UNIT 1

SITE INVESTIGATION AND SOIL EXPLORATION

Site investigations and sub-surface explorations are done to obtain the information about subsurface conditions at the site of proposed construction. Information about the surface and subsurface feature is essential for the design of structures and for planning construction techniques. Site investigations consist of determining the profile of the natural soil deposits at the site, taking the soil samples and determining the engineering properties of the soil. It also includes in-situ testing of the soils.

OBJECTIVES OF SITE INVESTIGATIONS

1. To select the type and depth of foundation for a given structure.
2. To determine the bearing capacity of the soil.
3. To determine the probable maximum and differential settlements.
4. To establish the ground water table and to determine the properties of water.
5. To predict the lateral earth pressure against the retaining walls and abutments.
6. To select suitable construction techniques.
7. To predict and to solve potential foundation problems.
8. To ascertain the suitability of the soil as construction material.
9. To investigate the safety of the existing structures and to suggest the remedial measures.

PLANNING A SUBSURFACE EXPLORATION PROGRAMME

A subsurface exploration programme depends upon the type of the structure to be built and upon the variability of the strata at the proposed site. The extent of sub surface exploration is closely related to the relative cost of the investigation and that of the entire project for which is undertaken.

In general, the more detailed investigations are done, the more is known about the subsurface conditions. As a result, a greater economy can be achieved in the construction of the project because the element of uncertainty is considerably reduced.

The extent of investigation would also depend upon the location of the project. A small house in an already built-up area would not require much exploration on the other hand if the house is to be built in a newly developed area, detailed investigation is necessary. If a multi-storied building is to be constructed, extensive sub surface exploration is necessary. These buildings impose very heavy loads and the zone of influence is also very deep. It would be therefore more desirable to invest some amount on sub surface exploration than to overdesign the buildings and make it costlier.

STAGES IN SUB SURFACE EXPLORATION

1) **Reconnaissance**

Site reconnaissance is the first step in an investigation process. It includes a visit to the site and to study the maps and other relevant records. It helps in deciding future programme of site investigations, scope of work, methods of exploration to be adapted, types of samples to be taken and the laboratory testing and in-situ testing. The information about following features is obtained in reconnaissance: -

- The general topography of the site, the existence of drainage ditches and dumps of debris and sanitary fills.
- Existence of settlement cracks in the structure already built near the site.
- The evidence of landslides, creep of slopes and the shrinkage cracks.
- The stratification of soils as observed from deep cuts near the site.
- The location of high food marks on the nearby building and bridges.
- The depth of ground water table as observed in the wells.
- Existence of springs, swamps etc. at the site.
- The drainage pattern existing at the site.
- Type of vegetation existing at the site.
- Existence of underground water mains, power conduits etc. at the site.

The information obtained during reconnaissance is helpful in evolving a suitable sub surface investigation programme. In addition to making site visits, the geotechnical engineer should study geological maps, aerial photographs, toposheet, and soil maps, blue prints of the existing buildings.

Study of maps:

- Topographical maps called toposheets – survey of India and geological survey of India.
- Soil conservation maps may also be available.
- Faults, folds, cracks, fissures, dikes, skills and caves and such other defects in rock and soil strata may be indicated.
- Maps showing the earthquake zones of different zones of different degree of vulnerability is available.
- Seismic potential is a major factor in structural design especially in the construction of major structures such as dams and nuclear power plants.

2) **Preliminary exploration**
   - The aim of a preliminary exploration is to determine the depth, thickness, extent and composition of each soil stratum at the site.
   - The depth of bed rock and ground water table is also determined.
   - The preliminary explorations are generally in the form of a few boring or test pits. Tests are conducted with cone penetrometers and sounding rods to obtain information about the strength and compressibility of soils.
   - Geophysical methods are also used in preliminary explorations for locating the boundaries of different strata.
3) **Detailed exploration**
   - The purpose of the detailed explorations is to determine the engineering properties of the soils in different strata. It includes an extensive boring programme, sampling and testing of the samples in the laboratory.
   - Field tests such as vane shear tests, plate load tests, permeability tests are conducted to determine the properties of the soils in the natural of state. The tests for the determination of dynamic properties are also carried out, if required.
   - For complex projects involving heavy structures such as bridges, dams, multi-storey buildings it is essential to have detailed explorations. However, for smaller projects, especially at sites where the strata are uniform, detailed investigation may not be required. The design of such projects is generally based on the data collected during reconnaissance and preliminary explorations.

DEPTH OF EXPLORATION

Depth of exploration required, depends on the type of the proposed structure, its total weight, the size, shape and disposition of the loaded area, the physical properties of the soil that constitute the different strata e site. Exploration, in general, should be carried out to a depth up to which the increase in pressure due to structural loading is likely to cause foundation failure. Such a minimum depth is known as critical depth or significant depth. The net loading intensity at any level below the foundation is obtained by approximately assuming a spread of load of, two vertical to one horizontal, from all sides of the foundation. Due allowance should be made for the overlapping effects of the load from closely spaced footings. It is rally safe to assume the significant depth up to a level at which the net increase in vertical pressure becomes less than10% of the initial overburden pressure.

