are to be corrected.

The standard penetration resistance i.e., N-value has been correlated to different soil properties by different investigators.

Some of the correlation is given in the following tables:

a) Dilatancy Correction.

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

$$N_c = 15 + \frac{1}{2} [N_R - 15]$$

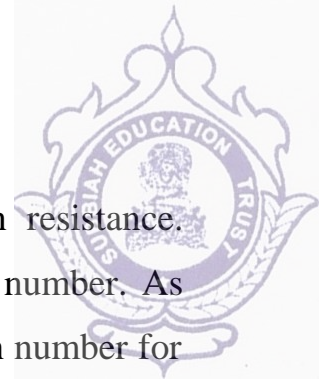
Where,

N_c – corrected value

N_R – Recorded Value

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

b) Over burden Pressure Correction:



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

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

$$N_c = N_R \times \frac{350}{\sigma_0 \cdot 70}$$

Where, N_c - corrected value

N_R - Recorded Value

σ_0 - effective overburden pressure

It is applicable for $\sigma_0 \leq 280 \text{ kN/m}^2$. Usually the overburden correction is applied first and then dilatancy correction is applied.

The correction given by Bazara & peck is

$$N = 4N_R \quad \text{if } \sigma_0 > 71.8 \text{ kN/m}^2 \quad 1 + 0.0418\sigma_0$$

$$N = 4N_R \quad \text{if } \sigma_0 > 71.8 \text{ kN/m}^2 \quad 3.25 + 0.0104\sigma_0$$

$$N = N_R \quad \text{if } \sigma_0 \leq 71.8 \text{ kN/m}^2$$

For cohesive soil:

Table 1 Relation between N and q_u

Penetration resistance N(blow/s)	Unconfined compressive strength (t/m ²) q_u	Consistency
<2	<2.4	Very soft
2-4	2.4-4.8	Soft
4-8	4.8-9.6	Medium
8-15	9.6-19.2	Stiff
15-30	19.2-38.8	Very Stiff
>30	>38.8	Hard



(From terzaghi and peck ,1948)

The relationship between q_u and N proposed by murthy1982

$$q_u = \frac{N}{7.5} Kg/cm^2$$

Sanglerat(1972) has proposed the following relationship between q_u and N

$$\text{For clay , } q_u = \frac{N}{4} Kg/cm^2$$

$$\text{For silty clay , } q_u = \frac{N}{5} Kg/cm^2$$

For cohesionless Soil:

Table 1 Relation between N and angle of shear resistance (ϕ)

Corrected N value	5	10	15	20	25	30	35	40	45	50
ϕ (degree)	28.5	30	32	33	35	36	37.5	39	40	41

(peck et al,1974)

N value	Density Index(%)	Degree of compaction
<4	0-15	Very loose
4-10	15-35	Loose
10-30	35-65	Medium
30-50	65-85	Dense

(Mitchell and Katti,1981)

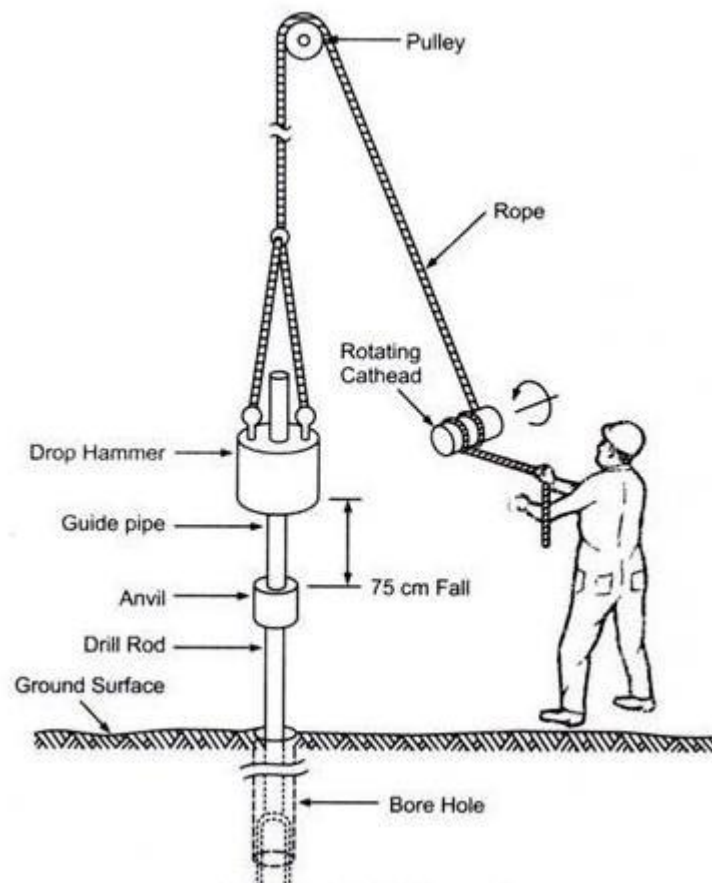


Fig 1 SPT

[Fig1 https://www.researchgate.net/figure/Standard-Penetration-Test-Arrangements_fig3_280572148]

Effect of Submergence:

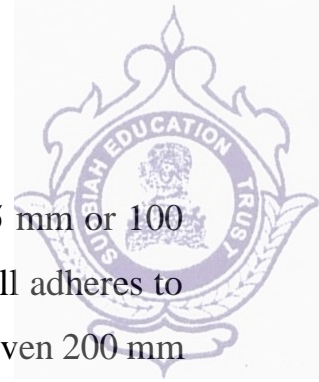
Terzaghi and Peck (1948) recommended that where the soil consists of very fine or silty sand below the water table, the measured N-value, if greater than 15, should be corrected for increased resistance due to excess pore water pressure set up during driving and unable to dissipate immediately. The corrected value of N, N_c is given by

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

where, both the overburden and submergence corrections are necessary, the overburden correction is applied first.

Effect of Rod Length:

Wave equation studies (Schmertman and Palacios, 1979) indicate that the theoretical maximum ratio decreases with decreasing rod length below a rod length of 10 m. The weight or stiffness of the rod stem, of a given length, appears to have little effect (Brown, 1977; Matsumoto and Matsubara, 1982).



Effect of Borehole Diameter:

In its original form the SPT was carried out from the bottom of 62.5 mm or 100 mm diameter wash borings (Skempton, 1986). The best modern practice still adheres to this dimension. In many countries 150 mm test boreholes are common and even 200 mm bore holes are permitted (Nixon, 1982). The effect of testing from relatively large bore holes in cohesive soils is probably negligible but in sands there is indication that appreciable lower N- values may results (Lake, 1974; Sanglerat and sanglerat, 1982). The minimum correction factors to allow for the effect of testing over large boreholes is suggested (Skempton, 1986) as given in Table 10.7.

Description	Correction factors
Rod Length	
10metres	1.00
6-10 metres	0.95
4-6 metres	0.85
3-4 metres	0.75
Standard Sampler	1.00
Us sampler without Lines	1.20
Bore hole Diameter	
65-115mm	1.00
150mm	1.05
200mm	1.15

Table 2. Approximate correction factor to measured N(after skempton1986)

Static Cone Penetration Test (CPT):

The static cone penetration test is normally called as the cone penetration test (CPT). CPT is a direct sounding test which gives a continuous record of variation of penetration resistance with depth. No sample is obtained from this test. A cone is used which has an apex angle of 60° and overall base diameter of 35.7 mm giving a cross-sectional area of 10 cm^2 .

It is made of steel and tip hardened. The cone is attached to the lower end of a 15 mm diameter steel sounding rod passing through a steel mantle tube of uniform or non-



identify the different strata.

In recent year, the static cone penetrometer had been modified to incorporate Piezo cone. Piezocone penetrometer gives simultaneous measurement of cone resistance, side friction and the pore water pressure as the cone is advanced in the soil. Piezocone penetrometer (CPTU) gives a more reliable determination of stratification and soil type than a standard CPT.

The CPT has three main applications:

1. To determine subsurface stratification and identify materials present.
2. To estimate geotechnical parameters.
3. To provide results for direct geotechnical design.

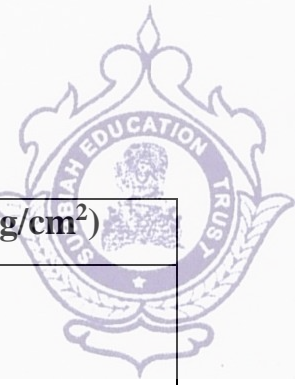
For fine grained soil as clay, the preliminary untrained shear strength (C_u) can be estimated from:

$$C_u = q_c / N_k$$

where

q_c = measured cone resistance

N_k = 17 to 18 for normally consolidated clays or,
20 for over consolidated clays.



S.No	Soil Type	q_u/N (q_u in Kg/cm ²)
1	Silts, Sandy silts, slightly cohesive, silt sand mixtures	2
2	Clean, fine to medium sands and slightly silty sands	3-4
3	Coarse sands and sands with little gravel	5
4	Sandy gravels and gravels	6

Dynamic Cone Penetration Test (Dcpt):

DCPT is similar to SPT as the use, except that there is no borehole for DCPT. This test is done by driving a standard 60° cone attached to a string of drill rods into the soil by blows of 65 kgs hammer falling from a height of 75 cm. The number of blows for every 30 cm penetration of the cone is recorded.

The number of blows required for 30 cm of penetration of cone is referred as cone resistances, N_c

DCPT is performed in two ways:

- (i) Using 50 mm cone without benetonite slurry (IS-4968, part I)
- (ii) Using 62.5 mm cone with bentonite slurry (IS-4968, part II)

For a 50 mm diameter cone without bentonite slurry, the cone is fitted to the driving rod (A-rod). The hammer head is joined to the other end of the A-rod with a A-rod coupling and a guide rod 150 cm long is connected to the hammer head. This assembly is kept vertical with the cone resting vertically on the ground at the point to be tested. The cone is then driven by the drop of the hammer and the driving is continued till the cone reaches the required depth.

For 62.5 mm cone with bentonite slurry, the setup should have arrangements for circulating slurry so that the friction on the driving rod is eliminated.

The N_c value of DCPT and N-value of SPT can be compared and an approximate correlation can be established for the site. With the help of these correlations, the data from DCPT at other locations can be deduced to know to the value of N. This type of



work is adequate for small structures and is useful in the preliminary exploration for extensive sites.

Geophysical Method:

(i) SEISMIC REFRACTION METHOD

General:

This method is based on the fact that seismic waves have different velocities in different types of soils and besides the wave refract when they cross boundaries between different types of soils. In this method an artificial impulse are produced either by detonation of explosive or mechanical blow with a heavy hammer at ground surface or at the shadow depth within a hole.

These shocks generate three types of waves.

- Longitudinal or compressive wave or primary (p) wave
- Transverse or shear waves or secondary (s) waves
- Surface waves

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

$$V_c = \sqrt{\frac{E(1 - \mu)}{(1 + \mu)(1 - 2\mu)\rho}}$$

$$V_s = \sqrt{\frac{E}{2\rho(1 + \mu)}}$$

Where E is the dynamic modulus of the soil μ is the Poisson's ratio

ρ is density

G is the dynamic shear modulus

These waves are classified as direct, reflected and refracted waves. The direct waves travel in approximately straight line from the source of impulse. The reflected and refracted wave undergoes a change in direction when they encounter a boundary separating media of different seismic velocities. This method is more suited to the shallow explorations for civil engineering purpose. The time required for the impulse to travel from the shot point to various points on the ground surface is determined by means of geophones which transform the vibrations into electrical currents and transmit



them to a recording unit or oscillograph, with a timing mechanism.

Assumptions

The various assumptions involved are

- All the soil layers are horizontal
- The layers are sufficiently thick to produce a response
- Each layer is homogeneous and isotropic
- Velocity should increase with depth following the Snell's law as given

i_1 is the angle of incidence

i_2 is the angle of refraction

v_1 and v_2 are velocity in two different mediums

Procedure

The detectors are generally placed at varying distance from the shot point but along the straight line. The arrival time of the first impulse at each geophone is utilized. If the successfully deeper strata transmit the waves with increasingly greater velocities the path travelled by the first impulse will be similar to those. Those recorded by the nearest recorders pass entirely through the overburden, whereas those first reaching the after detectors travel downward through the lower velocity material, horizontally within the higher velocity stratum and return to the surface.

(A T_1 and A T_2) as the function of the distances between the geophones and the shot points (L_1 and L_2). A curve obtained which indicates the wave velocity in each stratum and which may be used to determine the depths to the boundaries between the strata. Where H_1 and H_2 are the depths of the strata

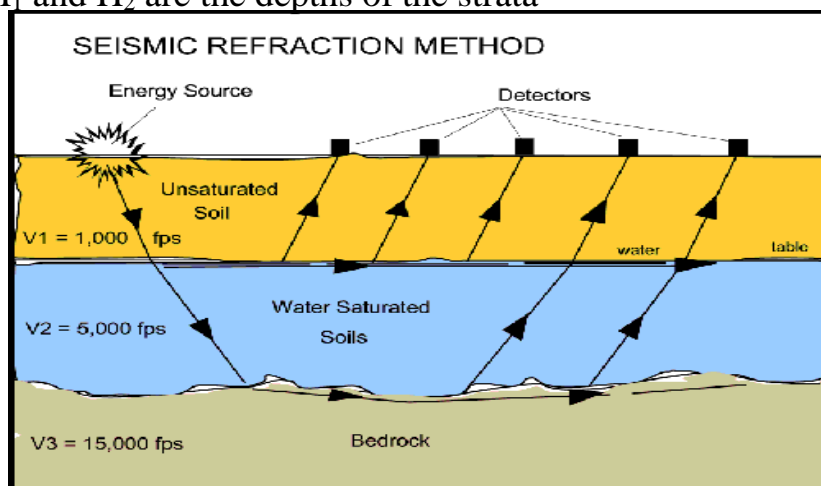
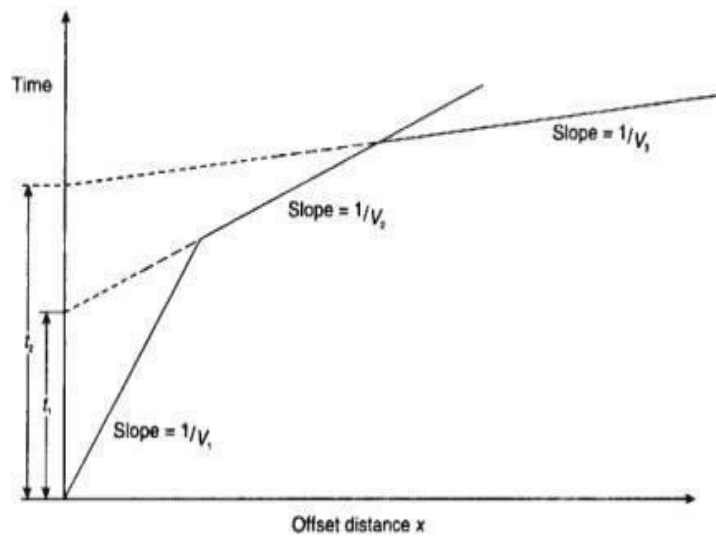


Fig 2 Seismic Refraction Method

[Fig 2 <https://www.sumoservices.com/seismic-refraction-case-study>]



$$H_1 = \frac{l_1 V_1}{2 \cos \alpha} = \frac{L_1}{2} \sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$

$$H = \frac{l_2 V_2}{2 \cos \beta} = \frac{L_2 - L_1}{2} \sqrt{\frac{V_3 - V_2}{V_3 + V_2}}$$

Where H_1 and H_2 are the depths of the strata

$$l_1 = AB_1$$

$$l_2 = AC_1 - AB_1$$

$$\sin \alpha = (V_1 - V_2) / V_3 \quad \sin \beta = (V_2 - V_1) / V_3$$

Applications

- Depth and characterization of the bed rock surfaces.
- Buried channel location.
- Depth of the water table.
- Depth and continuity of stratigraphy interfaces.
- Mapping of faults and other structural features.

Advantages

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

- Refraction observations generally employ fewer source and receiver location and





thus relatively cheap to acquire.

- Little processing is done on refraction observations with the exception of trace scaling or filtering to help in the process of picking the arrival times of the initial ground motion.
- Because such a small portion of the recorded ground motion is used developing models and interpretations is no more difficult than our previous efforts with other geophysical surveys.
- Provides seismic velocity information for estimating material properties.
- Provides greater vertical resolution than electrical, magnetic or gravity methods.
- Data acquisition requires very limited intrusive activity is non- destructive.

Disadvantages

- Blind zone effect: If $v_2 < v_1$, then wave refracts more towards normal then the thickness of the strata is neglected.
- Error also introduced due to some dissipation of the velocity as longer the path of travel, geophone receives the erroneous readings.
- Error lies in all assumptions.

(ii) ELECTRICAL RESISTIVITY METHOD

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

$$\rho = RA / L$$

ρ is resistivity in ohm-cm R is resistance in ohms

A is the cross sectional area (cm²)

L is the length of the conduction (cm)

Procedure:

In this method the electrodes are driven approximately 20 cms in to the ground and a dc or a very low frequency ac current of known magnitude is passed between the outer electrodes thereby producing within the soil an electrical field and the boundary conditions. The electrical potential at point C is V_c and at the point D is V_d which is measured by means of the inner electrodes respectively.



$$\rho = \frac{2\pi R r_1}{I}$$

Where,

Resistances $R = V_{CD}/I$

Thus the apparent resistivity of the soil to the depth approximately equal to the spacing r_1 of the electrode can be computed. The resistivity unit is often so designed that the apparent resistivity can be read directly on the potentiometer.

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

In resistivity sounding or depth profiling the center point of the set up is stationary whereas the spacing of the electrode is varied. A detailed evaluation of the results of the resistivity sounding is rather complicated, but preliminary indications of the subsurface conditions may be obtained by plotting the apparent resistivity as a function of electrode spacing. When the electrode spacing reaches a value equal to the depth to a deposit with a resistivity materially different from that of overlying strata, the resultant diagram will generally show a more or less pronounced break in the strata depth beyond A_2 .

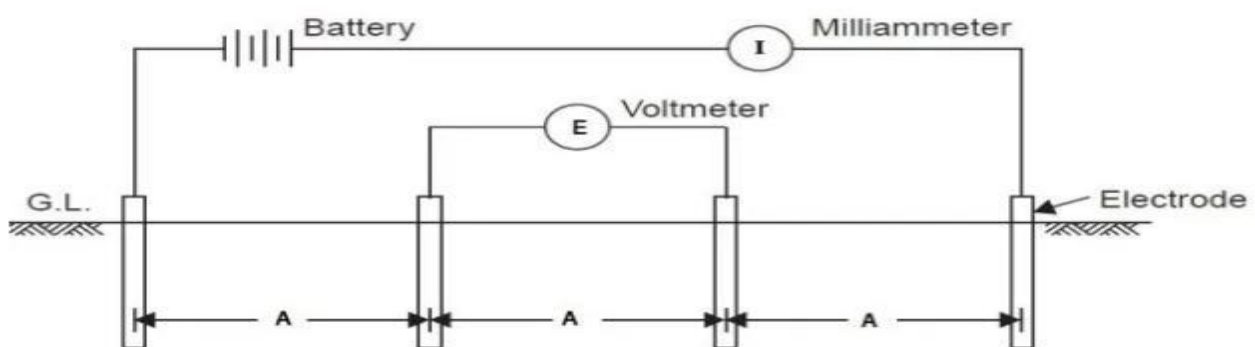


Fig 3 Electrical resistivity Method

[Fig 3 <https://civilblog.org/2015/04/18/electrical-resistivity-test-of-soil-geophysical-method-of-soil-exploration/>]

For simple sounding a Wenner array is used. Then the resistivity is given as



$$\rho = \frac{2\pi Ra}{I}$$

Where a is the spacing between the electrodes.

Applications

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

Resistivity profiling

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

Advantages of this method are

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

Disadvantages

- It can only detect absolutely different strata like rock and water.
- It provides no information about the sample.
- Cultural problems cause interference.
- Data acquisition can be slow compared to other geophysical methods, although

that difference is disappearing with the very latest techniques.





1.6 Bore log Report:

Information on subsurface conditions obtained from the boring operation is typically presented in the form of a boring record commonly known as 'boring log'. It is also known as sub-soil investigation report which should contain the data obtained from bore holes, site recommendations about the suitable type of foundation, soil pressure and expected settlements.

It is essential to give a complete and accurate record of data collected. All relevant data for the bore hole is recorded in a boring log. A boring log gives the description or classification of various strata encountered at different depths. Any additional information that is obtained in the field soil consistency, UCC strength, standard Penetration test, Cone penetration Test is also indicated on the boring log. It should also show the water table.

The data obtained from a series of bore holes is presented in the form of a vertical section through the ground along the line of exploration. It indicates the boundaries of different strata, along with their classification. It is important to remember that conditions between bore holes are estimated by interpolation, which may not be correct. Obviously, larger the number of holes, the more accurate the subsurface profile.

A soil exploration report generally consists of the following:

1. Introduction, which gives the scope of the investigation.
2. Description of the proposed structure, the location and the geological conditions at the site.
3. Details of the field exploration programme, indicating the number of borings, their location and depths.
4. Details of the method of exploration.
5. General description of the sub-soil conditions as obtained from in-sites tests, such as standard penetration Test, cone test.
6. Details of the laboratory test conducted on the soil samples obtained and the results obtained.
7. Date and weather condition during investigation.
8. Depth of ground water table and the change in water levels.



9. Discussion of the results.

10. Recommendation about the allowable bearing pressure, the type of foundation or structure.

11. Conclusion: The main findings of the bore hole investigations should be clearly stated.

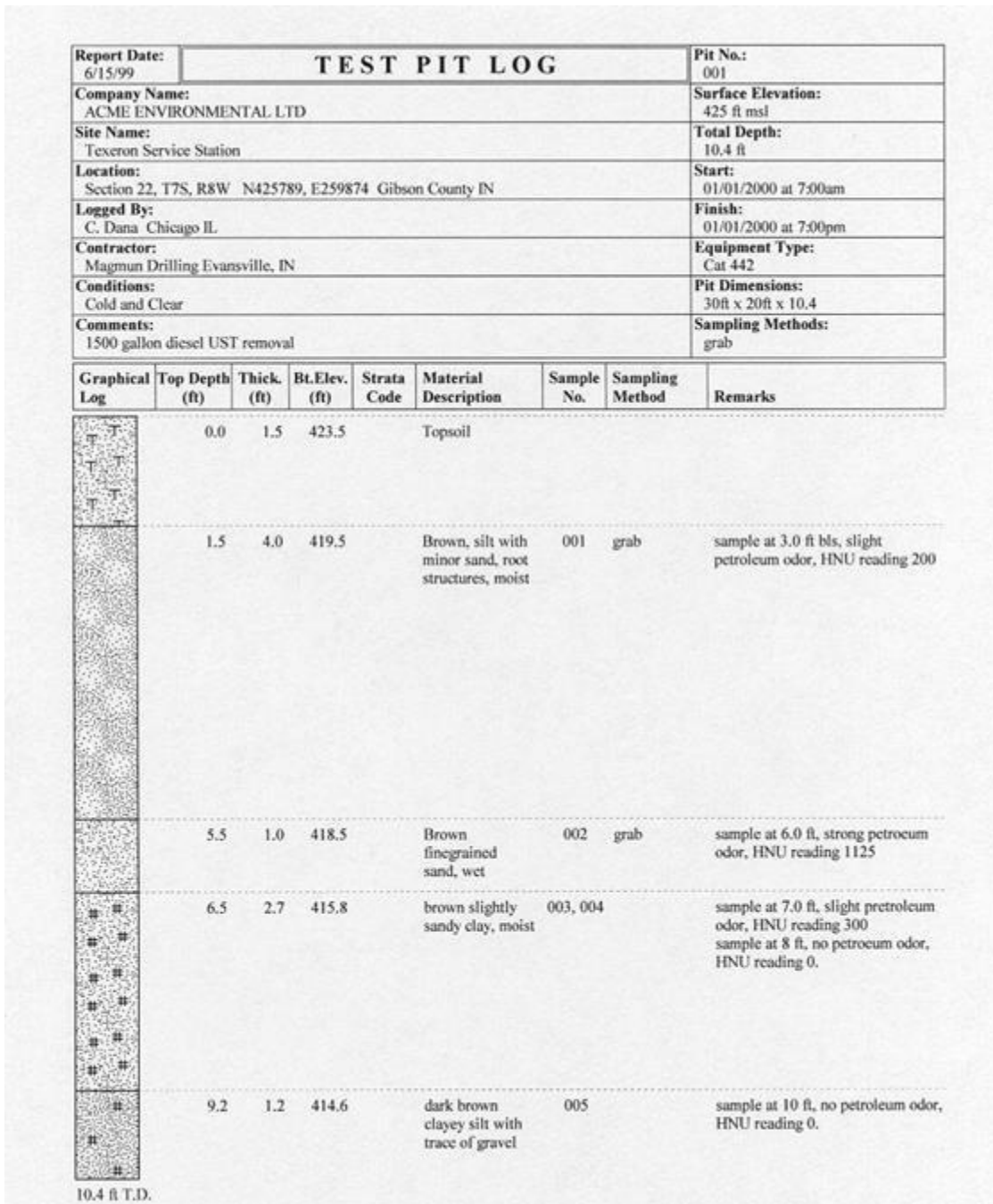


Fig 1 Sample of bore log
 [Fig1 http://www.easysolve.com/el_rpt.htm]
 Recordofboring[IS1892-1979]



2. Bearing capacity of soil
3. Depth of water level below the ground surface
4. Types of soil and depth of layers in case of layered soil
5. Depth of adjacent foundation

The minimum depth of foundation should be considered to ensure that the soil is having the required safe bearing capacity as assumed in the design. However, it is advised to carry out soil investigation before deciding on depth of foundation.

Soil investigation report will suggest the foundation depth based on the type of structure, soil properties, depth of water table, and all other variable that should be considered. Soil investigation report provides bearing capacity of soil at different levels and at different locations.

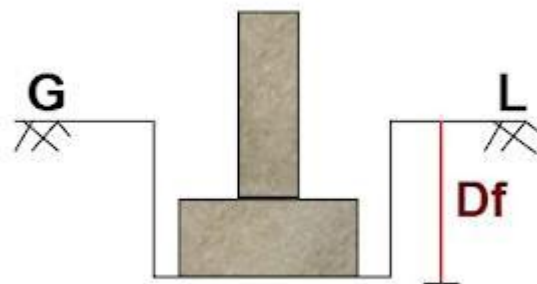


Fig 1. Depth of Foundation

[Fig 1 <https://www.paramvisions.com/2021/04/calculating-depth-of-foundation-by.html>]

When the soil investigation report is not available, the depth of foundation should be selected such that it is not affected by swelling and shrinking of soil due to seasonal changes. Depth of foundation should also consider the depth of water table to prevent and scour below the ground.

For foundation near existing foundation, it must be ensured that pressure bulbs of foundations do not coincide if the depth of new foundation has to be taken below the depth of existing foundation.



Rankine's Formula

$$h = \frac{p}{\gamma} \left(\frac{1 - \sin\phi}{1 + \sin\phi} \right)^2$$

Where, h = minimum depth of foundation

p= gross bearing capacity

γ = density of soil

ϕ = angle of repose or internal friction of soil.

The above formula does not consider the factors discussed above and just provides the guidance on minimum foundation depth, assuming that the foundations are not affected by factors such as water table, frost action, types and properties of soil etc. as discussed above. This formula does not consider the loads from the structure on the foundation.

In the Rankine's formula, it can be seen that foundation depth depends on the bearing capacity of soil, so, if the bearing capacity of soil increases, the depth of foundation also increases.

FACTORS AFFECTING DEPTH OF FOUNDATION

- Before calculating depth of shallow foundation, the following factors have to be considered well in advance.
- Foundation should be placed at such a depth so that it is safe against damages due to swelling, shrinkage or freezing of sub soil.
- Bearing capacity of soil beneath the foundation must be adequate to support the load coming from foundation.
- If foundation has to be placed on cohesive soil, then the settlement due to consolidation should not be excessive.
- Never place foundation on loose or disturbed soils which have a tendency to erode by wind or flood.
- If possible then foundation should be placed above ground water table as this can avoid cost of pumping, and can prevent instability of soil due to seepage of water into the bottom of an excavation.
- Make an investigation on foundation soil to know its physical and chemical properties, because presence of sulphate can damage foundation.



larger than the footing.

2. Frost depth:

The footing should be carried below the depth of frost penetration. If the footing is located at insufficient depth, it would be subjected to the frost damage due to formation of ice lenses and consequent frost heave. During summer, thawing occurs from the top downwards and the melted water is entrapped

3. Zone of soil volume change:

Some clay, especially clays having high plasticity, such as black cotton soil, undergoes excessive volume changes. Such soil shrinks upon drying and swells upon wetting. The volume changes are generally greater near the ground surface and decrease with increase in depth. Large volume change beneath a footing may cause lifting and dropping. The footing should be placed below strata that are subjected to large volume change.

4. Adjacent footing and property lines:

- The footing should be so located that no damage is done to the existing structure. The adjacent structure may be damaged by construction of a new footing due to vibrations, undermining or lowering of the water table. The new footing may also impose additional load on the existing footing which may cause settlement.
- In general, deeper the new footing and closer to the existing structure the greater is the potential damage to the existing structure. This is particularly more severe if the new footing is lower than the existing footing.
- As far as possible, the new footing should be placed at a small depth as the old ones and the sites of excavation adjacent to the existing structure should be suitably supported. If the footings are placed at the different levels, the slope of the line joining the two footings should not be steeper than two horizontal to one vertical as per IS: 1904-1978.

5. Sloping ground:

If a footing is located adjacent to a sloping ground, the sloping ground surface should not encroach upon a frustum of bearing material under the footing having sides



The footings located in streams, on water fronts or other locations where there is a possibility of scouring should be placed below the potential scour depth.

8. Underground defects:

The depth of footing is also affected by the presence of underground defects such as faults, causes and mines. If there are manmade discontinuities, such as sewer lines, water mains, underground cables, these should be shifted or footing should be relocated.

9. Root holes:

If there are root holes or cavities caused by burrowing animals or worms, the footing should be placed below such a zone of weakened soil.

10. Minimum depth:

IS 1904 – 1978 specifies that all foundations should extend to a depth of at least 50cm below the natural ground surface. However, in case of rocks, only its top soil should be removed and the surface should be cleaned and if necessary stepped.



The bearing capacity of soil is defined as the capacity of the soil to bear the loads coming from the foundation. The pressure which the soil can easily withstand against load is called allowable bearing pressure. The bearing capacity of soil is the maximum average contact pressure between the foundation and the soil which should not produce shear failure in the soil. There are three modes of failure that limit bearing capacity: general shear failure, local shear failure, and punching shear failure. It depends upon the shear strength of soil as well as shape, size, depth and type of foundation.

Basic definitions:

1. Ultimate bearing capacity or Gross bearing capacity (q_u):

It is the least gross pressure which will cause shear failure of the supporting soil immediately below the footing.

2. Net ultimate bearing capacity (q_{nu}):

It is the net pressure that can be applied to the footing by external loads that will just initiate failure in the underlying soil. It is equal to ultimate bearing capacity minus the stress due to the weight of the footing and any soil or surcharge directly above it.

3. Safe bearing capacity:

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

a. Safe net bearing capacity (q_{ns}):

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

(or)

Net ultimate bearing capacity is divided by certain factor of safety will give the net safe bearing capacity.

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

Where F = factor of safety = 3

b. Gross Safe bearing capacity (q_s):

It is the maximum gross pressure which the soil can carry safely without shear failure.

(or)



It is the maximum soil pressure without any shear failure or settlement failure.

5. Net safe settlement pressure (q_{np})

The pressure with which the soil can carry without exceeding the allowable settlement is called net safe settlement pressure.

6. Net allowable bearing pressure (q_{na})

This is the pressure we can use for the design of foundations. This is equal to net safe bearing pressure if $q_{np} > q_{ns}$. In the reverse case it is equal to net safe settlement pressure.

Methods of Improving the Bearing Capacity of soils:

The bearing capacity of a soil mainly depends on the closeness of its particles. The bearing capacity of a soil can be increased by the following methods:

1. By increasing the depth of foundation.

The compactness of the soil increases as we go below the ground level. As the bearing capacity directly depends on the compactness of the soil, it will go on increasing as the depth of foundation is increased.

2. By draining of the sub-soil under.

Water reduces the cohesive properties and hence reduces the bearing capacity of the soil. By draining off water from the sub-soil the bearing capacity of the soil is certainly increased.

3. By compacting the soil.

If the soil is compacted thoroughly, the voids are decreased and bearing capacity is increased.

4. By confining the soil and preventing it from spreading and lateral movement.

Spreading soils, if confined by sheet piling will resist more loads, that is, their bearing capacity will increase.

5. By increasing the width of foundation.

By increasing the width of foundations, the intensity of load is decreased and on the same soil more loads can be placed.

6. By hardening the soil by grouting, i.e. pumping in the cement-grout into the ground.



chloride etc.

Factors influencing bearing capacity of soils

1. Soil Strength
2. Foundation Width
3. Foundation Depth
4. Soil Weight and Surcharge
5. Spacing Between Foundations
6. Earthquake and Dynamic Motion
7. Frost Action
8. Subsurface Voids
9. Expansive and Collapsible Soils
10. Potential Heave
11. Soil Reinforcement
12. Soil Erosion and Seepage

1. Soil Strength:

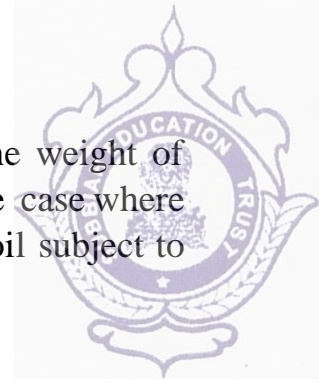
Bearing capacity of cohesionless soil and mixed soil increases unproportionally with the increase of in the effective friction angle. However, bearing capacity of cohesive soil varies linearly with the soil cohesion provided that the effective friction angle is zero.

2. Foundation Width

- Foundation width affects bearing capacity of cohesionless soil. The bearing capacity of a footing placed at the surface of cohesionless soil, where the soil shear strength is considerably dependent on internal friction, is proportional to the width of the foundation. Bearing capacity of cohesive soil of constant shear strength and infinite depth is independent of foundation width.

3. Foundation Depth

- The greater the bearing capacity the deeper the foundation. This is specifically obvious in a uniform cohesionless soil. In contrary, if the foundation is carried down to a weak soil layer, then bearing capacity is declined.



- Foundations placed at depths where the structural weight equals the weight of displaced soil usually assures adequate bearing capacity apart from the case where the structure supported by under-consolidated soil and collapsible soil subject to wetting.

4. Soil Weight and Surcharge

The contribution of subsurface and surcharge soil, which are influenced by water table, to the bearing capacity cannot be ignored. The water table should not be above the base of the foundation to avoid construction, seepage, and uplift problems. If the water table is below the depth of the failure surface, then it has no influence on the bearing capacity.

5. Spacing between foundations

It is recommended to consider minimum spacing between footings, which 1.5 times foundation width, during the design of foundation in order to avoid reduction in bearing capacity.

6. Earthquake and Dynamic Motion

Repeated movements could increase pore pressure in foundation soil and consequently bearing capacity is decreased. Sources of cyclic movements are earthquakes, vibrating machinery, and other sources like vehicular traffic, blasting, and pile driving.

The foundation soil can liquefy when pore pressures equal or exceed the soil confining stress. Liquefaction reduces effective stress to zero and causes gross differential settlement of structures and loss of bearing capacity.

7. Frost Action

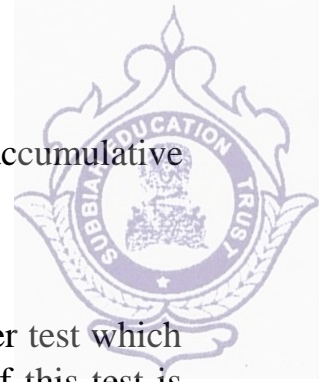
Frost heave in certain soils in contact with water and subject to freezing temperatures or loss of strength of frozen soil upon thawing can alter bearing capacity over time. Low cohesion materials containing a high percentage of silt-sized particle are mostly susceptible to frost action.

8. Subsurface Voids

Bearing capacity of soil decreases due to subsurface voids which are within a critical depth beneath the foundation. The critical depth is that depth below which the influence of pressure in the soil from the foundation is negligible.

9. Expansive and Collapsible Soils

Collapsible and expansive soil can have large strength and bearing capacity when they are fairly dry. However, the volume of these soils changes due to changes in water content. This leads to total and differential foundation movements. Seasonal wetting and drying cycles may cause soil movements that often lead to



excessive long-term deterioration of structures with substantial accumulative damage.

10. Potential Heave

The potential heave can be determined from results of consolidometer test which can be performed in accordance with ASTM D 4546. The results of this test is considered in determining preparation of foundation soils to reduce destructive differential movements and to provide a foundation of sufficient capacity to withstand or isolate the expected soil heave.

11. Soil Reinforcement

Bearing capacity of soft or weak soil can be increased greatly by installing various forms of reinforcement in the soil like metal ties, strips, or grids, geotextile fabrics, or granular materials.

12. Soil Erosion and Seepage

Erosion of soil around and under foundations and seepage can reduce bearing capacity and can cause foundation failure.

Types of shear failure of foundation soils:

Depending on the stiffness of foundation soil and depth of foundation, the following are the modes of shear failure experienced by the foundation soil.

- General shear failure (Fig.1(a))
- Local shear failure (Fig.1(b))
- Punching shear failure (Fig.1(c))

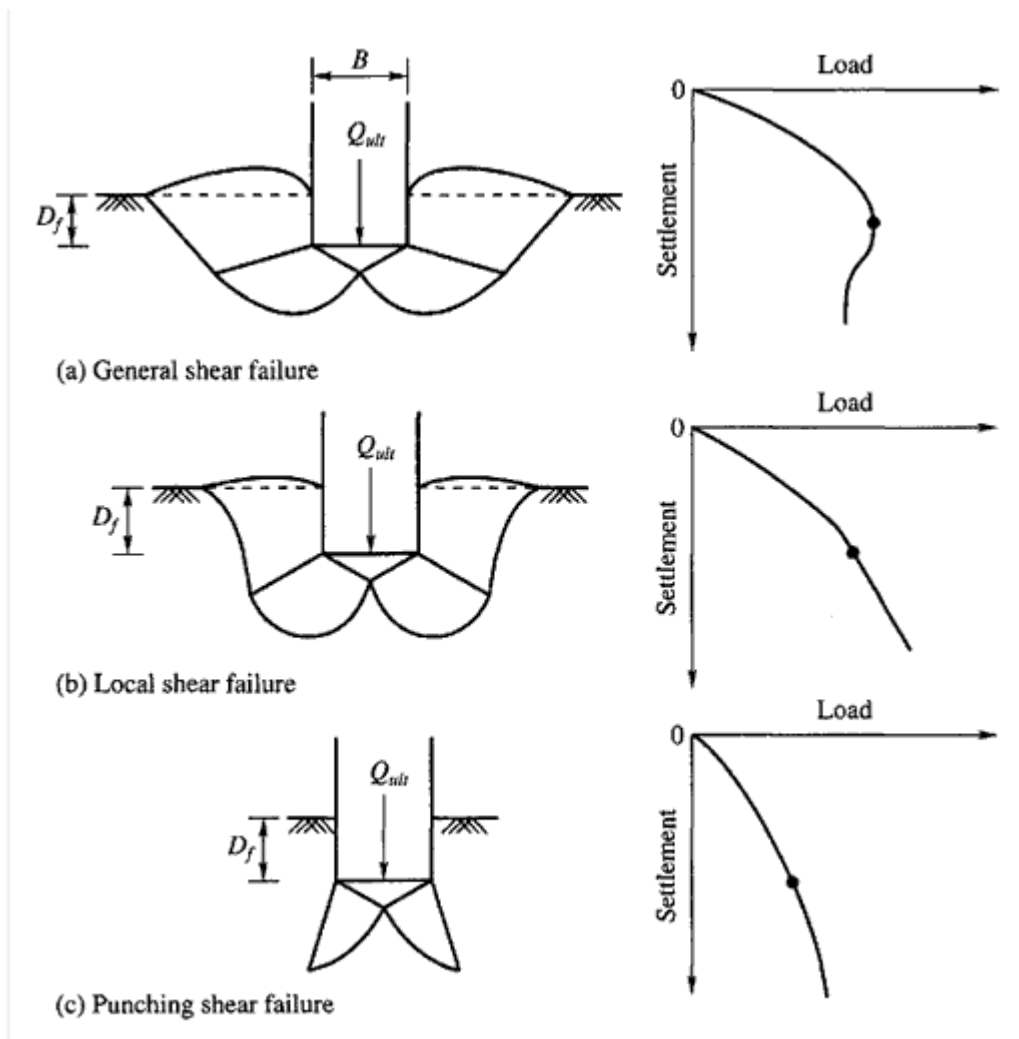


Fig 1 Different modes of failure

[Fig1 <http://www.abuildersengineer.com/2012/11/types-of-failure-in-soil.html>]

General Shear Failure:

- This type of failure is seen in dense and stiff soil. The following are some characteristics of general shear failure. Continuous, well defined and distinct failure surface develops between the edge of footing and ground surface.
- Dense or stiff soil that undergoes low compressibility experiences this failure.
- Continuous bulging of shear mass adjacent to footing is visible.
- Failure is accompanied by tilting of footing.
- Failure is sudden and catastrophic with pronounced peak in curve.
- The length of disturbance beyond the edge of footing is large.
- State of plastic equilibrium is reached initially at the footing edge and spreads gradually downwards and outwards.
- General shear failure is accompanied by low strain (<5%) in a soil with considerable (>36°) and large N ($N > 30$) having high relative density ($I_D > 70\%$).

**Local Shear Failure:**

- This type of failure is seen in relatively loose and soft soil. The following are some characteristics of general shear failure. A significant compression of soil below the footing and partial development of plastic equilibrium is observed.
- Failure is not sudden and there is no tilting of footing.
- Failure surface does not reach the ground surface and slight bulging of soil around the footing is observed.
- Failure surface is not well defined.
- Failure is characterized by considerable settlement.
- Well defined peak is absent in curve.
- Local shear failure is accompanied by large strain (> 10 to 20%) in a soil with considerably low ($< 28^\circ$) and low N ($N < 5$) having low relative density ($I_D > 20\%$).

Punching Shear Failure of foundation soils:

- This type of failure is seen in loose and soft soil and at deeper elevations. The following are some characteristics of general shear failure. This type of failure occurs in a soil of very high compressibility.
- Failure pattern is not observed.
- Bulging of soil around the footing is absent.
- Failure is characterized by very large settlement.
- Continuous settlement with no increase in P is observed in $p-\Delta$ curve.

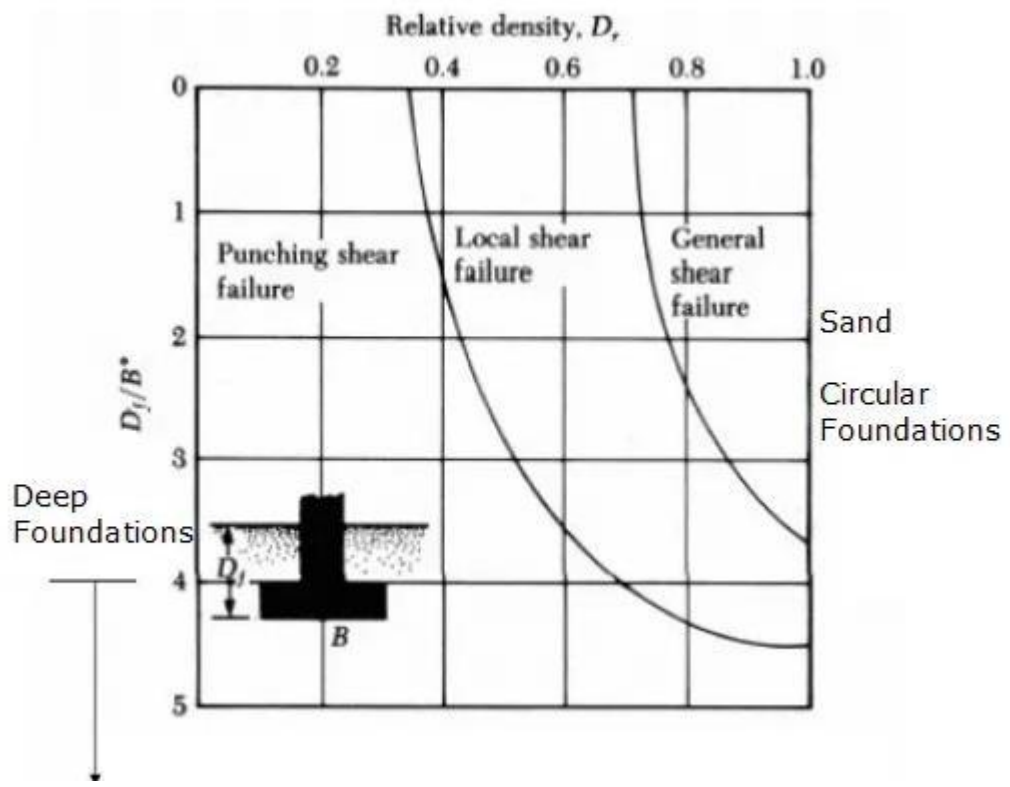
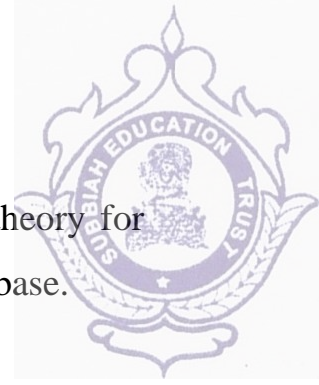


Fig. 2 presents the conditions for different failure modes in sandy soil carrying circular footing based on the contributions from Vesic (1963 & 1973)

[Fig 2 <https://theconstructor.org/geotechnical/types-of-shear-failure-of-foundation-soils/7492/>]



2.3 Terzaghi equation:

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

Assumptions:

1. Soil is homogeneous and Isotropic.
2. The shear strength of soil is represented by Mohr Coulombs Criteria.
3. The footing is of strip footing type with rough base. It is essentially a two dimensional plane strain problem.
4. Elastic zone has straight boundaries inclined at an angle equal to the horizontal.
5. Failure zone is not extended above, beyond the base of the footing. Shear resistance of soil above the base of footing is neglected.
6. Method of superposition is valid.
7. Passive pressure force has three components (P_{PC} produced by cohesion, P_{Pq} produced by surcharge and $P_{P\gamma}$ produced by weight of shear zone).
8. Effect of water table is neglected.
9. Footing carries concentric and vertical loads.
10. Footing and ground are horizontal.
11. The properties of foundation soil do not change during the shear failure
12. Limit equilibrium is reached simultaneously at all points. Complete shear failure is mobilized at all points at the same time.

Limitations:

1. The theory is applicable to shallow foundations
2. As the soil compresses, increases which is not considered. Hence fully plastic zone may not develop at the assumed.
3. All points need not experience limit equilibrium condition at different loads.
4. Method of superposition is not acceptable in plastic conditions as the ground is near failure zone.

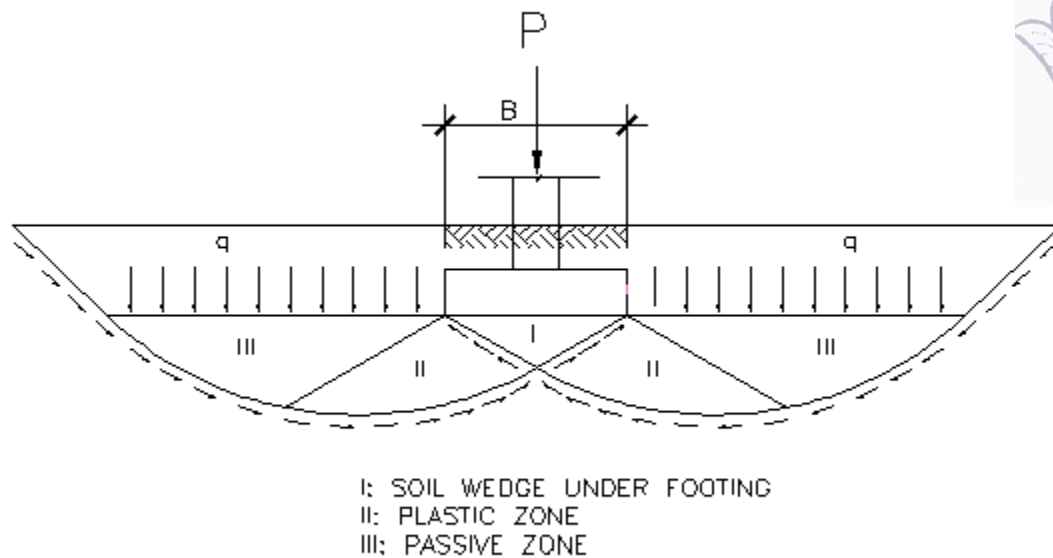


Fig 1 Shear stresses based on Terzaghi's soil bearing capacity theory.