The following guide rules may also be followed to decide the depth of exploration: -

Type of foundation	Depth of exploration
Isolated spread footing or raft	1.5 times the width
Adjacent footings with clear spacing less than twice the width	1.5 times the length
pile foundations	10 to 30 m and more or at least 1.5 times the width of the structure
Base of retaining walls	One and a half times the base width or the exposed height, whichever is greater
For black cotton areas, from the consideration of weathering. The exploration should be carried to a minimum depth of 4 m.	

METHODS OF SOIL EXPLORATION

Mainly there are three methods. They are

1. Direct methods
2. Semi-direct methods
3. Indirect methods

1) Direct methods

It includes open excavation methods of exploration. They are-

(a) Pits and Trenches

Pits and trenches are excavated at the site to inspect the strata. The size of the pits should be sufficient to provide necessary working space.IS:4453-1967 recommends a clear working space of 1.2m x 102m at the bottom of the pit. The depth of the pit depends upon the requirement of the investigation. Shallow pits up to a depth 3m can be made without providing any lateral support. For deeper pits, especially below water table, the lateral support should be provided in the form of sheeting and bracing system. For depths greater than 6m, bore holes are economical than pits. Deep pits should be properly ventilated to prevent accumulation of dead air. If water is encountered in the pit, it should be properly dewatered. Trenches are long shallow pits. A trench is continuous over a considerable length; it provides exposure along a line. The trenches are more suitable than pits for exploration on slopes.

(b) Drifts and Shafts

Drifts are horizontal tunnels made in the hill side to determine the nature and structure of geological formation. A drift should have minimum clear dimensions of 1.5m width and 2m height in hard rock. Shafts are large size vertical holes made in the geological formation. These may be rectangular or circular in section. The minimum width of rectangular shaft is 204m and for circular, the minimum diameter is 204m.

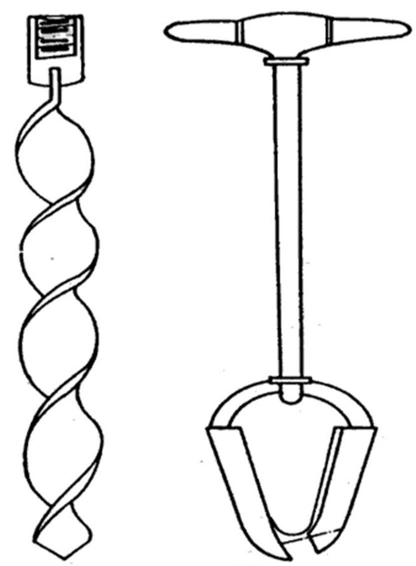
2) Semi-direct methods

When the depth of exploration is large, borings are used for the explorations. A vertical hole is drilled in the ground to get the information about the sub soil strata. Samples are taken from the bore hole and tested in the laboratory.

BORING METHODS

Depending upon the type of soil and the purpose of boring, the following methods are used for drilling the holes: -

1) Auger Boring



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

Augers are used in cohesive and other soft soils above water table. Hands augers are used for depth up to 6m. Mechanically operated augers are used for greater depths and they can also be used in gravelly soils. Samples recovered from the soil brought up by augers are badly disturbed nature of soil sample; it becomes difficult to locate the exact changes in the soil strata. Augers consist of a shank with a cross-wise handle for turning and having central tapered feed screw. It can be operated manually or mechanically.

Mechanical augers are driven by power. These are used for making holes in hard strata to a great depth. Even mechanical augers become inconvenient for depth greater than 12m and other methods of boring are used. The hand augers used in boring are about 15to 20cm in diameter. It is attached to the lower end of the pipe of about 18mm diameter. The pipe is provided with cross arm at top. The hole is advanced by turning the cross arm manually and at the same time applying thrust in the downward direction. When the auger is filled with soil, it is taken out. If the hole is already driven, another type of auger known as post hole auger is used for taking soil samples.

Disadvantages: -

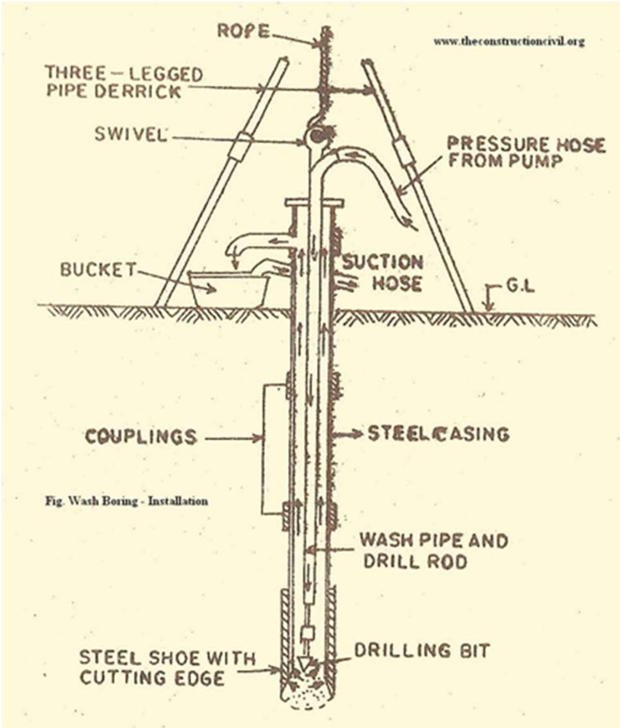
- Sandy soil below water table, a casing is normally required. For such soils, the method of auger boring becomes slow and expensive.
- It cannot be used when there are large cobbles, boulders or other obstructions which prevent drilling of the hole.
- Auger boring is fairly satisfactory for highways, railways, airfield exploration at shallow depth. The sub- surface explorations are done quite rapidly and economically by auger boring.

2) Auger and Shell boring

Cylindrical augers and shell with cutting edge on teeth at the lower end can be used for making deep borings. Hand operated rings are used for depth up to 2.m and the mechanical ring up to 50m. Augers are suitable for soft to stiff clays, shells for very stiff and hard clays and shells or sand pumps for sandy soils. Small boulders, thin soft strata or rock or cemented gravel can be broken by chisel bits attached to drill rods. The hole usually requires a casing.

3) Wash boring

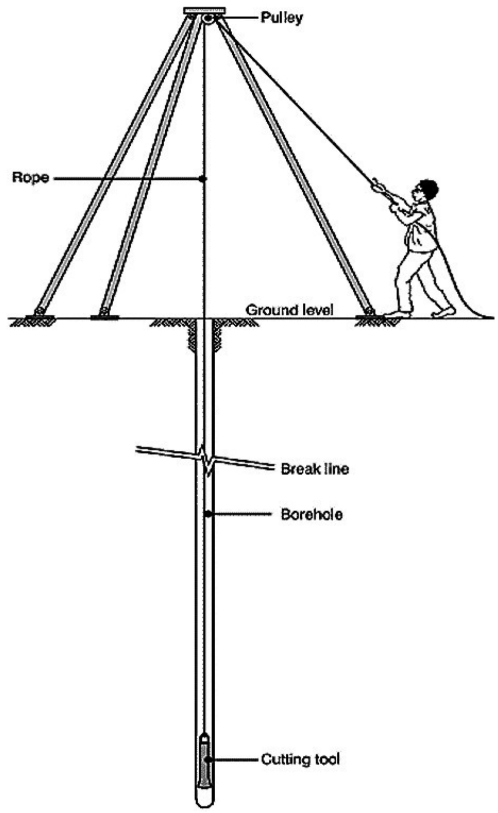
In wash boring, the hole is drilled by first driving a casing about 2 to 3m long and then inserted into a hollow drill rod with a chisel shaped chopping bit at its lower end. Water is pumped down the hollow drill rod, which is known as wash pipe. Water emerges as a strong jet through a small opening of the chopping bit. The hole is advanced by a combination of chopping action and the jetting action as the drilling bit and the accompanying water jet disintegrates the soil. The water and chopped soil particles rise upward through the annular space between the drill rod and the casing. The return water also known as wash water which is collected in a tub through a T-shaped pipe fixed at the top of the casing. The hole is further advanced by alternately raising and dropping the chopping by a winch. The swivel joint provided at the top of the drill rod facilitates the turning and twisting of the rod. The process is continued even below the costing till the hole begins to cave in. At that stage the bottom of the casing can be extended by providing additional pieces at the top. However, in stable, cohesive soils the casing is required only in the top portion. Sometimes instead of casing, special drilling fluids made of suspension or emulsion of fat clays or bentonite combined with some special additives are used for supporting walls of the hole. The change in strata is provided by the reaction of the chopping bit as the hole is advanced. It is also indicated by a change in color of the wash water. The wash boring is mainly used for advancing a hole in the ground. Once the hole has been drilled, a sampler is inserted to obtain soil samples for testing in the laboratory.



Disadvantages: -

- The equipment used in wash boring is relatively light and inexpensive. The main disadvantage of the method is that it is slow in stiff soils and coarse-grained soils. It cannot be used efficiently in hard soils, rocks and the soil containing boulders.
- The method is not suitable for taking good quality undisturbed samples above ground water table, as the wash water enters the strata below the bottom of the hole and causes an increase in its water content.