[Fig 1 <https://civilengineeringbible.com/subtopics.php?i=1>]

Terzaghi's concept of Footing with five distinct failure zones in foundation soil

- The soil is semi-infinite, homogeneous and isotropic
- The problem is two-dimensional
- The base of the footing is rough
- The failure is by general shear
- the load is vertical and symmetrical
- The ground surface is horizontal
- the overburden pressure at foundation level is equivalent to a surcharge load
- the principle of superposition is valid,

Coulomb's law is strictly valid, that is

$$r = C + \sigma \tan \phi$$

Mechanism of Failure:

- The shapes of the failure surfaces under ultimate loading conditions are given in Fig.
- The zones of plastic equilibrium represented in this figure by the area *gedcf* may be subdivided into three zones:
 - 1 . Zone I of elastic equilibrium
 2. Zones II of radial shear state



3. Zones III of Rankine passive state

- When load q_u per unit area acting on the base of the footing of width B with a rough base is transmitted into the soil, the tendency of the soil located within zone I is to spread but this is counteracted by friction and adhesion between the soil and the base of the footing.
- Due to the existence of this resistance against lateral spreading, the soil located immediately beneath the base remains permanently in a state of elastic equilibrium, and the soil located within this central Zone I behaves as if it were a part of the footing and sinks with the footing under the superimposed load.

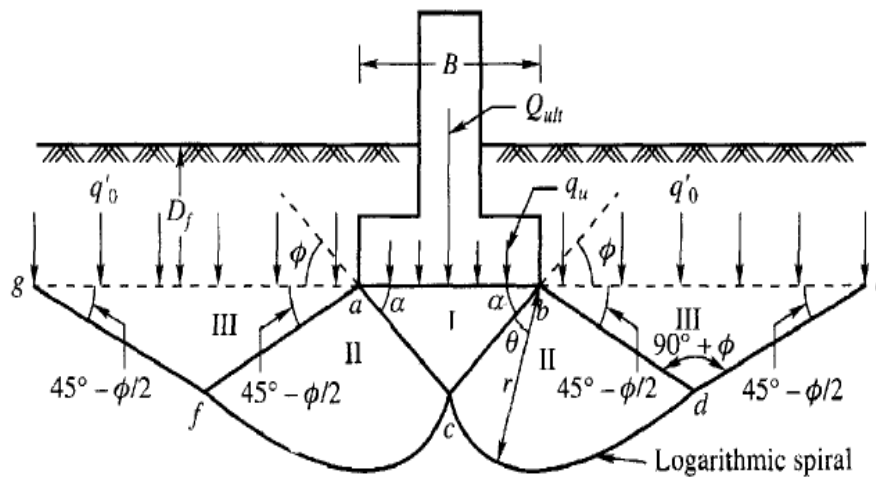
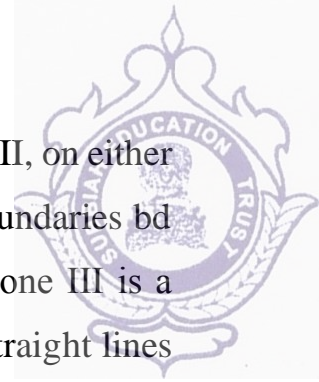


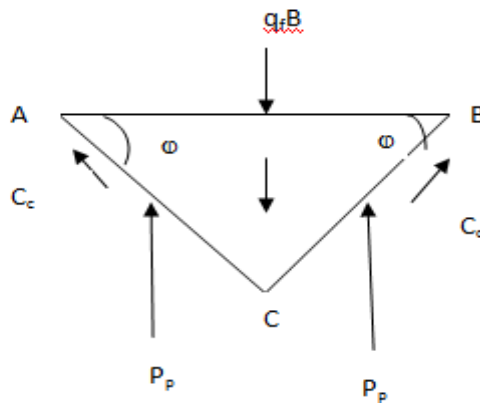
Fig 2 Shear stresses based on Terzaghi's soil bearing capacity theory

[Fig 2 <https://www.pinterest.com/pin/682013937310128083/>]

- The depth of this wedge shaped body of soil abc remains practically unchanged, yet the footing sinks.
- This process is only conceivable if the soil located just below point c moves vertically downwards. This type of movement requires that the surface of sliding cd (Fig.) through point c should start from a vertical tangent. The boundary bc of the zone of radial shear bed (Zone II) is also the surface of sliding.
- As per the theory of plasticity, the potential surfaces of sliding in an ideal plastic material intersect each other in every point of the zone of plastic equilibrium at an angle $(90^\circ - \phi)$. Therefore, the boundary be must rise at an angle ϕ to the horizontal provided the friction and adhesion between the soil and the base of the footing suffice to prevent a sliding motion at the base.



- The sinking of Zone I creates two zones of plastic equilibrium, II and III, on either side of the footing. Zone II is the radial shear zone whose remote boundaries bd and af meet the horizontal surface at angles $(45^\circ - \phi/2)$, whereas Zone III is a passive Rankine zone. The boundaries de and fg of these zones are straight lines and they meet the surface at angles of $(45^\circ - \phi/2)$. The curved parts cd and cf in Zone II are parts of logarithmic spirals whose centers are located at b and a respectively.



Downward force:

- i) weight of soil wedge ABC

$$w = \frac{1}{4} \gamma B^2 \tan \phi$$

- ii) Total load on footing $q_f B$

Upward force:

- i) passive force
- ii) cohesion (c)

Length of AC and CB

$$\cos \phi = \frac{\text{adj}}{\text{hypo}} = \frac{\frac{B}{2}}{AC}$$

$$AC = \frac{B}{2 \cos \phi}$$

Vertical component $C = \left(\frac{B/2}{\cos \phi} \cdot C \right) \sin \phi$



$$C = \left(\frac{B}{2} \cdot C\right) \tan\phi$$

vertical component of $C = \frac{B}{2} C \tan\phi$

i) $2P_p$

ii) $\frac{B}{2} \times \tan\phi \times 2C$

Upward = Downward

$$2P_p + BC \tan\phi = q_f B + \frac{1}{4} \gamma B^2 \tan\phi$$

$$q_f B = 2P_p + BC \tan\phi - \frac{1}{4} \gamma B^2 \tan\phi$$

The resultant passive earth pressure has 3 component

i) $P_{P\gamma} \rightarrow$ Produced by weight of shearzone BCDE

ii) $P_{Pc} \rightarrow$ Produced by cohesion

iii) $P_{Pq} \rightarrow$ Produced by surcharge q

$$q B = P_f + P_{Pc} + P_{Pq} + BC \tan\phi - \frac{1}{4} \gamma B^2 \tan\phi$$

$$q B = P_f + P_{Pc} + 2P_{Pq} + BC \tan\phi - \frac{1}{4} \gamma B^2 \tan\phi$$

$$q B = [2P_{P\gamma} - \frac{1}{4} \gamma B^2 \tan\phi] + P_{Pc} + P_{Pq}$$

Let, $2P_{P\gamma} - \frac{1}{4} \gamma B^2 \tan\phi = Bx \frac{1}{2} \gamma B N_\gamma$

$$2P_{Pc} + BC \tan\phi = Bc N_c$$

$$2P_{Pq} = B\gamma D N_q$$

Substitute in above equation

$$q_f B = Bx \frac{1}{2} \gamma B N_\gamma + Bc N_c + B\gamma D N_q$$

$$q_f B = B \left[\frac{1}{2} \gamma B N_\gamma + c N_c + \gamma D N_q \right]$$

$$q_f = \left[\frac{1}{2} \gamma B N_\gamma + c N_c + \gamma D N_q \right]$$

$$q_{nf} = q_f - \bar{\sigma}$$





$$q_{nf} = q_f - \gamma D$$

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

N_c, N_q, N_γ = Bearing Capacity factor which are dimensionless depend on angle of shear resistance

$$N_q = \left[\frac{a^2}{2 \cos^2 \left(45 + \frac{\phi}{2} \right)} \right]$$

$$a = e^{(3\pi/4 - \phi) \tan \phi}$$

$$a = e^{4.2}$$

$$N_c = (N_q - 1) \cos \phi$$

$$N_\gamma = \frac{1}{2} \left[\frac{K_p}{\cos \phi} - 1 \right] \tan \phi$$

Ultimate bearing capacity,

$$q_f = cN_c + \gamma D N_q + 0.5 \gamma B N_\gamma$$

If the ground is subjected to additional surcharge load q , then

$$q_f = cN_c + (\gamma D + q) N_q + 0.5 \gamma B N_\gamma$$

Net ultimate bearing capacity,

$$q_n = cN_c + \gamma D (N_q - 1) + 0.5 \gamma B N_\gamma - \gamma D$$

$$q_n = cN_c + \gamma D (N_q - 1) + 0.5 \gamma B N_\gamma$$

Safe bearing capacity,

$$q_s = cN_c + \gamma D (N_q - 1) + 0.5 \gamma B N_\gamma / F + \gamma D$$

Here, F = Factor of safety (usually 3)

c = cohesion

γ = unit weight of soil

D = Depth of foundation

q = Surcharge at the ground level

B = Width of foundation

N_c, N_q, N_γ = Bearing Capacity factors

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

$$N_q = e^{2(3\pi/4 - \phi/2)} \tan \phi / [2 \cos^2(45 + \phi/2)]$$

$$N_\gamma = (1/2) \tan \phi (K_{pr} / \cos^2 \phi - 1)$$



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

K_p = coefficient of passive earth pressure.

Strip footings: $q_f = c N_c + \gamma D N_q + 0.5 \gamma B N_\gamma$

Square footings: $q_f = 1.3 c N_c + \gamma D N_q + 0.4 \gamma B N_\gamma$

Circular footings: $q_f = 1.3 c N_c + \gamma D N_q + 0.3 \gamma B N_\gamma$

Rectangular footing: $q_f = \left[1 + 0.3 \frac{B}{L}\right] c N_c + \gamma D N_q + \left[1 - 0.3 \frac{B}{L}\right] \gamma B N_\gamma$

Note:

Local shear failure ($\Phi < 28^\circ$) ----- N'_c, N'_q, N'_γ

General shear failure ($\Phi > 36^\circ$) ----- N_c, N_q, N_γ

Terzaghi's Problems:

1. A square footing 2.5 m x 2.5 m is built in a homogeneous bed of sand of unit weight 20 KN/m³ and having an angle of shearing resistance of 36°. The depth of the base of footing is 1.5 m below the ground surface. Find the safe load that can be carried by a footing with a factor of safety of 3 against complete shear failure. Use Terzaghi's analysis.

Given Data;

$$L = B = 2.5 \text{ m}$$

$$D = 1.5 \text{ m}$$

$$\text{Unit weight } (\gamma) = 20 \text{ KN/m}^3$$

$$\Phi = 36^\circ$$

$$C = 0$$

$$\text{FOS} = 3$$

From the graph

$$N'_c = 60, N'_q = 42, N'_\gamma = 50$$

To find:

$$\text{Safe load} = ?$$

Solution:

Here $\Phi = 36^\circ$ therefore it is general shear failure. The equation can be written as



$$q_f = [cN'_c + \gamma DN'_q + 0.4\gamma BN'_\gamma]$$

$$q_f = [(0 \times 60) + (20 \times 1.5 \times 42) + (0.4 \times 20 \times 2.5 \times 50)]$$

$$= 2260 \text{ KN/m}^2$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 2260 - (20 \times 1.5)$$

$$= 2230 \text{ KN/m}^2$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{q_{nf}}{F} + \gamma D$$

$$q_s = \frac{2230}{3} + 20 \times 1.5$$

$$= 743.33 + 30$$

$$= 773.3 \text{ KN/m}^2$$

$$q_s = \frac{\text{load}}{\text{Area}}$$

$$\text{safe load}(W) = q_s \times \text{Area}$$

$$\text{Area} = B^2 = (2.5)^2 = 6.25 \text{ m}^2$$

$$W = 773.3 \times 6.25 = 4833.3 \text{ KN}$$

2. A square footing located at a depth of 1.5 m below the ground surface in Cohesion less soil carries a column load of 1280 kN. The soil is submerged having an effective unit weight of 11.5 kN/m³ and an angle of shearing resistance of 30°. Show and find the size of the footing for Fos = 3 by Terzaghi's theory of general shear failure.

Given Data;

B=?

D=1.5m



$$\Phi = 30^\circ$$

Cohesion less, $C=0$

Load = 1280 kN

From the graph

$$N_c = 37.2, N_q = 22.5, N_\gamma = 19.7$$

To find:

Size of footing (B) = ?

Solution:

Bearing capacity of soil,

$$q_f = [cN_c + \gamma DN_q + 0.4\gamma BN_\gamma]$$

$$q_f = [0 \times 37.2 + 11.5 \times 1.5 \times 22.5 + 0.4 \times 11.5 \times B \times 19.7]$$

$$q_f = 388.125 + 90.62B$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 388.125 + 90.62B - (11.5 \times 1.5)$$

$$q_{nf} = 388.125 + 90.62B - 17.25$$

$$q_{nf} = 370.875 + 90.62B$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{370.875 + 90.62B}{3} + (11.5 \times 1.5)$$

$$q_s = \frac{370.875 + 90.62B}{3} + 17.25$$

$$q_s = \frac{370.875 + 90.62B + (3 \times 17.25)}{3}$$

$$q_s = \frac{370.875 + 90.62B + 51.75}{3}$$



$$q_s = \frac{422.625 + 90.62B}{3}$$

$$q_s = \frac{\text{load}}{\text{Area}}$$

$$\text{safe load}(W) = q_s \times \text{Area}$$

$$\text{safe load}(W) = q_s \times B^2$$

$$1280 = \frac{422.625 + 90.62B}{3} \times B^2$$

$$3840 = (422.65 + 90.62B)B^2$$

$$3840 = 422.65B^2 + 90.62B^3$$

$$90.62B^3 + 422.65B^2 + 0B - 3840 = 0$$

$$B = 2.44\text{m}$$

$$\text{Size of footing} = 2.44 \times 2.44\text{m}$$

3. A rectangular footing (2x3m) rests on a C-φ soil which is 1.5m below the ground surface. Calculate the safe bearing capacity using FOS=3, C=10KN/m³, φ=30 degree. N_c = 31.2, N_q = 22.5 and N_γ = 19.7 and also soil has following properties voids ratio=0.55, degree of saturation=50%, specific gravity=2.67.

Given data:

$$B = 2\text{m}$$

$$L = 3\text{m}$$

$$D = 1.5\text{m}$$

$$\text{FOS} = 3$$

$$C = 10\text{KN/m}^3$$

$$\phi = 30$$

$$N_c = 31.2, N_q = 22.5 \text{ and } N_\gamma = 19.7$$

$$\text{voids ratio}(e) = 0.55,$$

$$\text{degree of saturation}(S_r) = 50\% = 0.5$$

$$\text{specific gravity}(G) = 2.67$$

To find:

$$\text{Safe bearing capacity}(q_s) = ?$$

Solution:



$$\gamma = \frac{(G + eS_r)\gamma_w}{1 + e}$$

$$\gamma = \frac{(2.67 + 0.55 \times 0.5)9.81}{1 + 0.55}$$

$$\gamma = 18.639 \text{ KN/m}^3$$

Bearing capacity of soil,

$$q_f = [(1 + 0.3) \frac{B}{L} c N_c + \gamma D N_q + (1 - 0.3) \frac{B}{L} \times 0.5 \gamma B N_\gamma]$$

$$q_f = [(1 + 0.3) \frac{2}{3} 10 \times 31.2 + 18.639 \times 1.5 \times 22.5 + (1 - 0.3) \frac{2}{3} \times 0.5 \times 18.639 \times 2 \times 19.7]$$

$$q_f = 1158.82 \text{ KN/m}^2$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 1158.82 - (18.639 \times 1.5)$$

$$q_{nf} = 1130.86 \text{ KN/m}^2$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{1130.86}{3} + (18.639 \times 1.5)$$

$$q_s = 404.90 \text{ KN/m}^2$$

Effect of water table:

Ultimate bearing capacity with the effect of water table,

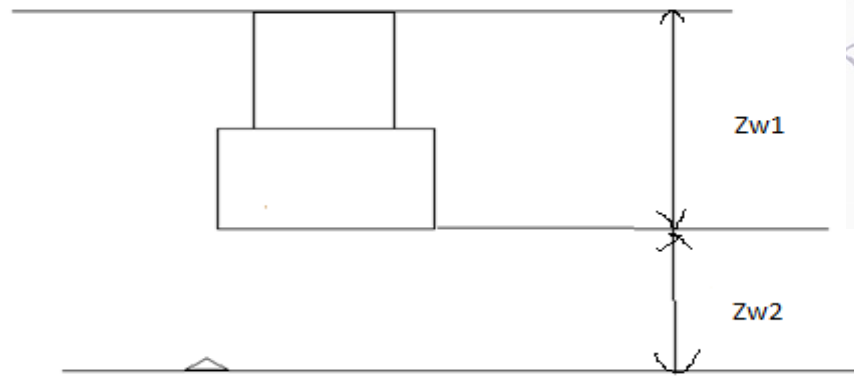
$$q_f = c N_c + \gamma D N_q R_{w1} + 0.5 \gamma B N_\gamma R_{w2}$$

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

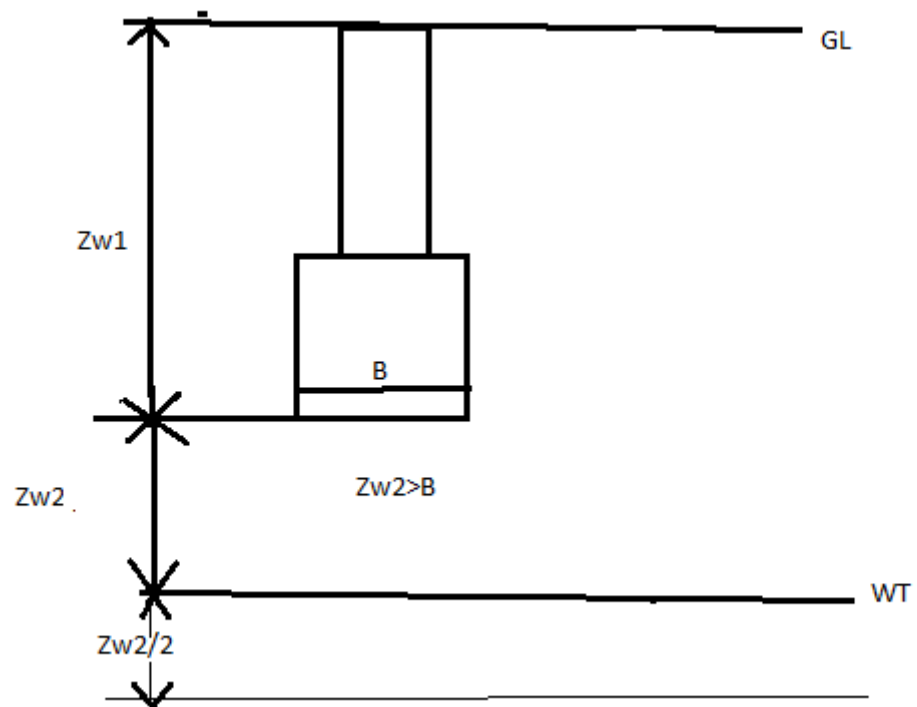
$$R_{w2} = \frac{1}{2} \left[1 + \frac{Z_{w2}}{B} \right]$$

Case1: water level Below the Footing:

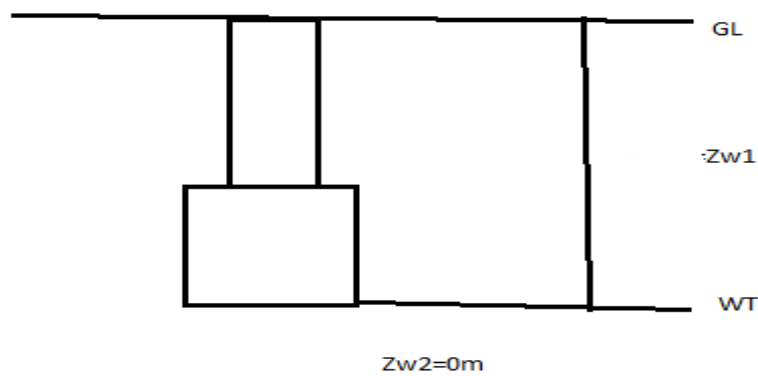
a) $Z_{w2} < B$



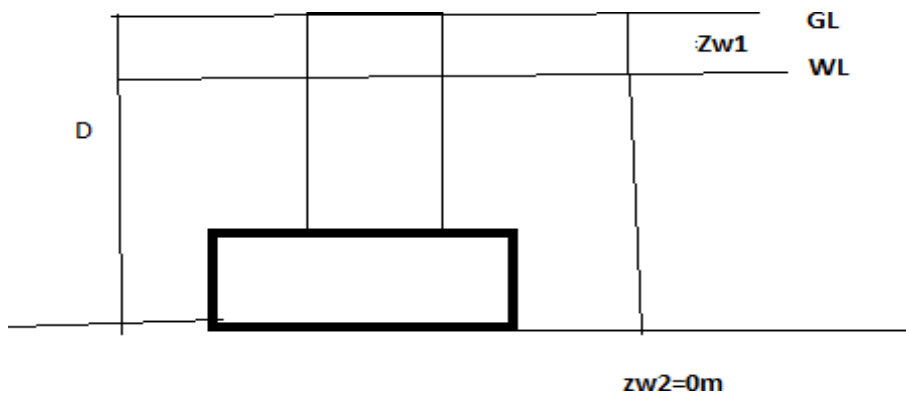
b) $Z_{w2} > B$



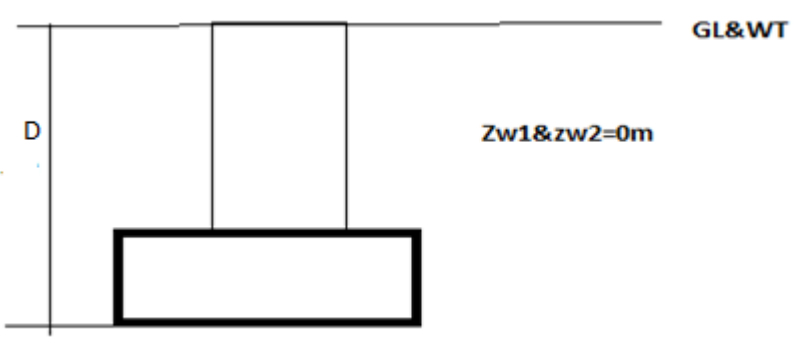
Case2: water level at the base of Footing:



Case3: water level above the Footing:



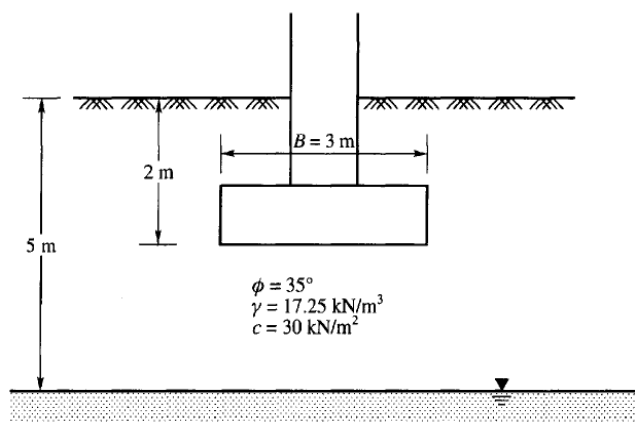
Case4: water level at Ground level:



Terzaghi’s Problem with water table:

1. A strip footing of width 3 m is founded at a depth of 2 m below the ground surface in a (c - ϕ) soil having a cohesion $c = 30 \text{ kN/m}^2$ and angle of shearing resistance $\phi = 35^\circ$. The water table is at a depth of 5 m below ground level. The moist weight of soil above the water table is 17.25 kN/m^3 .

Determine (a) the ultimate bearing capacity of the soil, (b) the net bearing capacity, and (c) the net allowable bearing pressure and the load/m for a factor of safety of 3. Use the general shear failure theory of Terzaghi.



**Given data:**

strip foundation

Width=3 m

Depth of foundation $D = 2\text{m}$ $\phi = 35^\circ$ $C = 30\text{KN/m}^3$ $\gamma = 17.25\text{KN/m}^3$

Fos=3

 $N_c = 57.8$, $N_q = 41.4$ and $N_\gamma = 42.4$ **To find:**

- the ultimate bearing capacity of the soil,
- the net bearing capacity, and
- the net allowable bearing pressure and the load/m

Solution:

$$q_f = \left[\frac{2}{3} c N_c + \gamma D N_q R_{w1} + 0.5 \gamma B N_\gamma R_{w2} \right]$$

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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{2}{2} \right]$$

$$R_{w1} = 1\text{m}$$

 Z_{w1} = depth of foundation from GL = 2m Z_{w2} = depth of foundation to water level = 5 - 2 = 3m

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

$$R_{w2} = \frac{1}{2} \left[1 + \frac{3}{3} \right]$$

$$R_{w2} = 1\text{m}$$

$$q_f = \left[\left(\frac{2}{3} \times 30 \times 57.8 \right) + \left(17.25 \times 2 \times 41.4 \times 1 \right) + \left(0.5 \times 17.25 \times 3 \times 42.2 \times 1.25 \right) \right]$$

$$q_f = [1156.6 + 1428.3 + 1097.1]$$

$$q_f = 3681 \text{KN/m}^2$$





Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 3681 - (17.25 \times 2)$$

$$q_{nf} = 3681 - 34.5 = 3646.5 \text{ KN/m}^2$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{3646.5}{3} + (17.25 \times 2)$$

$$q_s = 1250 \text{ KN/m}^2$$

2. A square foundation of size $1.8\text{m} \times 1.8\text{m}$ is to be built at a depth of 1.6m on a uniform clay strata having the following properties $\phi = 0^\circ$, $c = 30 \text{ KN/m}^3$ and $\gamma = 18.2 \text{ KN/m}^3$. Find the safe load that the foundation can carry with a factor of safety of 3. Use Terzaghi's bearing capacity theory. If the ground water table subsequently rises from depth of 6m to the ground surface, find the load carrying capacity of the foundation. The submerged density of the soil is 10.5 KN/m^3 .

Given data:

Square foundation size = $1.8\text{m} \times 1.8\text{m}$

Depth of foundation = 1.6m

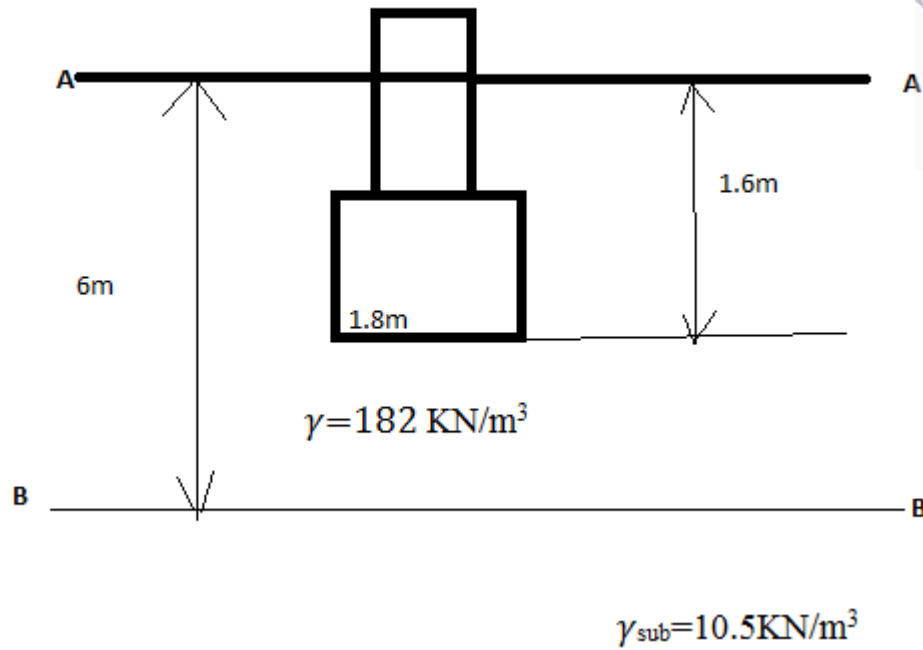
$\phi = 0$

$c = 30 \text{ KN/m}^3$

$\gamma = 18.2 \text{ KN/m}^3$

Fos = 3

$\gamma_{\text{sub}} = 10.5 \text{ KN/m}^3$



To find:

- i) case-i: water table at 6m from G.L. safe load=?
- ii) case-ii: water table at the ground surface safe load=?

Solution:

Case (1): water table at 6m from ground surface.

Bearing capacity of soil

$$q_f = [cN_c + \gamma DN_q R_{w1} + 0.4\gamma_{avg} BN_\gamma R_{w2}]$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{1.6}{1.6} \right]$$

$$R_{w1} = 1m$$

Z_{w1} = depth of foundation from GL = 1.6m

Z_{w2} = depth of foundation to water level = 6 - 1.6 = 4.4m, Z_{w2} > B



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

$$R_{w2} = \frac{1}{2} \left[1 + \frac{4.4}{1.8} \right]$$

$$R_{w2} = 1.72\text{m}$$

For $\phi=0$ [Terzaghi's Bearing capacity factor]

$$N_c=5.7, N_q=1.0, N_f=0$$

$$\gamma_{avg} = \frac{(182 \times 4.4) + \left[\frac{(4.4)}{2} \times 10.5 \right]}{\left[4.4 + \frac{4.4}{2} \right]}$$

$$\gamma_{avg} = 124.8 \text{KN/m}^3$$

Bearing capacity of soil

$$q_f = [cN_c + \gamma DN_q R_{w1} + 0.4 \gamma_{avg} B N_\gamma R_{w2}]$$

$$q_f = [(30 \times 5.7) + (182 \times 1.6 \times 1 \times 1) + (0.4 \times 124.8 \times 1.8 \times 0 \times 1.875)]$$

$$q_f = 462.2 \text{KN/m}^2$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = 462.2 - (182 \times 1.6)$$

$$q_{nf} = 171 \text{KN/m}^2$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$171$$

$$q_s = \frac{171}{3} + (182 \times 1.6)$$

$$q_s = 348.2 \text{KN/m}^2$$

$$q_s = \frac{\text{load}}{\text{Area}}$$

$$\text{safe load}(W) = q_s \times \text{Area}$$

$$\text{safe load}(W) = q_s \times B^2$$

$$\text{safe load}(W) = 348.2 \times (1.8)^2$$

$$W = 1128.2 \text{KN}$$

Case (2) if water table at the ground surface



Bearing capacity of soil

$$q_f = [cN_c + \gamma DN_q R_{w1} + 0.4\gamma BN_\gamma R_{w2}]$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$R_{w1} = \frac{1}{2} \left[1 + \frac{0}{D} \right]$$

$$R_{w1} = \frac{1}{2} \left[1 + \frac{0}{1.6} \right]$$

$$R_{w1} = 0.5m$$

Zw1= depth of foundation from GL=0m

Zw2= depth of foundation to water level=0m

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

$$R_{w2} = \frac{1}{2} \left[1 + \frac{0}{1.8} \right]$$

$$R_{w2} = 0.5m$$

$$q_f = [cN_c + \gamma DN_q R_{w1} + 0.4\gamma BN_\gamma R_{w2}]$$

$$q_f = [(30 \times 5.7) + (182 \times 1.6 \times 1 \times 0.5) + (0.4 \times 182 \times 1.8 \times 0.5)]$$

$$q_f = [171 + 145.6 + 65.52]$$

$$q_f = 391.12 \text{KN/m}^2$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_{nf} = 391.12 - (182 \times 1.6)$$

$$= 99.92 \text{KN/m}^2$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$



$$q_s = \frac{99.92}{3} + (182 \times 1.6)$$

$$q_s = 324.5 \text{ kN/m}^2$$

$$\text{safe load}(W) = q_s \times \text{Area}$$

$$\text{safe load}(W) = q_s \times B^2$$

$$\text{safe load}(W) = 324.5 \times (1.8)^2$$

$$W = 1051.38 \text{ kN}$$

3. A foundation, 2.0 m square of depth 1.2 m is installed 1.2 m above the water table and a submerged density of 10 kN/m^3 . The strength parameters with respect to effective stress $c' = 0$ and $\phi = 30^\circ$. Find the gross ultimate bearing capacity for the following conditions.

1. Water table is 1.2 m below the base of the foundation.
2. Water table raise to the level of the base of the foundation and
3. The water table rise to ground level. (For $\phi = 30^\circ$, Assume $N_q = 22$ and $N_\gamma = 20$).

Solution:

Square footing ($2 \text{ m} \times 2 \text{ m}$)

$$C = 0, \phi = 30^\circ$$

$$N_q = 22, N_\gamma = 20$$

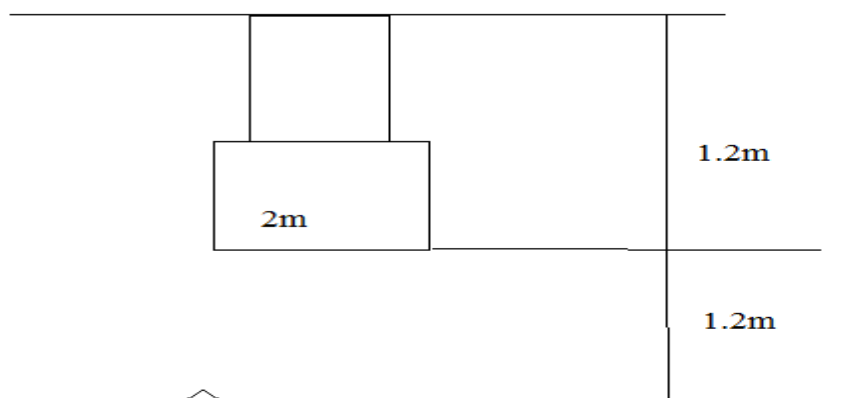
$$\gamma_{\text{sub}} = 10 \text{ kN/m}^3$$

$$\gamma_{\text{sub}} = \gamma_{\text{sat}} - \gamma_w$$

$$\gamma_{\text{sat}} = \gamma_{\text{sub}} + \gamma_w$$

$$\gamma_{\text{sat}} = 10 + 9.81 = 19.81 \text{ kN/m}^3$$

i) Water table is 1.2 m below the base of the foundation:





$$q_f = 1.3 c N_c + \gamma D N_q R_{w1} + 0.4 \gamma B N_r R_{w2}$$

Bearing capacity of soil

$$q_f = [c N_c + \gamma D N_q R_{w1} + 0.4 \gamma_{avg} B N_r R_{w2}]$$

Net Bearing capacity of soil,

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

Safe Bearing capacity of soil,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{1.2}{1.2} \right]$$

$$R_{w1} = 1 \text{ m}$$

Z_{w1} = depth of foundation from GL = 1.2 m

Z_{w2} = depth of foundation to water level = 1.2

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

$$R_{w2} = \frac{1}{2} \left[1 + \frac{1.2}{2} \right]$$

$$R_{w2} = 0.8 \text{ m}$$

$$q_f = 1.3 c N_c + \gamma D N_q R_{w1} + 0.4 \gamma B N_r R_{w2}$$

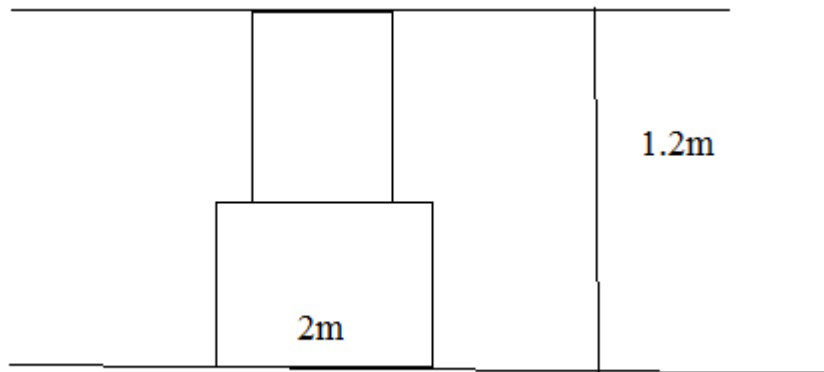
$$q_f = 0 + (19.8 \times 1.2 \times 22 \times 1) + (0.4 \times 19.8 \times 2 \times 20 \times 0.8)$$

$$q_f = 776.16 \text{ kN/m}^2$$

Net ultimate bearing capacity (q_{nf})

$$q_{nf} = q_f - \gamma D = 776.16 - (19.8 \times 1.2) = 752.4 \text{ kN/m}^2$$

ii) Water table at base of the foundation:



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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{1.2}{1.2} \right]$$

$$R_{w1} = 1m$$

Zw1= depth of foundation from GL=1.2 m

Zw2= depth of foundation to water level=0m

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

$$\frac{1}{2} \left[1 + \frac{0}{2} \right]$$

$$R_{w2} = \frac{1}{2} \left[1 + \frac{0}{2} \right]$$

$$R_{w2} = 0.5m$$

$$q_f = 1.3 cN_c + \gamma D N_q R_{w1} + 0.4 \gamma_{sub} B N_r R_{w2}$$

$$q_f = 0 + (19.8 \times 1.2 \times 22 \times 1) + (0.4 \times 10 \times 2 \times 20 \times 0.5)$$

$$q_f = 602.72 \text{ kN/m}^2$$

$$q_{nf} = q_f - \gamma D = 602.72 - (19.8 \times 1.2) = 578.96 \text{ kN/m}^2$$

iii) water table rises the ground level

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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{0}{1.2} \right]$$

$$R_{w1} = 0.5m$$

Zw1= depth of foundation from GL=0 m

Zw2= depth of foundation to water level=0m



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

$$R_{w2} = \frac{1}{2} \left[1 + \frac{0}{2} \right]$$

$$R_{w2} = 0.5$$

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

$$q_f = 1.3 c N_c + \gamma_{sub} D N_q R_{w1} + 0.4 \gamma_{sub} B N_r R_{w2}$$

$$q_f = 0 + (10 \times 1.2 \times 22 \times 1) + (0.4 \times 10 \times 2 \times 20 \times 0.5)$$

$$q_f = 344 \text{ kN/m}^2$$

$$q_{nf} = q_f - \gamma_{sub} D$$

$$= 344 - 10 \times 1.2 = 332 \text{ kN/m}^2$$

4. A footing 2.m square carries a gross pressure of 350kN/m² at a depth of 1.2m in sand. A saturated unit weight of sand is 20 kN/m² and the unit weight of sand above water table is 16 kN/m³. The shear strength parameters are C' =0, $\phi = 30^\circ$ (for $\phi = 30^\circ, N_q=22, N_\gamma=20$). Determine the factor of safety with respect to shear failure for the following cases

i) W.T is 5m below the ground level

ii) W.T is 1.2m below the ground level solution:

We will follow IS code method and Terzaghi

For square footing in soil having $c=0$ $q_f = \bar{\sigma} N_q + 0.4 \gamma B N_\gamma W'$

case i): W.T at 5 m below G.L

$$\bar{\sigma} = 16 \times 1.2$$

$$= 19.2 \text{ kN/m}^2$$

$$D_w = 5 \text{ m}$$

$$D + B = 3 + 1.2 = 4.2 \text{ m}$$

Since $D_w > (D + B), W' = 1$

$$\text{Also } \gamma = 16 \text{ kN/m}^2$$

$$q_f = \bar{\sigma} N_q + 0.4 B \gamma N_\gamma W''$$



$$=19.2 \times 22 + 0.4 \times 16 \times 3 \times 20 \times 1$$

$$=806.4 \text{ kN/m}^2$$

$$q_{nf} = q_f - \gamma D$$

$$=806.4 - 16 \times 1.2$$

$$=787.2 \text{ kN/m}^2$$

Safe bearing capacity,

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$=787.2/F + 16 \times 1.2$$

$$350 = 787.2/F + 19.2F = 2.38$$

Case ii): water table at 1.2m below the G.L

$$D_w = D \Rightarrow W^* = 0.5$$

$$\gamma = \gamma_{\text{sat}} = 20 \text{ kN/m}^3$$

$$\sigma = 16 \times 1.2 = 19.2 \text{ kN/m}^2$$

$$q_f = \bar{N}_q + 0.4 B \gamma N_\gamma W''$$

$$= 19.2 \times 22 + 0.4 \times (20 - 9.81) \times 3 \times 20$$

$$q_f = 666.96 \text{ kN/m}^2$$

$$q_{nf} = q_f - \gamma D$$

$$= 666.96 - 16 \times 1.2$$

$$= 647.76 \text{ kN/m}^2$$

Safe bearing capacity

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$350 = 647.76/F + 19.2$$

$$F = 1.96$$



5. A circular footing is resting on a stiff saturated clay with unconfined compression strength of 250 kN/m^2 . The depth of foundation is 2 m . Determine the diameter of the footing if the column load is 700 kN .

Assume a factor of safety as 2.5 . The bulk unit weight of soil is 20 kN/m^3 . What will be the change in ultimate, net ultimate and safe bearing capacity if the water table is at ground level?

For stiff saturated clay, $\phi = 0$

$N_c = 5.7, N_q = 1$ and $N_\gamma = 0$

$q_u = 250 \text{ kN/m}^2$

$$c = \frac{q_u}{2}$$

$$\therefore c = 250/2 = 125 \text{ kN/m}^2$$

$$q_f = 1.3 c N_r + \gamma D N_r + 0.4 \gamma B N_r$$

$$= 966 \text{ kN/m}^2$$

$$q_{nf} = q_f - \gamma D$$

$$= 966 - 20 \times 2 = 926 \text{ kN/m}^2$$

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$= 926/2.5 + 40$$

$$= 410.4 \text{ kN/m}^2$$

$$W = q_s \times A$$

$$700 = 410.4 \times \frac{\pi \times d^2}{4}$$

$$d = \sqrt{\frac{4 \times 700}{\pi \times 410.4}} = 1.47 \text{ m}$$

$$q_{nf} = 1.3 c N_c + \gamma^* D N_q$$

$$= 1.3 \times 125 \times 5.7 + 10 \times 2 \times 1$$

$$= 946.25 \text{ kN/m}^2$$



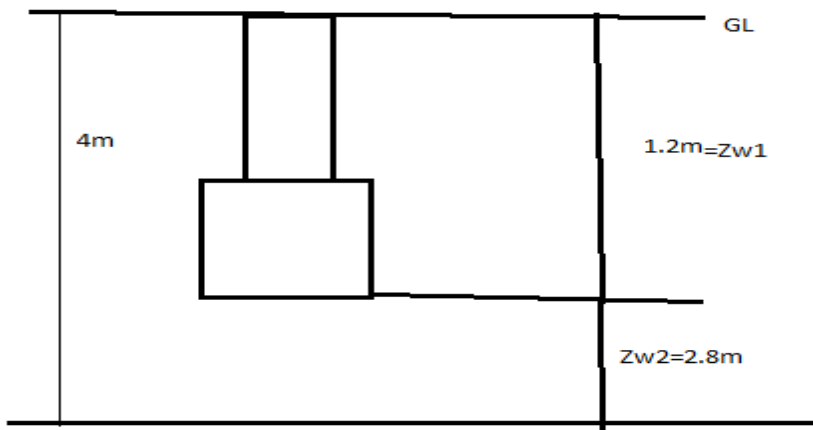
$$q_n = 946.25 - 20$$

$$= 926.25 \text{ KN/m}^2$$

$$q_s = 526.25 / 2.5 = 390.5 \text{ KN/m}^2$$

6. A strip footing 2 m wide carries a load intensity of 400 KN/m^2 at a depth of 1.2 m on sand. A saturated unit weight of sand is 19.5 KN/m^3 and unit weight above water table is 16.8 KN/m^3 . The shear strength parameter $C=0, \phi=36^\circ$. Determine the factor of safety for a following condition.