4) Percussion Drilling

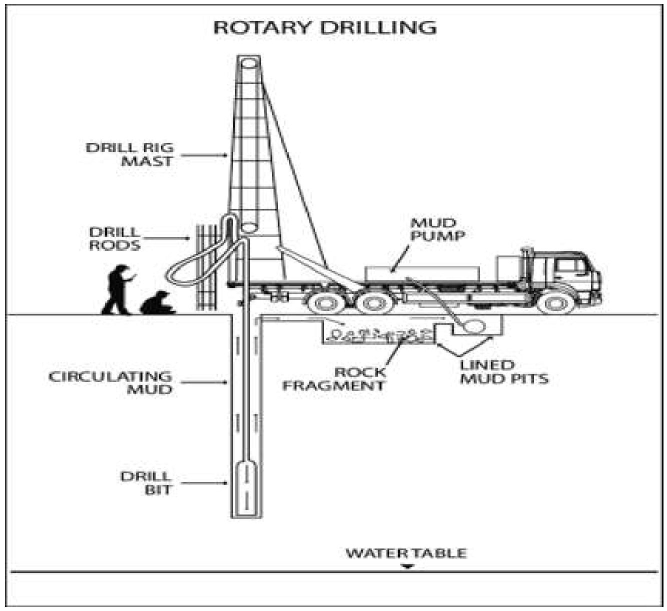


The percussion drilling method is used for making holes in rocks, boulders and other hard strata. In this method a heavy chisel is alternately lifted and dropped in a vertical hole. The material gets pulverized. If the point where chisel strikes is above the water table, water is added to the hole. The water forms slurry with the pulverized material which is removed by a sand pipe. Percussion drilling may require a casing. It is also used for drilling tube wells. The main advantage of the percussion drilling method is that it can be used for all types of materials. It is particularly useful for drilling holes in is glacial tills containing boulders.

Disadvantages: -

- One of the major disadvantages is that the material at the bottom of the hole is disturbed by heavy blows of the chisel.
- It is not possible to get good quality undisturbed samples. This method is generally more expensive.

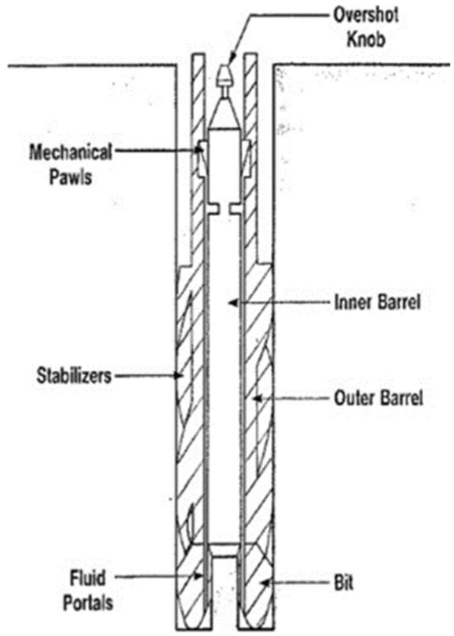
5) Rotary drilling



Rotary boring or drilling is a very fast method of advancing hole in the both rocks and soils. A drill bit, fixed to the lower end of the drill rods, is rotated by a suitable chunk and is always kept in firm contact with the bottom of the hole. A drilling mud, usually a water solution of bentonite with or without other admixtures is continuously forced down the hollow drill rods. The mud entering upwards brings the cuttings to the surface. This method is also known as Mud Rotary Drilling and the hole usually requires no casing. When the soil sample is required to be taken the drilling rod is raised and the drilling bit is replaced by the samples. Rotary drilling can be used in clay, sand and rocks. Bore holes of diameter 50mm to 200mm can be easily made by this method.

6) Core drilling

The core drilling method is used for drilling holes and for obtaining rock cores. In this method a core barrel fitted with a drilling bit is fixed to a hollow drilling rod. As the drilling rod is rotated, the bit advances and cuts an annular hole an intact hole. The core is then removed from its bottom and is retained by a core –lifter and brought to the ground surface. The core drilling may be done using either a diamond studded bit or cutting edge consists of chilled shot. The diamond driller is superior to the other type of drilling, but it is costlier Water is pumped continuously into the drilling rod to keep the drilling bit cool and to carry the disintegrated materials to the ground surface.



TYPES OF SOIL SAMPLES

Two types of soil samples can be obtained during sampling disturbed and undisturbed.

1) Disturbed sample

In disturbed sampling, the natural structures of soils get partly or fully modified or destroyed, although with suitable precaution the natural water content may be preserved. Disturbed sample can be obtained by direct excavations by auger and thick wall samplers. The disturbances can be classified in following basic types:

- Change in the stress condition.
- Change in the water content and the void ratio.
- Disturbance of the soil structure, Chemical changes.
- Mixing and segregation of soil constituents.

If all the constituents are present in the sample which represents the same soil type from any place, then it is called a **representative sample**. In the **remolded sample**, the engineering properties get changed due to remolding.

2) Undisturbed sample

In undisturbed sample, the natural structure and properties remain preserved. These samples are used to tests for shear, consolidation and permeability. For undisturbed sample the stress changes cannot be avoided.

The following requirements are looked for:

- No change due to disturbance of the soil structure.
- No change in void ratio and water content.
- No change in constituents and chemical properties.

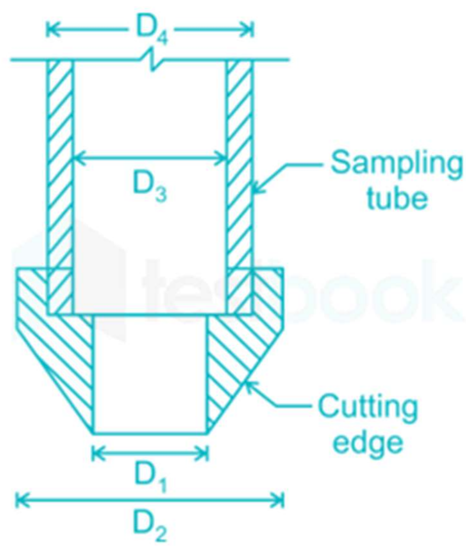
Sample Disturbances

This depends on the design of samplers and methods of samplings.

Design factors governing the degree of disturbances

Cutting edge

A typical cutting edge of a sampler is shown in the figure.



The important design features of the cutting edge are

a) Area ratio

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

Where,

D_1 - internal diameter of the cutting edge

D_2 - external diameter of the cutting edge

The area ratio should not exceed 25%. For good quality undisturbed samples, it should not exceed 10%.

b) Inside clearance

It allows elastic expansion of sample when it enters the tube.

$$\text{Inside Clearance} = \frac{D_3 - D_1}{D_1} \times 100$$

Where,

D3 - inside diameter of the sample tube

The inside clearance must lie between I to 10%, for undisturbed sample it should be between 0.5 and 3%.

c) Outside clearance

$$\text{Outside Clearance} = \frac{D_2 - D_4}{D_4} \times 100$$

Where,

D4 - the external diameter of the sample tube

It should not be much greater than the inside clearance. Normally it lies between 0 and 2 percent. It helps in reducing the force required to withdraw the tube.

d) **Inside wall friction**

The walls of the sampler should be smooth and kept properly oiled.

e) Non-return valve

The non-return valve should permit easy and quick escape of water and air when the sample is driven.

f) Recovery ratio

$$R = \frac{L}{H} \times 100$$

Where,

L - length of the sample within the tube

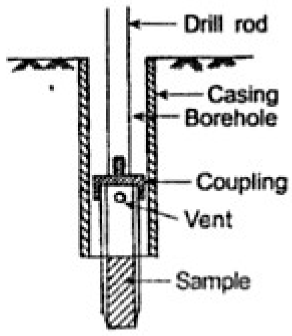
H - depth of penetration of the sampling tube

It represents the disturbance of the soil sample. For good sampling the recovery ratio should be 96 to 98 %.

SOIL SAMPLES AND SAMPLERS

DIFFERENT TYPES OF SAMPLERS

1. Open Driver Sampler



Open driver sampler can be of the thick wall type as well as of the thin wall type. The head of the sampler is provided with values to permit water and air to escape during driving. The check valve helps to retain the sample when the sampler is lifted. The tube may be seamless. If the tube splits in two parts which is called as split tube or split spoon sampler.