- 1) WT below 4m from GL
 - 2) WT 1.2 m from GL
 - 3) WT 2.5 m from GL
 - 4) WT 0.5 m from GL
 - 5) WT at GL
- 1) WT below 4m from GL



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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{1.2}{1.2} \right]$$

$$R_{w1} = 1.0$$

Z_{w1} = depth of foundation from GL = 1.2m

Z_{w2} = depth of foundation to water level = 2.8m

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



$$R_{w2} = \frac{1}{2} \left[1 + \frac{2.8}{2} \right]$$

$$R_{w2} = 1.2 \text{ m}$$

$$q_f = \left[\frac{2}{3} c N_c + \gamma D N_q R_{w1} + 0.5 \gamma B N_\gamma R_{w2} \right]$$

$$q_f = [16.8 \times 1.2 \times 40.4 \times 1 + 0.5 \times 16.8 \times 2 \times 33.4 \times 1.2]$$

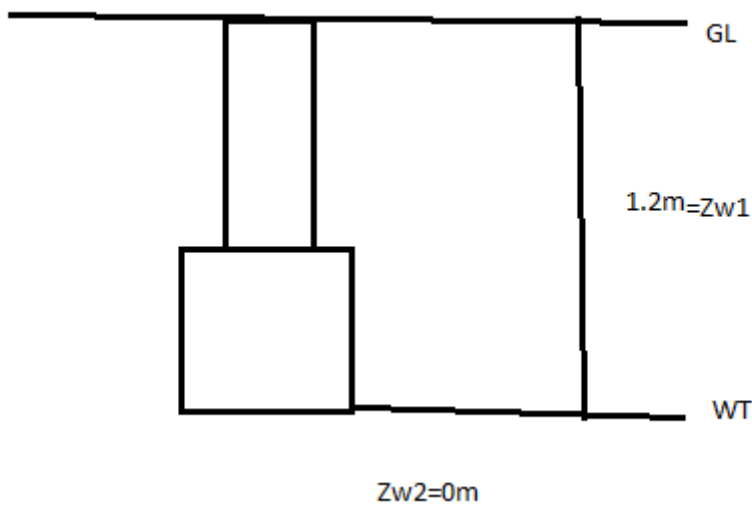
$$= 814.464 + 673.344$$

$$= 1487.8 \text{ KN/m}^2$$

$$F = \frac{q_f}{q_a}$$

$$F = \frac{1487.8}{400} = 3.7$$

2) WT 1.2 m from GL



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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{1.2}{1.2} \right]$$

$$R_{w1} = 1 \text{ m}$$

Zw1 = depth of foundation from GL = 1.2m

Zw2 = depth of foundation to water level = 0m

$$R_{w2} = \frac{1}{2} \left[1 + \right]$$

Z_{w2}
 B]

—





$$R_{w2} = \frac{1}{2} \left[1 + \frac{0}{2} \right]$$

$$R_{w2} = 0.5 \text{ m}$$

$$q_f = \left[\frac{2}{3} cN_c + \gamma DN_q R_{w1} + 0.5 \gamma B N_\gamma R_{w2} \right]$$

$$q_f = \left[\frac{2}{3} cN_c + 16.8 \times 1.2 \times 40.4 \times 1 + 0.5 \times 16.8 \times 2 \times 33.4 \times 0.5 \right]$$

$$= 1095 \text{ KN/m}^2$$

$$F = \frac{q_f}{q_a}$$

$$F = \frac{1095}{400} = 2.7$$

3) WT 2.5 m from GL

$$1 \quad Z_{w1}$$

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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{1.2}{1.2} \right]$$

$$R_{w1} = 1 \text{ m}$$

Zw1 = depth of foundation from GL = 1.2m

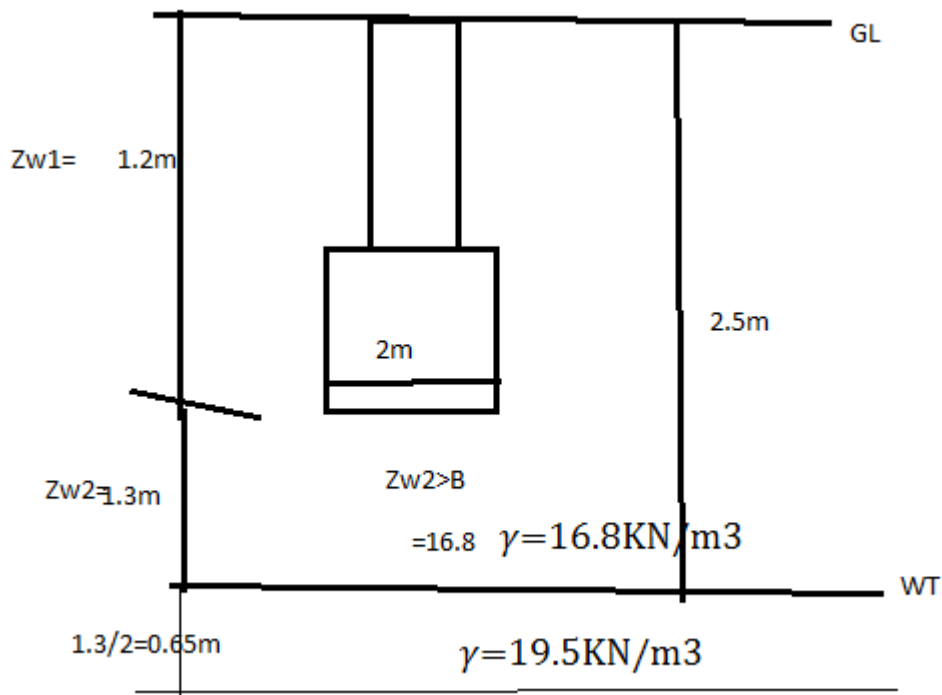
Zw2 = depth of foundation to water level = 1.3m

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

$$R_{w2} = \frac{1}{2} \left[1 + \frac{1.3}{2} \right]$$

$$R_{w2} = 0.82 \text{ m}$$

$$q_f = \left[\frac{2}{3} cN_c + \gamma DN_q R_{w1} + 0.5 \gamma_{avg} B N_\gamma R_{w2} \right]$$



$$\gamma_{avg} = \frac{(1.3 \times 16.8) + (0.65 \times 19.5)}{(1.3 + 0.65)} = 17.7 \text{ kN/m}^3$$

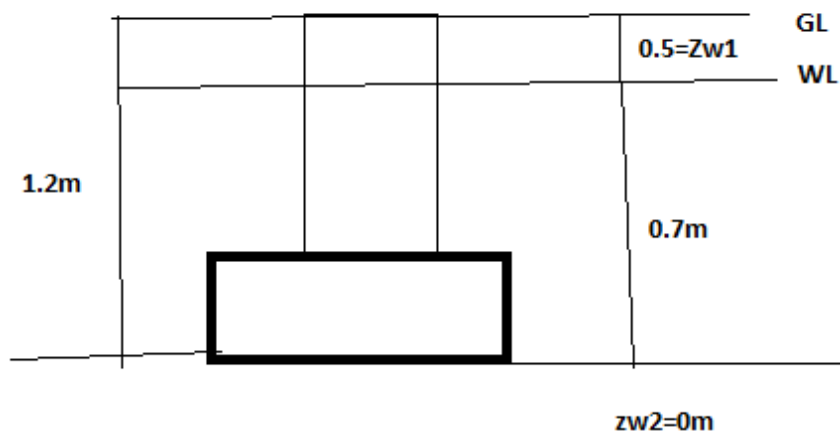
$$q_f = [(18.37 \times 1.2 \times 40.4 \times 1) + (0.5 \times 17.7 \times 2 \times 33.4 \times 0.82)]$$

$$q_f = 1375.3 \text{ kN/m}^2$$

$$F = \frac{q_f}{q_a}$$

$$F = \frac{1375.3}{400} = 3.4$$

4) WT 0.5 m from GL





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

$$R_{w1} = \frac{1}{2} \left[1 + \frac{0.5}{1.2} \right]$$

$$R_{w1} = 0.708m$$

Zw1= depth of foundation from GL= 0.5m

Zw2= depth of foundation to water level=0m

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

$$R_{w2} = \frac{1}{2} \left[1 + \frac{0}{2} \right]$$

$$R_{w2} = 0.5 m$$

$$\gamma_{avg} = \frac{(0.5 \times 16.8) + (0.7 \times 19.5)}{(0.5 + 0.7)} = 18.37 \text{KN/m}^3$$

$$q_f = \left[\frac{2}{3} cN_c + \gamma_{avg} DN_q R_{w1} + 0.5 \gamma B N_\gamma R_{w2} \right]$$

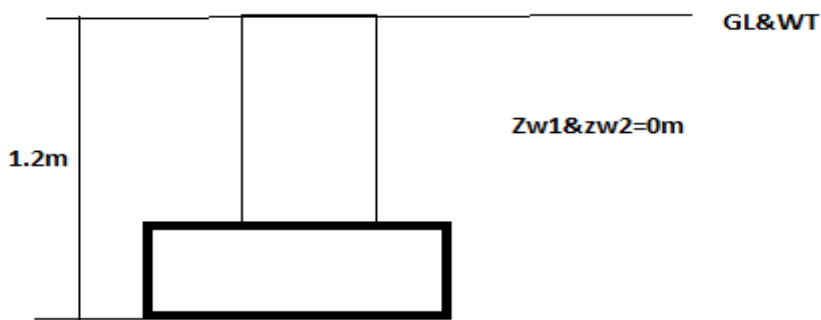
$$q_f = [(18.37 \times 1.2 \times 40.4 \times 0.708) + (0.5 \times 19.5 \times 2 \times 33.4 \times 0.5)]$$

$$q_f = 956.178 \text{KN/m}^2$$

$$F = \frac{q_f}{q_a}$$

$$F = \frac{956.178}{400} = 2.39$$

5) WT At GL



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



Zw1= depth of foundation from GL= 0m

Zw2= depth of foundation to water level=0m

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

$$R_{w2} = \frac{1}{1} \left[1 + \frac{0}{2} \right]$$

$$R_{w2} = 0.5 m$$

$$q_f = \left[\frac{2}{3} c N_c + \gamma D N_q R_{w1} + 0.5 \gamma B N_\gamma R_{w2} \right]$$

$$q_f = [(19.5 \times 1.2 \times 40.4 \times 0.5) + (0.5 \times 19.5 \times 2 \times 33.4 \times 0.5)]$$

$$q_f = 798.33 \text{ KN/m}^2$$

$$F = \frac{q_f}{q_a}$$

$$F = \frac{798.33}{400} = 1.99$$



1. Standard Penetration Test (SPT)

2. Cone Penetration Test (cPT)

3. Plate load Test

1. Standard Penetration Test (SPT)

The standard penetration test is an in-situ test that is coming under the category of penetrometer tests. The standard penetration tests are carried out in borehole. The test will measure the resistance of the soil strata to the penetration undergone. A penetration empirical correlation is derived between the soil properties and the penetration resistance.

The test is extremely useful for determining the relative density and the angle of shearing resistance of cohesionless soils. It can also be used to determine the unconfined compressive strength of cohesive soils.

The requirements to conduct SPT are:

- Standard Split Spoon Sampler
- Drop Hammer weighing 63.5kg
- Guiding rod
- Drilling Rig.
- Driving head (anvil).

Procedure for Standard Penetration Test

The test is conducted in a bore hole by means of a standard split spoon sampler. Once the drilling is done to the desired depth, the drilling tool is removed and the sampler is placed inside the bore hole.

By means of a drop hammer of 63.5kg mass falling through a height of 750mm at the rate of 30 blows per minute, the sampler is driven into the soil. This is as per IS - 2131:1963.

The number of blows of hammer required to drive a depth of 150mm is counted. Further it is driven by 150 mm and the blows are counted.

Similarly, the sampler is once again further driven by 150mm and the number of blows recorded. The number of blows recorded for the first 150mm not taken into

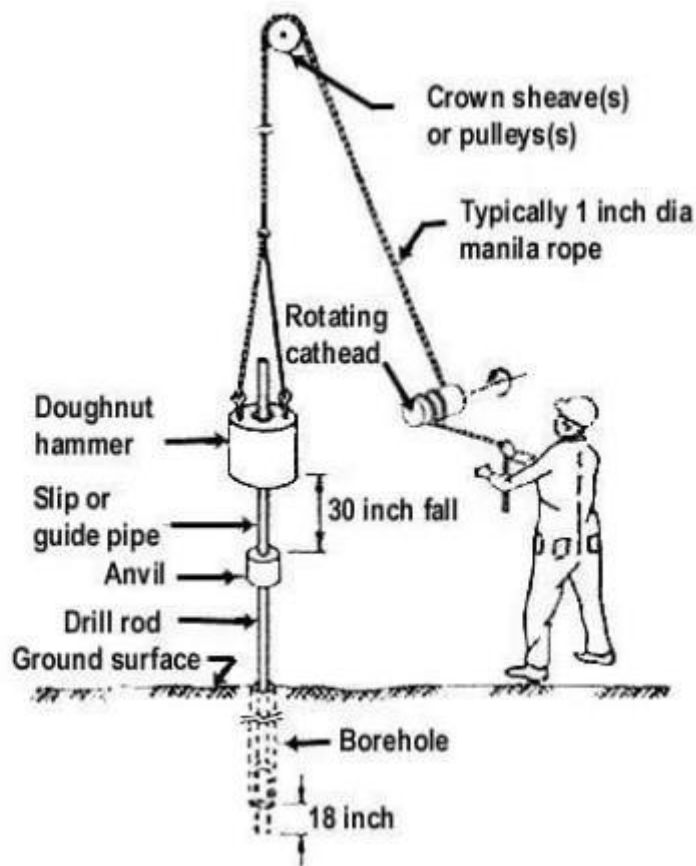


Fig.1: Standard penetration Test

[Fig 1 <https://theconstructor.org/geotechnical/standard-penetration-test-procedure-precautions-advantages/4657/>]

If the number of blows for 150mm drive exceeds 50, it is taken as refusal and the test is discontinued. The standard penetration number is corrected for dilatancy correction and overburden correction.

Corrections in Standard Penetration Test:

Before the SPT values are used in empirical correlations and in design charts, the field 'N' value have to be corrected as per IS 2131 – 1981. The corrections are:

1. Dilatancy Correction
2. Overburden Pressure Correction

1. Dilatancy Correction



Terzaghi and Peck (1967) recommend the following correction in the case of silty fine sands when the observed value is N exceeds 15.

The corrected penetration number,

$$N_c = 15 + 0.5 (N_R - 15)$$

Where N_R is the recorded value and N_C is the corrected value.

If N_R less than or equal to 15, then $N_c = N_R$

2. Overburden Pressure Correction

From several investigations, it is proven that the penetration resistance or the value of N is dependent on the overburden pressure. If there are two granular soils with relative density same, higher 'N' value will be shown by the soil with higher confining pressure.

With the increase in the depth of the soil, the confining pressure also increases. So the value of 'N' at shallow depth and larger depths are underestimated and overestimated respectively.

Hence, to account this the value of 'N' obtained from the test are corrected to a standard effective overburden pressure.

The corrected value of 'N' is

$$N_c = C_N N$$

Here C_N is the correction factor for the overburden pressure.

Precautions taken for Standard Penetration Test

- Split spoon sampler must be in good condition.
- The cutting shoe must be free from wear and tear
- The height of fall must be 750mm. Any change from this will affect the 'N' value.
- The drill rods used must be in standard condition. Bent drill rods are not used.
- Before conducting the test, the bottom of the borehole must be cleaned.

Advantages of Standard Penetration Test

The advantages of standard penetration test are:



Disadvantages of Standard Penetration Test

- The limitations of standard penetration tests are:
- The results will vary due to any mechanical or operator variability or drilling disturbances.
- Test is costly and time consuming.
- The samples retrieved for testing is disturbed.
- The test results from SPT cannot be reproduced
- The application of SPT in gravels, cobbles and cohesive soils are limited

Cone Penetration Test:

Dynamic Cone Penetrometer, or DCP, is a tool used for evaluating the strength of soils on site. It also helps with monitoring the condition of granular layers and subgrade soils in pavement sections over time. It can be used to determine the right solutions for the sites, especially when soft soils are involved.

It is also applied when the CBR value of compacted soil sub-grade beneath the existing road pavement is to be determined. Continuous readings can be taken down to a depth of 800 mm or, when an extension rod is fitted, to a depth of up to 1200 mm.

The DCP is a simple and portable instrument. It consists of a hardened conical tip, standard diameter steel rod, and a standard weight hammer(8kg), which is dropped from the top of the rod against an anvil to advance the tip into the ground.

Apparatus for DCP

The apparatus of the instrument involves the following parts:

- Handle
- Top Rod
- Hammer(8kg)

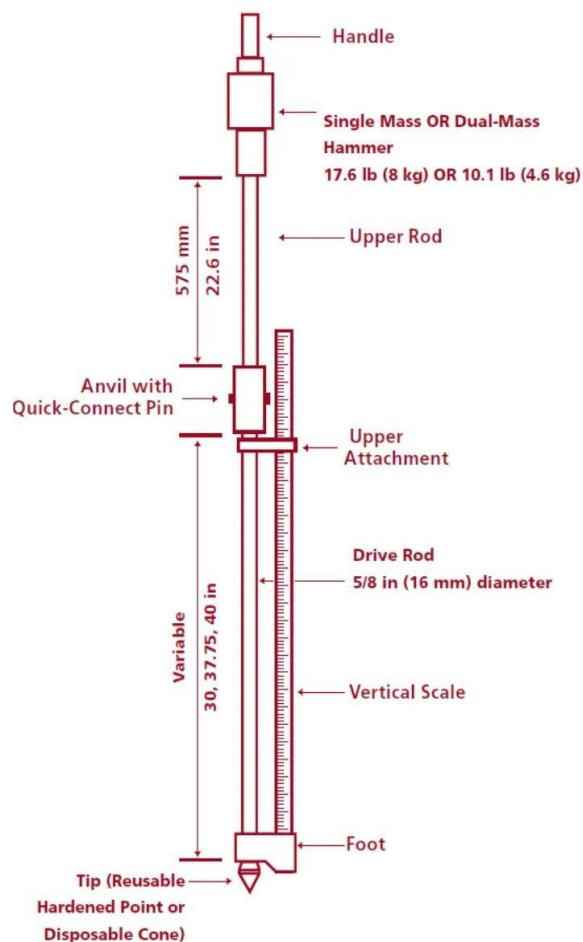


Fig 2 Dynamic Cone Penetrometer

[Fig 2 <https://theconstructor.org/geotechnical/soils/what-dynamic-cone-penetrometer/40239/>]

The following joints should be secured with a strong adhesive or similar non-hardening thread-locking compound prior to use:

- (i) Handle/top rod
- (ii) Anvil/bottom rod
- (iii) Bottom rod/cone

The hammer is lifted to the top of the rod and released in order to drive the rod into the ground. With the help of the embedded vertical scale, the penetration (in inches or millimeters) is recorded after the blows of the hammer.



down the zero reading.

The instrument is held vertical, and the weight is carefully raised to the handle. The weight should not touch the handle before it is allowed to drop, and that the operator should let it fall freely and does not lower it with his hands.

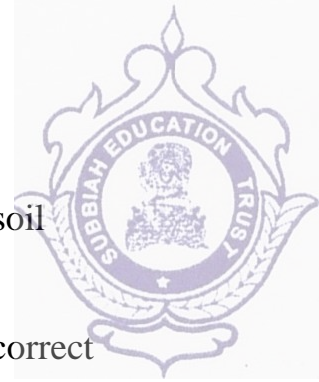
It is advised that a reading should be taken at increments of penetration of about 10mm. However, it is usually easier to take a scale reading after a set number of blows. It is, therefore, necessary to change the number of blows between readings according to the strength of the layer being penetrated. For good quality granular bases, readings after every 5 or 10 blows are normally satisfactory, but for weaker sub-base layers and sub-grades, readings after every 1 or 2 blows may be appropriate.

After the completion of the test, DCP is removed by gently tapping the weight upwards against the handle. It should be done with caution as if done vigorously, the life of the instrument will be reduced.

Benefits of DCP:

- Soil information is often limited, and is often collected from within the extents of the foundation area, but one may also need to assess the soils somewhere else on the site.
- Information regarding the variation of soil strength with depth can be obtained, which can be critical for developing the best solution for unsuitable subgrade soils.
- One can collect information from a lot of points relatively quickly, so you can see how soil conditions vary across the site and respond accordingly.
- One gets accurate and precise information on the soil conditions in the field and at construction time.

Advantage:



1. Continuous resistance with depth is recorded
2. Static resistance is more appropriate to determine static properties of soil

Disadvantage:

1. If a small rock piece is encountered resistance shown is erratic and incorrect
2. 3.involves handling heavy equipment

Plate Load Test:

The allowable bearing pressure can be determined by conducting a plate load test at the site. To conduct a plate load test, a pit of the size $5B_p \times 5B_p$, where B_p is the size of the plate, is excavated to the depth equal to the depth of foundation (D_f). The size of the plate is usually 0.3m square. It is made of steel and is 25mm thick. Occasionally circular plates are also used. Sometimes large size plates of 0.6m square are used.

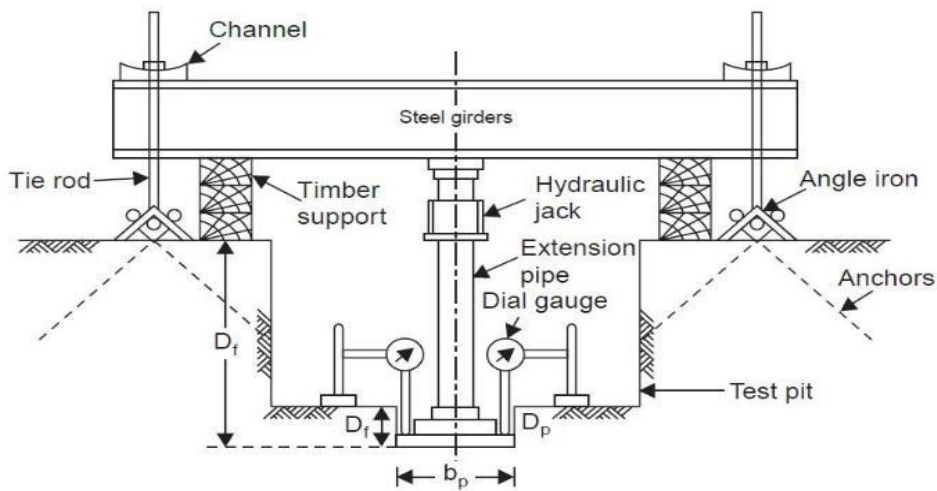


Fig 3 plate load test setup

[Fig3 <https://structville.com/2021/03/how-to-determine-the-bearing-capacity-of-soils-from-plate-load-test.html>]

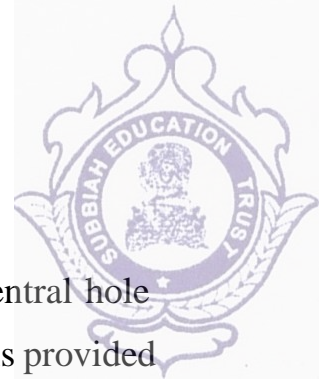
A central hole of size $B_p \times B_p$ is excavated in the pit the depth of the central hole (D_p) is obtained from the following relation

$$\frac{D_p}{B_p} = \frac{D_f}{B_f}$$

$$D_p = (D_f / B_f) B_p$$

$$= (B_p / B_f) D_f$$

Where,



B_f -width of the pit

B_p -size of plate

The conducting the plate load test, the plate is placed in the central hole and the load is applied by means of a hydraulic jack. the reaction to the jack is provided by means of a reaction beam. Sometimes truss is used instead of a reaction beam to take up the reaction. Alternatively, a loaded platform can be used to provide reaction.

Varieties in Plate load test and their durations: -

Plate load test is performed under two variations:

- 1) Gravity load test (Reaction Loading method)
- 2) Reaction truss method

The total duration required to perform a complete test varies from 6-7 days which includes installations, test, dismantling. The results of the test in case of soft strata can be obtained within a few hours whereas in case of hard strata it might take close to a couple of days.

1. Gravity load test

In this type of method, a rigid platform is utilized to transfer loads through loading of sandbags or concrete blocks. These blocks and sandbags act as a dead weight, and whole arrangement rests upon vertical columns. The hydraulic jack is provided in between the rigid plate and top of the column to transfer the load properly.

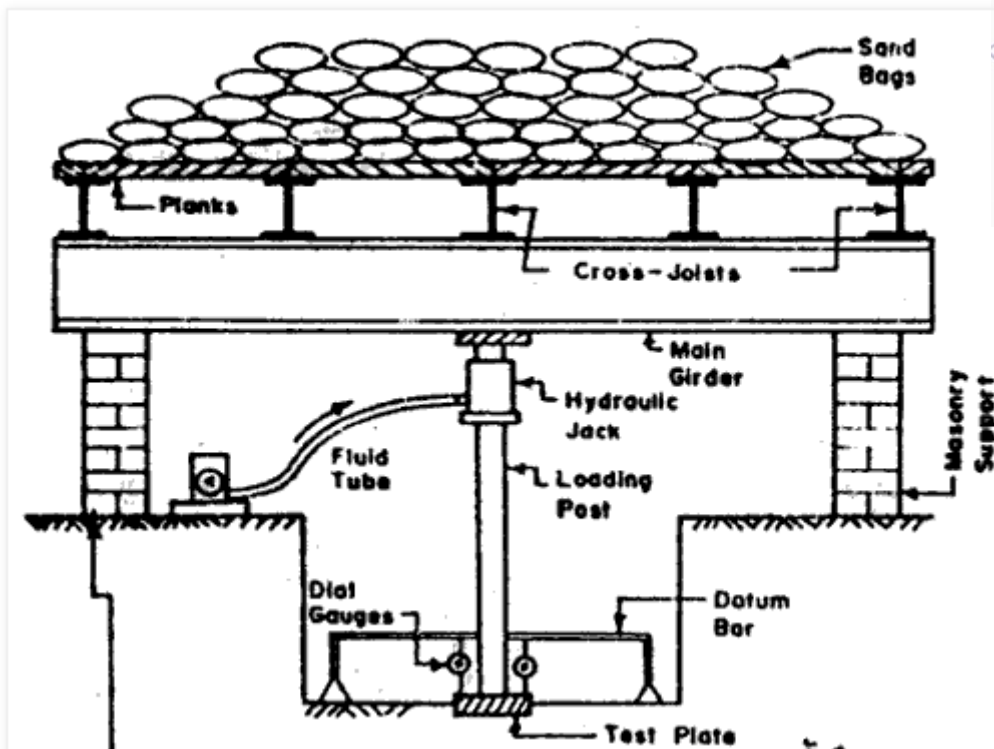


Fig 4 Sand bag method

[Fig4 <http://www.abuildersengineer.com/2012/10/plate-load-test-foundation-site.html>]

2. Reaction truss method

In this method, the reaction generated through jack is borne by reaction truss installed over it. The undesirable movement of truss is controlled by soil anchors or nails fixed into the soil with the help of hammers. The most commonly observed truss is made of mild steel sections. In order to curb later movement, truss is locked with guy ropes.

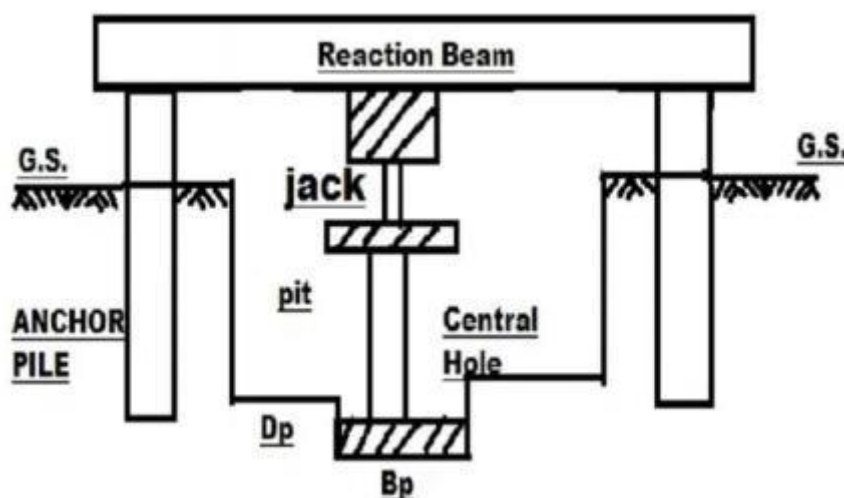
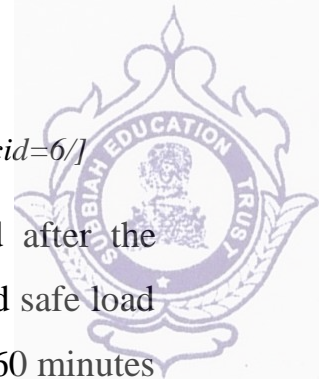


Fig 5 Reaction truss method



[Fig5 <https://www.wv-99.top/products.aspx?cname=plate+bearing+test&cid=6/>]

A seating load of KN/m^2 is first applied, which is released after the sometimes. The is then applied in increments of about 20% of the estimated safe load or $1/10^{\text{th}}$ of the ultimate load. The settlement is recorded after 1,5,10,20,40,60 minutes and further after an internal of one hour. These hourly observations are continued for clayey soils, until the rate of settlement is less than 0.2mm per hour. The test is conducted until failure or at least until the settlement of 25mm has occurred

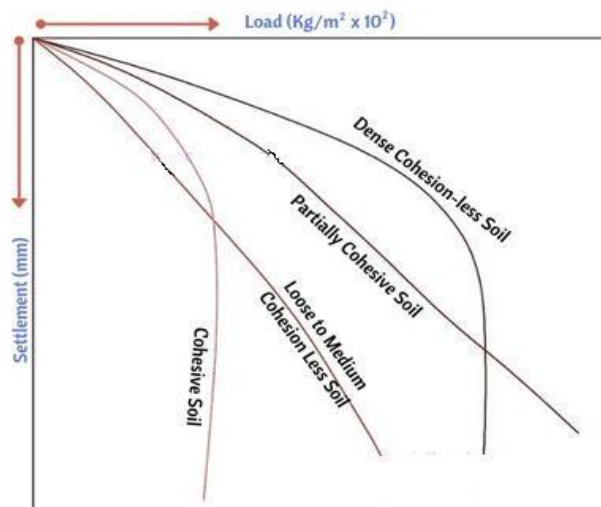


Fig 6 Load settlement Curve

[Fig 6 <https://civilread.com/plate-load-test/>]

The ultimate load for the plate is indicated by a break on the log-log between the load intensity q and the settlements. If the break is not will defined the ultimate load is taken as the corresponding to the settlement of $1/5^{\text{th}}$ of the plate width(B_p) onthe natural plot. The ultimate load is obtained from the intersection of the tangents drawn

Determination of bearing capacity:

1. The ultimate bearing capacity of the proposed foundation $q_u(f)$ can be obtained fromthe following relations

a) For sandy or gravel soil:

$$q_f = q_p \frac{B}{B_p}$$

b) For clay soil



$$q_f \cong q_p$$

c) For $C - \phi$ Soil:

$$Q = q \cdot A + PS$$

where,

B_f – foundation width

B_p – Plate width

q_f = bearing capacity of foundation

q_p = bearing capacity of plate

Q=Total load

A=Area of footing or plate

P=perimeter of footing or plate

Q=bearing Pressure

S=perimeter area

Determination of settlement:

2. The plate load test can also be used to determine the settlement for a given intensity of loading (q_0). The relations between the settlement of the plate (s_p) and that of the foundation (s_f) for the same load intensity

a) For clayey soils, $s_f = s_p(B_f/B_p)$ -----(3)

where s_p is obtained from the load intensity settlement curve for q_0

b) For sandy soils

$$S_f = S_p \left[\frac{B_f(B_p+30)}{B_p(B_f+30)} \right]^2 \text{ -----(4)}$$

Where B_f – width of foundation in meters

B_p – width of the plate in meters

3. For designing a shallow foundation for an allowable settlement of s_f , a trial and error procedure is adopted. First of all, a value of B_f is assumed and value of q_0 is obtained as



$$q_0 = Q/A_f \text{----- (5)}$$

where A_f - area of footing

Q – Load

For the computed value of q_0 the plate settlement (s_p) is determined from the load – settlement curve obtained from the plate load test the values of s_f is computed equation 3 if the soil is clay and using 4 if sand. The computed with the allowable settlement. The procedure is repeated till the computed value is equal to the allowable settlement

The plate load test is can be also be used for the determination of the influence factor I_f ,

$$S_i = \left(\frac{1 - \mu^2}{E} \right) q B I_f$$

The above graph shows a plot between settlements and the load qB , The slope of the line is equal to $\frac{1 - \mu^2}{E}$

LIMITATIONS OF PLATE LOAD TEST:

1. SIZE EFFECT:

The results of the plate load test reflect the strength and the settlement characteristics of the soil within the pressure bulbs. As the pressure bulb depends upon the size of the loaded area it is much deeper for the actual foundation as compared to that of plate. The plate load test does not truly represent the actual conditions to a large depth.

2. SCALE EFFECT:

The ultimate bearing capacity of saturated clays is independent of the size of the plate but for cohesionless soils. It increases with the size of the plate to reduce scale effect, it is desirable to repeat the plate load test with plates of two or three different sizes and the average of the bearing capacity values obtained.

3. TIME EFFECT:

A plate load test is essentially a test of short duration for clayey soils it



does not give the ultimate settlement. The load settlement curve is not truly representative.

4. INTERPRETATION OF FAILURE:

The failure load is not well defined except in the case of a general shear failure an error of personal interpretation may be involved in other type of failures

5. REACTION LOAD:

It is not practicable to provide a reaction of more than 250KN. Hence the test on a plate of size larger than 0.6m width is difficult.

6. WATER TABLE:

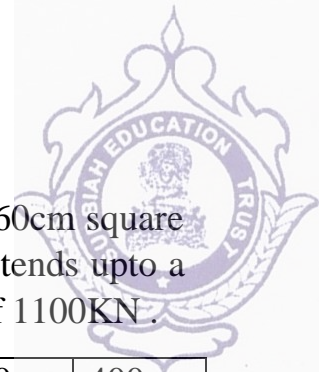
The level of water table affects the bearing capacity of the sandy soils. If the water table is above the level of the footing it has to be lowered by pumping before placing at the water table level if it is within about 1m below the footing.

Advantages of Plate Load Test:

- Bearing able to evaluate the actions of the base under loading conditions.
- Assessing soil capability at a certain depth and predicting settlement over a certain load.
- A shallow foundation could be determined on the basis of the permissible bearing size, which can be estimated in the context of a plate load test.
- Time and cost-effective
- It's easy to execute.

Disadvantages of Plate Load Test:

- Depth of impact is small and can hardly offer soil power.
- It does not have details on the prospects for long-term consolidation of the base soil.
- The scale of the test plate is smaller than the real base, hence why there is a scale impact.
- Significant land disturbance happens after drilling is finished.



Problems:

1. The following data were obtained from a plate load test carried out on a 60cm square test plate at a depth of 2m below ground surface on a **sandy** soil which extends upto a large depth. Determine the settlement of foundation 3x3m carrying a load of 1100KN.

Load intensity(KN/m ²)	50	100	150	200	250	300	350	400
Settlement mm	2.0350	4.0	7.5	11.0	16.3	23.5	34.0	45.0

Given data:

$$B_p = 60\text{cm} = 0.6\text{m}$$

$$D = 2\text{m}$$

sandy soil

$$B = 3\text{m}$$

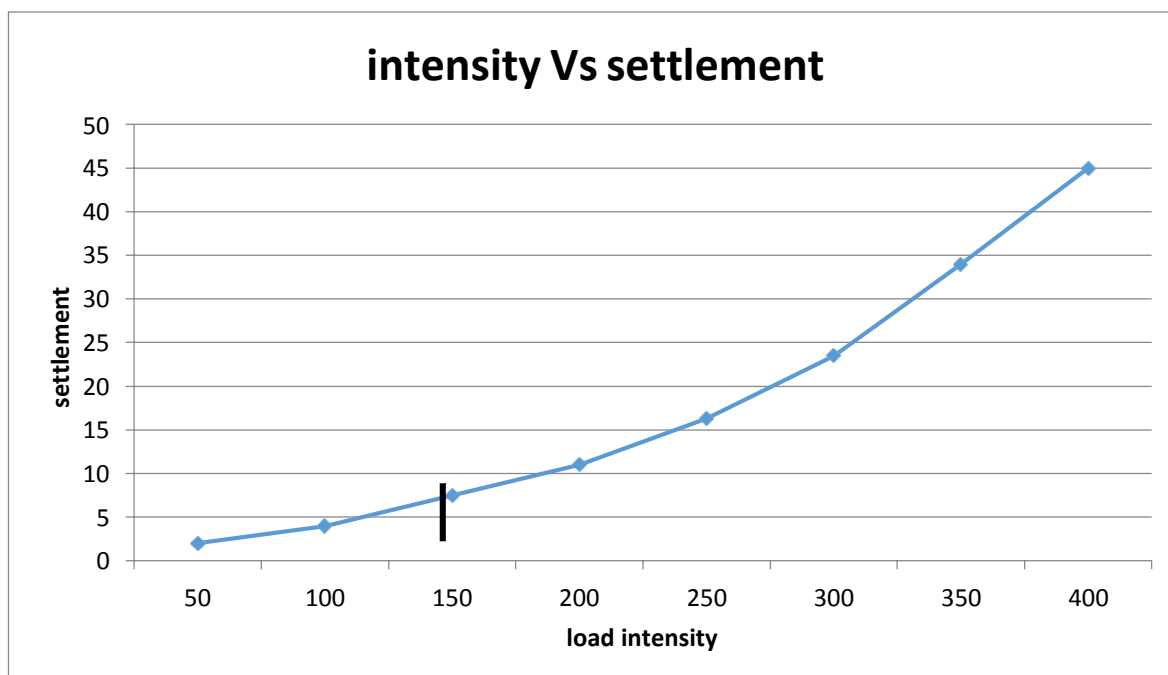
$$\text{load} = 1100\text{KN}$$

To find:

$$\text{Settlement} = ?$$

Solution:

$$\text{intensity} = \frac{\text{Load}}{\text{area}} = \frac{1100}{3^2} = 122.22\text{Kn/m}^2$$



For load intensity 122.22 the settlement is 7mm



$$S_p = 7 \text{ mm}$$

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

$$S_f = 7 \left[\frac{3(0.6 + 0.3)}{0.6(3 + 0.3)} \right]^2$$

$$S_f = 13.01 \text{ mm}$$

2. A plate load test was conducted on a uniform deposit of sand at a depth of 1.5m below the natural ground level and the following data were obtained

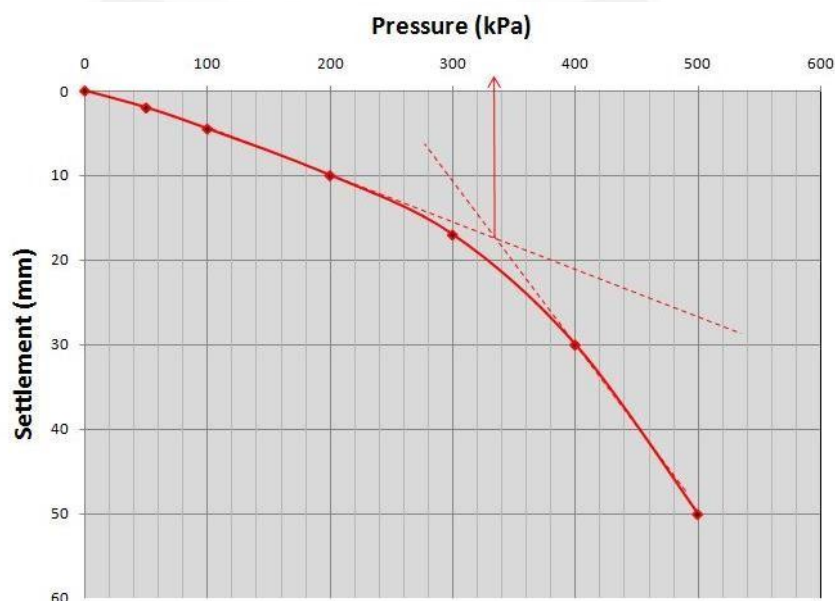
Pressure (Kpa)	0	50	100	200	300	400	500
Settlement (mm)	0	2	4.5	10	17	30	50

The size of plate was 600x600mm and that of pit 3mx3mx1.5m

- i) Plot the pressure settlement curve and determine the failure stress
- ii) A square footing, 1.5x1.5m is to be founded at 1.5m depth in this soil. Assuming the FOS against shear failure as 3 and maximum permissible settlement as 25mm Determine the allowable bearing pressure
- iii) Design of footing for a load of 600KN if the water table is at a great depth.

Solution:

Draw a graph between load and settlement. The failure is obtained by tangent the line. From the graph the failure pressure is $q_p = 335 \text{ KN/m}^2$.



For sandy or gravel soil:



$$q_f = q_p \frac{B}{B_p}$$

$$= 335 \times \frac{1.5}{0.6} = 837.5 \text{ KN/m}^2$$

$$q_a = \frac{q_f}{F} = \frac{837.5}{3} = 279.16 \text{ KN/m}^2$$

From settlement consideration:

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

$$= 25 \left[\frac{1.5(0.6 + 0.3)}{0.6(1.5 + 0.3)} \right]^2 = 16 \text{ mm}$$

From the load settlement curve, the settlement corresponds to a pressure of 290 KN/m²

$$\begin{aligned} \text{The maximum allowable service column load} &= 1.5 \times 1.5 \times 290 \\ &= 652.5 \text{ KN.} \end{aligned}$$

This shows that a column load of 600 KN can be safely supported on footing of 1.5 x 1.5 m on the soil

Net allowable bearing pressure:

For settlement 25 mm

$$q_p = 35(N - 3) \left(\frac{B + 0.3}{2B} \right)^2 \cdot R_{w2} \cdot R_d$$

For settlement 40 mm

$$q_p = 55(N - 3) \left(\frac{B + 0.3}{2B} \right)^2 \cdot R_{w2} \cdot R_d$$

N = standard penetration number

$$R_{w2} = 0.5 \left[1 + \frac{Z_w}{B} \right]$$

$$R_d = \text{depth factor} = \left[1 + 0.2 \frac{D}{B} \right] \leq 1.2$$

1. A strip footing 1.5 m wide is located at a depth of 2 m over a cohesionless soil. The standard penetration test was conducted having corrected N value of 20. If the depth of water table is 3 m below the ground level. Then determine the allowable bearing pressure

for the soil.





Given data:

strip footing

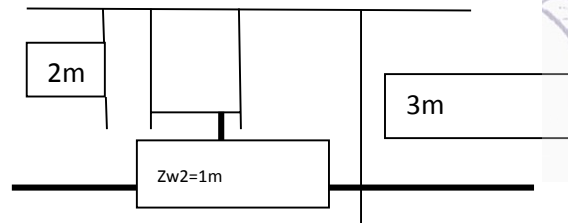
$B=1.5\text{m}$

$D=2\text{m}$

cohesionless soil

standard penetration test

$N=20$



To find:

$q_a=?$

Solution:

$$q_f = 35(N - 3) \left(\frac{B + 0.3}{2B} \right)^2 \cdot R_{w2} \cdot R_d$$

$$R_{w2} = 0.5 \left[1 + \frac{z_{w2}}{B} \right] = 1.3$$

$$R_d = \text{depth factor} = \left[1 + 0.2 \frac{D}{B} \right] \leq 1.2$$

$$R_d = \left[1 + 0.2 \frac{2}{1.5} \right] = 1.26$$

$$\begin{aligned} q_f &= 35(20 - 3) \left(\frac{2 + 0.3}{2 \times 1.5} \right)^2 \cdot 1.3 \times 1.26 \\ &= 573.35 \text{KN/m}^2 \end{aligned}$$



2.6 Settlement:

Settlement is the vertical downward movement to the loaded base. As a result of settlement, the original depth of soil mass decrease due to soil grains coming closer together. Uneven settlement leads to cracks. The amount of settlement is different for different type of soil or rock

Types of foundation settlement

- Differential foundation settlement
- Uniform foundation settlement

Differential foundation settlement

- Settlement that occurs at differing rates between different portions of a building is termed differential settlement.
- Differential settlement occurs if there is difference in soils, loads, or structural systems between parts of a building. in this case, different parts of the building structure could settle by substantially different amounts.
- Consequently, the frame of the building may become distorted, floors may slope, walls and glass may crack, and doors and windows may not work properly.
- Uneven foundation settlement may force buildings to shift out of plumb which lead to crack initiation in foundation, structure, or finish.
- Majority of foundation failures are attributable to severe differential settlement.
- Lastly, for conventional buildings with isolated foundations, 20mm differential settlement is acceptable. And 50mm total settlement is tolerable for the same structures.

Uniform foundation settlement:

- when foundation settlement occurs at neraly the same rate throughout all portions of a building, it is called uniform settlement.
- If all parts of a building rest on the same kind of soil, then uniform settlement the most probable type to take place.
- Similarly, when loads on the building and the design of its structural system are uniform throughout, the anticipated settlement would be uniform type.
- Commonly, uniform settlement has small detrimental influence on the building safety.
- However, it influences utility of the building for example damaging sewer; water supply; and mains and jamming doors and windows.

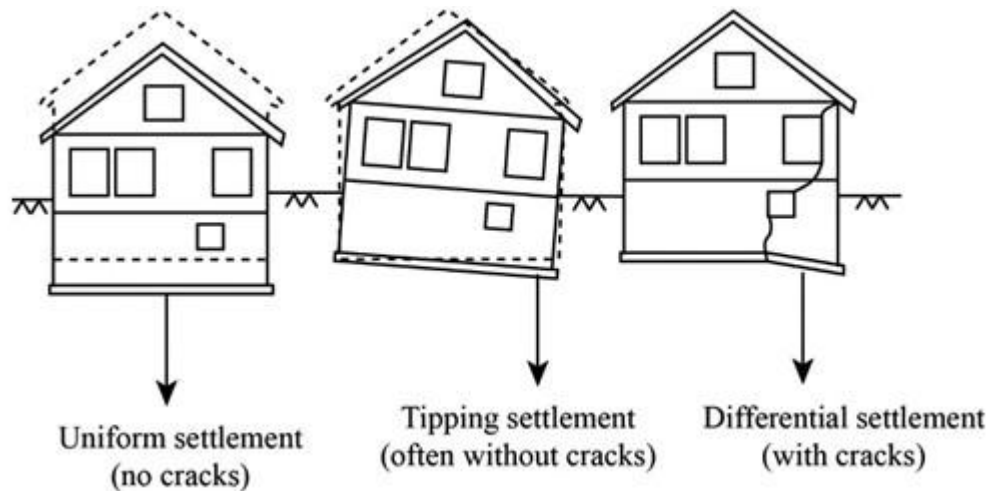


Fig.1: Difference between uniform and differential settlement

[Fig1 <https://www.chegg.com/homework-help/definitions/settlement-of-structure-8>]

Foundation settlement causes

Direct causes

The direct cause of foundation settlement is the weight of building including dead load and live load.

Indirect causes

- Failure of collapsible soil underground infiltration
- Yielding of excavation done adjacent to foundation
- Failure of underground tunnels and mines
- Collapse of cavities of limestones
- Undermining of foundation while flood
- Earthquake induced settlement
- Finally, due to extraction of ground water and oil.

Components of total settlement of foundations

1.Immediate settlement:

- It is also called short term settlement.
- Immediate settlement take place mostly in coarse grained soils of high permeability and in unsaturated fine-grained soils of low permeability.
- Lastly, it occurs over short period of time which about 7 days. So, it ends during construction time.

Cohesive soil:



$$s_i = qB \left[\frac{1 - \mu^2}{E_s} \right] I_f$$

Influence factor I_f :

$I_f=0.82$ for square footing

$I_f=0.88$ for circular footing

$I_f=1.06$ for rectangular footing $\frac{L}{B} = 1.5$

$I_f=1.7$ for square footing $\frac{L}{B} = 5$

Cohesionless Soil:

$$S_i = \frac{H}{C} \log_e \left(\frac{\bar{\sigma}_b + \Delta\sigma}{\bar{\sigma}} \right)$$

C=compressibility

H= depth of stratum

2.Primary settlement

- It also termed as primary consolidation
- Take place over long period of time that ranges from 1 to 5 years or more
- Primary settlement frequently occurs in saturated inorganic fine grain soil.
- Expulsion of water from pores of saturated fine grain soil is the cause of primary settlement.

$$S_c = \frac{HC_c}{1 + e_o} \log_{10} \left(\frac{\bar{\sigma}_b + \Delta\sigma}{\bar{\sigma}} \right)$$

$$S_c = m_v \bar{\Delta}H$$

$$S_c = \frac{\Delta e}{1 + e_o} H$$

i)compression index:

$$C_c = \frac{e_0 - e_1}{\log_{10} \left(\frac{\bar{\sigma}_b + \Delta\sigma}{\bar{\sigma}} \right)}$$

Or

$$C_c = 0.009(w_l - 10)$$

$W_l = \text{Liquid limit}$

ii) *Coefficient of volume change:*





$$m_v = \frac{\Delta e}{\Delta \sigma} \cdot \frac{1}{1 + e_0}$$

$$a_v = \frac{\Delta e}{\Delta \sigma}$$

$$C_v = \frac{K}{m_v \gamma_w}$$

$\bar{\sigma}_0$ = over burden pressure

$\bar{\sigma}$ = final stress or pressure

K = permeability

$$\bar{\sigma} = \bar{\sigma}_0 + \Delta \sigma$$

$$\Delta e = e_0 - e_f$$

e_0 = initial voids

e_f = final voids

3. Secondary settlement

Secondary settlement is the consolidation of soil under constant effective stress.

Frequently, it occurs in organic fine grain soil.

It continues over the life span of foundation structure similar to creep in concrete.

Total Settlement:

$$S = S_i + S_c + S_s$$

S_i = immediate or elastic settlement

S_c = Primary or consolidation settlement

S_s = secondary settlement

Causes of settlement are: -

- Uneven bearing capacity of soil at foundation level.
- Different loads on different parts of foundation.
- Varying ground water table height.
- Compressible foundation soil.
- Earthquakes and floods.
- Expansive soil such as black cotton soil.

Various remedial measures:

- Compaction of soil over the complete area at foundation level.



- Dewatering of foundation if ground water table interference with construction of foundation.
- Stabilization of soil of foundation level if it is compressible.
- Special type of foundation for expansive soils such as black cotton soil.
- Consideration of earthquake loads and other earthquake resisting methods during design and construction of buildings.

Problems:

1. A normal consolidated clay layer is 6m thick with a natural water content of 30% of clay has a saturated unit weight of 17.4 kN/m^3 , specific gravity of 2.67 and liquid limit of 40%. The ground water level is at surface of the clay. Determine the settlement of the foundation. If foundation level will subject to center of a clay layer to a vertical stress increase of 8 kN/m^3 .

Given data:

$$W = 30\%$$

$$H = 6 \text{ m}$$

$$\gamma_{sat} = 17.4 \text{ kN/m}^3$$

$$G = 2.67$$

$$W_l = 40\%$$

$$\text{Increase or additional } \Delta\sigma = 8 \text{ kN/m}^3$$

To find :

Settlement = ?

Solution:

$$S_c = \frac{HC_c}{1 + e_o} \log_{10} \left(\frac{\bar{\sigma}' + \Delta\sigma}{\bar{\sigma}'} \right)$$

$$C_c = 0.009(w_l - 10)$$

$$\bar{\sigma}' = \gamma' Z$$

$$\gamma' \text{ or } \gamma_{sub} = \gamma_{sat} - \gamma_w$$

$$= 17.4 - 9.81 = 7.59 \text{ kN/m}^3$$

$$Z = \frac{H}{2} = \frac{6}{2} = 3 \text{ m}$$

$$\bar{\sigma}' = 7.59 \times 3$$

$$= 22.7 \text{ kN/m}^3$$



$$C_c = 0.009(40 - 10) \\ = 0.27$$

$$e = \frac{0.3 \times 2.67}{1} = 0.801$$

$$S_c = \frac{6 \times 0.27}{1 + 0.801} \log_{10} \left(\frac{22.7 + 8}{22.7} \right)$$

$$S_c = 0.117m = 117mm$$

2. A rectangular footing 2m x 3m carries a column load of 600kN at a depth of 1m. The footing rests on a $C - \phi$ soil strata 6m thick having Poisson's ratio of 0.25 and young's modulus $E = 20000 \text{ kN/m}^2$. Calculate the immediate settlement of footing.

Given Data:

$$B = 2m$$

$$L = 3m$$

$$\text{Load} = 600 \text{ kN}$$

$$D = 1m$$

$$H = 6m$$

$$\mu = 0.25$$

$$E = 20000 \text{ kN/m}^2$$

$C - \phi$ soil (cohesive soil)

To find:

immediate settlement ($S_i = ?$)

Solution:

$$s_i = qB \left[\frac{1 - \mu^2}{E_s} \right] I_f$$

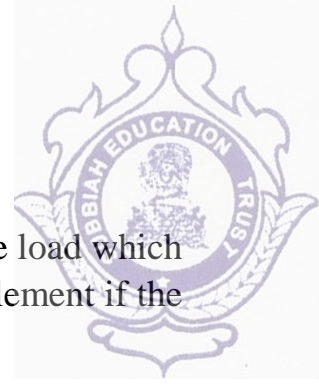
$$q = \frac{\text{load}}{\text{area}} = \frac{\text{Load}}{B \times L}$$

$$L = 3$$

$$\frac{L}{B} = \frac{3}{2} = 1.5m$$

$$I_f = 1.06 \text{ for rectangular footing } \frac{L}{B} = 1.5$$





40KN/m². A soil has a liquid limit of 30% water content 23% Determine the load which the footing carrying safe with FOS=3 against shear. Also determine the settlement if the footing is loaded with safe load use Terzaghi analysis $\gamma = 17.8\text{KN}/\text{m}^2$

Given Data:

$$B=1.2\text{m}$$

$$L=1.5\text{m}$$

$$D=1\text{m}$$

$$H=4\text{m}$$

normally consolidated

$$\text{Strength (q)}=40\text{KN}/\text{m}^2$$

$$W_l=30\%$$

$$W=23\%$$

$$\text{FOS}=3$$

$$\text{Here } \phi = 0, N_c = 5.7, N_q = 1, N_\gamma = 0$$

$$\gamma = 17.8\text{KN}/\text{m}^2$$

To find:

Load=?

Settlement=?

$$q_f = \left[1 - 0.3 \frac{B}{L}\right] c N_c + \gamma D N_q + \left[1 + 0.3 \frac{B}{L}\right] \gamma B N_\gamma$$

$$q_{nf} = q_f - \bar{\sigma}$$

$$q_{nf} = q_f - \gamma D$$

$$q_s = \frac{q_{nf}}{F} + \bar{\sigma}$$

$$q_s = \frac{\text{Load}}{\text{area}}$$

$$H C_c \quad \bar{\sigma}_b + \Delta \bar{\sigma}$$

$$S_c = \frac{H C_c}{1 + e_o} \log_{10} \left(\frac{\bar{\sigma}_b + \Delta \bar{\sigma}}{\bar{\sigma}_b} \right)$$

$$C_c = 0.009(w_l - 10)$$



$$\tau = \gamma Z$$

$$e = \frac{wG}{S_r} [\text{consider it as fully saturated } S_r=1]$$

$$\Delta\sigma = \frac{\text{maximum safe load}}{\text{area}}$$

4. A 30cm square bearing plate settles by 10 mm in the plate load test conducted on sandy soil. The intensity of load applied on the plate causing the settlement is 200KN/m². Estimate the possible settlement of a square shaped shallow foundation of side 2m under the same intensity of loading.

Given data:

$$B_p = 30\text{cm} = 0.3\text{m}$$

$$S_p = 10\text{mm} = 0.01\text{m}$$

$$\text{Intensity}(q) = 200\text{KN/m}^2$$

$$B = 2\text{m}$$

To find:

$$S_f = ?$$

Solution:

For sandy or granular soil:

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

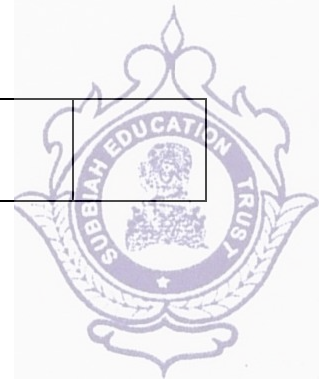
$$S_f = 0.01 \left[\frac{2(0.3 + 0.3)}{0.3(2 + 0.3)} \right]^2$$

$$S_f = 0.030\text{m}$$

5. The following data were obtained from a plate load test carried out on a 60cm square test plate at a depth of 2m below ground surface on a **sandy** soil which extends upto a large depth. Determine the settlement of foundation 3x3m carrying a load of 1100KN .

Load intensity(KN/m ²)	50	100	150	200	250	300	350	400
Settlement mm	2.0350	4.0	7.5	11.0	16.3	23.5	34.0	45.0

--	--	--	--	--	--	--	--	--	--



**Given data:**

$$B_p = 60\text{cm} = 0.6\text{m}$$

$$D = 2\text{m}$$

sandy soil

$$B = 3\text{m}$$

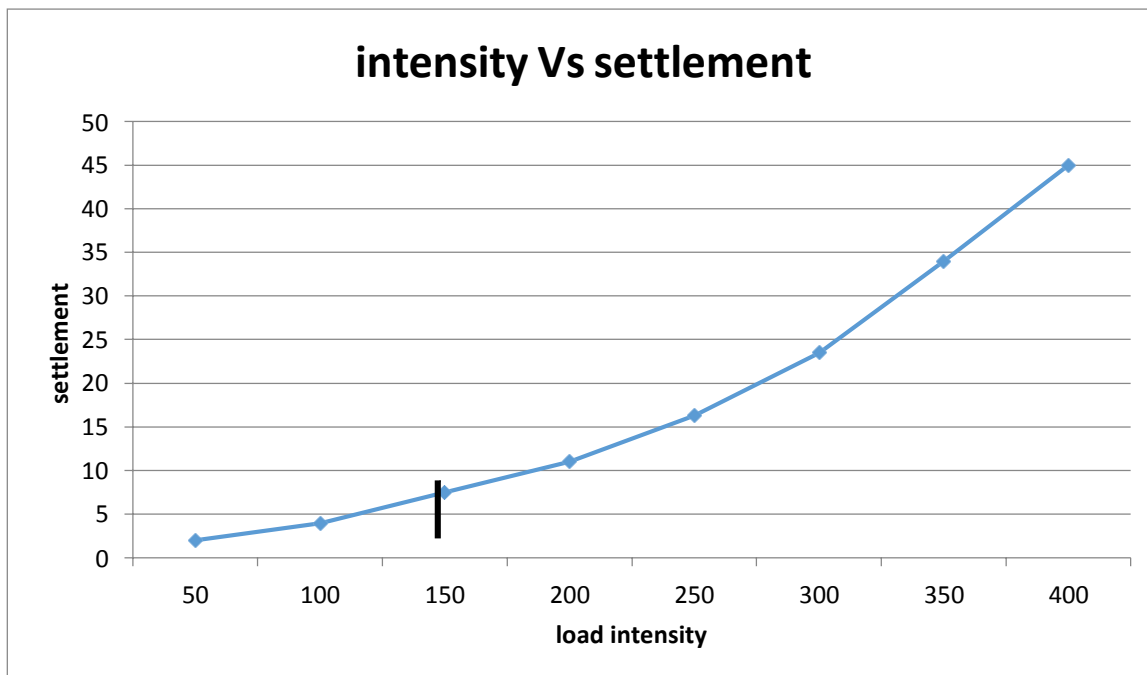
$$\text{load} = 1100\text{KN}$$

To find:

$$\text{Settlement} = ?$$

Solution:

$$\text{intensity} = \frac{\text{Load}}{\text{area}} = \frac{1100}{3^2} = 122.22\text{Kn/m}^2$$



For load intensity 122.22 the settlement is 7mm

$$S_p = 7\text{mm}$$

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

$$S_f = 7 \left[\frac{3(0.6 + 0.3)}{0.6(3 + 0.3)} \right]^2$$

$$S_f = 13.01\text{mm}$$



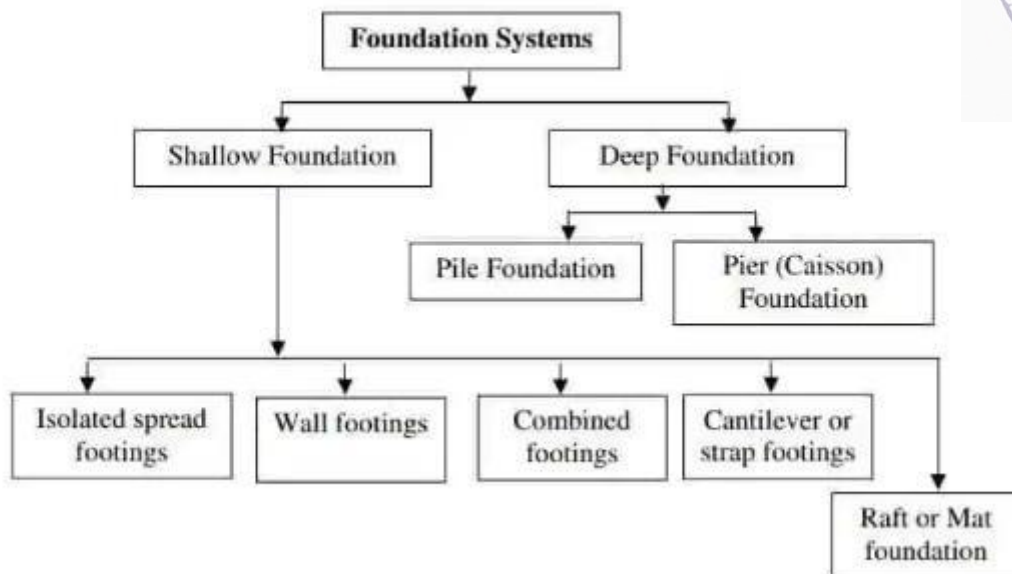


Fig 1 Types of footing

[Fig1 <https://civiconcepts.com/blog/types-of-foundation/>]

1. Types of Isolated Footings

There are various types of isolated footings such as spread footing, stepped footing, sloped footing etc. They are usually square, rectangular or circular in shape. Each type of footing is selected based on the soil condition and configuration of imposed loads. Isolated footings are one of the most economical types of footings and are used when columns are spaced at relatively long distances.

Isolated or single footings are structural elements used to transmit and distribute loads of single columns to the soil without exceeding its bearing capacity, in addition to preventing excessive settlement and providing adequate safety against sliding and overturning. Furthermore, they are used in the case of light column loads, when columns are not closely spaced and in the case of good homogeneous soil.

Use of Isolated Footing: Isolated footings are used as shallow foundation in order to transfer concentrated loads to the ground. To know the basic information, read Isolated footing.

Types of Isolated Footings

a. Flat, Pad, Plain, or Reinforced Isolated Footing

It is constructed under each column independently and is usually square, rectangular, or circular in shape. The thickness of flat isolated footing is uniform. It is provided so as to reduce the bending moments and shearing forces at their critical sections. It can be

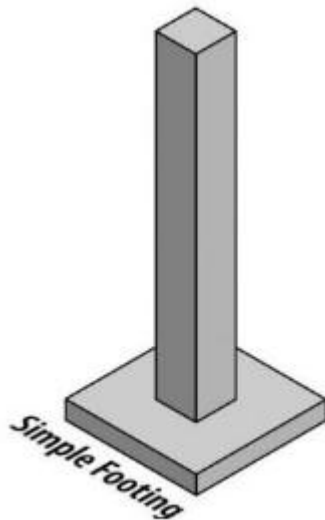


Fig. 2: Flat, Plain, or Reinforced Isolated Footing

[Fig 2 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

b. Sloped Isolated Footing

Sloped or trapezoidal footings are designed and executed with utmost attention to maintain a top slope of 45 degrees from all sides. The amount of reinforcement and concrete used in the sloped footing construction is less than that of plain isolated footing. Therefore, it decreases the utilization of concrete and reinforcement.

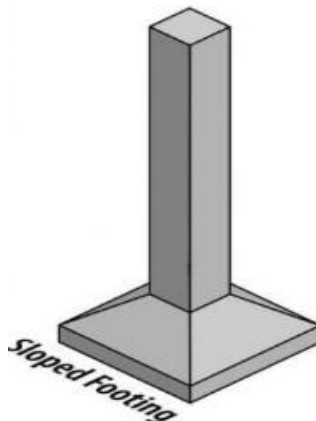


Fig. 3: Sloped Isolated Footing

[Fig 3 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

c. Stepped isolated Footing

Previously, the construction of this type of isolated footing was popular, but its application has declined nowadays. It is generally used in the construction of residential



buildings. Stepped footings are stacked upon one another as steps. By and large, three concrete cross-sections are stacked upon each other to create steps.

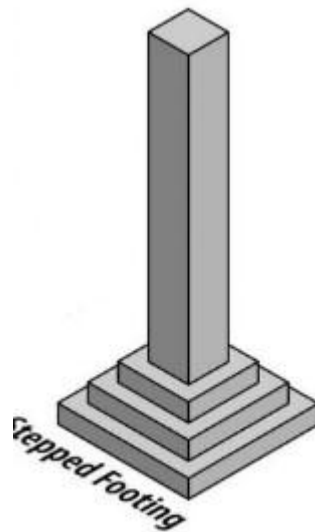


Fig. 4: Stepped Isolated Footing

[Fig 4 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

d. Shoe or eccentric footing

Shoe footing is the half cut-out from the original footing and it has a shape of shoe. They are constructed on property boundary, where there is no provision of setback area. It is constructed at the corner of the plot when the exterior column is close to the boundary or property line and hence there is no scope to project footing much beyond the column face. Column is provided or loaded at the edges of shoe footing. Shoe footings are constructed when the soil bearing capacity is 24KN/m^2

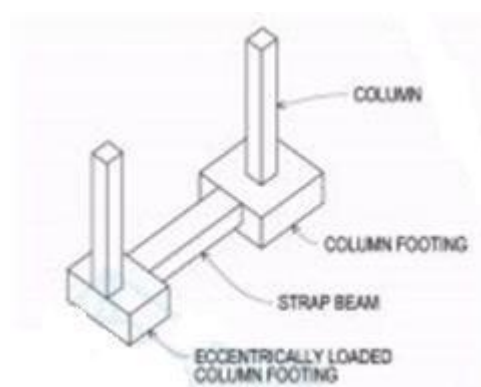
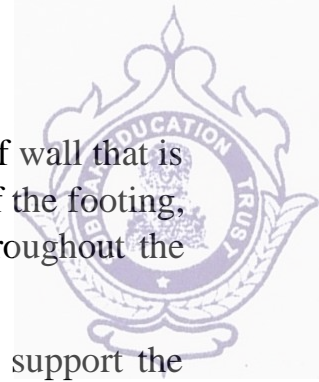


Fig5 Shoe or eccentric footing

[Fig 5 <https://civilread.com/different-types-footings/>]

2. Continuous Wall Footing:

The footing which supports a long masonry or RCC wall is known as a continuous footing. It can be either simple or stepped.



Generally, width of the footing should be at least equal to twice the width of wall that is rested on it. In this case, the width of the footing is smaller than the length of the footing, offering continuous vertical support to the structure. Basically, it runs throughout the length of the wall. This type of footing is not economical.

Use of Continuous Wall Footing: Continuous wall footings are used to support the foundation walls and load-bearing walls.

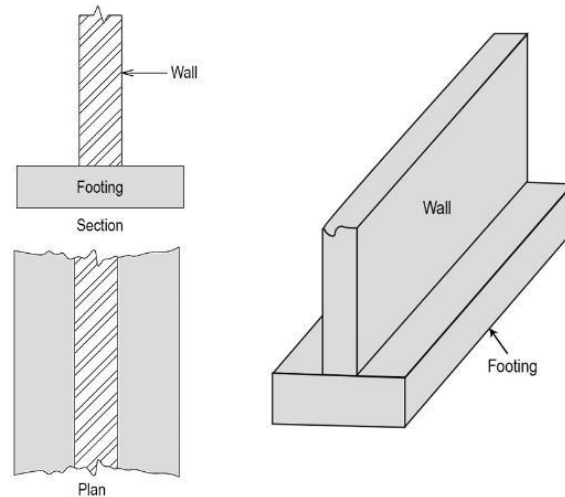


Fig 6 Continuous Wall Footing

[Fig 6 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

3. Combined footing: -

A footing which has more than one column is called as combined footing. This kind of footing is adopted when there is a limited space. Due to lack of space we cannot cast individual footing, therefore footings are combined in one footing. They are classified into two types based on their shape:

Use of Combined Footing: Combined footings are used to transfer loads of closely spaced column to the ground or when the column face the boundary of plot.

Rectangular combined footing: This rectangular footing is provided under two columns where the column is equal load.

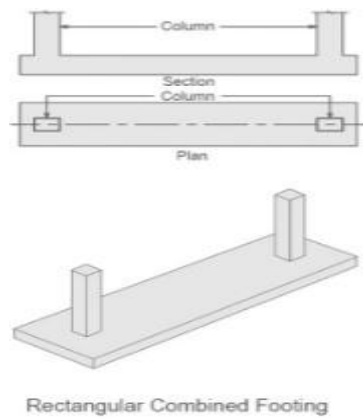


Fig7 Rectangular combined footing

[Fig 7 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

Trapezoidal combined footing: This trapezoidal footing are provided when the two columns are unevenly loaded.

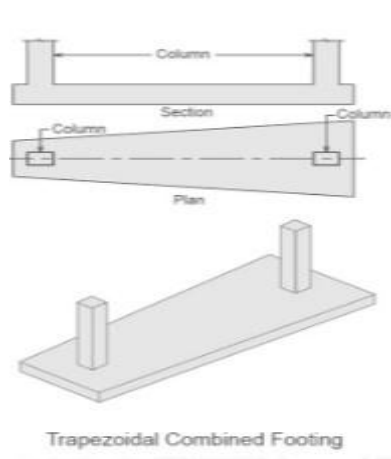


Fig8 Trapezoidal combined footing

[Fig8 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

4.Strap or Beam Combined Footing:

When a distance between the two columns supported on combined footing becomes large, the cost increases rapidly. The strap footing is an economical option in such cases.

Use of Strap Footing: Generally, strap footings are used in conjunction with columns of adjoining property.

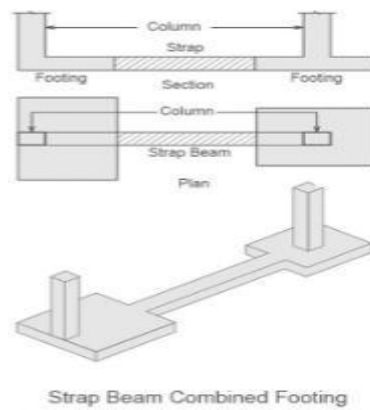


Fig 9 Strap beam combined footing

[Fig9 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

5. Raft footing

If loads transmitted by the columns in a structure are heavy and the allowable soil pressure is small, then footing requires more area. In such a case, it may be better to provide continuous footing under all columns and walls. Such kind of footing is called a Raft Footing.

Use of raft footing: It is widely used when soil has low load bearing capacity. To know more, read the basic information of raft foundation and also know the various types of raft foundation.

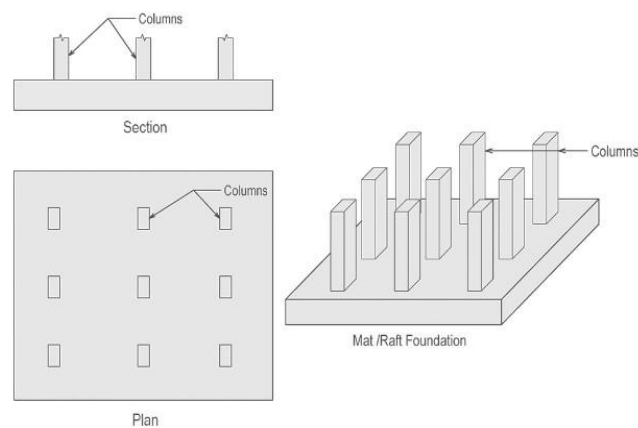


Fig 10 Raft footing

[Fig 10 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

When the soil has a low bearing capacity or the ground water level is high, pile footings are applied. Piles are common while building foundation for bridges, dam etc. in walls.

Use of Pile Footing: Piles are used as deep foundation where the soil is very weak and has higher groundwater table.

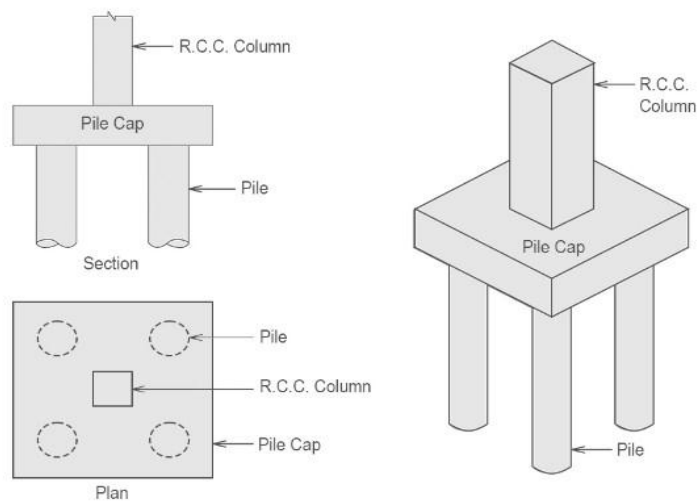


Fig 11 Pile Footing

[Fig 11 <https://gharpedia.com/blog/various-types-of-footings-for-your-house/>]

Drilled Shafts or Caisson Foundation:

Drilled shafts, also called as caissons, is a type of deep foundation and has an action similar to pile foundations discussed above, but are high capacity cast-in- situ foundations. It resists loads from structure through shaft resistance, toe resistance and/or combination of both of these. The construction of drilled shafts or caissons are done using an auger.

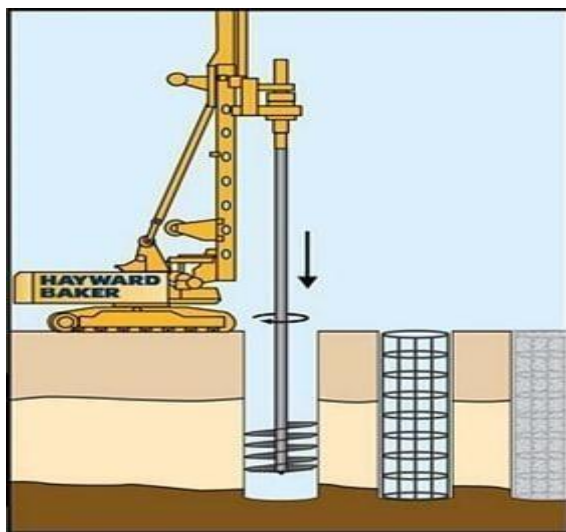


Fig:12 Drilled Shafts or Caisson Foundation (Source: Hayward Baker)

[Fig12<https://theconstructor.org/geotechnical/soil-foundation-contact-pressure-distribution/5647/>]

Drilled shafts can transfer column loads larger than pile foundations. It is used where the depth of hard strata below ground level is located within 10m to 100m (25 feet to 300 feet).



Drilled shafts or caisson foundation is not suitable when deep deposits of soft clays and loose, water-bearing granular soils exist. It is also not suitable for soils where caving formations are difficult to stabilize, soils made up of boulders, artesian aquifer exists.



together to get required stability. So, it is important to know about the contact pressure developed between soil and foundation and its distribution in different conditions which is briefly explained below.

Generally, loads from the structure are transferred to the soil through footing. A reaction to this load, soil exerts an upward pressure on the bottom surface of the footing which is termed as contact pressure.

Contact Pressure Distribution under Footings:

The distribution of contact pressure under different types of footings on different types of soils are explained below.

1. Under Flexible Footing
2. Under Rigid Footing

1. Contact Pressure Distribution under Flexible Footing:

cohesive soil:

- For flexible footing on cohesive soil, settlement is maximum at center of footing and minimum at the edges which forms bowl like shape as shown in below figure. But the contact pressure is distributed uniformly along the settlement line or deflected line.

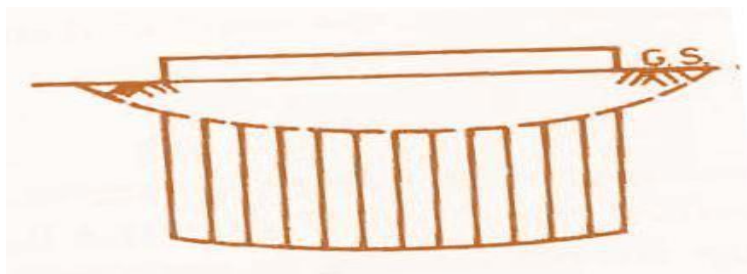


Fig 1: Contact Pressure Distribution - Flexible Footing - Cohesive Soil

[Fig1 <https://theconstructor.org/geotechnical/soil-foundation-contact-pressure-distribution/5647/>]

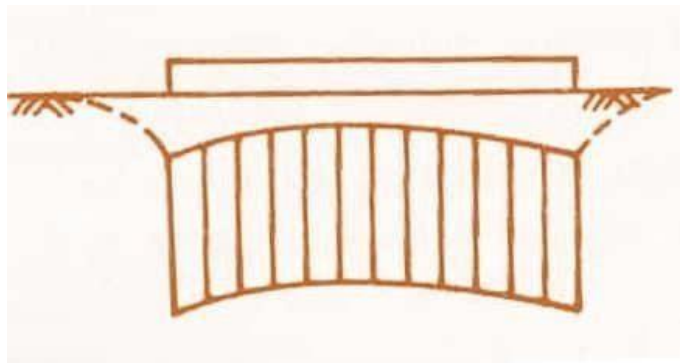


Fig 2: Contact Pressure Distribution - Flexible Footing - Cohesionless Soil

[Fig 2 <https://theconstructor.org/geotechnical/soil-foundation-contact-pressure-distribution/5647/>]

2. Contact Pressure Distribution under Rigid Footing

cohesive soils:

- For rigid footings resting on cohesive soils, settlement is uniform but contact pressure varies. At edges contact pressure is maximum and at center it is minimum which forms inverted bowl shape as shown in below figure.
- The values of stresses at edges becomes finite when plastic flow occurs in real soils.

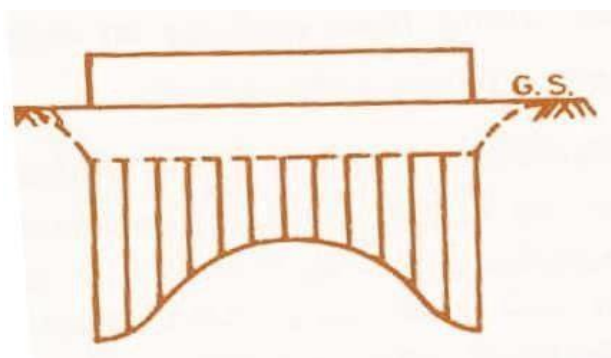


Fig 3: Contact Pressure Distribution - Rigid Footing - Cohesive Soil

[Fig 3 <https://theconstructor.org/geotechnical/soil-foundation-contact-pressure-distribution/5647/>]

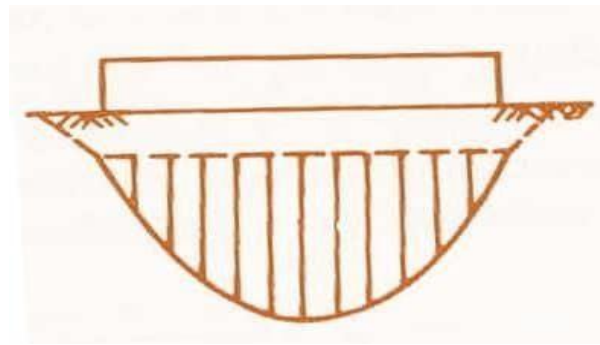


Fig 4: C.P Distribution - Rigid Footing - Cohesionless Soil

[Fig4 <https://theconstructor.org/geotechnical/soil-foundation-contact-pressure-distribution/5647/>]

Factors Effecting Contact Pressure Distribution:

Following are the factors effecting contact pressure distribution

- Stiffness of Footing
- Compressibility of soil
- Type of loading

1. Stiffness of Footing

- Contact pressure is uniform in case of flexible footings such as earth embankments. Contact pressure varies in case of rigid foundations such as R.C.C pad foundations etc. If the footing is partly flexible and partly rigid like raft foundation, contact pressure slightly varies.



Fig 5: Flexible and Rigid Footings of a Structure

[Fig 5 <https://theconstructor.org/geotechnical/soil-foundation-contact-pressure-distribution/5647/>]



- If concentrated loading is applied at the center of foundation resting on cohesive soil, contact pressure is not uniform irrespective of stiffness of foundation.
- For flexible foundation, contact pressure is maximum exactly under the load application.
- For rigid foundations, contact pressure is maximum at edges. So, application of point load on rigid foundations can be comparable to the application of uniform loading on rigid foundation resting on cohesive soil.

b) Uniform Loading

- Contact pressure distribution under uniform loading and deformed patterns of flexible and rigid foundations are already explained above with figures 1, 2, 3 and 4.

General Assumption of Contact Pressure Distribution

- In the design of foundations, Contact pressure is assumed to be uniform which is not a problem for flexible foundations since they have uniform contact pressure irrespective of stiffness of soil.
- But when it comes to rigid foundation, this assumption may lead to unsafe design since contact pressure is not uniform in this case. This happens when the soil acts as elastic material.
- However, the soil under footing acts as elasto-plastic material just before failure occurs. Hence, this assumption can be justified at the ultimate stage.



3.3 Design of footings:

Design Procedure for rectangular footing:

Step1) To find column load

$$Q = Q_1 + Q_2$$

Q_1 = load in exterior column

Q_2 = load in interior column

Step2) Find the area of footing

$$A = \frac{Q}{q_{na}} = \frac{Q}{q_s}$$

q_{na} = Allowable bearing pressure

Step3) Locate the line of action of the column loads measured from the centre of the exterior column

$$\bar{x}_1 = \frac{Q_2 x_2}{Q_1 + Q_2}$$

X_2 = centre to centre distance between the column

Step4) Define the total length of the footing

$$L = 2(\bar{x} + e_1)$$

e_1 = projection of footing

Step 5) Find the width of the footing

$$B = \frac{A}{L}$$

Step6) Find the actual area provided (A_o)

Step 7) Find the actual pressure

$$q_o = \frac{Q}{A_o}$$

$$Q \quad 6e$$

$$q_{max} = \frac{Q}{A_o} \left(1 + \frac{6e}{L}\right)$$

$$q_{min} = \frac{Q}{A_o} \left(1 - \frac{6e}{L}\right)$$

Step 8) Draw the shear force and BM diagram

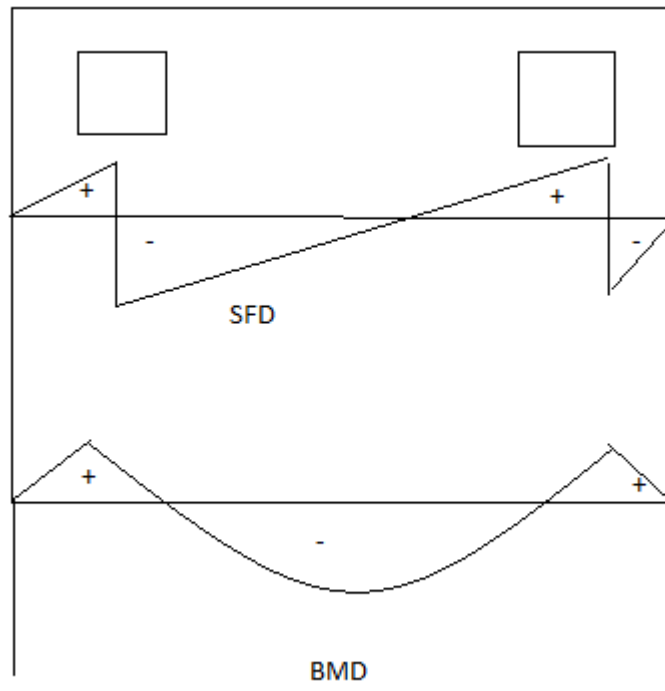




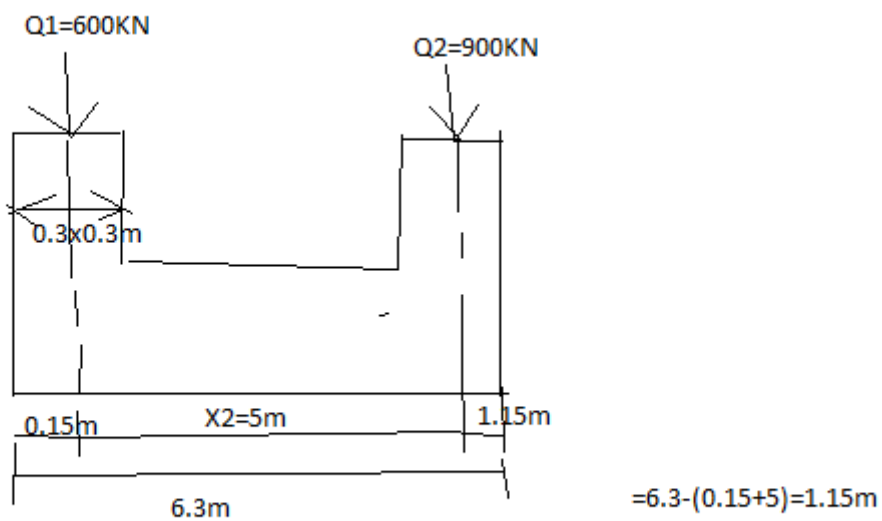
Step9) Determine the BM at the face of the column of maximum BM at the point of zero stress.

Step 10) Find the thickness of footing

Step11) Determine the required reinforcement for the maximum bending moment



1. Design a rectangular combined footing two column shown in figure. Take allowable soil pressure as 100KN/m^2 .



Step1) To find column load



$$Q = Q_1 + Q_2 = 600 + 900 = 1500 \text{ KN}$$

Step 2) Find the area of footing

$$A = \frac{Q}{q_{na}} = \frac{Q}{q_s} = \frac{1500}{100} = 15 \text{ m}^2$$

$q_{na} = \text{Allowable bearing pressure}$

Step 3) Locate the line of action of the column loads measured from the centre of the exterior column

$$\bar{x}_1 = \frac{Q_2 x_2}{Q_1 + Q_2} = \frac{900 \times 5}{600 + 900} = 3 \text{ m}$$

$X_2 = \text{centre to centre distance between the column}$

Step 4) Define the total length of the footing

$$L = 2(\bar{x}_1 + e_1)$$

$$e_1 = \frac{B}{2} = \frac{0.3}{2} = 0.15 \text{ m}$$

$$L = 2(3 + 0.15) = 6.3 \text{ m}$$

$e_1 = \text{projection of footing}$

Step 5) Find the width of the footing

$$B = \frac{A}{L} = \frac{15}{6.3} = 2.38 \text{ m} = 2.4 \text{ m}$$

Step 6) Find the actual area provided (A_o)

$$A_o = B \times L = 2.4 \times 6.3 = 15.12 \text{ m}^2$$

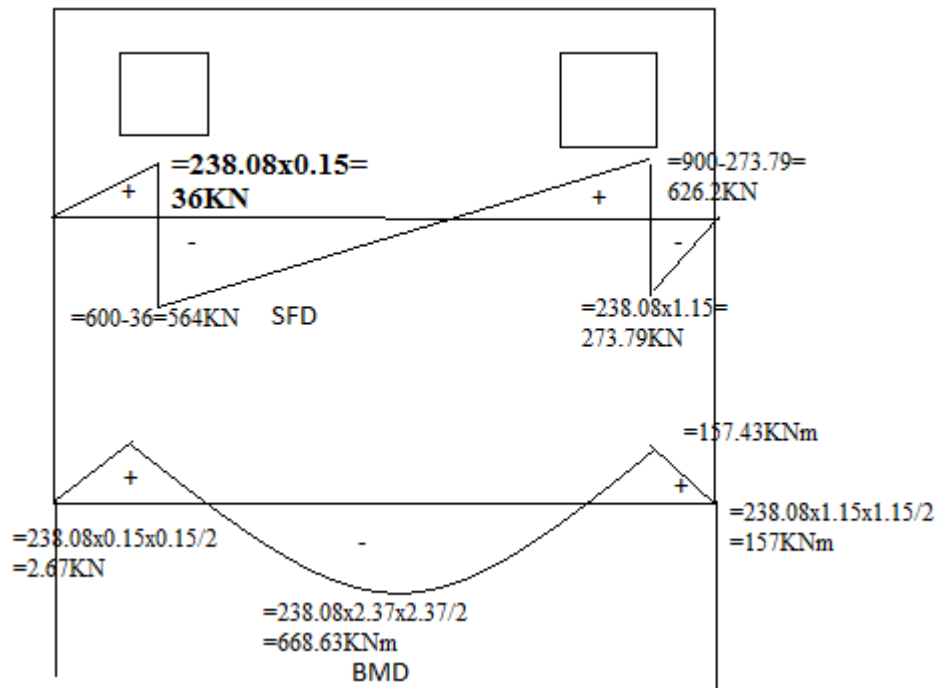
Step 7) Find the actual pressure

$$q_o = \frac{Q}{A_o}$$

$$q_o = \frac{1500}{15.12} = 99.2 \text{ KN/m}^2$$

Step 8) Actual pressure per meter

$$q_o = 99.2 \times 2.4 = 238.08 \text{ KN/m}$$



Design Procedure for trapezoidal footing:

Step1) To find column load

$$Q = Q_1 + Q_2$$

Q_1 = load in exterior column

Q_2 = load in interior column

Step2) Find the area of footing

$$A = \frac{Q}{q_{na}} = \frac{Q}{q_s}$$

q_{na} = Allowable bearing pressure

Step3) Locate the line of action of the column loads measured from the centre of the exterior column

$$\bar{x}_1 = \frac{Q_2 x_2}{Q_1 + Q_2}$$

X_2 = centre to centre distance between the column

Step4) Find x'

$$x' = \bar{x} + \frac{b_1}{2}$$

Step 5) Find $\frac{L}{b}$





$$\frac{L}{3} < x' < \frac{L}{2}$$

Step 6) Find the width of the footing

$$B_2 = \frac{2A}{L} \left(\frac{3x'}{L} - 1 \right)$$

$$B_1 = \frac{2A}{L} - B_2$$

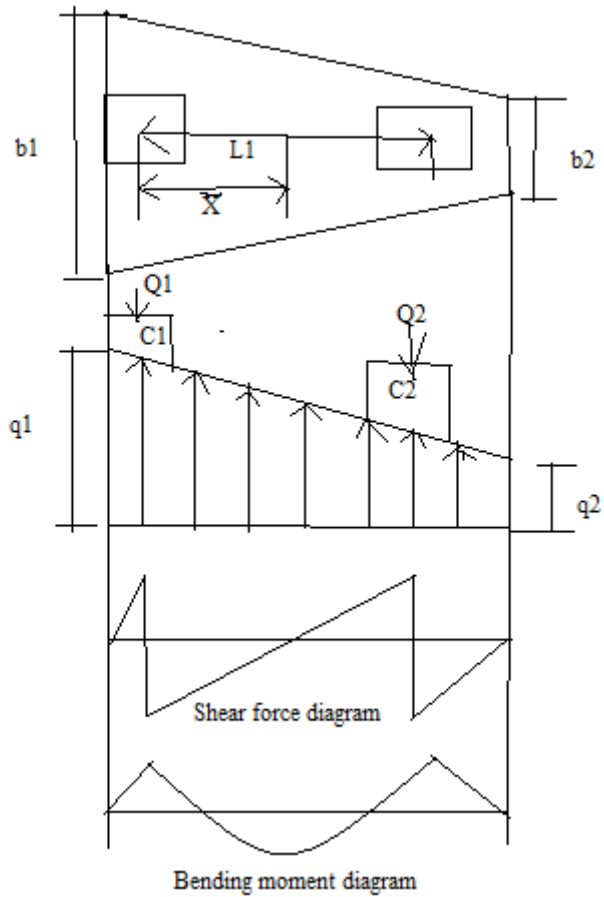
Step 7) Find the actual pressure

$$q_o = \frac{Q}{A_o}$$

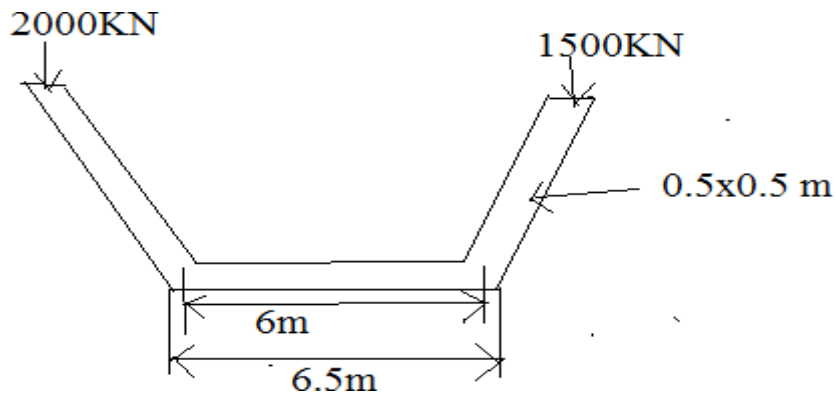
$$q_{max} = \frac{Q}{A_o} \left(1 + \frac{6e}{L} \right)$$

$$q_{min} = \frac{Q}{A_o} \left(1 - \frac{6e}{L} \right)$$

Step 8) Draw shear force and bending moment Diagram



3. Design a trapezoidal footing for the two column shown in figure .Take allowable soil pressure is 200KN/m^2 .



Step1) To find column load

$$Q = Q_1 + Q_2 = 2000 + 1500 = 3500 \text{ kN}$$

Q_1 = load in exterior column

Q_2 = load in interior column

Step2) Find the area of footing

$$A = \frac{Q}{q_{na}} = \frac{Q}{q_s} = 17.5 \text{ m}^2$$

q_{na} = Allowable bearing pressure

Step3) Locate the line of action of the column loads measured from the centre of the exterior column

$$\bar{x}_1 = \frac{Q_2 x_2}{Q_1 + Q_2}$$

$$\bar{x}_1 = \frac{3000 \times 6}{3500} = 2.57 \text{ m}$$

X_2 = centre to centre distance between the column

Step4) Find x'

$$x' = \bar{x} + \frac{b_1}{2}$$

$$x' = 2.57 + \frac{0.5}{2} = 2.82 \text{ m}$$

Step 5) Find $\frac{L}{4}$, $\frac{L}{2}$

2 3

$$\frac{L}{4} < x' < \frac{L}{2}$$

3

2

2.167<2.82<3.25





Step 6) Find the width of the footing

$$B_2 = \frac{2A}{L} \left(\frac{3x'}{L} - 1 \right)$$

$$B_2 = \frac{2 \times 17.5}{6.5} \left(\frac{3 \times 2.82}{6.5} - 1 \right) = 1.62m$$

$$B_1 = \frac{2A}{L} - B_2$$

$$B_1 = \frac{2 \times 17.5}{6.5} - 1.62 = 3.76$$

Step 7) Find the actual pressure

$$q_o = \frac{Q}{A_o}$$

$$q_o = \frac{3500}{17.42} = 200.17 \text{KN/m}^2$$

Step 7) Find the actual pressure per meter

$$q_o = 200.17 \times B_1$$

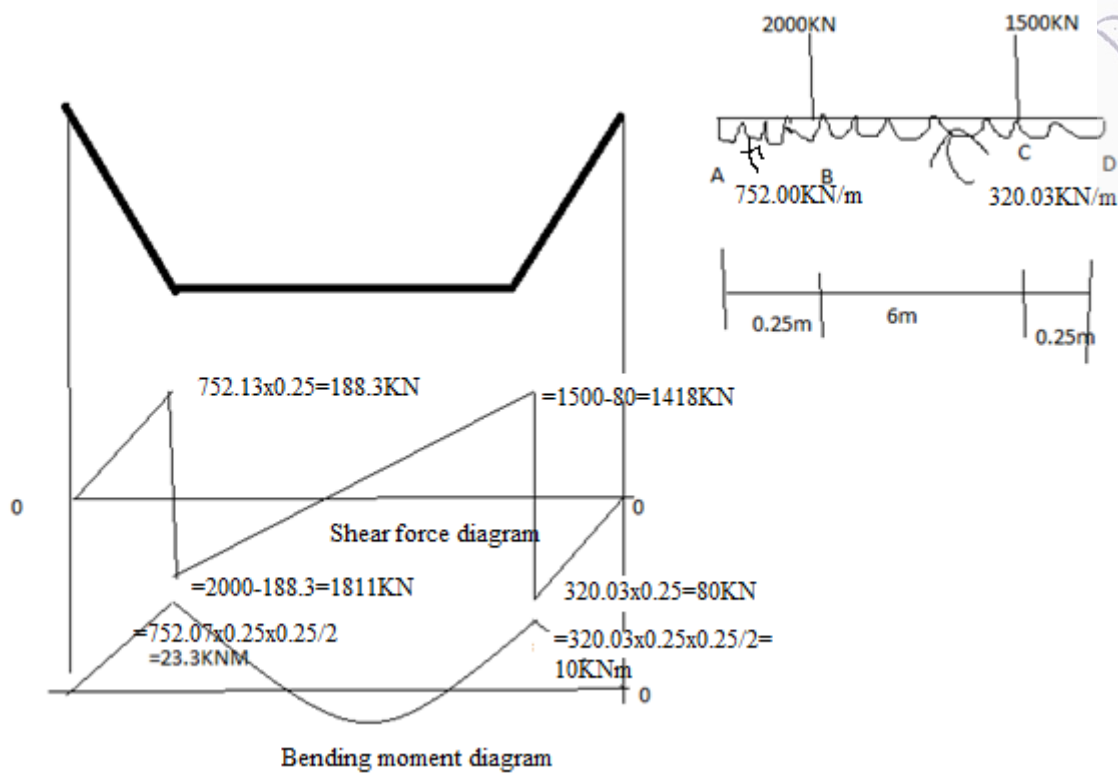
$$q_o = 200.17 \times 3.76$$

$$= 752.63 \text{KN/m}$$

$$q_o = 200.17 \times B_2$$

$$q_o = 200.17 \times 1.62$$

$$= 320.03 \text{KN/m}$$



Strap footing:

Design Procedure for strap footing:

Step 1: calculate length

$$L_1 = 2(e + 0.5b_1)$$

Step 2) calculate R_1

$$R_1 = \frac{Q_1 x_2}{S}$$

Step 3) calculate R_2

$$R_2 = (Q_1 + Q_2) - R_1$$

Step 4) calculate A_1

$$A_1 = \frac{R_1}{q_{na}}$$

$$A_2 = \frac{R_2}{q_{na}}$$

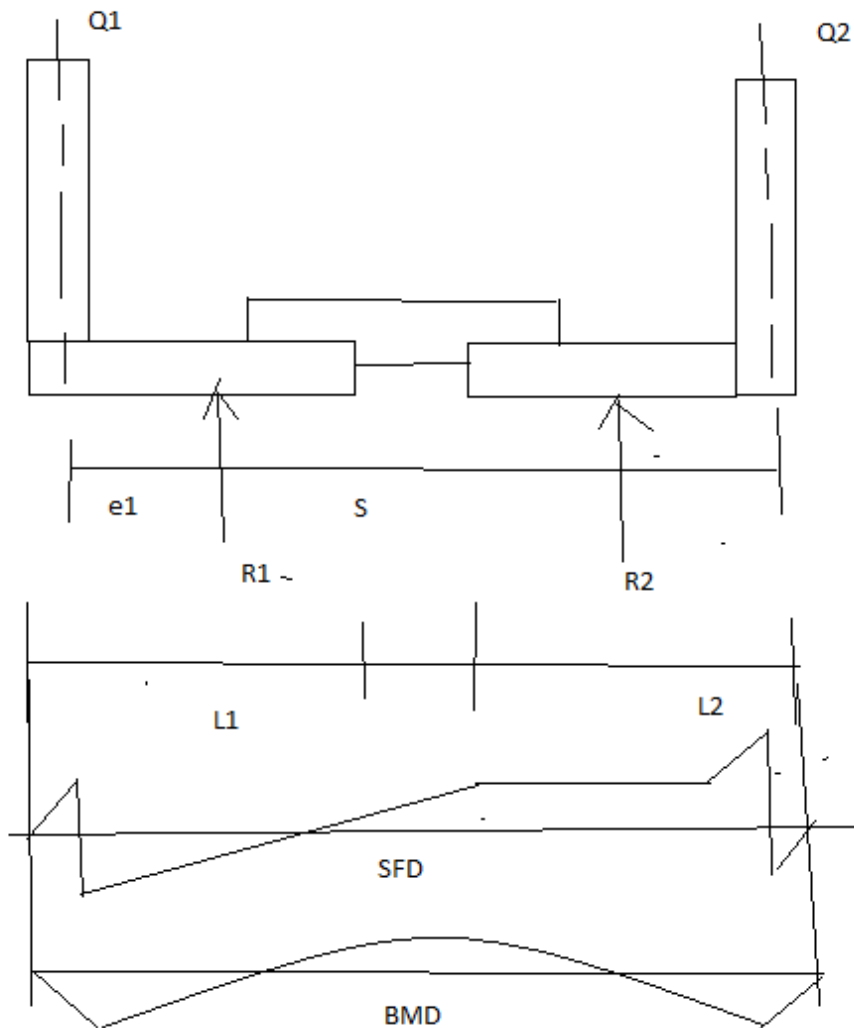
Step 5) calculate Pressure intensity

$$B_1 = \frac{A_1}{L_1}$$

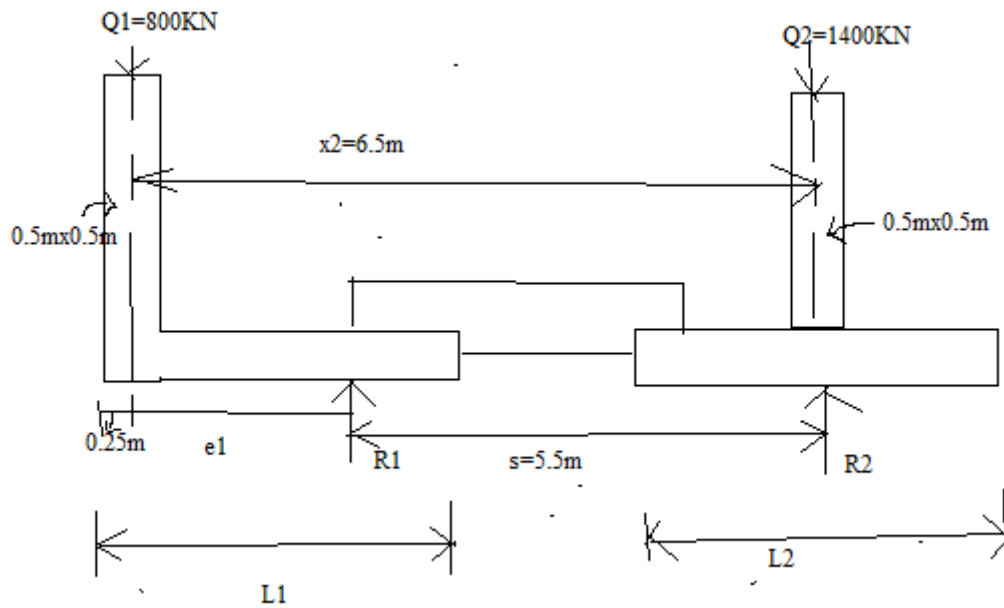


$$q_1 = \frac{R_1}{L_1 x B_1} = \frac{R_1}{A_1}$$

$$q_2 = \frac{R_2}{L_2 x B_2} = \frac{R_2}{A_2}$$



3. Design a strap footing for two columns with Centre to center distance 6.5m and distance between the reactions is 5.5 m. The allowable soil pressure is 120KN/m². take eccentricity of footing of column is 1. The size of the column is 0.5mx0.5m



Step 1: calculate length

$$L_1 = 2(e + 0.5b_1)$$

$$L_1 = 2(1 + 0.5 \times 0.5) = 2.5m$$

Step 2) calculate R_1

$$R_1 = \frac{Q_1 x_2}{s}$$

$$R_1 = \frac{800 \times 0.5}{5.5} = 945.45KN$$

Step 3) calculate R_2

$$R_2 = (Q_1 + Q_2) - R_1$$

$$R_2 = (800 + 1400) - 945.45 = 1254.55KN$$

Step 4) calculate A_1

$$A_1 = \frac{R_1}{q_{na}} = \frac{945.45}{120} = 7.878m^2$$

$$A_2 = \frac{R_2}{q_{na}} = \frac{1254.5}{120} = 10.45m^2$$

Step 5) calculate Pressure intensity

$$B_1 = \frac{A_1}{L_1} = \frac{7.878}{2.5} = 315m$$

$$q = \frac{R_1}{L_1 \times B_1} = \frac{R_1}{A_1} = \frac{945.45}{7.878} = 120 \text{ KN/m}^2$$





$$q = \frac{R_2}{L_2 \times B_2} = \frac{R_2}{A_2} = \frac{1254.55}{10.45} = 120 \text{KN/m}^2$$

$$B_2 = \sqrt{A_2} = \sqrt{10.45} = 3.23$$

$$B_2 = \frac{A_2}{L_2}$$

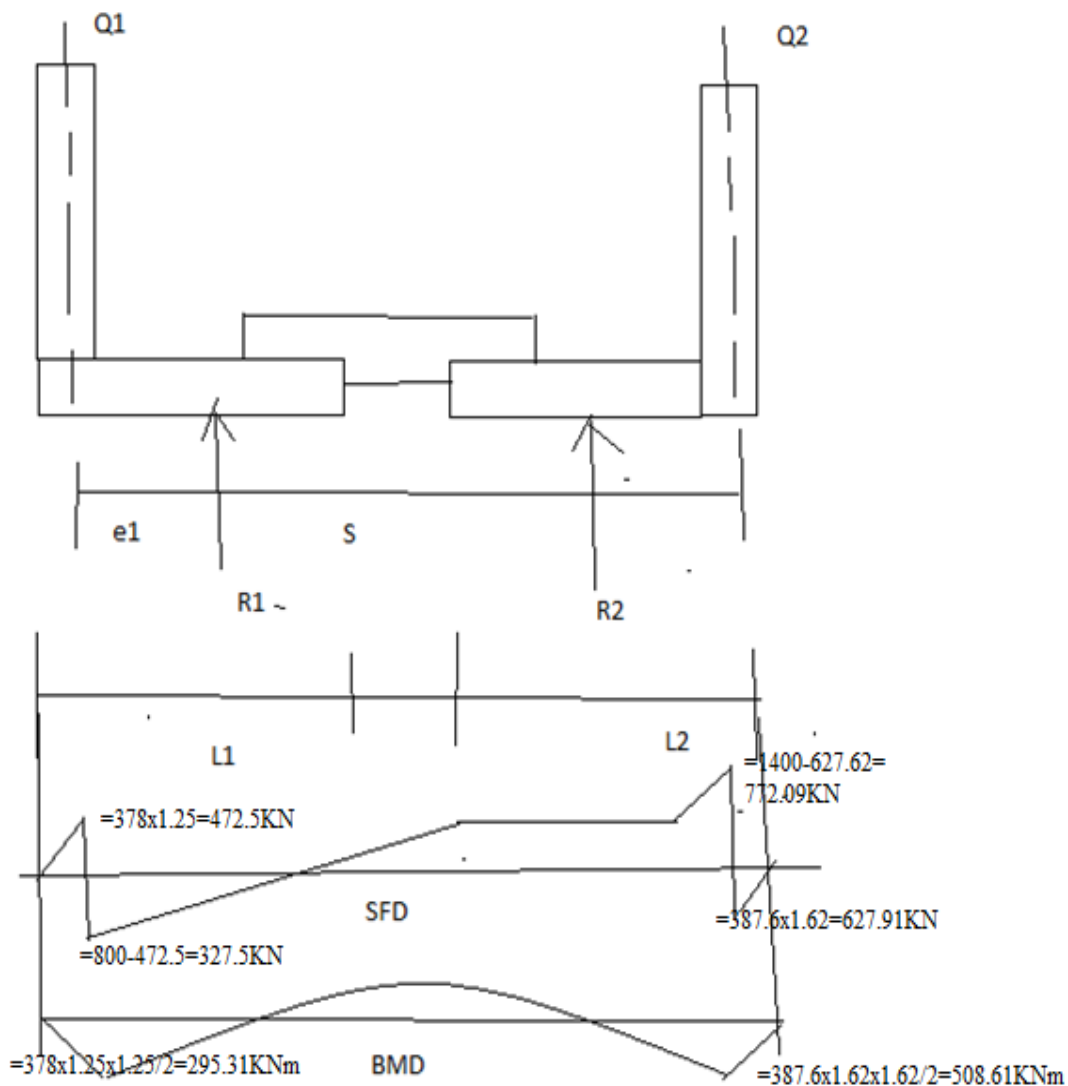
$$3.23 = \frac{10.45}{L_2}$$

$$L_2 = 3.23 \text{m}$$

Pressure intensity per meter,

$$q_1 = 120 \times 3.15 = 378 \text{KN/m}$$

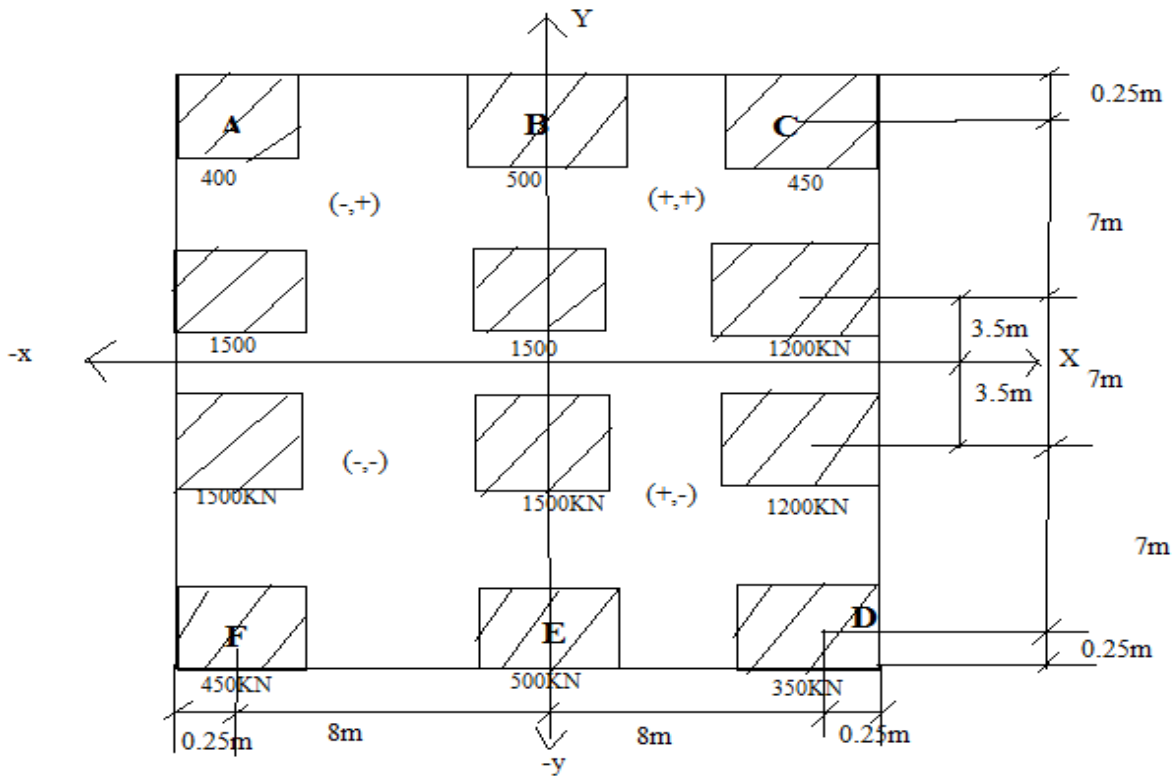
$$q_2 = 120 \times 3.2 = 387.6 \text{KN/m}$$





Mat Foundation:

4. A plan of raft foundation with column load as shown in figure .Calculate the soil pressure at point A,C,D,F The size of mat is16.5x21.5m all column are 0.5x0.5m in the section .The allowable soil pressure is 60KN/m².Determine the soil pressure at the point.



Solution:

Step1) Area of mat

$$A = bxd = 16.5 \times 21.5 = 354.75m^2$$

Step2) Calculate the moment of inertia

$$I_{xx} = \frac{bd^3}{12} = \frac{16.5 \times 21.5^3}{12} = 13665.26m^4$$

$$I_{yy} = \frac{db^3}{12} = \frac{21.5 \times 16.5^3}{12} = 8048.3m^4$$

Step 3) Calculate Moment:

$$M_y = Qxe_x$$

$$e_x = x' - \frac{B}{2}$$



$$x' = \frac{Q_1x_1 + Q_2x_2 + \dots + Q_nx_n}{Q}$$

$$= \frac{1}{11000} [(400 + 1500 + 1500 + 400)x0.25]$$

$$+ [(500 + 1500 + 1500 + 500)x8.25]$$

$$+ [(450 + 1200 + 1200 + 30)x16.25]$$

$$x' = 7.81m$$

$$e_x = 7.81 - \frac{16.5}{2} = -0.44m$$

$$M_y = 11000x(-0.44) = -4840KNm$$

$$M_x = Qxe_y$$

$$e_y = y' - \frac{d}{2}$$

$$y' = \frac{Q_1y_1 + Q_2y_2 + \dots + Q_ny_n}{Q}$$

$$= \frac{1}{11000} [(400 + 500 + 450)x0.25] + [(1500 + 1500 + 1200)x7.25]$$

$$+ [(1500 + 1500 + 1200)x14.25] + [(400 + 500 + 350)x21.25]$$

$$y' = 10.65m$$

$$e_y = 10.65 - \frac{21.5}{2} = 0.1m$$

$$M_x = 11000x0.1 = 1100KNm$$

Step 4) Calculate soil Pressure:

$$q = \frac{Q}{A} \pm \frac{M_x}{I_y} \pm \frac{M_y}{I_x}$$

$$q = \frac{11000}{354.75} \pm \frac{4840x}{8048.39} \pm \frac{1100y}{13665.27}$$

$$q = 31 \pm 0.6x \pm 0.08y$$

Soil pressure at A = $31 - 0.6x8.25 + 0.08x10.75 = 26.91KN/m^2$

Soil pressure at C = $31 + 0.6x8.25 + 0.08x10.75 = 31.81KN/m^2$

Soil pressure at D = $31 + 0.6x8.25 - 0.08x10.75 = 35.09KN/m^2$

Soil pressure at E=31 – 0.6x8.25 – 0.08x10.75 = 25.19KN/m²





3.4 Proportion of footings:

- A structure is usually supported on a number of column .These column usually carry a different load depending on their location with respect to structure.
- Differential settlements are minimized by proportioning the footing for the various columns so as to equalize the average bearing pressure under all columns.
- But each column load consists of dead load (DL)+Live load(LL).The full LL does not act all the time(wind load)
- Hence DL+ full LL is not a realistic criterion for producing equal settlement.
- For ordinary building the actual load expected on the building is D.L+50% L.L.

Procedure:

- i. DL inclusive self weight of column and estimated value for footing is noted for each column footing.
- ii. LL for each column is calculated(IS code)
- iii. The ratio of LL to DL is calculated for each column footing and the maximum value of ratio is noted.
- iv. The allowable bearing pressure is calculated by Terzaghi equation.
- v. For the footing with largest LL to DL ratio the area of footing required is calculated by total load by allowable bearing pressure.

$$A = \frac{Q}{\text{allowable pressure}}$$

- vi. The service load for the column is calculated by adding appropriate fraction LL to DL.
- vii. The design bearing capacity(q_d) is obtained by dividing the service load of maximum LL to DL ratio by the area of footing

$$q_d = \frac{\text{Service load}}{A}$$
- viii. This pressure is less than the pressure computed in(iv)
- ix. The area of footing for each of the column is obtained by dividing the corresponding service load by the allowable bearing pressure



$$A = \frac{\text{service load for that column}}{q_d}$$

Design Procedure for Proportioning of rectangular footing:

Consider dead load + reduced live load

Step1) To find column load

$$Q = Q_1 + Q_2$$

Q_1 = load in exterior column

Q_2 = load in interior column

Step2) Find the area of footing

$$A = \frac{Q}{q_{na}} = \frac{Q}{q_s}$$

q_{na} = Allowable bearing pressure

Step3) Locate the line of action of the column loads measured from the centre of the exterior column

$$\bar{x}_1 = \frac{Q_2 x_2}{Q_1 + Q_2}$$

x_2 = centre to centre distance between the column

Step4) Define the total length of the footing

$$L = 2(\bar{x} + e_1)$$

e_1 = projection of footing

Step 5) Find the width of the footing

$$B = \frac{A}{L}$$

Step6) Find the actual area provided (A_o)

Step 7) Find the actual pressure



Consider dead load +full live load

$$q_{max} = \frac{Q}{A_o} \left(1 + \frac{6e}{L}\right)$$

$$q_{min} = \frac{Q}{A_o} \left(1 - \frac{6e}{L}\right)$$

Step 8) Check the Pressure

Actual pressure < allowable pressure

1. Proportion a rectangular combined footing for a uniform pressure under DL+reduced LL with the following data allowable pressure

DL+reduced LL=180 KN/m²

DL+LL=270 KN/m²

Column Load

Load	Column A	Column B
DL	500KN	660KN
LL	400KN	840KN

C/C distance of column 5m, Projection of footing is 0.5m

Soln:

Column load	Column A	Column B	Total load
DL+reducedLL	=500+0.5x400 =700KN	= 660 + $\frac{50}{100}$ 840 = 1080KN	=700+1080=1780KN
DL+LL	=500+400=900KN	=660+840=1500KN	=900+1500=2400KN

Consider full dead load+50% reduced live load

Step1) To find column load





$$Q = Q_1 + Q_2 = 700 + 1080 = 1780 \text{ KN}$$

Step 2) Find the area of footing

$$A = \frac{Q}{q_{na}} = \frac{Q}{q_s} = \frac{1780}{180} = 9.88 \text{ m}^2$$

q_{na} = Allowable bearing pressure

Step 3) Locate the line of action of the column loads measured from the centre of the exterior column

$$\bar{x}_1 = \frac{Q_2 x_2}{Q_1 + Q_2} = \frac{1080 \times 5}{700 + 1080} = 3.03 \text{ m}$$

X_2 = centre to centre distance between the column

Step 4) Define the total length of the footing

$$L = 2(\bar{x}_1 + e_1)$$

$$L = 2(3.03 + 0.5) = 7.06 \text{ m}$$

e_1 = projection of footing

Step 5) Find the width of the footing

$$B = \frac{A}{L} = \frac{9.88}{7.06} = 1.39 \text{ m} = 1.4 \text{ m}$$

Step 6) Find the actual area provided (A_o)

$$A_o = B \times L = 1.4 \times 7.06 = 9.88 \text{ m}^2$$

Step 7) Find the actual pressure

$$q_{max} = \frac{Q}{A_o} \left(1 + \frac{6e}{L}\right)$$

$$q_{min} = \frac{Q}{A_o} \left(1 - \frac{6e}{L}\right)$$

$$e = \bar{x} - \bar{x}$$





Find uniform pressure under full DL+LL

$$\bar{x}_2 = \frac{Q_2 x_2}{Q_1 + Q_2}$$

$$\bar{x}_2 = \frac{1500 \times 5}{900 + 1500} = 3.125 \text{ m}$$

$$e = 3.03 - 3.125 = 0.095 = 0.1 \text{ m}$$

$$q_{max} = \frac{(900 + 1500)}{9.89} \left(1 + \frac{6 \times 0.1}{7.06}\right) = 263.04 \text{ KN/m}^2 < 270 \text{ KN/m}^2$$

$$q_{min} = \frac{(900 + 1500)}{9.89} \left(1 - \frac{6 \times 0.1}{7.06}\right) = 222.7 < 270 \text{ KN/m}^2$$

Hence ok

Provide rectangular footing of size 7.06x1.4m

Proportioning of trapezoidal footing:

2. Proportion a trapezoidal combined footing for uniform pressure under a dead load + reduced live load with the following data

Allowable bearing pressure:

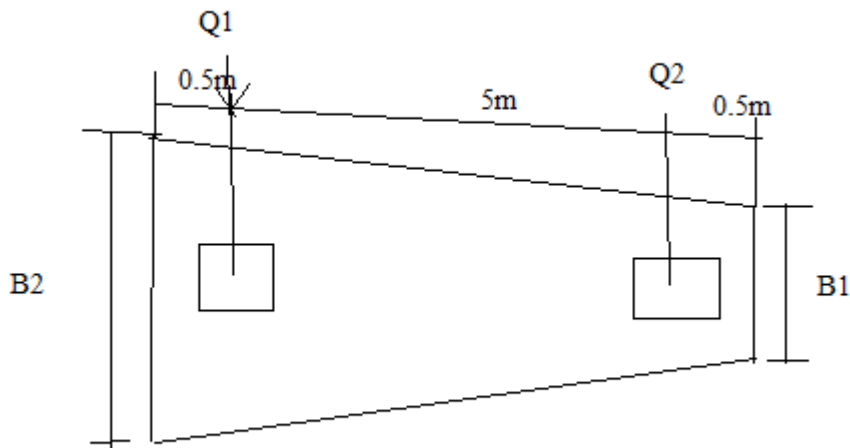
DL+reduced LL = 180 KN/m²

DL+LL = 280 KN/m²

Column load	A	B
DL	500KN	660KN
LL	400KN	840KN

Distance between c/c column = 5m

Projection beyond column A = 0.5m



Solution:

Column load	A	B	total
DL + reduced LL	700KN	1080KN	1780KN
DL+LL	900KN	1500KN	2400KN

For uniform pressure under DL+ reduced LL

Area:

$$A = \frac{Q}{q_{na}} = \frac{1780}{180} = 9.89m^2$$

$$\bar{x} = \frac{Q_2 x_2}{Q_1 + Q_2}$$

$$= \frac{1080 \times 5}{1780} = 3.03m$$

Length of footing:

Length=C/c distance + projection on both sides

$$= 5 + 0.5 + 0.5 = 6m$$

$$A = \frac{L}{2} (B_1 + B_2)$$

$$9.89 = \frac{6}{2} (B_1 + B_2)$$

$$B_1 + B_2 = 3.30 \dots (1)$$

We know that



$$\frac{L}{3} \left(\frac{B_1 + 2B_2}{B_1 + B_2} \right) = \bar{x} + e$$

$$\frac{6}{3} \left(\frac{B_1 + 2B_2}{B_1 + B_2} \right) = 3.03 + 0.5$$

$$\left(\frac{B_1 + 2B_2}{B_1 + B_2} \right) = 1.77$$

$$-B_1 + 0.43 B_2 = 0 \text{ --- (2)}$$

Solve (1)&(2)

$$B_1 = 1\text{m}$$

$$B_2 = 2.40\text{m}$$

$$\text{Total area provided} = \frac{2.40 + 1}{2} \times 6 = 10.2 \text{ m}^2$$

For dead load +live load calculations

$$\bar{x} = \frac{6}{3} \left(\frac{1 + 2 \times 2.4}{1 + 2.4} \right) = 3.41\text{m}$$

Location of resultant DL+LL for exterior column

$$= \frac{Q_2 x_2}{Q} = \frac{1500 \times 5}{2400} = 3.125\text{m}$$

Location of resultant from the outer edge of the footing

$$= 3.125 + 0.5 = 3.625\text{m}$$

$$\text{Eccentricity } e = 3.625 - 3.43 = 0.195\text{m}$$

Determination of pressure:

$$\text{moment of inertia} = \left[\frac{B_1^2 + 4B_1B_2 + B_2^2}{36(B_1 + B_2)} \right] L^3$$

$$\text{moment of inertia} = \left[\frac{1^2 + 4 \times 1 \times 2.40 + 2.4^2}{36(1 + 2.4)} \right] 6^3$$

$$= 28.87\text{m}^4$$

$$q_{max} = \frac{Q}{A} + \left[\frac{QxexX}{I} \right]$$

$$= \frac{2400}{10.2} + \left[\frac{2400 \times 0.195 \times (6 - 3.41)}{28.87} \right] = 277 < 280 \text{KN/m}^2$$

$$q_{max} = \frac{Q}{A} + \left[\frac{QxexX}{I} \right]$$





$$= \frac{2400}{10.2} - \left[\frac{2400 \times 0.195 \times (6 - 3.41)}{28.87} \right]$$

$$= 193.3 < 280 \text{KN/m}^2$$

Hence OK

Proportioning of strap footing:

3. Proportioning of strap footing for the following data:

Allowable pressure=150KN/m² for DL+reduced LL

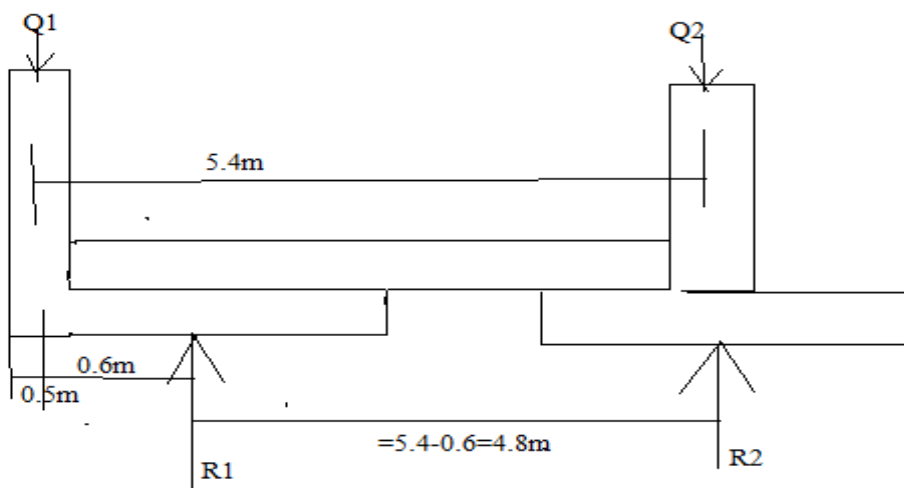
Allowable pressure=225KN/m² for DL+ LL

Column Load	A	B
DL	500KN	600KN
LL	450KN	800KN

Proportion the footing for uniform pressure under DL+reduced LL centre to centre spacing between column =5.4m.projection beyond column A should not exceed 0.5m

Solution:

Step 1: Assume eccentricity e=0.6m



Column load	A	B	total
DL+reduced LL	725KN	1000KN	1725KN
DL+LL	950KN	1400KN	2350KN

Step 2) Determine the length of footing of exterior column



$$L_1 = 2(e + 0.5b_1)$$

$$L_1 = 2(0.6 + 0.5 \times 1) = 2.2m$$

Consider DL+reduced LL

Steps 3) compute the reaction R_1 by taking moment about the line of action of the reaction

$$R_1 = \frac{Q_1 x_2}{S} = \frac{725 \times 5.4}{4.8} = 815KN$$

Steps 4) compute R_2

$$\begin{aligned} R_2 &= (Q_1 + Q_2) - R_1 \\ &= 1725 - 815 = 910KN \end{aligned}$$

Steps 5) compute the area of footing A_1

$$A_1 = \frac{R_1}{q_{na}} = \frac{815}{150} = 5.4m^2$$

$$A_2 = \frac{R_2}{q_{na}} = \frac{910}{150} = 6.07m^2$$

Step 6) calculate width of footing

$$B_1 = \frac{A_1}{L_1} = \frac{5.4}{2.2} = 2.45m$$

Provide a width of 2.50m

Consider $B_1 = B_2 = 2.5m$

$$B_2 = \sqrt{A_2}$$

$$B_2 = \frac{A_2}{L_2}$$

$$L_2 = \frac{A_2}{B_2} = \frac{6.07}{2.5} = 2.43m$$

Provide the length is 2.5m

Step 7) calculate Pressure intensity

$$q_1 = \frac{R_1}{L \times B}$$

$$\begin{aligned} &= \frac{815}{2.2 \times 2.5} \\ &= \frac{148.18 \text{KN}}{\text{m}^2} < 150 \text{KN/m}^2 \end{aligned}$$





$$\begin{aligned}q_2 &= \frac{R_2}{L \times B} \\ &= \frac{910}{2.5 \times 2.5} \\ &= \frac{145.6 \text{ KN}}{\text{m}^2} < 150 \text{ KN/m}^2\end{aligned}$$

Provide the strap footing of size
2.2x2.5m and 2.5x2.5m



3.5 Mat Foundation:

Mat foundation is also known as the raft foundation. It is a continuous thick concrete slab on the soil that extends the entire footprint of the building and increases the earth-bearing capacity power. This foundation supports the whole building loads and safely transfers them to the ground.

Raft/Mat foundation is used in those places where we have less bearing capacity of the soil. At those places, we use mat foundations to distribute the whole loads of the structure to the earth (When the footing area increases, then the soil load-bearing capacity will also increase) because this foundation reduces the stress on the soil at the same place.

USE MAT FOUNDATION:

- There is a lot the critical reason for the use of mat foundations. We use this foundation if the bearing capacity of the soil is weak and not capable of transferring the load of the building to the ground.
- A column is placed near the property line, and walls are so close that individual footing would overlap.
- If a deep foundation (Pile foundation) cost is higher than the raft foundation, we use it to make the structure economical.
- When a spread footing, columns can cover up to 50% of the foundation area.

TYPES OF MAT/RAFT FOUNDATION:

There are the following types of mat foundation,

1. Flat plate raft
2. Plate thickened under columns.
3. Two-way slab and beam
4. Piled mat
5. Rigid frame raft
6. Cellular mat foundation

1: FLAT PLATE RAFT:



- A flat plate raft is used in the small and lightweight load structure. This type of foundation is suitable when the soil is not compressible. The reinforcement bars are provided in both directions, top and bottom, in the form of a mesh (cage).
- A minimum of 6 inches of thick RCC slab is used in this foundation.

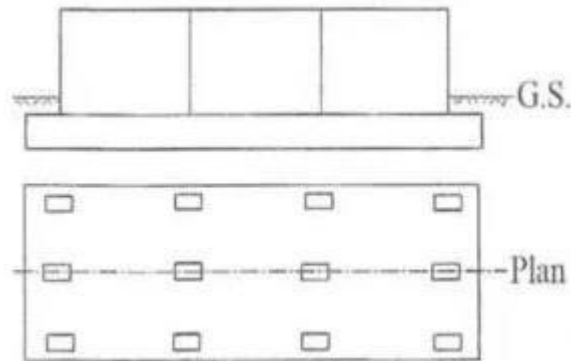


Fig 1 FLATPLATERAFT

[Fig1 <https://www.civilclick.com/mat-foundation/>]

2: THICKENED PLATE UNDER COLUMNS:

- The Slab thickness should be increased when the upcoming column loads are weighty. The flat plate foundation is not suitable in those structures where the column loads are very high, then the thickened flat plate is used.
- The heavy loads create the diagonal shear in the slab and create a negative bending moment on columns, so the thickness of the RCC slab under the column should be thickened.

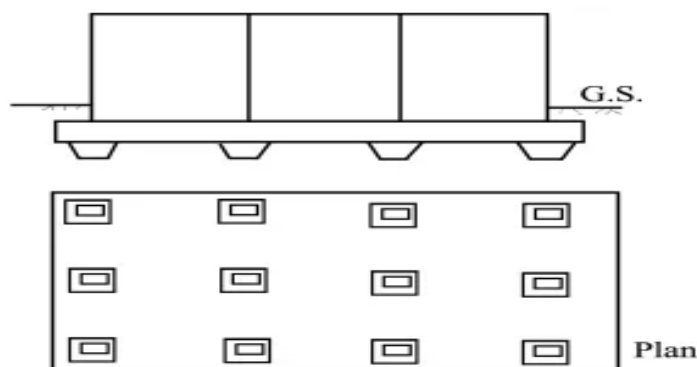


Fig2 ThickenedPlateUnderColumns

[Fig 2 <https://bestengineeringprojects.com/mat-foundation-types-of-mat-foundation/>]



3: TWO-WAY SLAB AND BEAM:

- In this type of mat foundation, beams are placed in perpendicular directions, and all the beams are connected by an RCC slab. The columns are placed at the intersection of beams.
- A two-way slab and beam raft foundation is suitable when the columns are carrying unequal loads and the spacing is large between them.

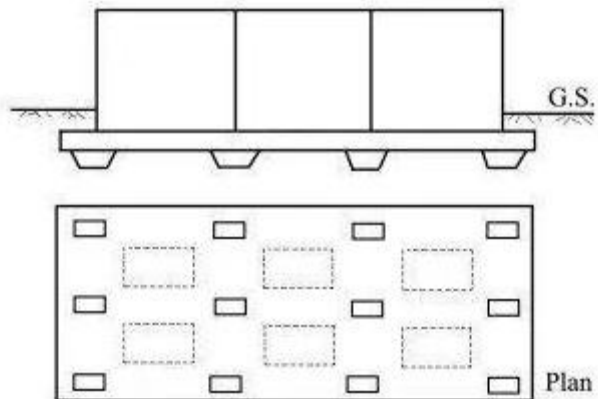


Fig3 Two-Way Slab and Beam

[Fig 3 <https://www.civilclick.com/mat-foundation/>]

4: PILED RAFT FOUNDATION:

- This foundation is supported by piles in the soil. The piled raft foundation is suitable for the ground of high compressibility and where the water table is high. Piled raft foundation is mainly used for high-rise buildings.
- The piles were used to reduce the amount of soil settlement(With time) and increase the soil load-bearing capacity.

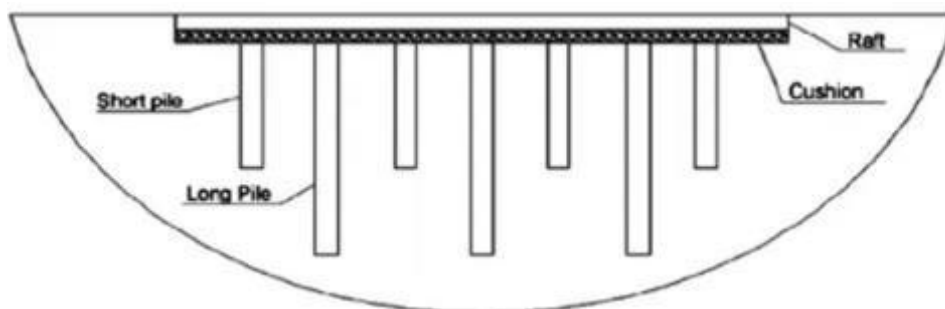


Fig 4 Piled Raft Foundation

[Fig 4 <https://www.civilclick.com/mat-foundation/>]

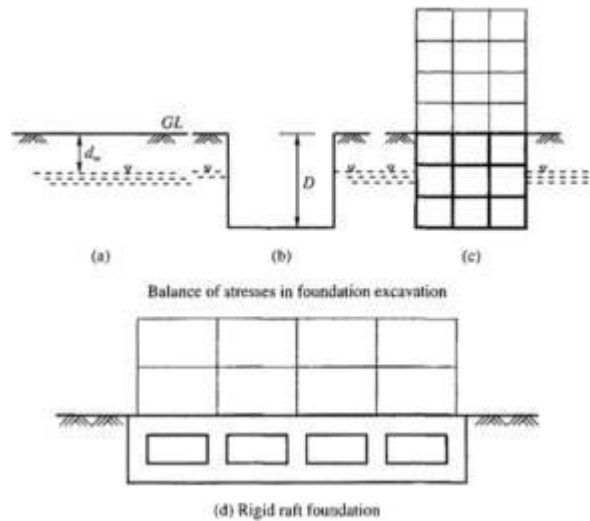


Fig 5 RigidFrameRaft

[Fig 5 <https://www.civilclick.com/mat-foundation/>]

6: CELLULAR MAT FOUNDATION:

- It is also termed a box mat foundation. In the cellular mat foundation, the structures of boxes are formed, and the walls of every box act as beams. The walls are connected by slabs at the top and bottom.
- This type of foundation is suitable for loose soil.

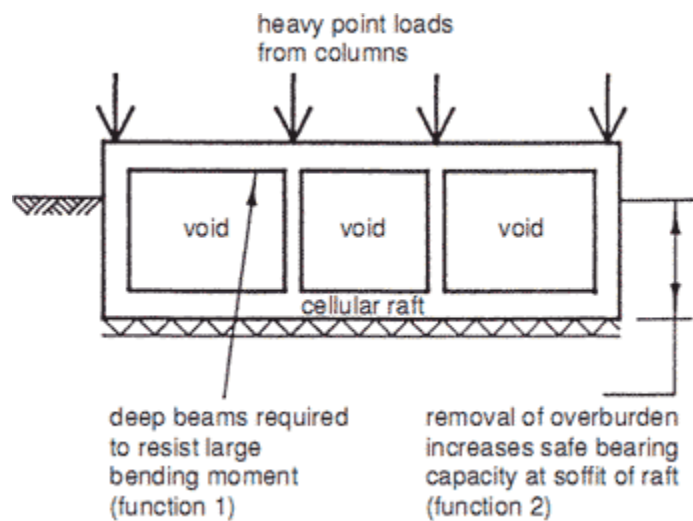


Fig6 CellularMatFoundation

[Fig 6 <https://www.civilclick.com/mat-foundation/>]



- It is provided where the shallow foundation is possible, but the condition of the soil is poor.
- Reduces the cost of constructing a floor slab (But not entirely economical).
- It helps in the transferring of loads over a wide area.
- It shows good resistance and cannot slide during the flood.
- We can handle more heavy loads as compared to other types of foundations..

Disadvantages:

- Raft foundation requires a large quantity of steel and concrete.
- This foundation is costly (The volume of footing was increasing).
- It is not suitable and used for domestic home construction.
- Unique measurement is needed in the case of concentrated loads.
- In the mat foundation, skilled laborers are required.

Floating foundation:

A floating foundation is a type of foundation constructed by excavating the soil in such a way that the weight of structure built on the soil is nearly equal to the total weight of the soil excavated from the ground including the weight water in the soil before the construction of structure.

A Floating Foundation, also known as Balancing Raft is a type of foundation where the weight of the building is approximately equal to the full weight of the soil and water removed from the site of the building prior to construction.

Problems During in the Design of a Floating Foundation:

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

1. Excavation
2. De watering
3. Critical depth



It is better to examine the water table level prior to the excavation. If the depth of the excavation is below the water table then dewatering is essential. Care has to be taken to see that the adjoining structures are not affected due to the lowering of the water table.

Critical Depth:

If the shear strength of the soil is low and there is a theoretical limit to the depth to which an excavation can be made. Terzaghi (1943) has proposed the following equation for computing the critical depth D_c ,

$$D_c = \frac{5.7 s}{\gamma - (s/B)\sqrt{2}}$$

γ =unit weight of soil

s = Shear strength

B =width of foundation

L =Length of foundation

- Skempton (1951) proposes the following equation for D_c which is based on actual failures in excavations,

$$D_c = N_c \frac{S}{\gamma}$$

Where N_c = Skempton's bearing capacity factor.

- By using any one the above two equations, the critical depth or maximum depth of excavation can be determined.



Bottom Heave

When the soil is excavated up to some depth, the pressure of the soil below this depth is lowered which results the formation of heave.

The formed heave causes settlement to the structure or foundation. We cannot prevent the formation of heave but there are some methods to minimize the formation of heave.

There are two possible causes of heave:

- Plastic inward movement of the surrounding soil.
- Elastic movement of the soil as the existing overburden pressure is removed.

Principle of Floating Foundation

- The main principle of floating foundation is to balance the weight of removed soil by a structure of same weight which causes zero settlement to the structure. So, this foundation is also called as balancing raft foundation.
- Let's consider a ground with water table at the top. The ground is excavated up to certain depth which is below water table. Now in the next step, a building is constructed which is as same weight as of the removed soil and water.
- Even the depth of excavation is below the table the total vertical pressure in the soil below the foundation is unchanged because of its balancing weight. But here one point is to be noted that we cannot build a structure immediately after the excavation.
- During the time of construction, the effective vertical pressure under the depth of excavation may slightly increase because of unbalancing weight. So, this type of foundations can also be called as partly compensated foundations instead of fully floating or compensated foundations.

Advantages of floating foundation:-

Low Load-bearing Capacity

Floating foundations are best in areas of low load-bearing capacity — when constructing a building over loose soil — or if the soil has varying degrees of compression compatibility. Construction of deep foundations will not be viable in new fill, sand and loose soil areas but, because floating foundations spread the support over a large area, there are no points of pressure taking a heavy load. To make the floating foundation stronger it might contain beams or ribs.

**Nearby buildings:**

A floating foundation can be poured when other building foundations are close. To build deep foundations would interfere with the structure of other buildings.

Moisture:

Floating foundations can be used on high moisture soils. The positioning of the foundation above the earth, rather than in it, helps create a moisture barrier between the ground and the structure.

Trenches

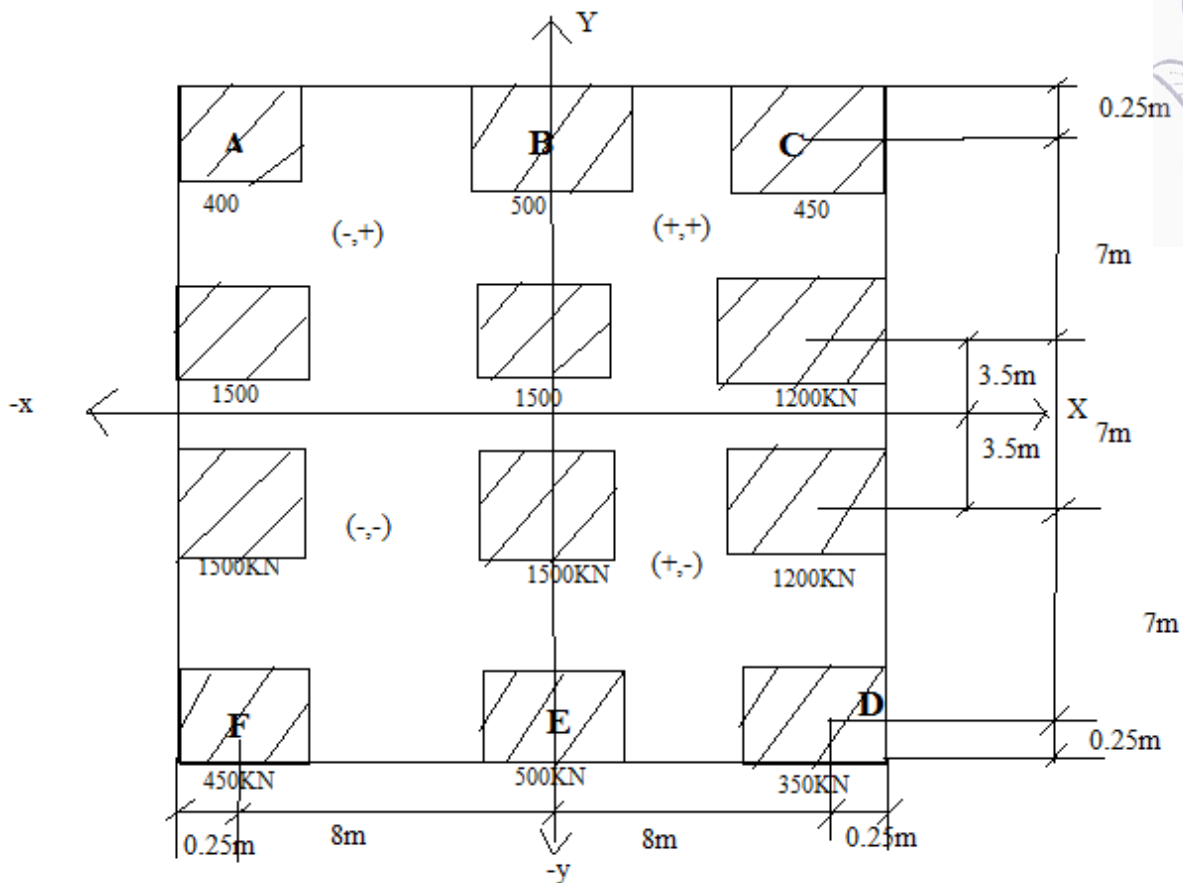
Floating foundations require far less digging because deep footer trenches are not needed. In addition, there is no need to disturb the earth beneath the building where there might be long-established tree roots or ground water.

Movement

If the earth is expected to move due to high underground moisture or high levels of vibration – as in the case of mining areas or heavily used highways -floating foundations will not be compromised.

Problem:

1. A plan of raft foundation with column load as shown in figure. Calculate the soil pressure at point A, C, D, F. The size of mat is 16.5x21.5m. All columns are 0.5x0.5m in the section. The allowable soil pressure is 60 kN/m². Determine the soil pressure at the point.



Solution:

Step1) Area of mat

$$A = bxd = 16.5 \times 21.5 = 354.75 \text{m}^2$$

Step2) Calculate the moment of inertia

$$I_{xx} = \frac{bd^3}{12} = \frac{16.5 \times 21.5^3}{12} = 13665.26 \text{m}^4$$

$$I_{yy} = \frac{db^3}{12} = \frac{21.5 \times 16.5^3}{12} = 8048.3 \text{m}^4$$

Step 3) Calculate Moment:

$$M_y = Qxe_x$$

$$e_x = x' - \frac{B}{2}$$

$$x' = \frac{Q_1x_1 + Q_2x_2 + \dots + Q_nx_n}{Q}$$



$$= \frac{1}{11000} [(400 + 1500 + 1500 + 400)x0.25] \\ + [(500 + 1500 + 1500 + 500)x8.25] \\ + [(450 + 1200 + 1200 + 30)x16.25]$$

$$x' = 7.81m$$

$$e_x = 7.81 - \frac{16.5}{2} = -0.44m$$

$$M_y = 11000x(-0.44) = -4840KNm$$

$$M_x = Qxe_y$$

$$e_y = y' - \frac{d}{2}$$

$$y' = \frac{Q_1y_1 + Q_2y_2 + \dots + Q_ny_n}{Q}$$

$$= \frac{1}{11000} [(400 + 500 + 450)x0.25] + [(1500 + 1500 + 1200)x7.25] \\ + [(1500 + 1500 + 1200)x14.25] + [(400 + 500 + 350)x21.25]$$

$$y' = 10.65m$$

$$e_y = 10.65 - \frac{21.5}{2} = 0.1m$$

$$M_x = 11000x0.1 = 1100KNm$$

Step 4) Calculate soil Pressure:

$$q = \frac{Q}{A} \pm \frac{M_x}{I_y} \pm \frac{M_y}{I_x}$$

$$q = \frac{11000}{354.75} \pm \frac{4840x}{8048.39} \pm \frac{1100y}{13665.27}$$

$$q = 31 \pm 0.6x \pm 0.08y$$

Soil pressure at A = $31 - 0.6x8.25 + 0.08x10.75 = 26.91KN/m^2$

Soil pressure at C = $31 + 0.6x8.25 + 0.08x10.75 = 31.81KN/m^2$

Soil pressure at D = $31 + 0.6x8.25 - 0.08x10.75 = 35.09KN/m^2$

Soil pressure at E = $31 - 0.6x8.25 - 0.08x10.75 = 25.19KN/m^2$



4.1 CLASSIFICATION OF PILES:

Piles can be classified according to

1. The material used
2. The mode of transfer of load
3. The method of construction
4. The use and
5. Displacement of soil

1. Classification according to material used

There are four types of piles according to materials used

- a) Steel piles
- b) Concrete piles
- c) Timber piles
- d) Composite piles

a) Steel piles:

- Steel piles are generally either in the form of thick pipes or rolled steel H- section. Pipe steel piles are driven into ground with their ends open or closed. Piles are provided with a driving point or shoe at the lower end.
- Epoxy coatings are applied in the factory during manufacture of pipes to reduce corrosion of the steel pipes. Sometimes concrete encasement at site is done as a protection against corrosion. To take into account the corrosion, an additional thickness of the steel section is usually recommended.

b) Concrete piles

- Cement concrete is used in the construction of concrete piles. Concrete piles are either precast or cast in-situ. Precast concrete piles are prepared in a factory or a casting yard. The reinforcement is provided to resist handling and driving stresses. Precast piles can also be pre-stressed using high strength steel pre-tensioned cables.
- A cast in-situ pile is constructed by making a hole in the ground and then filling it with concrete. A cast in situ pile may be cased or uncased. A cased pile is constructed by driving a steel casing into the ground and filling it with concrete. An uncased pile is constructed by driving to the desired depth and gradually



withdrawing casing when fresh concrete is filled. An un-casted pile may have a pedestal.

c) Timber piles

- Timber piles are made from tree trunks after proper trimming. The timber used should be straight, sound and free from defects.
- Steel shoes are provided to prevent damage during driving. To avoid damage to the top of the pile, a metal bond or a cap is provided. Splicing of timber piles is done using pipe sleeve or metal straps and bolts. The length of the pipe sleeve should be at least five times the diameter of the pile.
- Timber piles below the water table have generally long life. However, above the water table, these are attacked by insects. The life of the timber piles can be increased by preservatives such as creosote oil. Timber piles should be used in massive environment where these are attacked by various.

d) Composite piles

- A composite pile is made of two materials. A composite pile may consist of the lower portion of steel and the upper portion of cast in-situ concrete.
- A composite may also have the lower portion of timber below the permanent water table and the upper portion of the concrete.
- As it is difficult to provide a proper joint between two dissimilar materials, composite piles are rarely used in practice.