2. Split Spoon Sampler

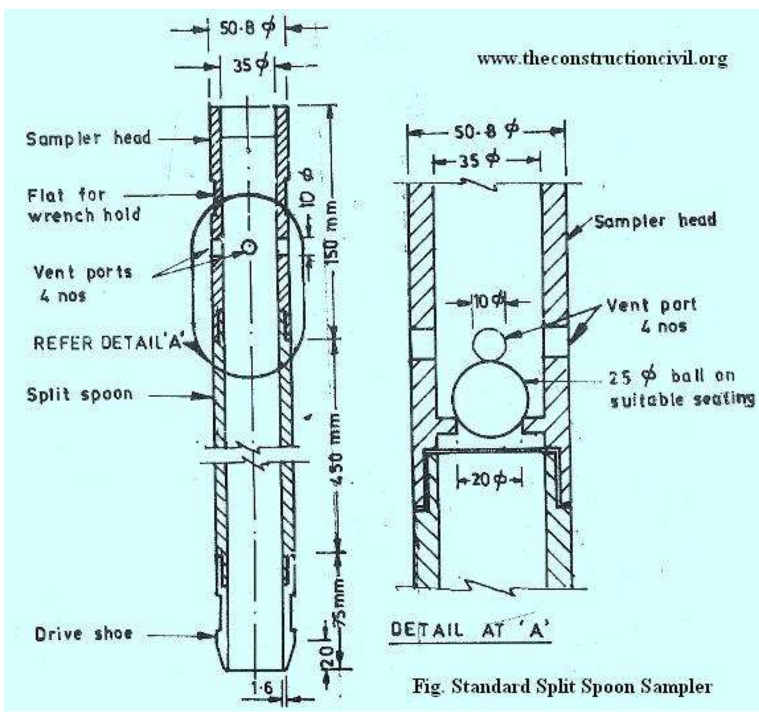
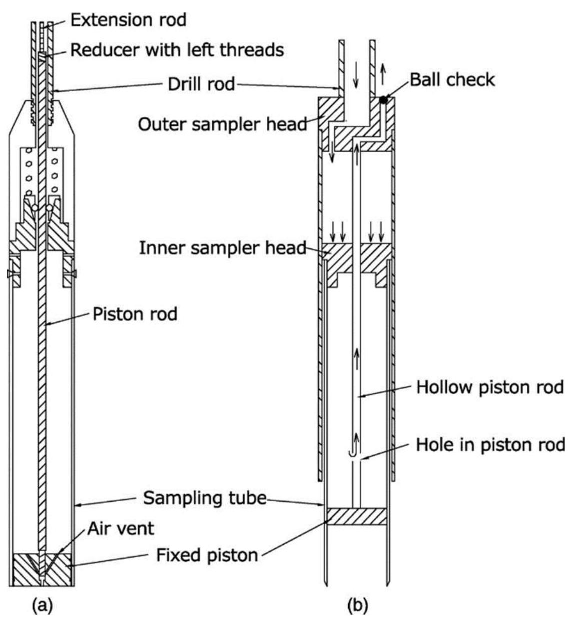


Fig. Standard Split Spoon Sampler

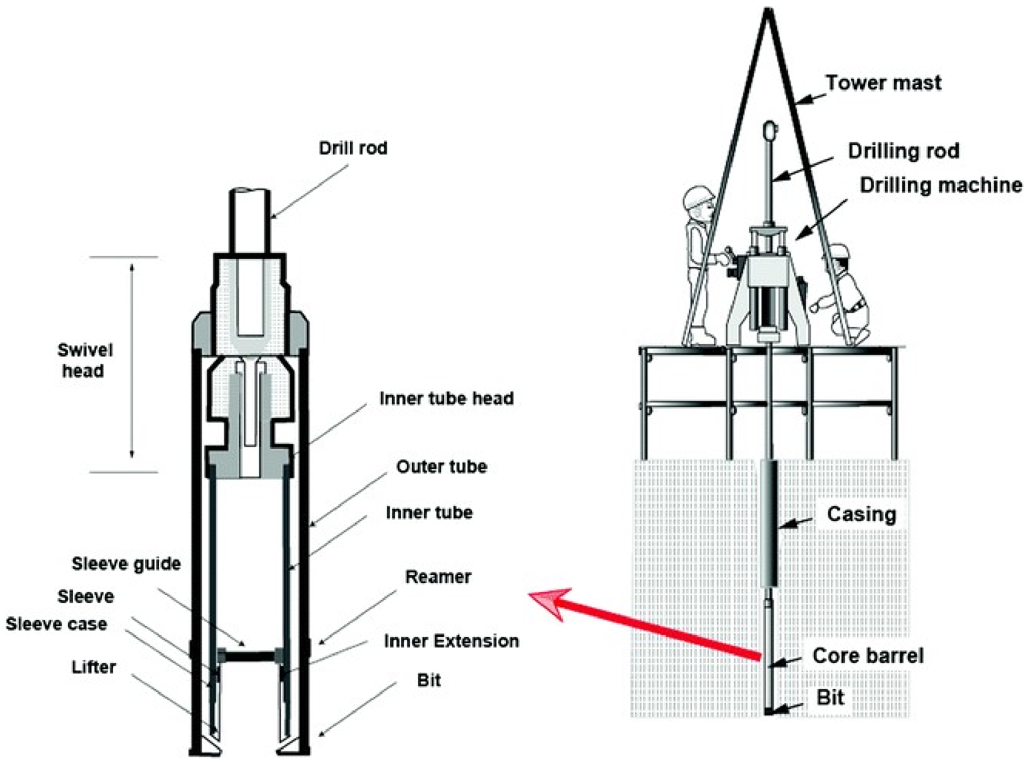
It consists of tool-steel driving shoe at the bottom, a steel tube (that is split longitudinally in to halves) in the middle, and a coupling at the top. The steel tube in the middle has inside and outside diameters of34.9mm and 50.8mm, respectively. When the bore hole is advanced to a desired depth, the drilling tools are removed. The split spoon sampler is attached to the drilling rod and then lowered to the bottom of the bore hole. The sampler is driven in to the soil at the bottom of the bore hole by means of hammer blows. The hammer blows occur at the top of the drilling rod. The hammer weights 623N. For each blow, the hammer drops a distance of O.762m. The number of blows required for driving the sampler through three 152.4mm interval is recorded. The sum of the number of blows required for driving the last two 152.4mm intervals is referred to as the standard penetration number; N. it is also commonly called the blow count. After driving is completed, the sampler is withdrawn and the shoe and coupling are removed. The soil sample collected inside the split tube is then removed and transferred to the laboratory in small glass jars. Determination of the standard penetration number and collection of split-spoon samples are usually done at 1.5m.