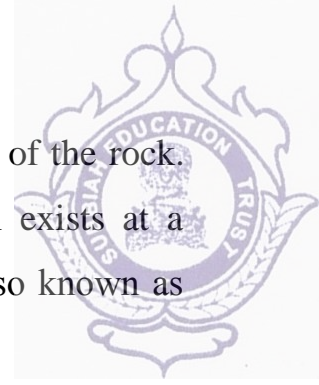
2. Classification based on mode of transfer of load

a) Based on the mode of transfer of loads, the pile can be classified into three categories:

1. End bearing piles
2. Friction piles
3. Combined end bearing and friction piles

i. End bearing piles

- End bearing piles transmit the loads through their bottom tips. Such piles act as columns and transmit the load through a weak material to a firm stratum below. If bed rock is located within a responsible depth, piles can be extended to the rock.



- The ultimate capacity of the pile depends upon the bearing capacity of the rock. If instead of bed rock, a fairly compact and hard stratum of soil exists at a reasonable depth, piles can be extended a few minutes' piles are also known as "point-bearing piles".
- The ultimate load carried by the pile (Q_u) is equal to the load carried by the point or bottom end (Q_p)

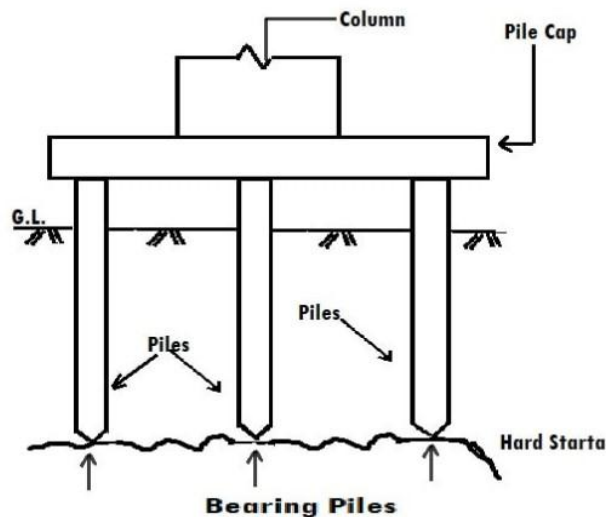


Fig1. End bearing pile

[Fig1 <https://www.civilknowledges.com/pile-foundation/>]

ii. Friction piles

- Friction piles do not reach the hard stratum. These piles transfer the loads through skin friction between the embedded surface of the pile and the surrounding soil. Friction piles are used when a hard stratum does not exist at a reasonable depth.
- The ultimate load (Q_u) carried by the pile is equal to the sum of the load carried by the pile is equal to the load transferred by skin friction (Q_s).
- Friction piles are known as floating piles as these do not reach the hard stratum.

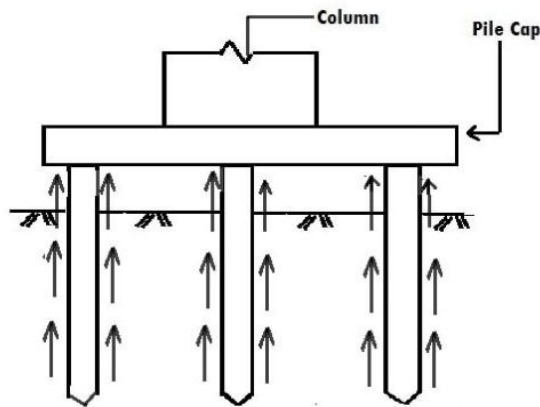


Fig2.FrictionPile

[Fig2 <https://www.civilknowledges.com/pile-foundation/>]

iii. Combined end bearing and friction piles

- The piles transfer load by a combination of end bearing at the bottom of the pile and friction along the surface of the pile shaft, the ultimate load carried by the pile is equal to the sum of the load carried by the pile point (Q_p) and the load carried by the skin friction (Q_s).

3) Classification based on method of installation

Based on the method of construction, the piles may be classified into the following 5 categories

- i. Driven pile
- ii. Driven and cast in situ piles
- iii. Bored and cast in situ piles
- iv. Screw piles
- v. Jacked piles

i) Driven piles

These piles are driven into the soil by applying blows of a heavy hammer on their tops.

ii) Driven and cast in situ piles

These piles are formed by drawing a casing with a closed bottom end into the soil. The casing is later filled with concrete. The casing may or may not be withdrawn.

iii) Bored and cast in situ pile



These piles are formed by a hole into the ground and then filling it with concrete.

iv) Screw piles

These piles are screwed into soil.

v) Jacked piles

These piles are jacked into the soil by applying a downward force with the help of a hydraulic jack.

4) Classification based on use

The piles can be classified into the following 6 categories depending upon their use.

- i. Load bearing piles
- ii. Compaction piles
- iii. Tension piles
- iv. Sheet piles
- v. Fender piles
- vi. Anchor piles

i) Load bearing piles

These piles are used to transfer the load of the structure to a suitable stratum by end bearing by friction or by both.

ii) Compaction piles

These piles are driven into the loose granular soil to increase the relative density. The bearing capacity of the soil is increased due to densification caused by vibrations.

iii) Sheet piles

Sheet piles form a continuous wall or bulk head which are used for retaining earth or water.

iv) Fender piles

Fender piles are sheet piles which are used to protect water front structures from impact of ships and vessels.

v) Anchor piles

These piles are used to protect anchorage for anchored sheet piles. These piles provide resistance against horizontal pull for a sheet pile wall.

5) Classification based on displacement of soil:



All driven piles are displacement piles as the soil is displaced laterally when the pile is installed. The soil gets densified. The installation may cause heaving of the surrounding ground. Precast concrete pile and closed end pipe pile are high displacement piles. Sheet H- piles are low displacement piles.

2. Non- displacement piles

Bored piles are non- displacement piles. As the soil is removed when the hole is bored, there is no displacement of the soil during installation. The installation of these piles causes very little change in the stresses in the surrounding soil.

Use of pile foundation is preferred:

- The load coming from the user structure is heavy and its distribution is uneven.
- The structure is located on a sea-shore or river bed when the foundation is likely to be affected by the scouring action of water. Hence they are useful in marine structures.
- The pile foundation is used for the structures in the area where canals, deep drainage lines, etc. are to be constructed near the foundation.
- The top soil has a poor bearing capacity.
- The construction of the raft foundation or grillage foundation is likely to be very costly or is practically impossible.
- The sub-soil water level is high so that the pumping of water from the open trenches for the shallow foundation is difficult and uneconomical.

Selection of Foundation:

The selection of pile foundation depends on the soil investigation data received from soil exploration bore holes at different depths. Selection of appropriate pile for the desired strength and requirement plays an important role in cost reduction and efficiency. In this article we discuss about the selection of type of piles based on soil conditions.



- Number of piles to be used
- Cost of construction
- Type, size, and weight of the structure to be supported.
- Physical properties of the soil at the site.
- Depth to a stratum capable of supporting the piles.
- Possibility of variations in the depth to a supporting stratum.
- Availability of materials for piles.
- Number of piles required.
- Facilities for driving piles.
- Durability required.
- Types of structures adjacent to the project.
- Depth and kind of water, if any, above the ground into which the piles will be driven.



4.2 Load carrying capacity of pile:

- The ultimate Load carrying capacity of pile or ultimate Load bearing resistance of pile is the maximum load which it can be carry without failure.
- The pile transfer the load in two ways
 - 1) Through the tip in compression is called end bearing or point bearing
 - 2) By shear along the surface is called as skin friction
- All type of pile behave both end bearing and skin friction

The Load carrying capacity of pile can be determined by following method

1. Dynamic Analysis
2. Static Analysis
3. Pile load test
4. Penetration test

1. Dynamic Analysis:

The load carrying capacity of a driven pile can be estimated from the resistance against penetration developed during driving operation with a hammer.

i) Engineering news formula

$$Q_a = \frac{WH}{F(S + C)}$$

$$Q_a = \text{allowable load}$$

H= height of fall

W=weight of hammer in kg

F=Factor of safety=6

S=settlement or penetration

C=empirical constant=2.5cm

ii) Drop hammer

$$Q_a = \frac{WH}{b(S + 2.5)}$$

a) single acting hammer

$$Q_a = \frac{WH}{6(s + 0.25)}$$

b) double acting hammer



$$Q_a = \frac{(W + ap)H}{6(s + 0.25)}$$

iii) Hiley's formula

$$Q_f = \frac{WH\eta_h\eta_b}{s + \frac{C}{2}}$$

$Q_f = \text{ultimate load on pile}$

$C = \text{elastic compression}$

$$C = c_1 + C_2 + C_3$$

$c_1, C_2, C_3 = \text{temporary elastic constant}$

C_1 value :

Precast=0.12-0.5

Steel=0.04-0.16

Timber=0.05-0.2

$$C_2 = \frac{Q_a L}{AE}$$

C_3 value:

$C_3 = 0.1 = \text{average value}$

$C_3 = 0 = \text{hard soil}$

$C_3 = 0.3 = \text{residual soil}$

$$\eta_b = \frac{W + e^2 P}{W + p}, W > ep$$

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

$P = \text{weight of pile helmet/cap}$

$e = \text{coefficient of restitution value}$

For timber pile $e = 0$

For steel pile $e = 0.5$

$\eta_h = 100\% = \text{drop hammer}$

$\eta_h = 75 - 85\% = \text{single acting hammer}$

$$\eta_h = 100\% = \text{Diesel hammer}$$

1. For single acting hammer





$e=0$ to 0.25

2. Double acting hammer

$$C_1 = 1.77 \cdot \frac{Q_u}{A}$$

$$C_2 = 0.0657 \frac{Q_u L}{A}$$

$$C_3 = 3.55 \frac{Q_u}{A}$$

A=Area of pile in cm^2

L=length of pile in m

R= resistance

Safe load on pile:

$$Q_s = \frac{Q}{F}$$

Problems:

Dynamic Analysis:

1. A wooden pile is being driven with a drop hammer weight 20KN and having a free fall of 1m.the penetration in the last blow is 5mm.Determine the load carrying capacity of pile according to engineering news formula.

Given data:

W=20KN

H=1m=100cm

S=5mm=5/10cm=0.5cm

To find:

$Q_a=?$

Soln:

i)Engineering news formula

$$Q_a = \frac{WH}{F(S + C)}$$

W.K.T $F=6, C=2.5$

$$Q_a = \frac{20 \times 100}{6(0.5 + 2.5)} = 111.11 \text{KN}$$



2. A reinforced concrete pile weighing 30 kN inclusive of pile cap and a dolly driven by drop hammer of weight 40 kN and having an effective fall of 0.8 m. The average set per blow is 1.4 cm. The temporary elastic compression is 1.8 cm. Assume the coefficient of restitution as 0.25 and FOS of 2. Determine the allowable load.

Given data:

concrete pile weighing (p) = 30 kN

drop hammer of weight (W) = 40 kN

$H = 0.8 \text{ m} = 80 \text{ cm}$

$S = 1.4 \text{ cm}$

$C = 1.8 \text{ cm}$

$e = 0.25$

FOS = 2

To find:

Allowable load (Q) = ?

Soln:

Hiley's formula

$$Q_f = \frac{WH\eta_h\eta_b}{s + \frac{c}{2}}$$

$$\eta_b = \frac{W + e^2P}{W + p}, W > ep$$

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

$$ep = 0.25 \times 30 = 7.5 \text{ kN}$$

$$W = 40 \text{ kN}$$

$W > ep$, therefore

$$\eta_b = \frac{W + e^2P}{W + p}$$

$$\eta_b = \frac{(40 + (0.25^2) \times 30)}{30 + 40} = \frac{41.87}{70} = 0.598$$

$$\eta_h = 100\% = 1 = \text{drop hammer}$$

$$Q_f = \frac{40 \times 80 \times 1 \times 0.598}{s + \frac{c}{2}}$$

$$\frac{1.4 + \frac{1.8}{2}}{2} = \frac{1913.6}{2} = 956.8$$





$$Q_a = \frac{Q_f}{F} = \frac{832}{2} = 416 \text{ kN}$$

3. A reinforced concrete pile weighing 30 kN is driven by a drop hammer weighing 40 kN and having an effective fall of 0.80m. The average set per blow is 1.40 cm. The total temporary elastic compression is 1.80 cm, assuming the coefficient of restitution as 0.25 and factor of safety of 2. Determine the ultimate bearing capacity and the allowable load for the pile.

Given:

$$P = 30 \text{ kN}$$

$$W = 40 \text{ kN}$$

$$\eta_n H = 0.8 \text{ m} = 80 \text{ cm}, s = 1.4 \text{ cm}$$

$$C = 1.8 \text{ cm}$$

$$\eta_n = 1 \text{ for drop hammer.}$$

$$P \times e = 30 \times 0.25 \\ = 7.5 \text{ kN}$$

$$\therefore w > p \text{ e}$$

Efficiency hammer blow

$$\therefore \eta_b = \frac{w + e \cdot 2p}{w + p} \\ = \frac{40 + 0.25 \times 30}{40 + 30} \\ \eta_b = 0.598$$

Ultimate load on pile

$$Q_f = \eta_n w H \eta_b / (s + c/2) \\ = 1 \times 40 \times 80 \times 0.598 / (1.4 + 1.8/2) \\ = 1914.286 / (1.4 + 0.9) \\ Q_f = 832.3 \text{ kN}$$

Allowable load = Q_f / FOS

$$= 832.3 / 2.0$$

$$Q_a = 416.2 \text{ kN}$$



4.3 Methods to determine load carrying capacity of pile:

- The ultimate Load carrying capacity of pile or ultimate Load bearing resistance of pile is the maximum load which it can be carry without failure.
- The pile transfer the load in two ways
 - 1) Through the tip in compression is called end bearing or point bearing
 - 2) By shear along the surface is called as skin friction
- All type of pile behave both end bearing and skin friction

The Load carrying capacity of pile can be determined by following method

1. Dynamic Analysis
2. Static Analysis
3. Pile load test
4. Penetration test

1. Dynamic Analysis:

The load carrying capacity of a driven pile can be estimated from the resistance against penetration developed during driving operation with a hammer.

2. Static Analysis:

Sum of end bearing pile/point bearing pile and friction pile

$$Q_{up} = A_s R_f + R_p A_p$$

A_s = Surface area of pile

A_p = Area of cross section of pile

r_f = Average skin friction

r_p = end/point/tip bearing of pile

For circular Pile:

$$A_p = \frac{\pi}{4} D^2$$

$$A_s = \pi D L$$

For rectangular Pile:

$$A_p = B \times D$$

$$A_s = 2(B + D)L$$

i) Cohesive soil:

$$r_f = \alpha \cdot C \text{ or } mc$$



$$r_p = C_p N_c, N_c = 9$$

$$r_p = 9C_p$$

$$Q_{up} = \alpha C A_s + 9C_p A_p$$

Where, α =Reduction factor

$$Q_a = \frac{Q_{up}}{F}$$

$$Q_a = \frac{\alpha C A_s + 9C_p A_p}{F}$$

ii) Cohesionless soil

$$r_f = k \tan \phi (\gamma \cdot Z + q)$$

For circular pile:

$$r_p = 0.3 \gamma B N_\gamma$$

For rectangular and square pile:

$$r_p = \frac{\gamma_B}{2} N_\gamma$$

Where,

r_f =average skin friction

γ = density of soil.

q = surcharge on the ground

ϕ = angle of internal friction.

Static Analysis: Problems

1.A reinforced concrete square pile of size 30x30cm and 10cm long is driven into saturated sand extending to great depth. The average effective unit weight =16KN/m³.average FS=2.5 .Find Q_s

Given data:

square pile of size 30x30cm=0.3x0.3m

Z=10cm

$\gamma = 16KN/m^3$

F=2.5

To find:



Safe Load $Q_s = ?$

Solution:

Assume $K=1.5, N \gamma = 25, \tan \phi = 0.6$

$$Q_{up} = A_s R_f + R_p A_p$$

cohesionless soil

$$r_f = k \tan \phi (\gamma \cdot Z + q)$$

$$r_f = 1.5 \times 0.6 \times (16 \times 10 + 0) = 144$$

For rectangular and square pile:

$$r_p = \frac{\gamma B}{2} N_\gamma$$

$$r_p = \frac{16 \times 0.3}{2} \times 25 = 60$$

$$A_p = 0.3 \times 0.3 = 0.09 \text{ m}^2$$

$$A_s = b \times Z = 0.3 \times 10 = 3 \text{ m}^2$$

$$Q_{up} = A_s R_f + R_p A_p$$

$$Q_{up} = 0.09 \times 60 + 144 \times 3 = 437.4 \text{ KN}$$

$$Q_a = \frac{Q_{up}}{F} = \frac{437.4}{2.5} = 174.96 \text{ KN}$$

2. A pile is driven in a uniform clay of large depth. $UCC=90 \text{ KN/m}^2$, 30 cm dia and 6m long, $FS=3, \alpha=0.7$. Determine the frictional resistance.

Given data:

$$UCC = 90 \text{ KN/m}^2$$

$$D = 30 \text{ cm} = 0.3 \text{ m}$$

$$Z = 6 \text{ m}$$

$$F = 3$$

$$\alpha = 0.7$$

To find :

frictional resistance = ?

Soln:

$$Q_{up} = A_s R_f + R_p A_p$$

$$Q_{up} = \text{frictional resistance} + \text{end/point/tip bearing pile}$$



$$A_s R_f = \text{frictional resistance}$$

$$R_p A_p = \text{end bearing pile}$$

Given clay soil therefore it is cohesive soil

For circular Pile:

$$A_s = \pi DL = \pi \times 0.3 \times 6 = 5.6m^2$$

Cohesive soil:

$$r_f = \alpha \cdot C \text{ or } mc$$

UCC:

$$C = \frac{q_u}{2} = \frac{90}{2} = 45 \text{KN/m}^2$$

$$R_f = 0.7 \times 45 = 31.5$$

$$\text{frictional resistance} = A_s R_f = 5.6 \times 31.5 = 176.4 \text{KN}$$

$$\text{Safe frictional resistance} = \frac{A_s R_f}{F} = \frac{176.4}{3} = 58.8 \text{KN}$$

3. A 30cm diameter concrete pile is driven normally consolidated clay deposit 15m thick. Estimate the safe load. Take $C_u = 70 \text{KN/m}^2$.

Given data:

Diameter $D = 30 \text{cm} = 0.3 \text{m}$

Clay-cohesive

$Z = 15 \text{m}$

$C_u = 70 \text{KN/m}^2$.

To find:

Safe load = ?

Solution:

$$Q_{up} = A_s R_f + R_p A_p$$

For circular Pile:

$$A_p = \frac{\pi}{4} D^2$$

$$= \frac{\pi}{4} 0.3^2 = 0.070 \text{m}^2$$

$$A_s = \pi DL = \pi \times 0.3 \times 15 = 14.13 \text{m}^2$$



Cohesive soil:

Assume $\alpha = 0.9, F = 2.5$

$$\begin{aligned} r_f &= \alpha \cdot C \text{ or } mc \\ &= 0.9 \times 70 \\ &= 63 \text{KN/m}^2 \end{aligned}$$

$$r_p = C_p N_c, N_c = 9$$

$$r_p = 9 \times 70 = 630 \text{KN}$$

$$Q_{up} = A_s r_f + r_p A_p$$

$$\begin{aligned} Q_{up} &= 14.13 \times 63 + 630 \times 0.070 \\ &= 934.29 \text{KN} \end{aligned}$$

$$Q_{up} = \text{working load}$$

Where, α = Reduction factor

$$Q_a = \frac{934.29}{2.5} = 373.716 \text{KN}$$

Q_a or Q_s = safe load or allowable load

4. A concrete pile of 45cm dia is driven through a system of layered cohesive soil. The length of the pile = 16m

1. Stiff clay = 8m, $C_u = 30, \alpha = 0.9$

2. Medium Stiff clay = 6m, $C_u = 50, \alpha = 0.75$

3. silt stratum = to creator depth, $C_u = 105, \alpha = 0.5, A_s = 0.159 \text{m}^2$.

Given data:

$$D = 45 = 0.45 \text{m}$$

Cohesive soil

$$\text{Length } L = 16 \text{m}$$

$$A_s = 0.159 \text{m}^2$$

To find:

Safe load = ?

Solution:

$$\text{silt stratum = to creator depth} = 16 - (8 + 6) = 2 \text{m}$$

$$Q_{up} = A_s r_f + r_p A_p$$



$$r_f = \alpha \cdot C \text{ or } mc$$

$$= [(0.9 \times 30 \times 8) + (0.75 \times 50 \times 6) + (0.5 \times 105 \times 2)] = 357 \text{ KN/m}^2$$

$$r_p = C_p N_c, N_c = 9$$

$$r_p = 9 \times 105 = 945 \text{ KN}$$

$$A_p = \frac{\pi}{4} D^2$$

$$= \frac{\pi}{4} 0.45^2 = 0.158 \text{ m}^2$$

$$Q_{up} = 0.159 \times 357 + 945 \times 0.158 = 202.86 \text{ KN}$$

$$Q_a = \frac{202.86}{2.5} = 81.144 \text{ KN}$$

5. A group of 9 piles with 3 piles in a row is driven into soft clay extending from ground level to a great depth. The diameter and length of piles were 30 cm and 10 m respectively. The unconfined compression strength of clay is 70 kN/m². If the piles were spaced at 90 cm centre to centre, compute the allowable load on the pile group on the basis of shear failure criteria for a factor of safety of 2.5, neglect bearing at the tip of piles, take $m = 0.6$ for shear mobilization around each pile.

Given Data:

$$n = 9 \text{ piles with 3 piles in a row. } S = 90 \text{ cm} = 0.9 \text{ m c/c}$$

$$D = 30 \text{ cm} = 0.3 \text{ m } L = 10 \text{ m}$$

$$q_u = 70 \text{ kN/m}^2$$

$$c = \frac{q_u}{2} = \frac{70}{2} = 35 \text{ KN/m}^2$$

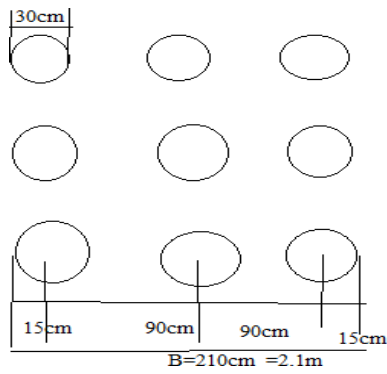
$$\text{F.S.} = 2.5, m = 0.6$$

To Find:

$$Q_a = ?$$

Solution:

Ultimate load on pile based on individual action::



Size of pile group = BXB

$$= 2.1\text{m} \times 2.1\text{m}$$

$$Q_{up} = A_s r_f + A_p r_p$$

$A_s r_f$ = Friction pile

$A_p r_p$ = end/point/tip bearing

In question neglect bearing at the tip of piles, therefore

$$Q_{up} = A_s r_f$$

$$A_s = \pi d L = \pi \times 0.3 \times 10 = 9.42\text{m}^2$$

$$r_f = \alpha C = mc = 0.6 \times 35 = 21$$

$$= 9.42 \times 21 = 197.82\text{ kN}$$

$$Q_{un} = n \times Q_{ug} = 9 \times 197.82 = 1780.38\text{ kN}$$

Ultimate load on pile based on group action:

$$Q_{up} = A_s r_f$$

$$A_s = 4BL = 4 \times 2.1 \times 10 = 84\text{m}^2$$

$$Q_{up} = 84 \times 21 = 1764\text{ kN}$$

Ultimate load on pile = least = 1780.3 kN

When the pile acting individually,

$$\text{Safe load on pile} = \frac{1780.3}{2.5} = 712.5\text{ kN}$$

6. A group of 16 friction piles is to support a column load of 4000kN. The piles will be driven in four rows with four numbers in each column. The piles are 35 cm diameter and the c/c spacing is 1m both ways. What set value must be attained by the piles when driven by a single acting 22.5kN steam hammer with 90cm stroke so that the pile group can carry the column load? Assuming L= 10m

Solution:



Case1)i)Load carried by group action

$$Q_{up}=A_s r_f + A_p r_p$$

$$A_s=4BL$$

$$=4 \times 3.35 \times 10$$

$$=134m^2$$

$$Q_{up}=84 \times 21 = 1764KN$$

$$A_p = B^2 = 3.35^2 = 11.22m^2$$

$$r_p = CN_c = 9C$$

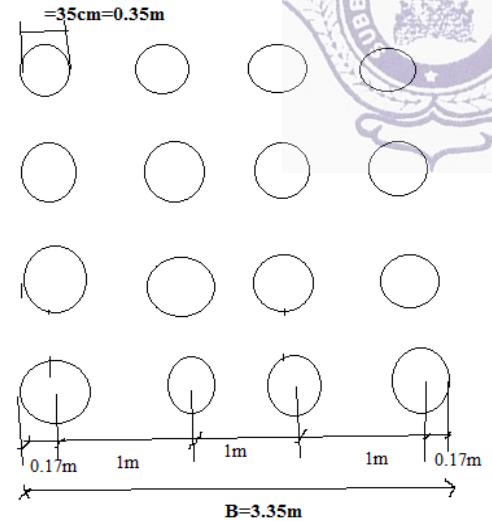
$$r_f = \alpha C = mc = 0.7 \times C$$

In group load is 4000KN

$$Q_{ug} = A_s r_f + A_p r_p$$

$$4000 = 134 \times 0.7 C + 11.22 \times 9C$$

$$C = 22KN/m^2$$



ii)Load carried by individual action:

$$Q_{up} = A_s r_f + A_p r_p$$

$$A_p = \frac{\pi d^2}{4} = \frac{\pi \times (0.35)^2}{4} = 0.096m^2$$

$$r_p = CN_c = 22 \times 9 = 198KN/m^2$$

$$r_f = \alpha C = mc = 0.7 \times 22 = 15.4KN/m^2$$

$$A_s = \pi dL = \pi \times 0.35 \times 10 = 10.99m^2$$

$$Q_{up} = 10.99 \times 15.4 + 0.096 \times 198 = 188.254KN$$

$$Q_{un} = n \times Q_{up}$$

$$= 16 \times 188.25$$

$$= 3012.064$$

Individual pile fails first.

Caseii)Engineering news formula

$$Q_u = WH/6(S+C) \text{ for stream hammer } C=0.254$$

$$2628 = [22.5 \times 0.9 \times 100] / [6(S + 0.254)]$$

$$2628 = 20.25 / [6S + 1.524]$$

$$2628(6S + 1.524) = 20.25 \times 100 \text{ (neglect the sign)}$$

$$6S + 1.524 = 0.77$$



$$6S = 0.752$$

$$S = 0.125 \text{ cm}$$

$$S = 1.25 \text{ mm}$$

7. Design a friction pile group to carry a load of 3000 kN including the weight of the pile cap at a site where the soil is uniform clay to a depth of 20 m underlain by rock. Average unconfined compressive strength of the clay is 70 kN/m^2 . The clay may be assumed to be of normal sensitivity and normally loaded, with liquid limit of 60%. A factor safety of 3 is required against shear failure.

Given

$$Q_{ug} = 3000 \text{ kN}$$

$$C = q_u/2 = 70/2 = 35 \text{ kN/m}^2$$

$$\text{Permission } C = \frac{c}{F}$$

$$\text{Permission } C = 35/3 \text{ kN/m}^2$$

Assume, Let the length of pile = 10 m

Diameter of the pile = 0.5 m

Spacing of pile = $3d = 3 \times 0.5 = 1.5 \text{ m} = 150 \text{ cm}$

Let the no. Of piles = n

$$Q_{up} = C \pi d L$$

$$Q_{ug} = n Q_{up}$$

$$Q_{ug} = n \times 35/3 \times \pi \times 0.5 \times 10$$

$$n = 16.37$$

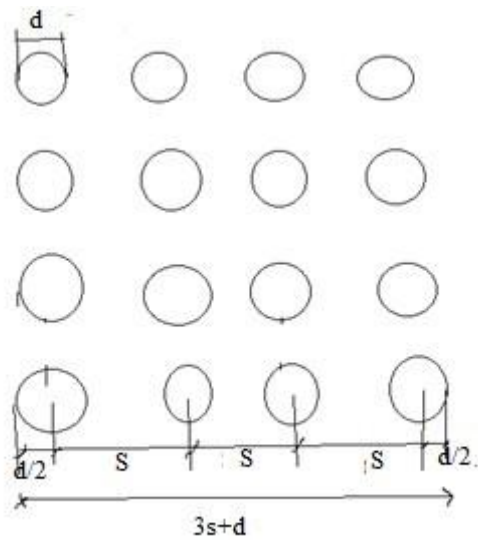
For square arrangement keep n = 16

The modified length L will then have to increase by the ratio $16.37/16$

$$L = 10 \times 16.37/16$$

$$L = 10.23 \text{ m} = 11 \text{ m}$$

Check for group action



$$B = 3s + d = 3 \times 150 + 50 = 500 \text{ cm} = 5 \text{ m}$$

Load taken by group action

$$= 4 BL \times C + A P. C N_c$$

$$= 4 \times 5 \times 11 \times (35/3) + [(5 \times 5) \times (39/3) \times 9]$$

$$= 2566.7 + 2625$$

$$Q_{ug} = 5191.7 \text{ kN} > 3000 \text{ kN}$$

Hence safe,



4.4 Capacity from insitu tests:

1. Standard Penetration Test (SPT)
2. Cone Penetration Test (CPT)
3. Pile load test

1. Standard Penetration Test (SPT): The load carrying capacity of a pile can be estimated from the SPT value (N)

i) For Driven pile:

$$q_p = 40N \left(\frac{D}{B}\right) \leq 400N \text{ --- (1)}$$

Where,

q_p = Point resistance (KN/m²)

D = Length of pile

B = width of pile

The value of q_p is usually limited to 400N

The average unit frictional resistance (f_s) is related to the average value of the blow count \bar{N}

For High displacement piles,

$$f_s = 2\bar{N} \text{ KN/m}^2$$

For low displacement piles,

$$f_s = 1\bar{N} \text{ KN/m}^2$$

Where \bar{N} is the average uncorrected value

ii) For bored pile in sand:

$$q_p = 14N \left(\frac{D_b}{B}\right) \text{ KN/m}^2 \text{ --- (2)}$$

D_b = actual penetration into the granular soil

For bored piles in sand,

$$f_s = 0.67\bar{N} \text{ KN/m}^2$$

2) Dutch cone test:

Meyerhof (1965) relates the unit point resistance (q_p) and the unit skin traction (f_s) of driven pile to cone point resistance (q_c)

$$\text{Point resistance, } q_p = q_{10} \left(\frac{D_b}{B} \right) \dots (3)$$





Unit skin friction:

$$a) f_s \text{ (dense sand)} = q_c/200 \text{ ----- (4)}$$

$$b) f_s \text{ (loose sand)} = q_c/400 \text{ ----- (5)}$$

$$c) f_s \text{ (silt)} = q_c/150 \text{ ----- (6)}$$

3. Pile Load test:

➤ The Pile Load Test is the most reliable method of determining the load carrying of a pile. This test can be performed either on a working pile that forms the foundation of the structure or on a test pile.

➤ Loads Acting on Piles

Following are the loads which are to be taken into account while designing a pile.

- Direct vertical load coming from the superstructure.
- Impact stresses developed during the process of pile driving.
- Stresses developed during handling operations.
- Bending stress developed due to the curvature of a pile.
- Bending stresses developed due to the eccentricity of loads coming on the pile.
- Lateral forces due to the wind, waves, currents of water, etc.
- Impact forces due to the ice sheets or bergs.
- Impact forces due to ships, in case of marine structures.
- Force due to the uplift pressure.
- Earthquake forces.

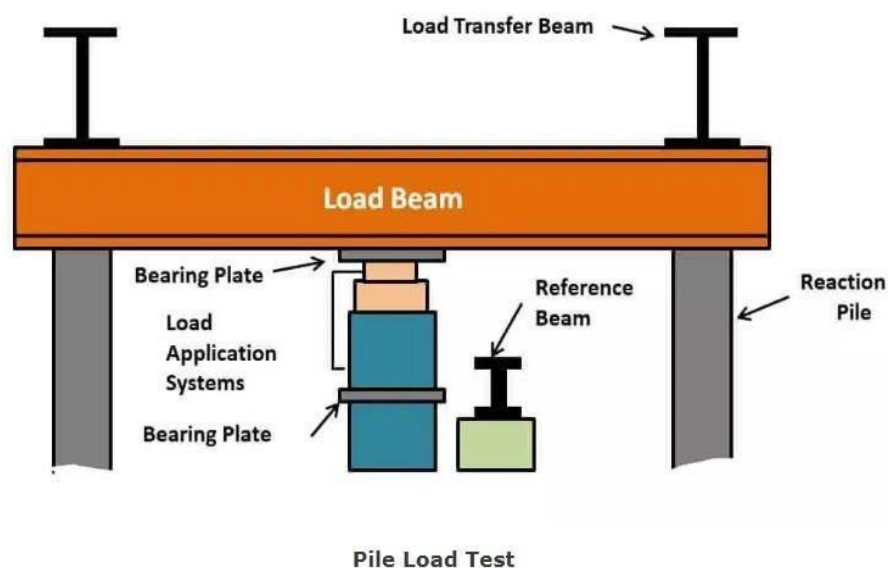




Fig1 Pile Load Test

[Fig1 <https://civiconcepts.com/blog/pile-load-test>]

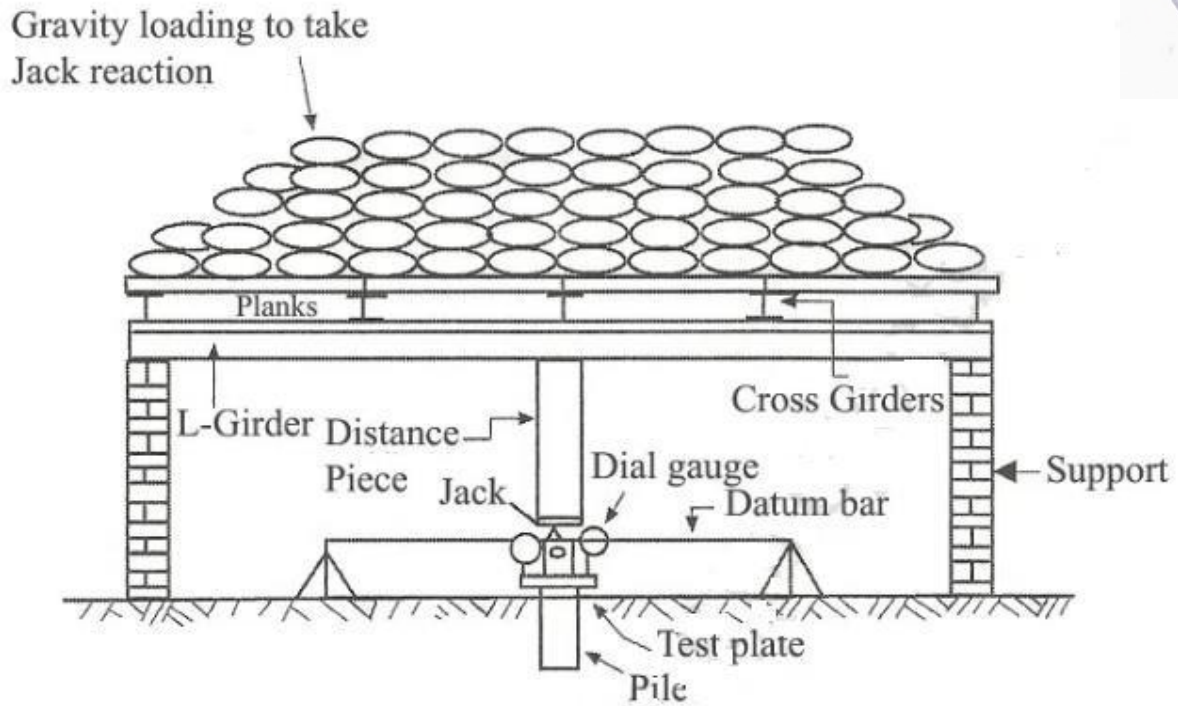


Fig 2 Jack Loading reaction by loading platform

[Fig 2 <https://bestengineeringprojects.com/pile-load-test/>]

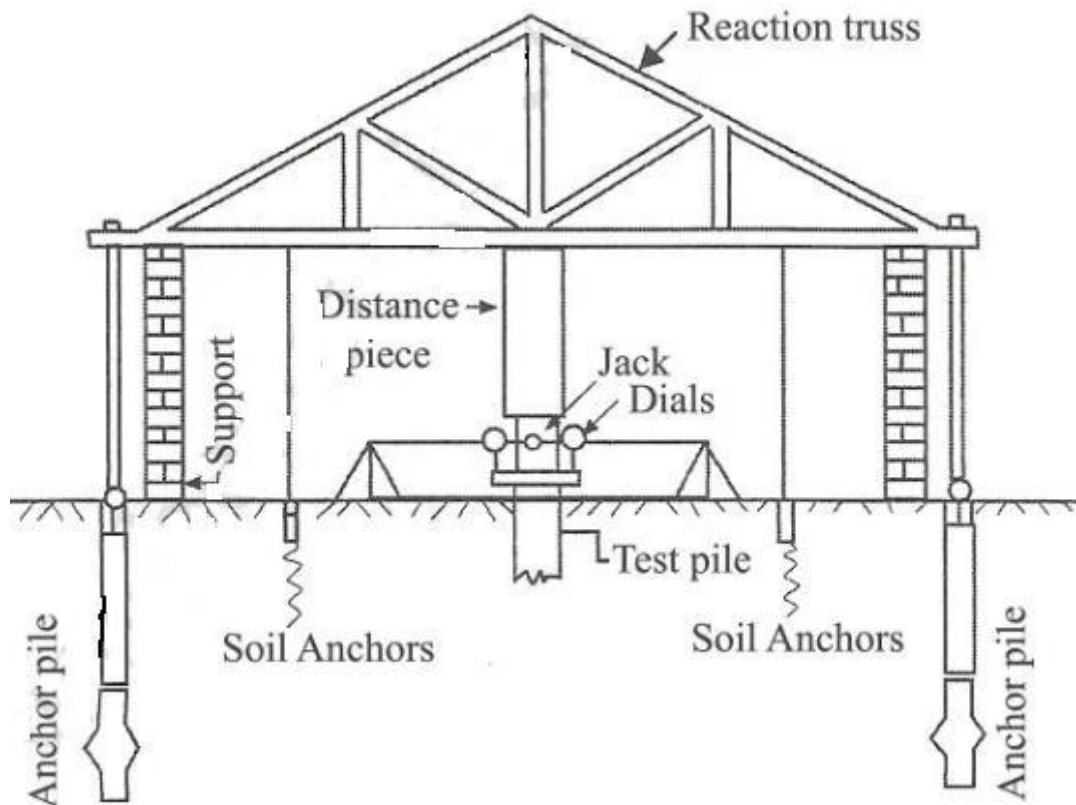


Fig 3 Jack Loading reaction by anchor

[Fig 3 <https://bestengineeringprojects.com/pile-load-test/>]



The following the procedure of pile load test,

- The sets up for the load test on a pile consist of two anchor piles provided with an anchor girder or a reaction girder at their top as shown in Figure above.
- The test pile is generally installed between two anchor piles in such a manner in which the foundation piles are to be installed.
- The test pit should be at least $3B$ or 2.5 m clear from the anchor piles.
- The load is applied through a hydraulic jack resting on the reaction girder. The measurements of the settlement of the pile are recorded with the help of three dial gauges, with respect to a fixed reference mark.
- The test is conducted after a period of 3 days after installation of the test pile in sandy soils, and after a period of one month after the installation of the test pile in silts and soft clays.
- This is because by driving the test pile the soil properties are altered and with the passage of time much of the original properties are restored.
- The load is generally applied in an equal amount of increment and that is about one fifth of the allowable load. Settlements should be recorded with three dial gauges.
- Each load increment is kept for sufficient time till the rate of settlement of the pile becomes less than 0.02 mm per hour.
- Each load increment is maintained till the rate of movement of the pile is not more than 0.1 mm per hour in sandy soils and 0.02 mm per hour in clay soils or a maximum of two hours (IS: 2911 — 1979).
- For each load increment settlements are observed at 0.5, 1, 2, 4, 8, 12, 16, 20, 60 minutes.
- The test pile is loaded until ultimate load is reached.
- The test load is increased to a value 2.5 times the estimated allowable load or to a load whichever failure occurs earlier.
- The load is removed in the same decrements at 1 hour interval and the final rebound is recorded after 24 hours after the entire load has been removed.

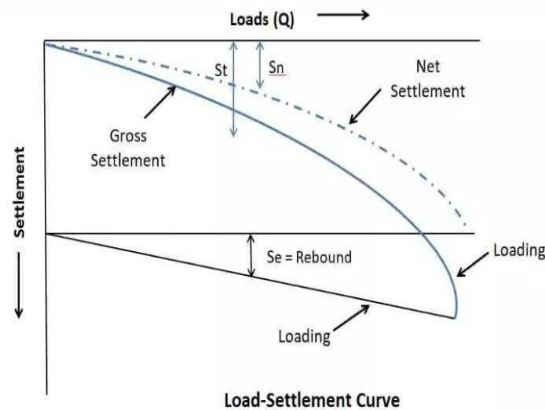


Fig 4 Load Settlement curve

[Fig4 <https://civiconcepts.com/blog/pile-load-test>]

- The ultimate load is clearly indicated by the load settlement curve approaching vertical.
- If the ultimate load cannot be obtained from the settlement curve the allowable load is taken as follows,
- one-half to one third the final load which cause settlement equal to 10% of the pile diameter.
- Two third of the final load which cause a total settlement of 12mm.
- Two third of final load which causes a net settlement (residual settlement after the removal of load) of 6mm.

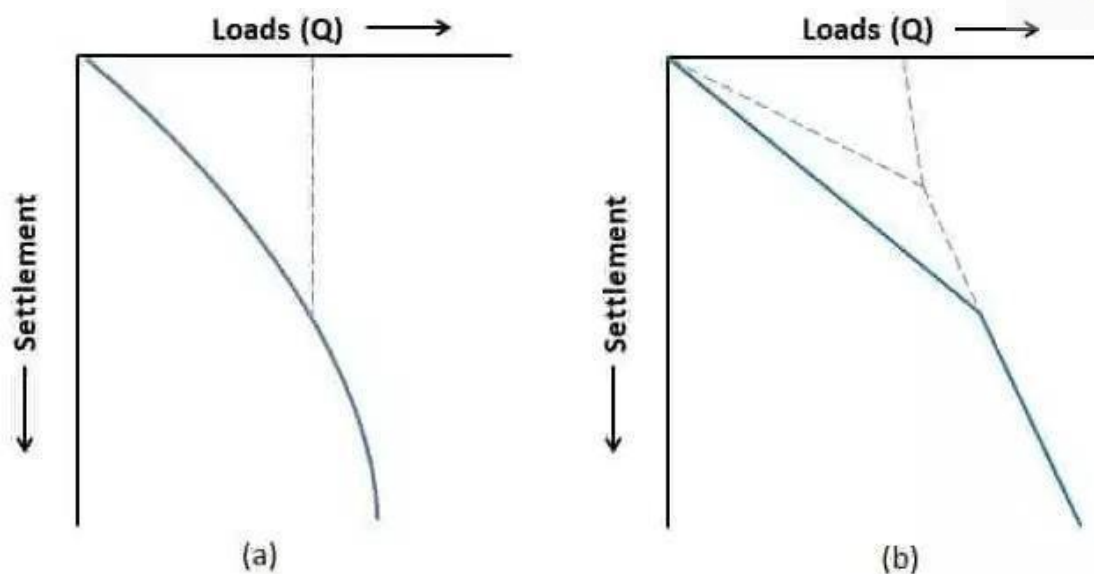
For given load, the net settlement (S_n) is given by, $S_n = S_t - S_e$

Where, S_n = Net Settlement

S_t = Total Settlement or Gross Settlement

S_e = Elastic Settlement (rebound)

- Fig. shows two loads-settlement curves obtained from a pile load tests on two different soils. The ultimate load Q_u may be determined as the abscissa of the point where the load settlement curve changes to a steep straight line.



Determination of Ultimate Load from Load Settlement Curve for Pile

Fig5 Settlement curve for pile

[Fig5 <https://civiconcepts.com/blog/pile-load-test/>]

Causes of Failure of Piles:

Following are the most common causes of failure of piles:

- Absence of statistical data regarding the nature of soil strata through which the pile is to be driven.
- The actual load coming on the pile is more than the design load.
- Bad workmanship in case of cast-in-situ concrete piles.
- Attack by insect etc. on wooden piles.
- Breakage due to over stress especially in case of the timber piles.
- Buckling of piles due to removal of side support, inadequate lateral support, etc.
- Lateral forces (wind, waves, currents, etc.)
- Damage due to abrasion resulting from the absence of suitable protective covering.
- Improper choice of types of piles.
- Improper choice of the method of driving the pile.



- Misinterpretation of the results obtained during the pile load test.
- Wrongful use of pile formula for determining its load-bearing capacity.



- Displacement Piles
- Non Displacement Piles

1. Displacement Piles

Methods of pile driving can be categorized as follows:

- Dropping weight
- Vibration
- Jacking (restricted to micro-piling)
- Jetting

2. Non Displacement Piles

- Under reamed Pile

Drop Hammer Method of Pile Driving:

A hammer with approximately the weight of the pile is raised a suitable height in a guide and released to strike the pile head. This is a simple form of hammer used in conjunction with light frames and test piling, where it may be uneconomical to bring a steam boiler or compressor on to a site to drive very limited number of piles.

There are four types of drop hammers:

1. Single-acting steam or compressed-air hammers
2. Double-acting pile hammers
3. Diesel hammer
4. Steam hammer

1. Single-acting steam or compressed-air:

Single-acting steam or compressed-air comprise a massive weight in the form of a cylinder. Steam or compressed air admitted to the cylinder raises it up the fixed piston rod. At the top of the stroke, or at a lesser height which can be controlled by the operator, the steam is cut off and the cylinder falls freely on the pile helmet.

2. Double-acting pile hammers

Double-acting pile hammers can be driven by steam or compressed air. A piling frame is not required with this type of hammer which can be attached to the top of the pile by leg-guides, the pile being guided by a timber framework. When used with a pile frame, back guides are bolted to the hammer to engage with leaders, and only short leg-



Double-acting hammers are used mainly for sheet pile driving.



Figure-1: Pile driving using hammer

[Fig1 <https://www.dreamstime.com/photos-images/drop-hammer.html>]

3. Diesel pile hammer

A modern diesel pile hammer is a large two-stroke diesel engine. The weight is the piston, and the apparatus which connects to the top of the pile is the cylinder. Pile driving is started by raising the weight; usually a cable from the crane holding the pile driver. This draws air into the cylinder. Diesel fuel is injected into the cylinder. The weight is dropped, using a quick-release. The weight of the piston compresses the air/fuel mixture, heating it to the ignition point of diesel fuel.

4. Steam Hammer

Air/Steam Hammer can be classified as either single-acting or double-acting. These external combustion hammers use an external power source such as air compressors or steam boilers to power the ram upward or downward.

Single –acting /Steam hammers allow air or steam to raise the movable portion of the hammer and allows it to free-fall. This type of impact hammer can be readily used in all soil conditions, with an average of 50-60 blows per minute.

Double–acting /steam hammers allow air or steam to raise the ram of the hammer, and adds additional energy during down stroke for a higher frequency of blows (90-150 per minute). The hammer applies additional air or steam pressure to the top of the piston to enable shorter strokes.



sides of the pile. Vibratory methods are best suited to sandy or gravelly soil. **Jetting:** to aid the penetration of piles in to sand or sandy gravel, water jetting may be employed. However, the method has very limited effect in firm to stiff clays or any soil containing much coarse gravel, cobbles, or boulders.

Jetting

Water jetting can be used to assist the infiltration of piles into sediment or gravel-filled soil. The method has, however, very limited success against stiff clays or any land that contains a lot of coarse gravel, cobbles, or pebbles.

Non Displacement Pile:

1. Under reamed Pile:

- Under reamed piles are bored cast in-situ concrete piles having one or more bulbs formed by enlarging the bore hole for the pile stem by an under reaming tool.
- These piles find applications in widely varying situations in different types of soils where foundation are required to be taken down to a certain depth to avoid the undesirable effect of seasonal moisture changes as in expansive soils or to reach strata or to obtain adequate capacity for downward, upward and lateral loads or to take the foundations below scour level and for moments.

Types of Under reamed pile:

1. Single-bulb cast-in-situ pile,
2. Multiple-bulb pile.

- A single bulb attached at the bottom end of a pile is called a single bulb under reamed pile. The pile with two bulbs is called a double bulb under reamed pile. And the pile with more than two bulbs is called a multiple bulb under reamed pile.
- Generally, the diameter of under –reamed bulbs is kept equal to 2.5 times the diameter of pile stem.



where the ground movements become negligible.

- In shallow depths of expansive soils and other poor soils depending upon the load requirements the length may be reduced and the piles may be taken up to at least 50 cm in stable zone pile length may be increased for higher loads.
- The diameter manually bored piles range from 20 cm to 37.5 cm.
- The spacing of the piles shall be considered in relation to the nature of the ground, the types of piles and the manner in which the piles transfer the loads to the ground.
- Generally, the center to center spacing for under-reamed piles should not be less than $3 D_u$.
- It may be reduced to $1.5 D_u$ when a reduction in load carrying capacity of 10 % should be allowed.
- For the spacing of $2 D_u$ the bearing capacity of pile group may be taken equal to the number of piles multiplied by the bearing capacity of individual pile.
- If the adjacent piles are of different diameters, an average value for spacing should be taken.
- The maximum spacing of the under-reamed pile should not normally exceed $2 \frac{1}{2}$ meters so as to avoid heavy capping beams.
- In building, the piles should generally be provided under all wall junctions to avoid point loads on beams.
- Position of intermediate piles are then decided trying to keep the door opening fall in between two piles as far as possible.
- In double and multi-under-reamed piles of size less than 30 cm dia., the center-to-center vertical spacing between the two under reams may be kept equal to $1.5 D_u$ while for piles of 30 cm and more this distance may be reduced to $1.25 D_u$. the upper bulb should not be less than 1.5m or $2 D_u$ whichever is greater.



- Under reamed piles can be made at a better also, for sustaining large lateral loads, thus making them suitable for tower footing, retaining walls and abutments. They have also been found useful for factory buildings, machine foundations and transmission line towers and poles.
- In black cotton soils and other expansive soils, the under reamed pile anchors the structures at a depth where the volumetric changes in soils due to seasonal and other variation is negligible.
- The under reamed pile is nominally reinforced with 10 to 12 mm dia. Longitudinal bars, and 6mm \emptyset rings. A clear cover of 4 cm is provided.

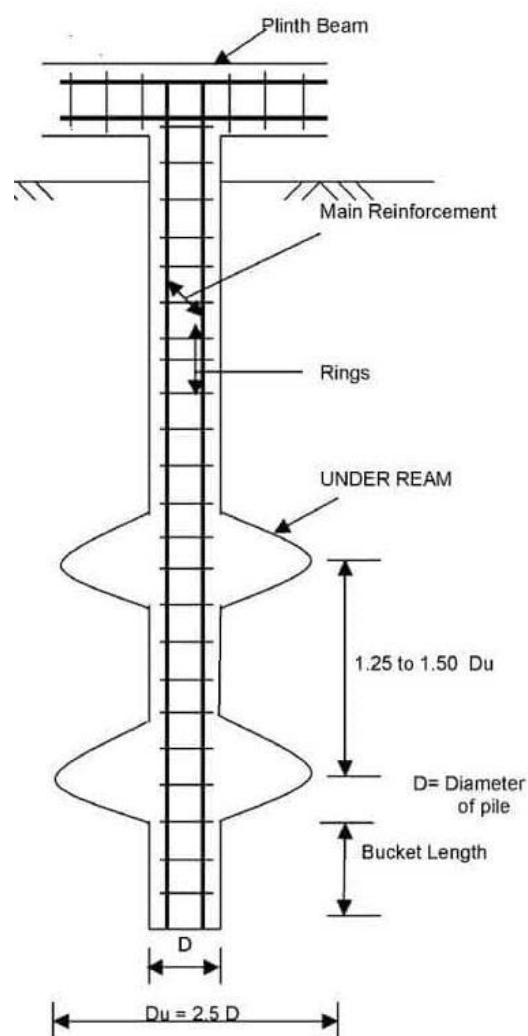
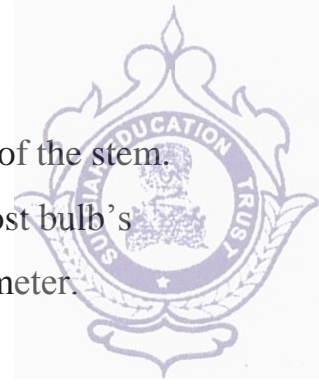


Fig 1 Under reamed pile with two under reamer

[Fig 1 <https://expertcivil.com/under-reamed-pile/>]

Under Reamed Piles Bulb Calculation



The width of the under-reamed bulbs might be 2 to 3 times the width of the stem. The distance between bulbs is 1.25 to 1.5 times the stem breadth. The topmost bulb's distance from the surface should not be less than 2 to 3 times the bulb's diameter.

Procedure:

Under-reamed piles are mostly artificially constructed. Tool for Such piles of construction are given below.

1. Spiral Auger
2. Under – reamer
3. Boring guide

- A spiral auger is used to drill a hole in the ground for an Under reamed piles. Cutter is attached at the end of Auger to easily dig ground. Below the spiral auger, the filling bucket is hung to remove soil. Auger handles are also used to increase depth.
- A special type of cutter is used to make the bulb; the diameter of the bulb can be increased by applying pressure on the handle of the auger.
- Excavation is carried out from the auger only after passing the handle of the auger through a special design made on the head of a tricycle placed on the ground so that the digging of the pile hole is done in the vertical direction only.
- After digging the pile hole and the bulb to the required depth, the auger is taken out and the Case of reinforcement is inserted in the hole. Then Concreting is done.
- All the piles are connected to each other by forming a beam at the head of the pile. The wall is constructed over the beams.

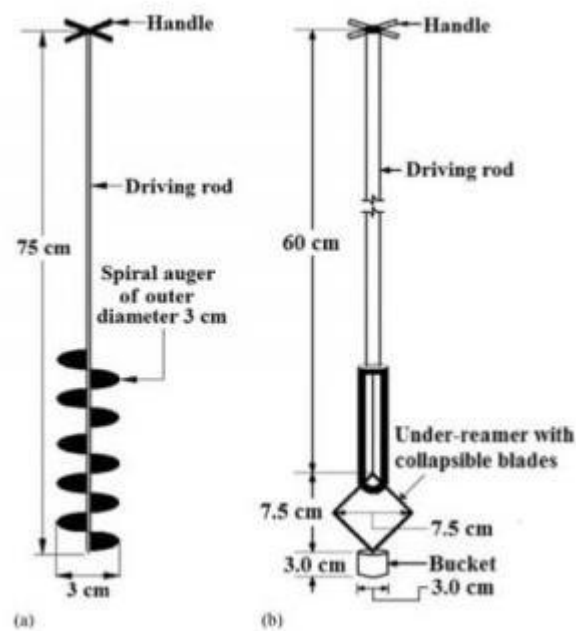


Fig1 Under reamed pile

[Fig1 <https://www.civilengineeringweb.com/2020/07/under-reamed-piles.html>]

Advantages of Under Reamed Piles:

- Such piles are 15 to 20% cheaper than strip footing.
- Less material is required for such a pile.
- No heavy digging is required, so operations can be carried out even in the rainy season.
- There is no need for back filling in such piles.
- Shoring is not required.
- Dewatering is not required.

Disadvantages of Under Reamed Piles:

- Required great workmanship.
- Skills required for placing of such type of piles.
- Maintaining verticality in ground is difficult. Because some time such pile are driven by hand-operated machine and it is very difficult.
- Required strict and regular supervision with great quality control.

Clayey soils:

$$Q_u = A_p N_c C_p + A_a N_c C_a' + C_a' A_s' + \alpha C_a A_s$$

Q_u = Ultimate bearing capacity



$A_p = C/s$ area of pile

$N_c =$ Bearing capacity factor

$C_p =$ Cohesion of the soil

$$A = \frac{\pi}{4} (D^2 - D_u^2)$$

$C_a =$ Average cohesion of soil around the bulb

$A_s =$ Surface area of the cylinder

$A =$ reduction coefficient (usually 0.5 for clays)

Sandy soils:

$$Q_u = \frac{\pi}{4} (D^2 - D_u^2) \left[\sum_{r=1}^{r=n} D_u n \cdot \gamma N_\gamma + \gamma N_q \right] + \frac{\pi}{4} (D^2 - D_u^2) \left[\sum_{r=1}^{r=n} D_u \gamma N_\gamma + \gamma D_f N_q \right] + \frac{1}{2} \pi D \gamma K \tan \delta (d_1 + d_f + d_n)$$

$D_u =$ Diameter of under reamed bulb

$D =$ diameter of stem

$D_f =$ Depth of pile

Pile settlements

Pile settlement can be estimated as follows.

Compute the average pile axial force in each segment of length L , average cross-section & A_{av} and shaft modulus of elasticity E_p from the pile butt to point. That is.

$$\Delta H_{s,s} = \frac{P_{av} \times \Delta L}{A_{av} \times E_p}$$

and sum the several values to obtain the axial total compression

$$\Delta H_a = \sum \Delta H_{s,s}$$

Compute the point settlement using the equation below

$$\Delta H_{pt} = \Delta q D \left(\frac{1 - \mu^2}{E_s} \right) m I_s I_F F_1$$



Where,

$$mI_s = 1$$

I_f = Fox embedment factor, with values as follows:

$$I_f = 0.55 \text{ if } L/D \leq 5$$

$$I_f = 0.5 \text{ if } L/D > 5$$

D = diameter of the pile μ = Poisson's ratio

q = bearing pressure at point = input load / $A_p E_s$ = Young's modulus

SPT: $E_s = 500 (N+15)$

CPT: $3-6 q_c$

F_1 is the reduction factor as follows

0.25 if the axial skin resistance reduces the point load $P_p \leq 0$

0.5 if the point load $P_p > 0$

0.75 if the point bearing

Problems:

1. In a load test conducted at a depth of 1 meter below ground with a square plate of 30cm side on a granular soil, load required to cause 25mm settlement was 72 kN.

Find out the size of a square column footing which will be having its base at a depth of 2.5 m below ground level and is required to take a load of 1750kN. The settlement of the footing is restricted to be 10mm only and factor is to be 3 only. Unit weight of soil 19kN/m². $N_c = 12$ and $N_r = 6$.

For 25mm of settlement, allowable load was 72kN for square plate of 30cm side.

$$\therefore \text{Allowable pressure} = \frac{7.2}{0.3} \times 0.3 = 800 \text{KN/m}^2$$

\therefore Allowable pressure for settlement S_q —

$$q_u = \frac{10}{25} \times 800 = 320 \text{KN/m}^2$$

For square footing, $q_u = 1.3 C N_c + \gamma D N_q + 0.4 \gamma B N_v$

$$\therefore 320 = (1.3 C \times 25) + (19 \times 1 \times 12) + (0.4 \times 19 \times B \times 6)$$

Solving, $B = 2.6$ m

\therefore Size of footing = 2.6m x 2.6m.



2. Complete the settlement of a rigid footing 2.6m x 2.6m carrying a load of 1800kN, supported on a sandy soil, if a plate load test gives a settlement of 8mm under a load of 320 kN/m². Size of plate 30cm x 30cm.

Given:

Size of footing = 2.6 x 2.6m

Load = 1800 kN

$$= 1800 / 2.6 \times 2.6$$

$$= 266.27 \text{ kN/m}^2$$

Settlement of plate $\rho_p = 8 \text{ mm}$

Plate size $B_p = 0.3 \times 0.3$

Load on the plate = 320 kN/m²

$$\begin{aligned} \rho_f &= \rho_p \left[\frac{B(B_p + 0.3)}{B_p(B + 0.3)} \right]^2 \\ &= 0.8 \left[\frac{2.6(0.3 + 0.3)}{0.3(2.6 + 0.3)} \right]^2 \\ &= 25.72 \text{ mm (for } 320 \text{ kN/m}^2 \text{ loading.)} \end{aligned}$$

∴ Settlement of footing for 266.27 kN/m²

$$= \frac{266.27}{320} \times 25.72$$

$$= \mathbf{21.4 \text{ mm}}$$

Group capacity by different methods:

1. Converse – Labarra formula:

$$\eta_g = 1 - \frac{\theta}{90^\circ} \left[\frac{(n-1)m + (m-1)n}{mn} \right]$$

m = number of rows

n = number of pile in a row

$$\theta \rightarrow \tan^{-1} \left(\frac{d}{s} \right)$$

d = Diameter of pile

S = spacing of pile

2. Seiler -Keeny formula:

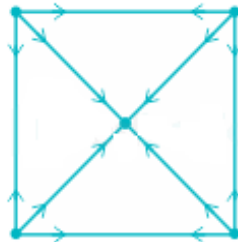


$$\eta_g = \left[1 - 0.479 \left(\frac{s}{s^2 - 0.093} \right) \left(\frac{m + n - 2}{m + n - 2} \right) \right] + \frac{0.3}{m + n}$$

S=Average spacing from center to center

3.Feld’s rule:

It is reduced by 1/16 on account of the nearest pile in each diagonal or straight row. The spacing of the pile is not considered.



Efficiency of corner pile(3adjacent pile), $\eta = \frac{16-3}{16} = 81.25\%$

Efficiency of central pile(3adjacent pile), $\eta = \frac{16-4}{16} = 75\%$

$$Average = \frac{(81.25 \times 4) + (75 \times 1)}{5} = 80\%$$

Group Action

Piles are generally used in groups with a common pile cap. A group may consist of two or three, or as many as ten to twelve piles depending on the design requirement. The load carrying capacity of a group of piles is given by

$$(Q_u)_g = Nq_u n$$

where,

$(Q_u)_g$ = Load carrying capacity of pile group

N = number of piles

q_u = allowable load per pile

n = group efficiency

- Its value for bearing or friction piles at sites where the soil strength increases with depth is found to be 1.
- For friction piles in soft clays the value on n is less than 1. The actual value of n depends on soil type, method of pile installation, and pile spacing.



- When piles are driven in loose, sandy soils, the soil is densified during driving, and $n > 1$ in such cases.
- It has been observed that if the spacing between piles is more than 2.5 times the pile diameter, the group efficiency is not reduced.
- The large pile to pile spacing will increase the overall cost of construction. The reduction in load capacity due to the group effect can be estimated empirically.
- The use of Feld's rule is probably the simplest. It states that the load capacity of each pile in a group is reduced by 1/16 on account of the nearest pile in each diagonal or straight row.

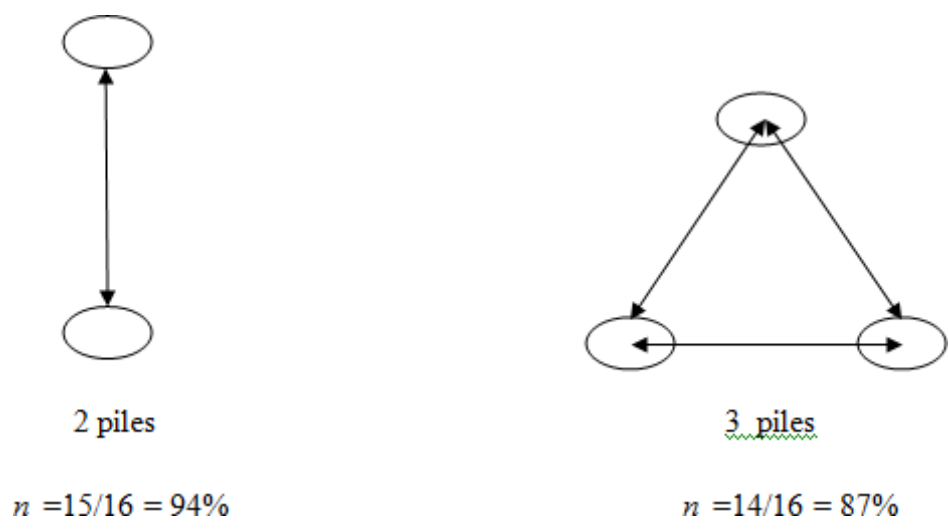


Fig. 6: Group action of piles- Feld's rule

[Fig6 <https://aits-tpt.edu.in/wp-content/uploads/2018/08/pile-foundations-theory.pdf>]

A group of piles may fail as a block, i.e., by sinking into the soil and rupturing it at the periphery of the group Fig.7.

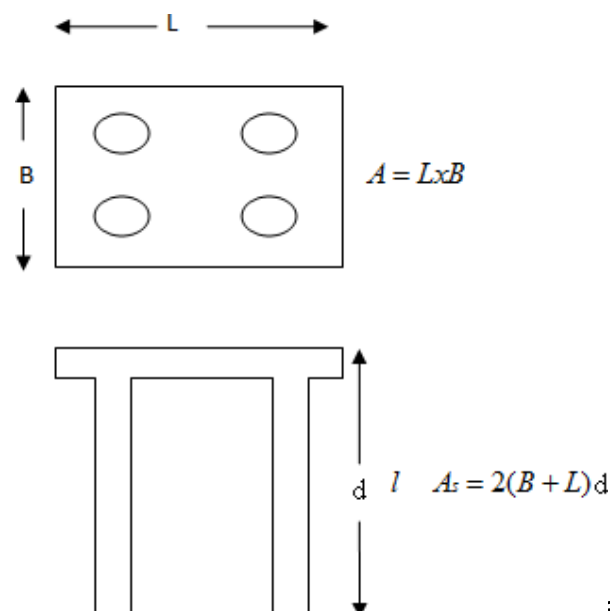




Fig7: Failure of a pile group as a block

[Fig 4 <https://aits-tpt.edu.in/wp-content/uploads/2018/08/pile-foundations-theory.pdf>]

Ultimate Load Carrying Capacity for the Pile Group

The ultimate load carrying capacity for the pile group taken as a block is given by

$$(Q_u)_g = C_u N_c A_b + C_u A_p$$

where A_p and A_b are the area of the base and the surface area of block. i.e. $A_b = LB$ where, L and B are the dimensions of the pile cap.

A_p is the perimeter of the block times the embedded length of the pile.

Efficiency of a Pile Group:

The efficiency of a pile group is defined as

$$\eta_g = \frac{\text{Ultimate bearing capacity of the group}}{n \times \text{ultimate bearing capacity of single pile in the group}}$$

where n = number of piles in the group

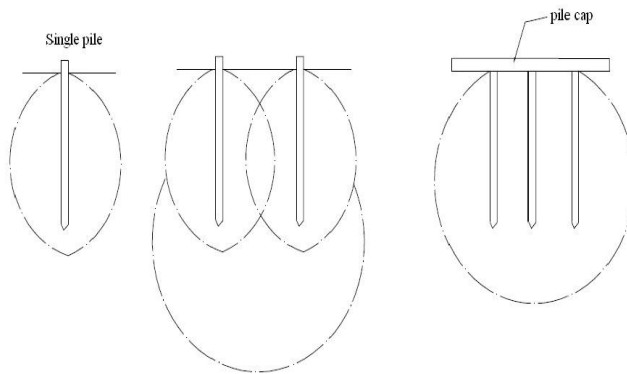


Fig. 8: Group action of Piles

[Fig8 <https://aits-tpt.edu.in/wp-content/uploads/2018/08/pile-foundations-theory.pdf>]

Settlement of Pile Groups

Due to group action, both immediate and consolidation settlement values of a pile group are greater than those for a single pile.

For bearing piles the total foundation load is assumed to act at the base of the piles on an imaginary foundation of the same size as the plan of the pile group as show in Fig 9

For friction piles it is virtually impossible to determine the level at which the structural load is effectively transferred to the soil. The level used in design is at a depth of two-thirds the penetration depth.

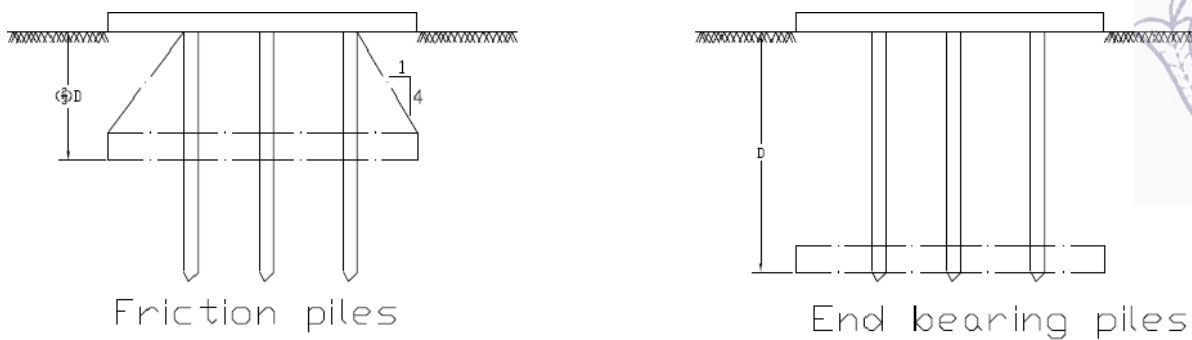


Fig.9: Equivalent foundations for pile

[Fig 9 <https://aits-tpt.edu.in/wp-content/uploads/2018/08/pile-foundations-theory.pdf>]

Uplift Resistance of Pile:

Clay:

$$Q_{uu} = W_p + \alpha C A_s$$

W_p =weight of pile

A =adhesion factor

C =average undrained shear strength of clay along the pile shaft.

Negative Skin Friction:

Negative skin friction is usually a downward shear drag acting on a pile or pile group because of downward movement of surrounding soil relative to the piles. This shear drag movement are anticipated to occur when a pile penetrates into compressible soil layer that can consolidate. It is reported that, a small relative movement between the soil and the pile of around 10 mm may be adequate for the full negative skin friction to materialize. Moreover, the time of ending the negative skin friction of piles is estimated to be around 2 years and the degree of consolidation of the soft soils reaches 90%. Finally, this article presents different aspect of negative skin friction on piles and pile group.

Factors that cause negative skin friction on piles and pile group:

- Newly placed fill material on compressible soil before the completion of consolidation.
- If fill material is loose cohesion less soil
- When fill material is deposited over layer of soft soil or peat.



- Lowering groundwater which increases the effective stress causing consolidation of soil with resultant settlement and friction force being developed on the pile.
- Effect of negative skin friction on piles and pile groups
- Negative skin friction contributes to the uneven settlement of piles or pile group.
- For piles in compressible soils where pile capacity is contributed by both point resistance and shaft adhesion, the problem of negative skin friction should be considered a settlement problem.
- In bearing piles where the settlement of the pile is negligible, negative skin friction becomes a pile capacity problem.

1. Negative skin friction in single pile

The negative skin friction in single piles can be computed using the following expressions:

a. Negative skin friction of piles in cohesive soil

$$F_n = PL_c C_a \text{ --- (1)}$$

Where: F_n : negative skin friction

P: perimeter of the pile

L_c : pile length in compressible soil

C_a : unit adhesion and can be computed using equation 2

$$C_a = \alpha C_u \text{ --- (2)}$$

Where: α : adhesion factor

C_u : Undrain Cohesion of the compressible layer

b. Negative skin friction of piles in cohesion less Soils:

$$F_n = 0.5 PL_c^2 \gamma K \tan \delta \text{ --- (3)}$$

Where:

K: lateral earth pressure coefficient

γ : unit weight of soil

δ : angle of friction between pile and soil, which may vary from $1/2$ to $2/3$.

2. Negative skin friction on pile groups

Negative skin friction in pile groups equal to the greater of equation 4 and equation 5:



$$F_{ng} = nF_n \text{ --- (4)}$$

$$F_{ng} = C_u L_c P_g + \gamma L_c A_g \text{ --- (5)}$$

Where:

n: number of piles in the group

P_g: perimeter of the group

γ: unit weight of the soil within the pile group up to a depth

A_g: area of pile group within the perimeter P_g

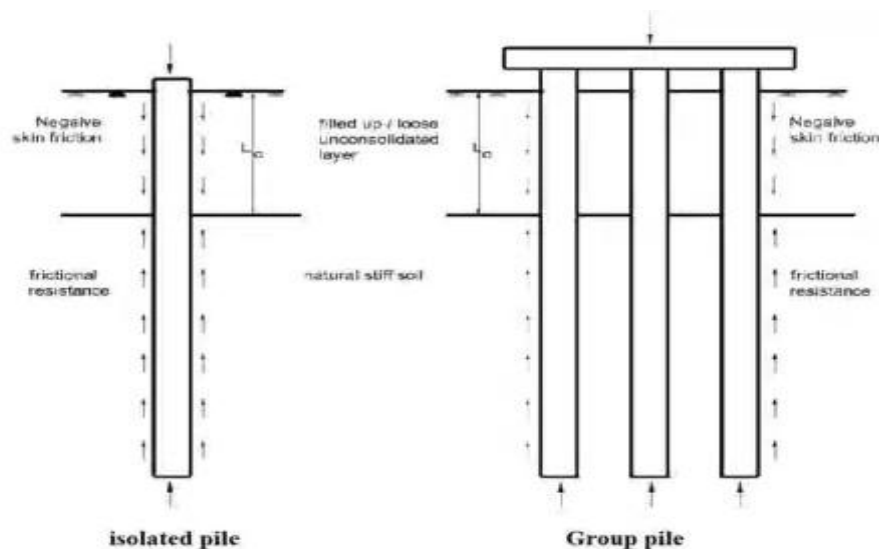


Fig.1: Negative skin friction on piles and group pile

[Fig1<https://theconstructor.org/geotechnical/negative-skin-friction-piles/3376/>]

The effect of negative skin friction on the factor of safety with respect to the ultimate load capacity of a pile or a pile group

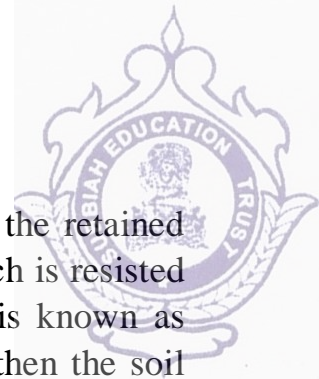
The influence of negative skin friction on safety factor with respect to ultimate load carrying capacity of piles or pile group is considered by introducing the factor of safety, so: ultimate load capacity of a pile or a group of pile=(working load + negative skin friction load)(Factor of safety) Where it is anticipated that negative skin friction would impose undesirable, large downward drag on a pile, it can be eliminated by providing a protective sleeve or a coating for the section which is surrounded by the settling soil.

If a soil subsides or consolidates around a group of piles these piles will tend to support the soil and there can be a considerable increase in the load on the piles.

The main causes for this state of affairs are

- Bearing piles have been driven into recently placed fill
- Fill has been placed around the piles after driving
- As a result of remolding of clay during driving of the pile





5.1 Lateral Earth Pressure: Types and Derivation

When a soil mass is retained at a higher level by a retaining wall, the retained mass of the soil tends to slide and assume a flat slope for equilibrium, which is resisted by the retaining wall. This exerts pressure on the retaining wall, which is known as lateral earth pressure. Usually, the retaining wall is constructed first and then the soil behind the wall is backfilled; hence, the retained soil is often called backfill. The back of the wall is either vertical or slightly inclined to the vertical and the lateral earth pressure is slightly inclined to the horizontal due to wall friction and inclination of the back of the wall.

The magnitude of the lateral earth pressure depends on the following factors:

- i. Type and extent of the movement of the wall and the resulting horizontal strain in the backfill.
- ii. Properties of the backfill material, including the density (γ), cohesion (c), and angle of shearing resistance (ϕ).
- iii. Groundwater conditions in the backfill such as depth of water table and provision for drainage.
- iv. Degree of roughness of the surface of the back of the retaining wall.
- v. Slope of the back of the retaining wall.
- vi. Depth of the retaining wall, that is, the height of the backfill to be retained.
- vii. Inclination of the backfill surface with the horizontal.
- viii. Additional loads on the backfill surface such as traffic loads or additional constructions, if any.

Types of Lateral Earth Pressure:

There are three basic types of lateral earth pressure.

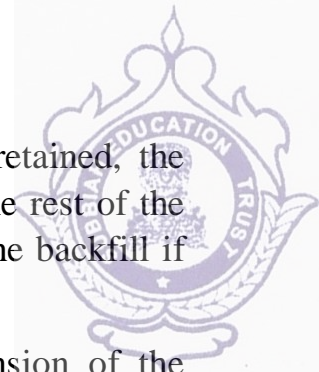
They are:

1. Active earth pressure.
2. Passive earth pressure.
3. Earth pressure at rest.

These three basic types of lateral earth pressures are discussed below:

1. Active Earth Pressure:

Figure 1(a) shows a retaining wall of height H with a backfill having a horizontal surface. If the retaining wall were not there, the backfill would assume a stable flat slope. We know that cohesion less soils assume a stable slope equal to the angle of



internal friction without any lateral support. Hence, when a backfill is retained, the wedge of soil above a certain slope tends to slide and move away from the rest of the backfill for equilibrium. This tends to push or rotate the wall away from the backfill if the wall is free to move or rotate.

The movement of the wall away from the backfill causes expansion of the backfill, resulting in stress release, thereby reducing the lateral earth pressure. Thus, the more is the movement of the wall away from the backfill, the more is the horizontal strain in the backfill, in the form of expansion, and the less is the lateral earth pressure. Initially when the wall is in a state of rest, a typical element of backfill at any depth is subjected to vertical stress due to self-weight of soil above the element and lateral earth pressure in the horizontal direction. The state of stress for the soil element is represented by Mohr's circle (I) in Fig. 1(b), where OB is the vertical stress and OA1 is the lateral earth pressure at rest.

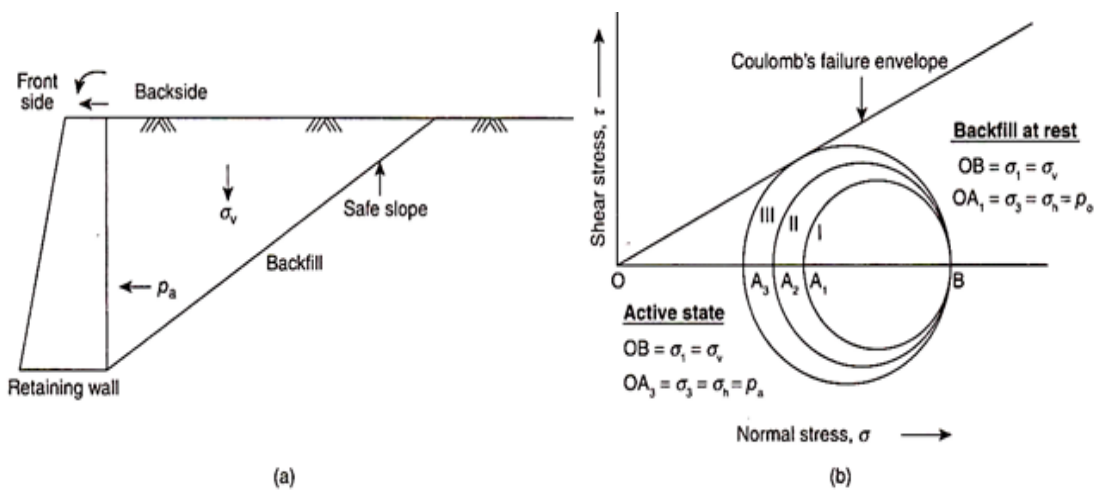


Fig 1(a) Retaining wall with a backfill Fig(b) Mohr's circle showing the gradual decrease of lateral earth pressure in active case

[Fig 1 <https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925>]

When the lateral earth pressure tends to push or rotate the wall away from the backfill, the movement of the wall away from the backfill causes expansion of the backfill, resulting in stress release, thereby reducing the lateral earth pressure. Thus, the more is the movement of the wall away from the backfill, the more is the horizontal strain in the backfill, in the form of expansion, and the less is the lateral earth pressure.

This is shown in Fig.1 (b), by Mohr's circle (II), in which $\sigma_h = \sigma_3 = OA_2$ is the reduced lateral earth pressure while the vertical stress, equal to $\sigma_v = \sigma_1 = OB$, remains constant. The decrease in the lateral earth pressure thus causes increase in the diameter of Mohr's circle, causing it to approach the Coulomb's failure envelope.

The decrease in the lateral earth pressure due to movement of wall away from the backfill and consequent expansion and stress release continues until Mohr's circle touches the Coulomb's failure envelope of the backfill material. When Mohr's circle



touches the failure envelope, as shown by Mohr's circle (III) in Fig.1(b), the backfill material is on the verge of failure (limiting equilibrium) and no further decrease in the lateral earth pressure can take place. The minimum lateral earth pressure exerted on the retaining wall, when the wall moves away from the backfill, and the backfill material is in the limiting equilibrium, is known as active earth pressure.

When the wall moves away from the backfill, the backfill is said to be in the active state and the minimum lateral earth pressure exerted by the backfill in the active state in its limiting equilibrium condition is known as active earth pressure. Active earth pressure occurs when Mohr's circle of stresses at any point in the backfill touches the Coulomb's failure envelope.

Active earth pressure is denoted by the symbol p_a , and its units are kN/m^2 , t/m^2 , or kgf/cm^2 . All retaining walls, which are free to move or rotate, are by default subjected to active earth pressure and are designed to resist the same.

2. Passive Earth Pressure:

All retaining walls are usually not placed on the ground surface on the front side but are laid at some depth. Hence, the retaining wall has soil to some depth on its front side. When the wall moves away from the backfill due to active earth pressure, it actually moves towards the soil on the front side.

The movement of the wall is resisted by the front soil and exerts a lateral pressure on the wall, in a direction opposite to that of active earth pressure, as shown in Fig.2. Also, the movement of the wall towards the front soil causes compression of the soil, which, in turn, increases the lateral pressure from the front soil.

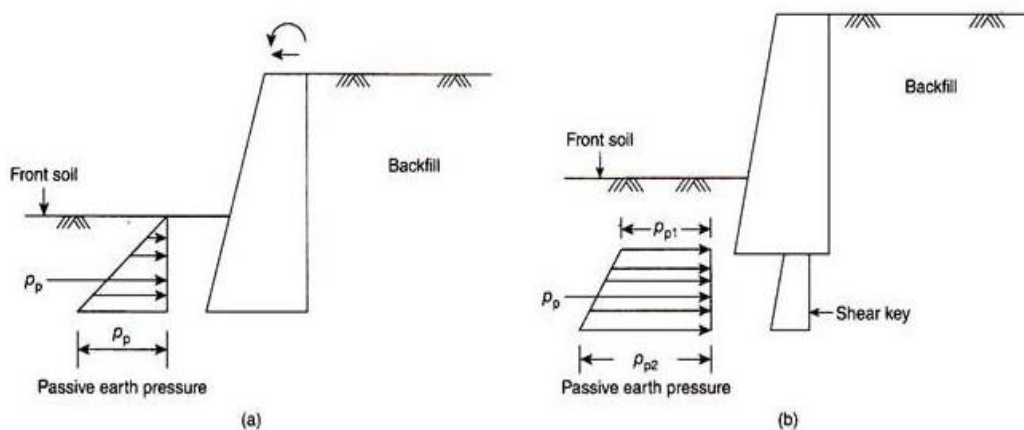
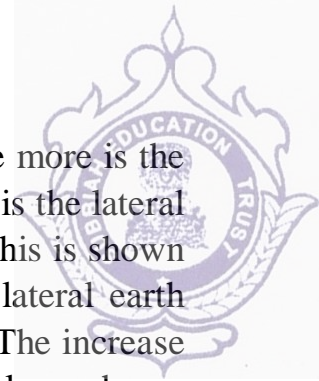


Fig 2 Practical cases of passive earth pressure a) In front of a retaining wall and b) on shear key below a retaining wall

[Fig 2 <https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925>]



Thus, the more is the movement of the wall toward the front soil, the more is the horizontal strain in the front soil, in the form of compression, and the more is the lateral earth pressure from the front soil opposite to that of active earth pressure. This is shown in Fig.3, by Mohr's circle (II), in which $\sigma_h = \sigma_3 = OA_2$ is the increased lateral earth pressure while the vertical stress, equal to $\sigma_v = \sigma_1 = OB$, remains constant. The increase in the lateral earth pressure causes decrease in the diameter of Mohr's circle as shown by Mohr's circles (II) and (III), and Mohr's circle reduces to a point, as represented by points A_4 and B, which become concurrent.

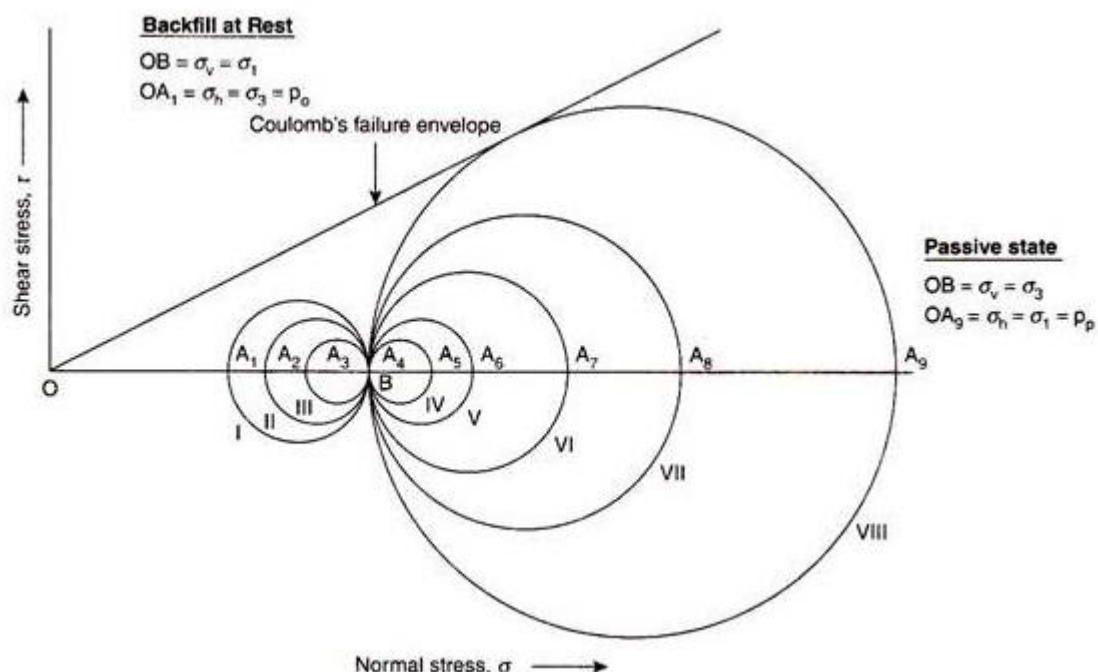


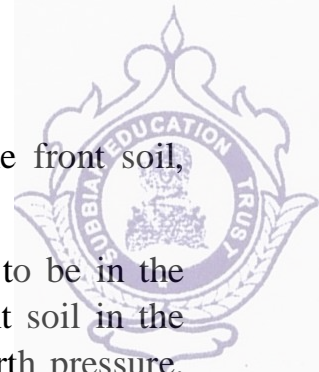
Fig 3 Mohrs circle showing the gradual decrease and then increase in lateral earth pressure in the passive case

[Fig 3 <https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925>]

Further increase of the lateral earth pressure from the front soil makes it higher than the vertical stress. At this stage, the lateral earth pressure becomes the major principal stress and the vertical stress becomes the minor principal stress. This is shown by Mohr's circles (IV), (V), (VI), etc., causing again an increase in the diameter of Mohr's circle.

The increase in the diameter of Mohr's circle leads it to approach the Coulomb's failure envelope. The increase in the lateral earth pressure due to the movement of wall towards the front soil and the consequent compression continues until Mohr's circle touches the Coulomb's failure envelope of the front soil.

When Mohr's circle touches the failure envelope, as shown by Mohr's circle (VIII) in Figure 3, the front soil is on the verge of failure (limiting equilibrium) and no further increase in the lateral earth pressure can take place. The maximum lateral earth



pressure exerted on the retaining wall, when the wall moves towards the front soil, while it reaches its limiting equilibrium, is known as passive earth pressure.

When the wall moves towards the front soil, the front soil is said to be in the passive state and the maximum lateral earth pressure exerted by the front soil in the passive state in its limiting equilibrium condition is known as passive earth pressure. Passive earth pressure occurs when Mohr's circle of stresses at any point in the front soil touches the Coulomb's failure envelope.

Another practical example of passive earth pressure is the case of shear key provided below the base of a retaining wall. A shear key shown in Fig.3 is provided to improve the stability of the wall against sliding. When the retaining wall moves away from the backfill due to active pressure, the shear key also moves in the same direction but toward the soil below the base of the wall on the front side.

This generates passive earth pressure on the shear key. It is denoted by the symbol p_p , and its units are kN/m^2 , t/m^2 , or kgf/cm^2 . Passive earth pressure is actually a stabilizing force improving the stability of the retaining wall, unlike active earth pressure.

3. Earth Pressure at Rest:

Figure 4 shows a basement retaining wall in which the wall is rigidly fixed to the basement slab. The basement retaining wall is therefore fixed in position and cannot move away from the backfill when subjected to lateral earth pressure. The lateral earth pressure exerted by the backfill on a retaining wall which is fixed in position and cannot move is known as earth pressure at rest.

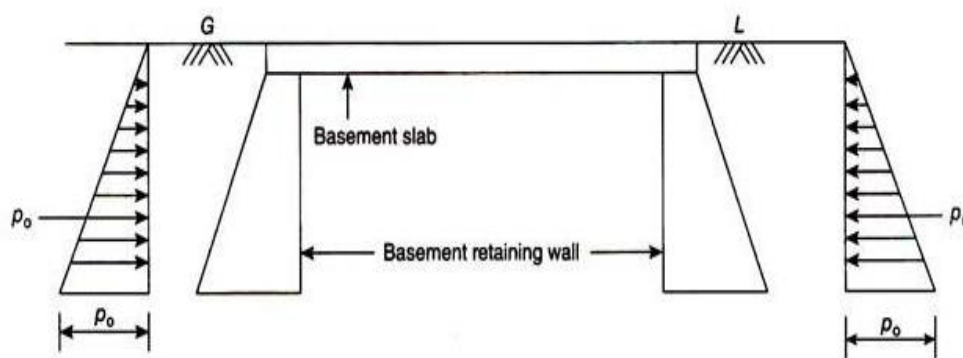


Fig 4 Earth pressure at rest on basement retaining walls

[Fig 4 <https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925>]

It is denoted by the symbol p_o , and its units are kN/m^2 , t/m^2 , or kgf/cm^2 . As the wall does not move, the earth pressure exerted does not cause any lateral strain, and hence, there is no expansion of the backfill and no stress release. Earth pressure at rest is therefore always more than active earth pressure for the same depth of soil.



The abutment of a bridge is rigidly attached to the deck slab of the bridge and is also similarly fixed in position and hence subjected to earth pressure at rest.

Thus, lateral earth pressure exerted on a retaining wall depends on the direction and extent of the movement of the wall. Figure 5 shows the variation in lateral earth pressure on the y-axis as a function of the wall movement. When the wall moves away from the backfill, lateral pressure decreases with the increase in the movement of the wall; the minimum lateral earth pressure exerted on the wall is known as active earth pressure.

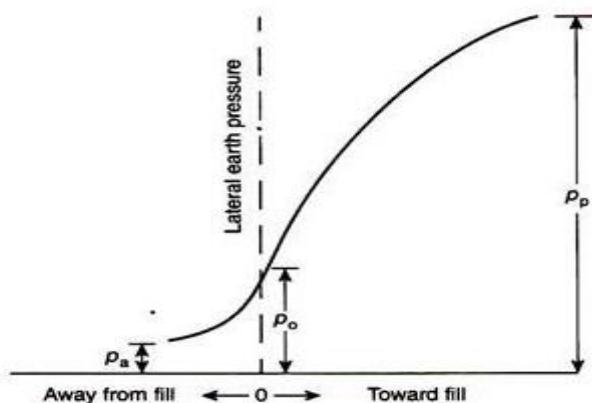


Fig 5 Variation in lateral earth pressure with the movement of the wall relative to the soil

[Fig 5 <https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925>]

When the wall moves toward the soil, the lateral earth pressure generated increases with the increase in the movement of the wall; the maximum lateral earth pressure generated on the wall is known as passive earth pressure. The lateral earth pressure exerted on the wall when the wall is fixed in position is known as earth pressure at rest.

Derivation of Expression for Earth Pressure at Rest:

When a material is subjected to three-dimensional (3D) stresses, σ_x , σ_y and σ_z , along the three coordinate axes, x, y, and z, respectively, the strain along the x-axis can be computed from the principles of mechanics of materials as –

$$e_x = 1/E[\sigma_x - \mu(\sigma_y + \sigma_z)] \text{-----(1)}$$

where e_x is the horizontal strain (in the X-direction), E is the modulus of elasticity of soil, and μ is the Poisson’s ratio. In the case of earth pressure at rest –

$$e_x = 0 \text{-----(2)}$$

$$\sigma_x = \sigma_y = P_0 \text{-----(3)}$$

Substituting these values in Eq. (1), we have –



$$e_x = 1/E [(p_0 - \mu(p_0 + \sigma_z)) = 0$$

$$\text{or } p_0 - \mu(p_0 + \sigma_z) = 0 \Rightarrow p_0 - \mu p_0 - \mu \sigma_z = 0 \Rightarrow p_0(1 - \mu) = \mu \sigma_z$$

$$p_0 = [\mu / (1 - \mu)] \sigma_z \dots (4)$$

$$p_0 = K_0 \sigma_z \dots (5)$$

where K_0 is the coefficient of the earth pressure at rest and σ_z is the vertical stress due to the self-weight of the soil at depth z , where the earth pressure at rest is to be computed

$$K_0 = \mu / (1 - \mu) \dots (6)$$

Equation (6) is valid for elastic materials but not for soils, since soils are not elastic. Table 1 gives typical values of K_0 for different types of backfills, as obtained from actual measurement of earth pressure at rest.

Table 1 Coefficient of earth Pressure at rest for different soil

S.No	Type of soil	K_0
1	Loose Sand	0.5-0.6
2	Dense Sand	0.3-0.5
3	Undrained Clay	0.8-1.1
4	Over consolidated Clay	1.0-3.0

Coefficients of earth pressure - Earth Pressure Coefficient:

On a small unit at depth Z in the back there are two kinds of pressure.

i) Vertical Earth pressure:

The pressure applied in the vertical direction due to the back fill lying above it.

ii) Horizontal Earth pressure:

The pressure applied in the horizontal direction due to backfill is called the horizontal pressure or lateral earth pressure

Coefficient of active earth pressure at rest:

When the retaining wall is at rest then the ratio between the lateral earth pressure and the vertical pressure is called the co-efficient of the earth pressure at rest,

$$K_0 = \text{lateral pressure} / \text{vertical pressure}$$

Co-efficient of active earth pressure:

When the retaining wall is moving away from the backfill the ratio between lateral earth pressure and vertical earth pressure is called coefficient of active earth pressure.



$K_a = \text{lateral pressure} / \text{vertical pressure}$

(or)

It is the ratio of horizontal and vertical principal effective stress when a retaining wall moves away from the retained soil

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2\left(45 - \frac{\phi}{2}\right)$$

Coefficient of passive earth pressure:

When the retaining wall is moving towards the backfill, then the ratio between the lateral earth pressure and the vertical earth pressure is called the coefficient of passive earth pressure.

$K_p = \text{lateral pressure} / \text{vertical pressure}$

(or)

It is the ratio of horizontal and vertical principal effective stress when a retaining wall is forced against a soil mass.

$$k_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \tan^2\left(45 + \frac{\phi}{2}\right)$$



5.2 RANKINE'S THEORY:

Rankine's theory of lateral earth pressure is applied to uniform cohesion less soils only. Later, it was extended to include cohesive soils, by Resal and by Bell. The theory has also been extended to stratified, partially immersed and submerged soils.