3. Stationary Piston Sampler



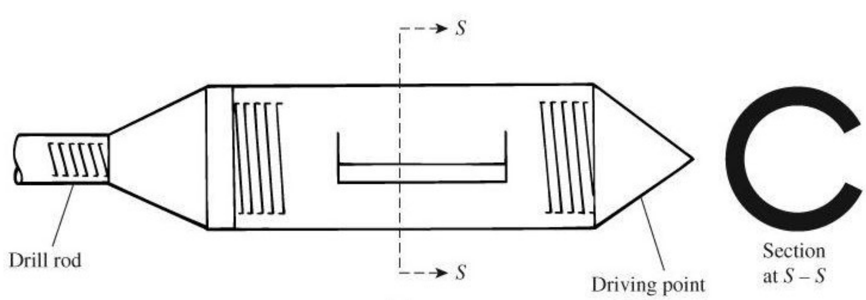
Stationary piston sampler consists of a sampler with a piston attached to a long piston rod extending up to the ground surface through the drill rods. The lower end of the sampler is kept closed with piston while the sampler is lowered through the bore hole. When the desired elevation reached, the piston rod is clamped; thereby keeping the piston stationary and the sampler tube is advanced further into the soil. The sampler is then lifted and the piston rod clamped in position. The piston prevents the entry of water and soil into the tube, when it is being lowered and also helps to retain the sample during the process of lifting the tube. The sampler is therefore very much being suited for sampling in soft soils and saturated sands.

4. Rotary Sampler



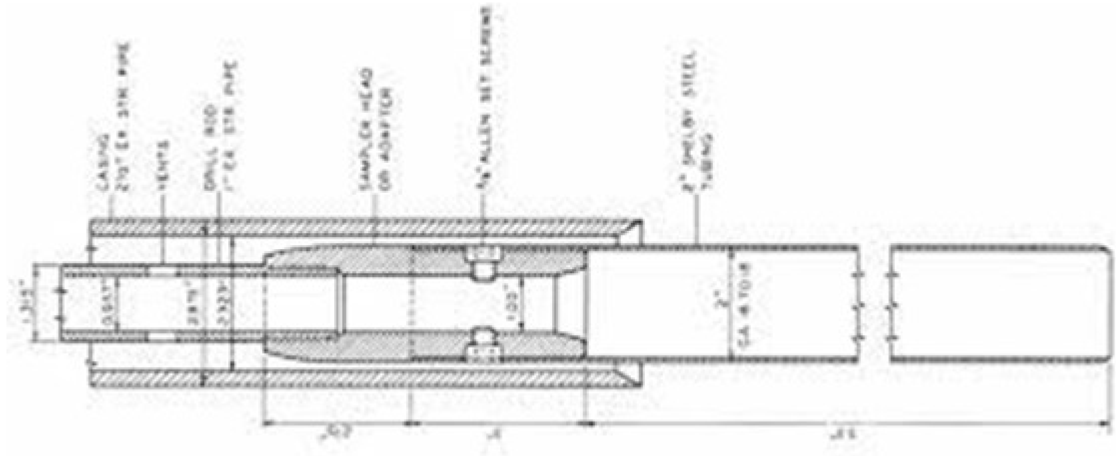
Rotary samplers are core barrel type with an outer tube provided with cutting teeth and a removable thin liner inside. It is used for sampling in stiff cohesive soils.

5. Scraper Bucket Sampler



If a sandy deposit contains pebbles, it is not possible to obtain samples by standard split spoon sampler by standard split spoon sampler or split spoon fitted with a spring core catcher. The pebbles come in between the springs and prevent their closure. For such deposits, a scraper bucket sampler can be used. A scraper bucket sampler can also be used for obtaining the samples of cohesion less soils below the water table.

6. Shelby Tubes and Thin-Walled Sampler



Shelby tubes are thin wall tube samplers made of seamless steel. The outside diameter of the tube may be between 40 to 125mm. The area ratio is less than 15% and the inside clearance is between 0.5 to 3%. The length of the tube is 5 to 10 times the diameter for sandy soils and 10 to 15 times the diameter for clayey soils. The diameter generally varies between 40 and 125mm and thickness varies from 1.25 to 3.15mm. The sampler tube is attached to the drilling rod and lowered to the bottom of the bore hole. It is then pushed into the soil. Care should be taken to push the tube into the soil by a continuous rapid motion without impact or twisting. The tube should be pushed to the length provided for the sample. At least 5 minutes after pushing the tube into its final position, the tube is turned revolutions to shear the sample off at the bottom before it is withdrawn. The tube is taken out and its ends are sealed before transportation. Shelby tubes are used for obtaining undisturbed samples of clay.