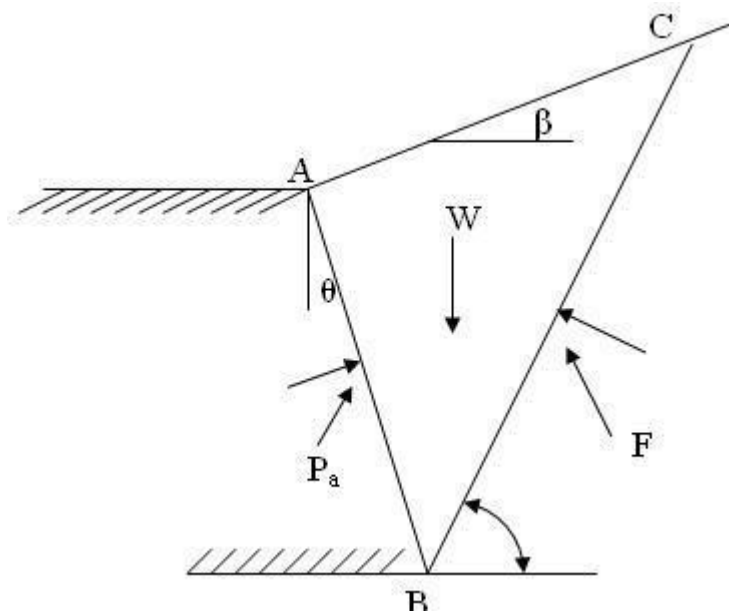
Following are the assumptions of the Rankine's theory:

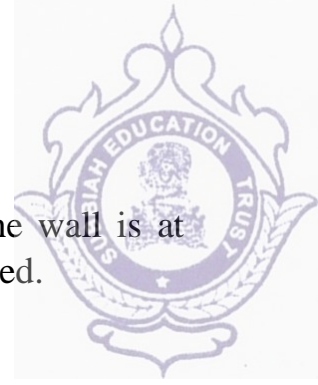
- The soil mass semi-infinite, homogeneous, dry and cohesion less.
- The ground surface is a plane which may be horizontal and inclined
- The back of the wall is vertical and smooth. In other words, there are no shearing stresses between the wall and the soil and the stress relationship for any element adjacent to the wall is the same as for any other element far away from the wall.
- The wall yields about the base and thus satisfies deformation condition for plastic equilibrium.

ACTIVE EARTH PRESSURE:

The following cases of cohesion less back fill will now be considered:

1. Dry or moist backfill with no surcharge.
2. Submerged backfill.
3. Backfill with uniform surcharge.
4. Backfill with sloping surface.
5. Inclined back and surcharge.





1. DRY OR MOIST BACKFILL WITH NO SURCHARGES:

Consider an element at a depth z below the ground surface. When the wall is at the point of moving outwards, the active state of plastic equilibrium is established.

Backfill is Cohesion less soil:

It is derived on the basis of the principal stress relationship on a failure plane.

$$\sigma_1 = 2C \tan \alpha + \sigma_3 \tan^2 \alpha$$

For active

$$\sigma_3 = \sigma_h$$

$$\sigma_1 = \sigma_v$$

Substitute in above equation

$$\sigma_v = 2C \tan \alpha + \sigma_h \tan^2 \alpha$$

Expression for active Pressure:

$$\sigma_v = \gamma Z$$

$$\sigma_h = P_a$$

According to principal stress relationship:

$$C=0$$

$$\sigma_v = \sigma_h \tan^2 \alpha \text{ --- (1)}$$

Substitute σ_v and σ_h value in eqn(1)

$$\gamma Z = P_a \tan^2 \alpha$$

$$P_a = \frac{\gamma Z}{\tan^2 \alpha} = \gamma Z \cot^2 \alpha$$

$$\text{W.KT } K_a = \frac{1 - \sin \phi}{1 + \sin \phi} = \cot^2 \alpha$$

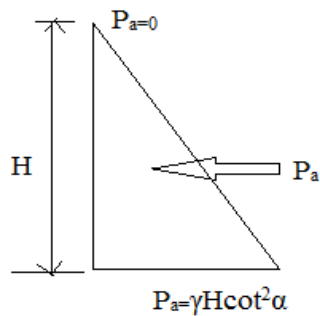
$$P_a = K_a \gamma Z \text{ --- (2)}$$

Pressure Diagram:



At top ----- $Z=0, P_a=0$

At bottom ----- $Z=H, P_a=\gamma H \cot^2 \alpha$



Consider for 1m run backfill

Total active earth pressure per m=Area of pressure diagram height

$$= \frac{1}{2}, K_a \gamma H. 1. H$$

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

P_a act as a distance $H/3$ from base.

If the soil is dry, γ is the dry weight of the soil, if wet, γ is the moist weight.

Backfill is Cohesive soil:

For active Pressure:

$$\sigma_v = \gamma Z$$

$$\sigma_h = P_a$$

According to principal stress relationship:

$$\sigma_v = 2C \tan \alpha + \sigma_h \tan^2 \alpha \text{ ----- (1)}$$

Substitute σ_v and σ_h value in eqn(1)

$$\gamma Z = 2C \tan \alpha + P_a \tan^2 \alpha$$

$$P_a \tan^2 \alpha = \gamma Z - 2C \tan \alpha$$



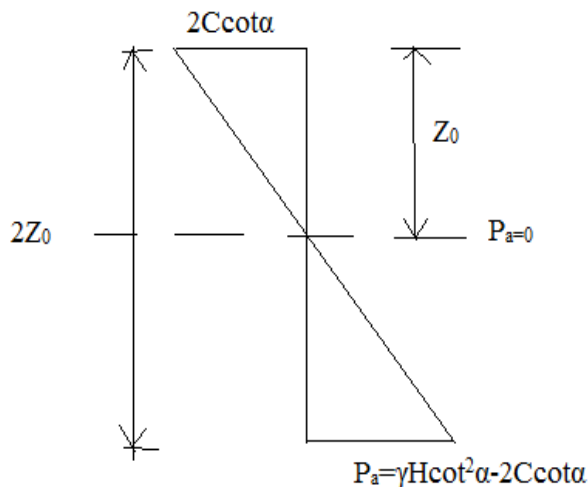
$$P_a = \frac{\gamma Z}{\tan^2 \alpha} - \frac{2C \tan \alpha}{\tan^2 \alpha}$$

$$P_a = \gamma Z \cot^2 \alpha - 2C \cot \alpha \text{ --- (2)}$$

Pressure Diagram:

At top ----- $Z=0, P_a = -2C \cot \alpha$

At bottom ----- $Z=H, P_a = \gamma H \cot^2 \alpha - 2C \cot \alpha$



If $P_a=0, Z=Z_0$

$$0 = \gamma Z_0 \cot^2 \alpha - 2C \cot \alpha$$

$$\gamma Z_0 \cot^2 \alpha = 2C \cot \alpha$$

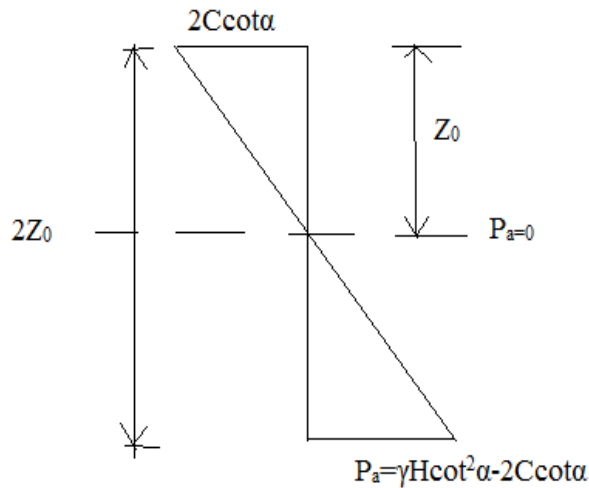
$$Z_0 c = \frac{2C \cot \alpha}{\gamma \cot^2 \alpha} = \frac{2C}{\gamma \cot \alpha} = \frac{2C \tan \alpha}{\gamma}$$

Z_0 indicates the soil can withstand comfortably without slip.

The depth upto which the cohesive soil can withstand without any support is known as critical height (H_c)

$$H_c = 2Z_0$$

$$H_c = 2x \frac{2C}{\gamma} \tan \alpha$$



$$H_c = \frac{4C}{\gamma} \sqrt{N\phi} \quad [\alpha = N\phi]$$

Height of backfill below $P_a=0, H-Z_0$

Consider for 1m run backfill

Total active earth pressure per m = Area of pressure diagram x Height

$$= \frac{1}{2}, (\gamma H \cot^2 \alpha - 2C \cot \alpha) x (H - Z_0)$$

2.SUBMERGED BACKFILL:

In this case, the sand fill behind the retaining wall is saturated with water. The lateral pressure is made up of two components:

For Active Pressure:

a) Backfill is fully submerged

Lateral pressure due to two component

Due to submerged unit weight of soil

Due to pore water

Consider a cohesionless soil with unit weight γ' and height Z

$$P_a = K_a \gamma' Z \text{ --- for submerged soil}$$

$$P_a = \gamma_w Z \text{ --- pore water}$$

Active earth pressure,

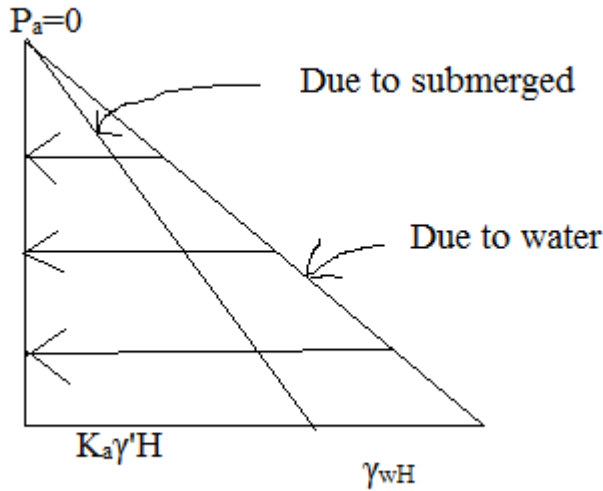


$$P_a = K_a \gamma' Z + \gamma_w Z$$

Pressure diagram,

At top ----- $Z=0, P_a=0$

At bottom ----- $Z = H, P_a = K_a \gamma' H + \gamma_w H$



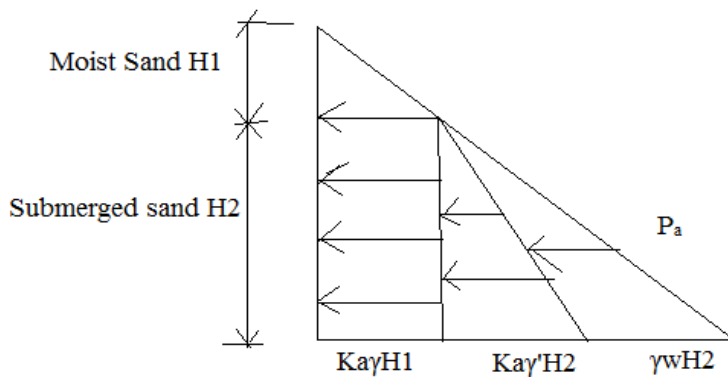
$$\text{Total Pressure } P_a = \frac{1}{2} K_a \gamma' H^2 + \frac{1}{2} \gamma_w H^2$$

Case b) If backfill is partially submerged

The pressure diagram

γ =unit weight of moist sand having depth H_1

γ' =unit weight of moist sand having depth H_2



$$P_a = K_a \gamma H_1 + K_a \gamma' H + \gamma_w H$$



Total Pressure per m

$$P_a = \frac{1}{2} K_a \gamma H_1^2 + K_a \gamma' H_2^2 + \frac{1}{2} \gamma_w H^2$$

$$\bar{Z} = \frac{P_1 \left(\frac{H}{2}\right) + P_2 \left(\frac{H}{3}\right)}{P_a}$$

3.BACKFILL WITH UNIFORM SURCHARGE:

If the backfill is horizontal and carries a surcharge of uniform intensity q per unit area. The vertical pressure increment, at any depth z , will increase by q . the increase in the lateral pressure due to this will be $K_a q$.

For active pressure:

Consider a surcharge load (q) is acting on the top of backfill. It act as vertical stress [$\sigma_v = q$]

For surcharge load alone $C=0$

According to principal stress relationship,

$$\sigma_v = 2C \tan \alpha + \sigma_h \tan^2 \alpha \text{ --- (1)}$$

$$\sigma_v = q$$

$$\sigma_h = P_a$$

Substitute σ_v and σ_h value in eqn(1)

$$q = P_a \tan^2 \alpha$$

$$P_a \tan^2 \alpha = \gamma Z - 2C \tan \alpha$$

$$P_a = \frac{q}{\tan^2 \alpha}$$

$$P_a = q \cot^2 \alpha = q K_a \text{ --- (2)}$$

Pressure Diagram:

At top ----- $Z=0, P_a = q K_a$

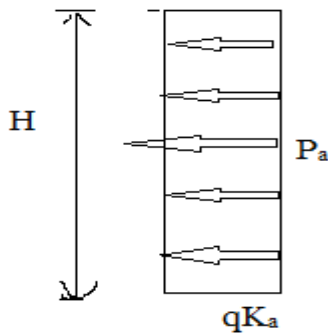
At bottom ----- $Z=H, P_a = q K_a$

Total active pressure per m run

$P_a = \text{Area of pressure diagram} \times \text{height}$



$$= qK_a \times 1 \times H = qcot^2 \alpha$$



Total active pressure = Pressure due to surcharge + pressure due to backfill

$$= qcot^2 \alpha + \frac{1}{2} K_a \gamma H^2$$

At the base of the wall, the pressure intensity is $P_a = 1/2 K_a \gamma H + K_a q$

4.BACKFILL WITH SLOPING SURFACE:

Let the sloping surface behind the wall be inclined at the angle β with the horizontal; β is called the surcharge angle. In finding out the active earth pressure for this case by Rankine's theory, an additional assumption that the vertical and lateral stresses are conjugate is made. It can be shown that if the stress on the given plane at a given point is parallel to the another plane, the stress on the latter plane at the same point must be parallel to the first plane. such planes are called the conjugate planes the stresses acting on them are called conjugate stresses.

Consider a soil element at point A at depth z with in a backfill with a sloping surface. The top plane of the element is parallel to the ground plane and the other plane conjugate to this is vertical. Let σ and p be the conjugate stresses, σ being vertical and p being the parallel to the sloping backfill. Being conjugate, both the vertical pressure and lateral pressure have the same angle of obliquity β , which is equal to the surcharge angle.

$$\sigma_1 - \sigma_3 / \sigma_1 + \sigma_3 = \sin \phi \text{-----(1)}$$

Mohr circle corresponding to the principal stress intensity σ_1 and σ_3 at A. OA_1 represents the resultant stress p and OA_2 represents the resultant stress σ . Draw OB perpendicular to $A_1 A_2$.

$$OB = OC \cos \beta \text{-----(2)}$$

$$BC = OC \sin \beta = \sin \beta (\sigma_1 + \sigma_3) / 2 \text{-----(3)}$$

$$A_1 B = BA_2 = \sqrt{(A_1 C^2 - BC^2)} = \sqrt{((\sigma_1 - \sigma_3) / 2)^2 - ((\sigma_1 + \sigma_3) / 2)^2 \sin^2 \beta}$$



From (1) ,

$$A_1B = BA_2 = (\sigma_1 + \sigma_3)/2 \sqrt{(\sin^2\phi - \sin^2\beta)} \text{-----(4)}$$

Now stress

$$\begin{aligned} \sigma &= OB + BA_2 \\ &= (\sigma_1 + \sigma_3)/2 \cos\beta + (\sigma_1 + \sigma_3)/2 \sqrt{(\sin^2\phi - \sin^2\beta)} \text{----- (5)} \end{aligned}$$

And stress p = OB - A₁B

$$= (\sigma_1 + \sigma_3)/2 \cos\beta - (\sigma_1 + \sigma_3)/2 \sqrt{(\sin^2\phi - \sin^2\beta)} \text{----- (6)}$$

Dividing (5) & (6) we get,

$$\begin{aligned} p/\sigma = K &= \cos\beta - \sqrt{(\sin^2\phi - \sin^2\beta)} / \cos\beta + \sqrt{(\sin^2\phi - \sin^2\beta)} \\ K &= \cos\beta - \sqrt{(\cos^2\beta - \cos^2\phi)} / \cos\beta + \sqrt{(\cos^2\beta - \cos^2\phi)} \end{aligned}$$

The ratio K is called conjugate ratio.

For the present case,

$$\begin{aligned} \sigma &= (\gamma \cdot z \cdot b \cos\beta / b) \\ &= \gamma \cdot z \cdot \cos\beta \\ P_a &= \gamma \cdot z \cdot \cos\beta (\cos\beta - \sqrt{(\cos^2\beta - \cos^2\phi)} / \cos\beta + \sqrt{(\cos^2\beta - \cos^2\phi)}) \end{aligned}$$

$$P_a = K_a \gamma Z$$

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right]$$

$$K_a = (1 - \sin\phi) / (1 + \sin\phi)$$

The total active pressure P_a for the wall of height H is given by P_a = 1/2 K_aγ.H²

If the backfill is submerged, the lateral pressure due to the submerged weight of the soil will act at β with horizontal, while the lateral pressure due to water will act normal to the wall.

5. INCLINED BACK AND SURCHARGE:

A retaining wall with an inclined back supporting a backfill with horizontal ground surface. The total active pressure P₁ is first calculated on a vertical plane BC passing through the heel. The total pressure P is the resultant of the horizontal pressure P₁ and the weight W of the wedge ABC:

Where,



$$P = \sqrt{P_1^2 + W^2}$$

$$P_1 = 1/2 K_a \gamma \cdot H^2$$

The active earth pressure is first calculated on a vertical plane passing through the heel and intersecting the surface of the backfill or its extension in point C. the height H of vertical plane is represented by BC. The resultant of P is the vector sum of P_1 and W, where W is the weight of the soil contained in the triangle ABC.

For Passive Pressure:

Case i) Dry or moisture backfill with no surcharge:

Backfill is Cohesion less soil:

$$\sigma_1 = \sigma_h, \sigma_3 = \sigma_v$$

$$\sigma_v = \gamma Z$$

$$\sigma_h = P_p$$

According to principal stress relationship:

$$C=0$$

$$\sigma_h = 2C \tan \alpha + \sigma_v \tan^2 \alpha \text{ --- (1)}$$

Substitute σ_v and σ_h value in eqn(1)

$$P_p = 2C \tan \alpha + \gamma Z \tan^2 \alpha$$

$$P_p = \gamma Z \tan^2 \alpha$$

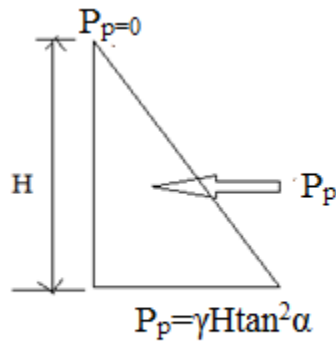
$$\text{W.KT } K_p = \frac{1 + \sin \phi}{1 - \sin \phi} = \tan^2 \alpha$$

$$P_p = K_p \gamma Z \text{ --- (2)}$$

Pressure Diagram:

At top ----- $Z=0, P_p=0$

At bottom ----- $Z=H, P_p=\gamma H \tan^2 \alpha$



Consider for 1m run backfill

Total passive pressure per m=Area of pressure diagram height

$$= \frac{1}{2}, K_p \gamma H \cdot 1 \cdot H$$

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

P_p act as a distance $H/3$ from base.

Backfill is Cohesive soil:

$$\sigma_1 = \sigma_h, \sigma_3 = \sigma_v$$

$$\sigma_v = \gamma Z$$

$$\sigma_h = P_p$$

According to principal stress relationship:

$$\sigma_h = 2C \tan \alpha + \sigma_v \tan^2 \alpha \text{ --- (1)}$$

Substitute σ_v and σ_h value in eqn(1)

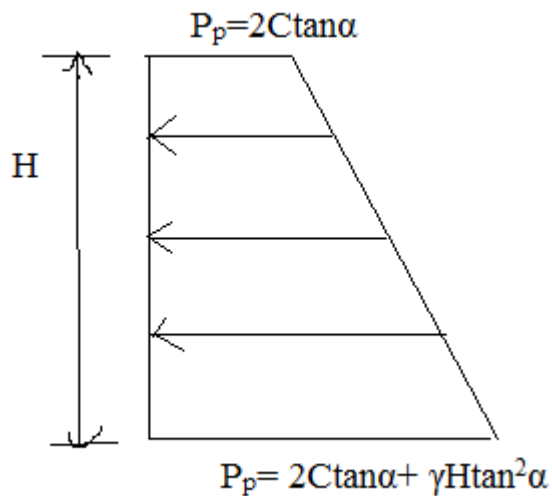
$$P_p = 2C \tan \alpha + \gamma Z \tan^2 \alpha$$

$$P_p = 2C \tan \alpha + \gamma Z \tan^2 \alpha \text{ --- (2)}$$

Pressure Diagram:

At top ----- $Z=0, P_p=2C \tan \alpha$

At bottom ----- $Z=H, P_p= 2C \tan \alpha + \gamma H \tan^2 \alpha$



Consider for 1m run backfill

Total passive earth pressure per m = Area of pressure diagram x Height

$$= \frac{1}{2} (2C \tan \alpha + \gamma H \tan^2 \alpha + 2C \tan \alpha) \times H \times 1$$

$$= \frac{1}{2} (4C \tan \alpha + \gamma H \tan^2 \alpha) \times H \times 1$$

Case iii) Surcharge Load:

For passive pressure:

Consider a surcharge load (q) is acting on the top of backfill. It act as vertical stress [$\sigma_v = q$]

For surcharge load alone C,

$$\sigma_1 = \sigma_h, \sigma_3 = \sigma_v$$

According to principal stress relationship,

$$\sigma_v = 2C \tan \alpha + \sigma_h \tan^2 \alpha \text{ --- (1)}$$

$$\sigma_v = q$$

$$\sigma_h = P_p$$

Substitute σ_v and σ_h value in eqn(1)

$$P_p = q \tan^2 \alpha$$

$$P_a = q \tan^2 \alpha = q K_p \text{ --- (2)}$$

Pressure Diagram:



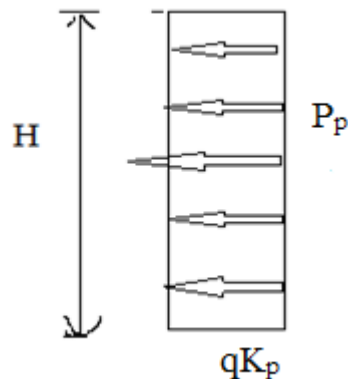
At top ----- $Z=0, P_p=qK_p$

At bottom ----- $Z=H, P_a=qK_p$

Total active pressure per m run

$P_p = \text{Area of pressure diagram} \times \text{height}$

$$= qK_p \times 1 \times H = q \tan^2 \alpha$$



Case iv) Effect of inclined surcharge or sloping backfill (or) Expression for earth pressure in case of sub charge angle.

$$\sigma_3 = \sigma_v$$

$$\sigma_1 = \sigma_h$$

For passive pressure:

$$P_p = K_p \gamma Z$$

$$K_p = \cos \beta \left[\frac{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \varphi}}{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \varphi}} \right]$$

Case v) Effect of inclined surcharge or sloping backfill (or) Expression for earth pressure in case of sub charge angle.

$$\sigma_3 = \sigma_h$$

$$\sigma_1 = \sigma_v$$

For active pressure:

$$P_a = K_a \gamma Z$$

$$K_a = \cos \beta \left[\frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \varphi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \varphi}} \right]$$



For passive pressure:

$$P_p = K_p \gamma Z$$

$$K_p = \cos\beta \left[\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \right]$$

Problems:

1. A gravity retaining wall retains 10 m of a backfill, unit weight of soil = 18 kN/m³, angle of shearing resistance = 30° with a horizontal surface. Assume the wall interface to be vertical, determine (i) the magnitude and point of application of the total active pressure (ii) if the water table is at a height of 5m, and how far do the magnitude and the point of the application of active pressure changed. Take submerged unit weight = 10kN/m³.

$$H = 10\text{m}, \phi = 30^\circ, \gamma = 18\text{kN/m}^2, \gamma_{\text{sub}} = 10\text{kN/m}^2$$

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30}{1 + \sin 30} = \frac{1}{3}$$

$$\text{Active Pressure at base of wall} = \frac{1}{3} \times 18 \times 10 = 60\text{KN/m}^2$$

$$\text{Total active thrust, } P_a = \frac{1}{2} K_a \gamma H^2$$

$$P_a = \frac{1}{2} \times \frac{1}{3} \times 18 \times 10 = \frac{60\text{KN}}{\text{m}^2}$$

ii) Water – table at 5m from surface:

$$P_a = K_a \gamma H_1 + K_a \gamma_{\text{sub}} H_2 + \gamma_w H$$

$$= \frac{1}{3} \times 18 \times 5 + \frac{1}{2} \times 10 \times 5 + 9.81 \times 5$$

$$= 104.05 \text{ kN/m}^2 \text{ at base}$$

$$P_1 = \frac{1}{2} \times \frac{1}{3} \times 18 \times 5 \times 5 = 75 \text{ kN}, Y_1 = \frac{5}{3} + 5 = 6.67\text{m}$$

$$P_2 = 5 \times \frac{1}{3} \times 18 \times 5 = 150 \text{ kN}, Y_2 = \frac{5}{2} = 2.5\text{m}$$

$$P_3 = \frac{1}{2} \times \frac{1}{3} \times 10 \times 5 \times 5 = 41.67 \text{ kN}, Y_3 = \frac{H}{3} = \frac{5}{3} = 1.67\text{m}$$

$$P_4 = \frac{1}{3} \times 9.81 \times 5 \times 5 = 122.63 \text{ kN}, Y_4 = \frac{H}{3} = 1.67 \text{ m}$$

$$\text{Total thrust, } p_a = p_1 + p_2 + p_3 + p_4$$

$$= 75 + 150 + 41.67 + 122.63$$



$P_a = 389.3$ kN per meter length of wall.

Taking moments about base

$$P \times \bar{y} = p_1 y_1 + p_2 y_2 + p_3 y_3 + p_4 y_4$$

$$389.3 \bar{y} = (75 \times 6.67) + (150 \times 2.5) + (41.67 \times 1.67) + (122.63 \times 1.67)$$

$$\therefore \bar{y} = 2.95 \text{ m}$$

\therefore Total thrust of 389.3 kN per meter length of wall will act at 2.95 m from base of wall.

2. A retaining wall is 4 m high. Its back is vertical and it has got sandy backfill up to its top. The top of the fill is horizontal and carries a uniform surcharge of 85 kN/m². Determine the active pressure on the wall per meter length of wall. Water table is 1 m below the top of the fill. Dry density of soil = 18.5 kN/m³. Moisture content of soil above water table = 12%. Angle of internal friction of soil = 30°, specific gravity of soil particles = 2.65. Porosity of backfill = 30°. The wall friction may be neglected.

$$e = \frac{n}{1-n}$$

$$= 0.43$$

$$\gamma = (1+w) \times \gamma_d = (1+0.12) \times 18.5 = 20.7 \text{ kN/m}^3$$

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

$$= \frac{1 - 0.5}{1 + 0.5} = 0.3$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = 20.7 - 9.81 = 11.52 \text{ kN/m}^3$$

(i) Due to soil above W.T

$$P_1 = \frac{1}{2} K_a \gamma H^2 + K_a \gamma H$$

$$= \frac{1}{2} \times 0.33 \times 18.5 \times 4^2 + 0.33 \times 18.5 \times 4$$

$$= 21.85 \text{ kN/m}$$

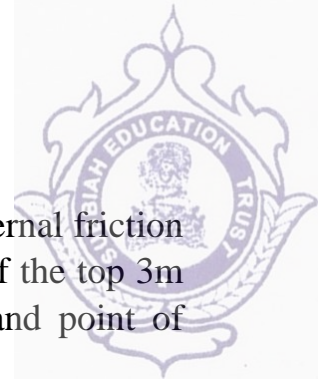
(ii) Due to submerged soil

$$P_2 = \frac{1}{2} \times 0.333 \times 11.52 \times 3^2 = 17.3 \text{ kN/m}$$

Due to water pressure, $P_3 = \frac{1}{2} \times 9.81 \times 3^2 = 45 \text{ kN/m}$.

$$\therefore \text{Total active pressure} = P_1 + P_2 + P_3 + P_4$$

$$P_a = 21.58 + 17.3 + 45 + 113.2$$



=197.08 kN/meter length of wall.

3. A retaining wall is 9 m high retain on a cohesion less soil with an angle internal friction 33° . The surface is level with the level with the top of wall. The unit weight of the top 3m of the fill is 21KN/m^3 and that the rest is 27KN/m^3 . Find the magnitude and point of application of resultant active thrust.

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

$$= \frac{1 - \sin 33^\circ}{1 + \sin 33^\circ} = 0.295$$

$$P_a @ 3\text{m} = K_a \gamma H = 0.295 \times 21 \times 3 = 18.6 \text{KN/m}^3$$

$$P_a @ 3\text{m} = K_a \gamma H = 0.295 \times (18.6 + 27 \times 6) = 66.4 \text{KN/m}^3$$

Total active thrust,

$$P_a = 1/2 \times 3 \times 18.6 + 18.6 \times 6 + 1/2 \times 6 \times 47.8 = 283 \text{KN}$$

$$\bar{z} = \frac{27.9(6 + 1) + 111.6 \times 3 + 143.4 \times 2}{283} = 2.89 \text{m}$$

4. A wall of 6m height sand having a unit weight of 20KN/m^3 and angle of internal friction of 30° . If the surface of the backfill slope upwards @ 15° to the horizontal. Find the active thrust per unit length of the wall using Rankine's theory. Solve the problem both analytically.

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

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right]$$

$$\cos 15 = 0.96, \cos^2 15 = 0.93, \cos^2 30 = 0.75$$

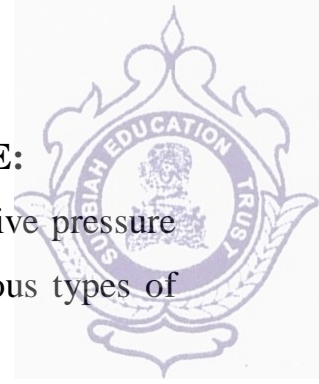
$$K_a = \cos 15 \left[\frac{\cos 15 - \sqrt{\cos^2 15 - \cos^2 30}}{\cos 15 + \sqrt{\cos^2 15 - \cos^2 30}} \right]$$

$$K_a = 0.96 \left[\frac{0.96 - \sqrt{0.93 - 0.75}}{0.96 + \sqrt{0.93 - 0.75}} \right]$$

$$= 0.96 \times \frac{0.5}{1.418} = 0.37$$

$$P_a = \frac{0.37 \times 20 \times 6^2}{2} = 131.2 \text{KN/m}$$





5.3 CULMANN'S GRAPHICAL METHOD FOR ACTIVE PRESSURE:

Culmann's (1866) also gave a graphical solution to evaluate the active pressure and can be conveniently used for ground surface of any shape, for various types of surcharging loads, and for a layered backfill of different densities.

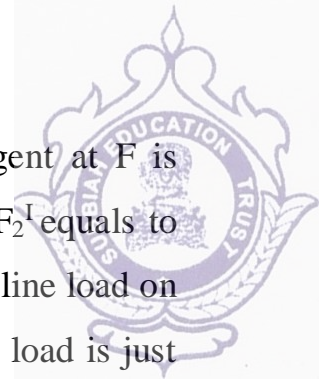
PROCEDURE:

1. Draw the ground line ϕ line and the ψ line as usual
2. Take a slip plane BC_1 . Calculate the weight of the wedge ABC_1 and plot it as BE_1 to some scale on the ϕ line.
3. Through E_1 , draw E_1F_1 parallel to the line ψ , to cut the slip plane BC_2 in F_1 .
4. Similarly take another slip plane BC_2 , calculate the weight of wedge ABC_2 and plot it as BE_2 on the line. Draw E_2F_2 parallel to the line cut the slip plane BC_2 in F_2 .
5. Take number of such slip planes BC_3, BC_4 . Plot the weight of the corresponding wedges on the ψ line and obtain points F_3, F_4 .
6. Draw a smooth curve through points B, F_1, F_2, F_3, F_4 etc. This curve is known as the Culmann's line.
7. Draw a tangent to the Culmann's line parallel to the ϕ line. The maximum value of the earth pressure is represented by the intercept EF , on the adopted scale. EF being drawn through the points of tangency parallel to the line ψ line. BFC represents the critical slip plane.
8. To locate the points of application of the resultant pressure, draw a line parallel to the critical slip plane BC , through the center of gravity of the sliding wedge ABC and obtain its intersection on the back AB .

When the ground line is a plane, the weights of the wedges $ABC_1, AC_1 = L_3$, etc. since the height of soil wedge is constant being equal to H_1 . Hence the weights of these wedges are plotted as their base lengths L_1, L_2, L_3 , etc. on the ϕ line.

$$P_a = 1/2 \gamma H_1 (EF)$$

If the backfill also carries a surcharge of intensity q , γ



gives the maximum pressure in the absence of the line load. If the tangent at F is prolonged to meet the modified Culmann's line in F_2^I the intercept $E_2^I F_2^I$ equals to FE . This means that if the line is placed beyond C_2 , there is no effect of the line load on the pressure for the other plotted. It will be seen that the maximum pressure is when the load is just at face of the wall, it remains constant with the position of q up to point c_1 and then decreases gradually to zero at C_2 .

For load positions beyond C_2 the pressure on the wall is not due to q . This method is very much used in locating the position of the railway line or the footing of building on the backfill at such a safe distance that the earth pressure on the (existing) wall does not increase.



5.4 COULOMB'S WEDGE THEORY

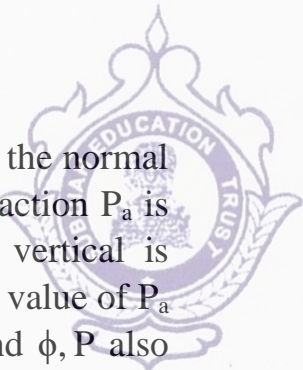
Instead of considering the equilibrium of an element within the mass of the material, Coulomb (1776) considered of equilibrium of whole of the material supported by a retaining wall when the wall is on the point of moving slightly away from the filling. The wedge theory of earth pressure is based on the concept of a sliding wedge which is torn off from the rest of the backfill on movement of the wall. In the case of active earth pressure, the sliding wedge moves downwards on a slip surface relative to the intact backfill and in the case of passive earth pressure, the sliding wedge moves upwards and inwards. The pressure on the wall is, in fact, a force of reaction which it has to exert to keep the sliding wedge in equilibrium. Factors such as wall friction, irregular soil surfaces and different soil strata can easily take into account in this method.

Following are the basic Assumptions of the wedge theory:

- The backfill is dry, cohesion less, homogenous, isotropic and elastically undeformable but breakable.
- The slip surface is plane which passes through the heel of the wall.
- The sliding wedge itself acts as a rigid body and the value of earth pressure is obtained by considering the limiting equilibrium of the sliding wedge as a whole.
- The position and direction of the resultant earth pressure are known. The resultant pressure acts on the back of the wall at one-third the height of the wall from the base and is inclined at an angle δ (called the angle of wall friction) to the normal to the back. (The assumption means that the pressure distribution is hydrostatic, i.e., triangular). The back of wall is rough and a relative movement of the wall and the soil on the back takes place which develops frictional forces that influence the direction of the resultant pressure.

The forces acting on a wedge of soil are: its weight W , the reaction R along the plane of sliding and the active thrust P_a against the retaining wall. R will act at an angle ϕ to the normal of the plane of sliding. The pressure P is inclined at an angle of wall friction δ to the normal which is considered positive as marked in Fig. 2 Both R and P will be inclined in a direction so as to oppose the movement of the wedge. For the condition of the yield of the wall from the backfill the most dangerous or the critical slip surface is that for which the wall reaction is maximum, i.e., the wall must resist the maximum lateral pressure before it moves away from the fill.

Condition for maximum pressure from a sliding wedge. BD shows a plane inclined at an angle ϕ to the horizontal at which the soil is expected to stay in the absence of any lateral support. The line BD , therefore, is called the natural slope line, repose line or the ϕ – line. AD , inclined at β to the horizontal, is called the ground line or surcharge line. Plane BC , inclined at angle λ (to be determined) is the line or rupture plane or slip plane; the



angle λ is called the critical slip angle. The reaction R inclined at an angle ϕ to the normal to the slip line; R is also inclined at an angle $(\lambda - \phi)$ to the vertical. The wall reaction P_a is inclined at an angle to the normal to the wall. The inclination of P_a to vertical is represented by angle $\psi = 90^\circ - \theta - \delta$ ($=$ constant for given value of θ and δ). The value of P_a depends upon the slip angle λ . P_a is zero when $\lambda = \phi$. As λ increases beyond ϕ , P also increases and after reaching a maximum value it again reduces to zero when λ equals $90 + \theta$. Thus, the critical slip plane lies between the line and back of the wall.

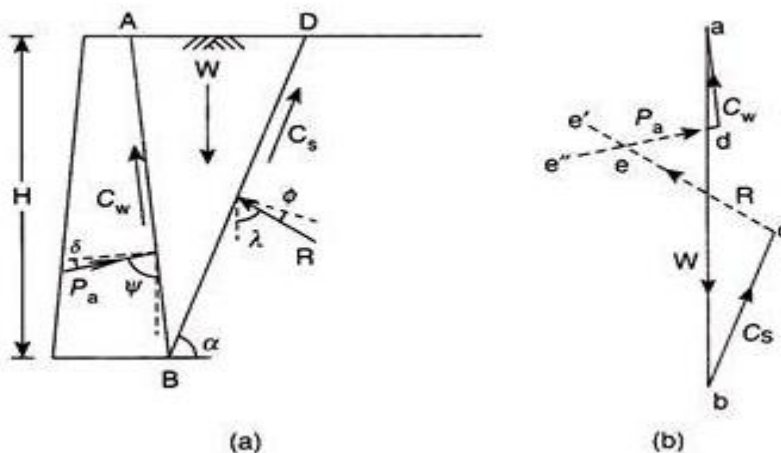


Fig1 Condition for maximum pressure from a sliding wedge

[Fig1 <https://www.soilmanagementindia.com/lateral-earth-pressure/coulombs-theory/coulombs-theory-for-earth-pressure-soil/13949>]

In order to derive the condition for maximum active pressure P_a from the sliding wedge, draw line CE at an angle ψ to the ϕ -line. Let x and n be the perpendicular distance of points C and A from the ϕ -line, and m be the length of line BD . It will be seen triangle BCE and the force triangle similar.

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

$$P_a = W \frac{CE}{BE} \text{ --- (1)}$$

From triangle CFE ,

$$\sin \phi = \frac{x}{CE}$$

$$CE = \frac{x}{\sin \phi} \text{ --- (2)}$$

Where $A_1 = \text{cosec } \phi$



$$BE = BD - FD + FE$$

From triangle CFD,

$$\tan(\varphi - \beta) = \frac{x}{FD}$$

$$FD = x \cot(\varphi - \beta)$$

From triangle CFE,

$$\tan(\varphi) = \frac{x}{FE}$$

$$FE = x \cot \Phi$$

$$\text{Hence, } BE = n - x[\cot(\Phi - \beta) - \cot \Phi]$$

or

$$BE = n - A_2 x \text{-----(3)}$$

$$\text{Where } A_2 = [\cot(\Phi - \beta) - \cot \Phi]$$

$$W = \gamma(\Delta ABC) = \gamma[\Delta ABD - \Delta BCD]$$

$$W = \frac{1}{2} \gamma(m - x)n \text{-----(4)}$$

Substituting equation 2,3&4 in 1

$$P_a = \frac{1}{2} \gamma(m - x)n \frac{A_1 x}{n - A_2 x} = \left(\frac{1}{2} \gamma n A_1\right) \left(\frac{mx - x^2}{n - A_2 x}\right)$$

In the above expression x is the only variable which depends upon the position of slip plane BC. For maxima $dP_a/dx = 0$

$$\frac{dP_a}{dx} = \left(\frac{1}{2} \gamma n A_1\right) \frac{(m - 2x)(n - A_2 x) - (-A_2)(mx - x^2)}{(n - A_2 x)^2} = 0$$

$$(m - 2x)(n - A_2 x) = -A_2 (mx - x^2)$$

$$mn - A_2 mx - 2nx + 2A_2 x^2 = -A_2 mx + A_2 x^2$$

$$mn - 2xn = -A_2 x^2$$

Rearranging,

$$mn - xn = xn - A_2 x^2 = x(n - A_2 x) = x BE$$

We can Write,



$$\frac{mn}{2} - \frac{xn}{2} = X \frac{BE}{2}$$

$$\Delta ABD - \Delta BCD = \Delta BCE$$

$$\Delta ABC = \Delta BCE$$

Thus the criterion for maximum active pressure is that the slip plane is so chosen that

ΔABC and ΔBCE are equal in area.



5.5 Stability Analysis of Retaining Wall:

Introduction:

Stabilization incorporates the various methods employed for modifying the properties of a soil to improve its engineering performance. Stabilization is used for a variety of engineering works, the most common application being in the construction of road & air- field pavements, where the main objective is to increase the strength or stability of soil & to reduce the construction cost by making best use of the locally available materials.

1.Mechanical stabilisation:

- Mechanical stabilization involves two operations :
 - (i) changing the composition of soil by addition or removal of certain constituents
 - (ii) Densification or compaction .the particle size distribution and composition are the important factors governing the engineering behaviour of a Soil. Significant changes in the properties can be made by addition or removal of suitable soil fractions. For mechanical stabilizations where the primary purpose is to have a soil resistant to deformation and displacement under the loads, soil materials can be divided in two fractions: The granular fraction retained on a 75 microns

IS sieve and the fine soil fraction passing a 75 –microns sieve. The granular fraction imparts strength and hardness. The fine fraction provides cohesion or binding property, water – retention capacity and also acts as a filler for the voids of the coarse fraction.

2.Cement stabilization:

a).Soil cements and its influencing factors

- The soil stabilized with cement (Portland) is known as soil cement.
- The cementing action is believed to be the result of chemical reaction of cement with the siliceous soil during hydration. The binding action of individual particles through cement may be possible only in coarse-grained soils .in fine grained,



cohesive soils, only some of the particles can be expected to have cement bonds, and the rest will be bonded through natural pollution. The important factors affecting soil cement are: nature of soil, cement content, condition of mixing, compaction and curing and admixtures.

b)Construction methods

The normal construction sequence for soil – cement bases is as follow:

- (i) shaping the sub-grade and scarifying the soil,
- (ii) Pulverising the soil,
- (iii) addition and mixing cements,
- (iv) adding and mixing water,
- (v) compacting,
- (vi) finishing,
- (vii) curing and
- (viii) adding wearing surfacing.

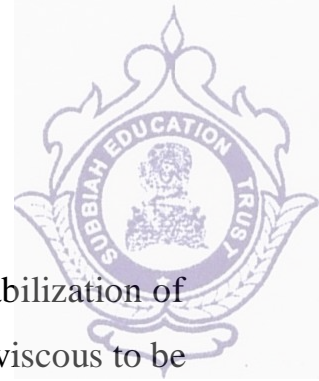
There are three methods of carrying out these operations:

- (i) mix-in place method,
- (ii) travelling plant method and
- (iii) stationary plant method.

3.Lime stabilization:

Hydrated (or slaked) lime is very effective in treating heavy, plastic clayey soils . Lime may be used alone, or in combination with cement, bitumen or fly ash. Sandy soils can also be stabilized with these combinations. Lime has been mainly used for stabilizing the road bases and sub- grades on addition of lime to soil , two main types of chemical reactions occurs:

- i. Alteration in the nature of absorbed layer through Base Exchange phenomenon,
- ii. Cementing or puzzolanic action. Lime reduces the plasticity index of highly plastic soils making them more friable and easy to be handled and pulverized. The plasticity index of soils of low plasticity generally increases. There is generally and increase in the optimum water content and a decrease in the maximum compacted



density, but the strength and durability increase.

4.Bitumen stabilization:

Asphalts and tars are the bituminous materials which are used for stabilization of soil, generally for pavement construction. These materials are normally too viscous to be incorporated directly with soil .the fluidity of asphalts is increased by either heating, emulsifying or by cut-back process. Tars are heated or cut back. The bituminous materials when added to a soil impart cohesion or binding action and reduced water absorption. Thus either the binding action or the water proofing action or both the actions, may be utilized for stabilization. depending upon these actions and the nature of soils , bitumen stabilization is classified under the following four types :

- (i) sand-bitumen ,
- (ii) (ii)soil-bitumen
- (iii) (iii) water-proofed mechanical stabilization and
- (iv) (iv) oiled earth

5.Chemical stabilization:

There are a great many chemicals which are used for stabilization. Only the chemicals which are commonly used for stabilizing moisture in the soil and for cementation of particles will be described here,

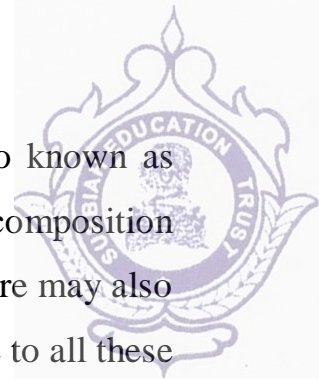
1. Calcium chloride
2. Sodium chloride
3. Sodium silicate

6.Stabilization by heating

Heating a fine grained soil to temperature of the order of 400-600°C causes irreversible changes in clay minerals. The soil becomes non – plastics, less water sensitive and non- expansive. Also the clay clods get converted into aggregates. Soil can be baked in kilns, or in –situ downwards draft slow moving furnaces. The artificial aggregates so produced can be used for mechanical stabilization.

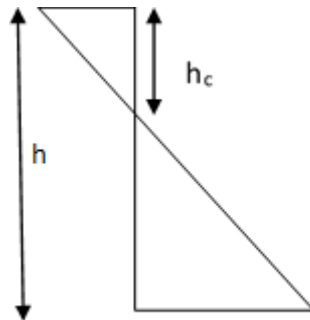
7.Electrical stabilization:

The stability or shear strength of fine-grained soils can be increased by draining



them with the passage of direct current through them. The process is also known as electro-osmosis. Electrical drainage is accompanied by electro-chemical composition of the electrodes and the deposition of the metal salts in the soil pores. There may also be some change in the structure of soil. The resulting cementing of soil due to all these reactions, is also known as electro-chemical hardening and for these purposes the use of aluminium anodes is recommended.

Tension cracks :



In clay under undrained condition at climatic temperature variation and due to water drain in season's soil volume shrinkage, cause compression due to soil self weight, surcharge and live load, cause tensile stress and spilt or fracture in the clay mass especially.

Hence in figure the depth of the tension zone was given the symbol h_c . It is possible for cracks to develop over this depth and a value for h_c is obtained as

From active earth pressure theory, $h_c = 2C / \gamma$

Different modes of failure of retaining wall:

1. Failure against sliding
2. Failure against overturning
3. Failure against bearing capacity

Failure against sliding

- This soil in front of the wall provides active and passive pressure resistance as the wall tends to slide.
- Use of a key beneath the base provides additional sliding stability.



- The sliding resistance along the base $F_R = \mu R$, where R includes all the vertical forces, including the vertical components of P_a , acting at the base and μ the coefficient of wall friction.

$$\text{Factor of safety} = \frac{\text{Sum of resisting force}}{\text{Sum of driving force}} =$$

- Factor of safety against sliding should be atleast 1.5 for sandy soil and 2.0 for clayey soil.

Failure against overturning:

- For a wall to be stable the resultant thrust must be within the base. Most walls are so designing that the thrust is within the middle third of the wall base. It is to avoid loss of contact of base with soil.

$$\text{Factor of safety} = \frac{\text{Sum of resisting force}}{\text{Sum of overturning force}}$$

- Overturning is usually considered with respect to toe and the factor of safety should be at least 1.5 for sandy soil and 2.0 for clayey soil. The resisting moments are normally due to vertical component of all the forces namely weight of wall, weight of soil over base, vertical component active pressure and passive pressure.

Failure against Bearing Capacity:

$$\text{Factor of safety} = \frac{\text{Allowable bearing pressure}}{\text{Maximum contact pressure}}$$

- Vertical load causes uniform contact pressure at the base. Overturning moment causes compressive pressure at toe and tensile pressure at heel. The sum contact pressure is maximum at toe. Factor of safety against bearing capacity should be atleast 2.5 for sandy soil and 3.0 for clayey soil.