7. Denison Sampler

The Denison sampler is a double walled sampler. The outer barrel rotates and cuts into the soil. The sample is obtained in the inner barrel. The inner barrel is provided with a liner. It may also be provided with a basket type core retainer. The Denison sampler is mainly used for obtaining samples of stiff to hard cohesive soils and slightly cohesive sands. However, it cannot be used for gravelly soils, loose cohesion less sands and silts below ground water table and very soft cohesive soils.

Hand-Curved Samples

Hand curved samples can be obtained if the soil is exposed, as in a test pit, shaft or tunnel. Hand curved samples are also known as chunk samples. The soil should have at least a trace of cohesion so that it can stand unsupported for sometimes. To obtain a sample, a column of soil is isolated in the pit. The soil is carefully removed from around the soil column and it is properly trimmed. An open-ended box is then placed over the soil column. The space between the box and the soil column is fitted with paraffin. A spade or a plate with sharp edges is inserted below the box and the sample is cut at its base. The box filled with the soil sample is removed. It is turned over and the soil surface in the box is trimmed and any depression is filled with paraffin. A chunk sample may be obtained without using the box if the soil is cohesive. A column of soil is isolated. The block of soil is carefully removed from the soil column with sharp knife. The chunk sample is then coated with paraffin wax to prevent loss of moisture. Samples from open pits can also be obtained by pressing a sampling tube provided with a cutting edge. The soil surrounding the outside of the tube is carefully removed while the tube is being pushed into the soil. Hand – curved samples are undisturbed.

Preservation of samples

Undisturbed samples which are to be tested after some time should be maintained in such a way that the natural water content is retained and no evaporation are allowed. Usually, two coats of 12mm thick paraffin wax and petroleum jelly are applied in molten state on either end, of the sample is preserved in a humidity-controlled room. In the absence of such facilities, the sampling tubes should be covered by Hessian bags and sprinkled with water from time to time. Block samples may be coated with 6mm thick paraffin wax and kept in air-tight box with saw dust filling the annular space between the box and the sample.

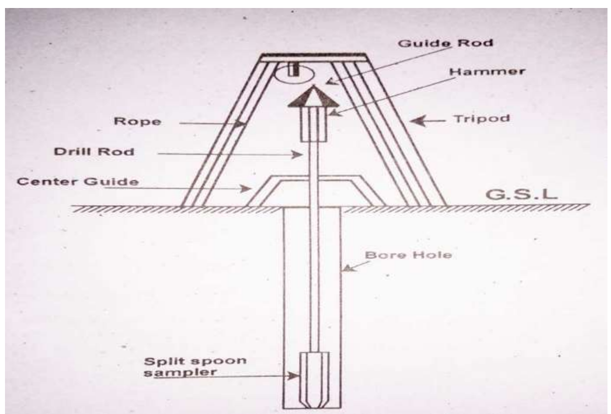
FIELD TESTS

The in-situ tests in the field have the advantage of testing the soils in their natural, undisturbed condition. Laboratory tests, on the other hand, make use of small size samples obtained from boreholes through samplers and therefore the reliability of these depends on the quality of the so called 'undisturbed' samples. Further, obtaining undisturbed samples from non-cohesive, granular soils is not easy, if not impossible. Therefore, it is common practice to rely more on laboratory tests where cohesive soils are concerned. Further, in such soils, the field tests being short duration tests, fail to yield meaningful consolidation settlement data in any case. Where the subsoil strata are essentially non-cohesive in character, the bias is most definitely towards field tests. The data from field tests is used in empirical, but time-tested correlations to predict settlement of foundations. The field tests commonly used in subsurface investigation are:

- Penetration Test
- Standard cone Penetration test
- Plate load test

STANDARD PENETRATION TEST

The standard penetration test is carried out in a borehole, while the DCPT and SCPT are carried out without a borehole. All the three tests measure the resistance of the soil strata to penetration by a penetrometer. Useful empirical correlations between penetration resistance and soil properties are available for use in foundation design. This is the most extensively used penetrometer test and employs a split-spoon sampler, which consists of a driving shoe, a split-barrel of circular cross-section which is longitudinally split into two parts and a coupling. IS: 2131-1981 gives the standard for carrying out the test.



Procedure:

- The borehole is advanced to the required depth and the bottom cleaned.
- The split-spoon sampler, attached to standard drill rods of required length is lowered into the borehole and rested at the bottom.
- The split-spoon sampler is driven into the soil for a distance of 450mm by blows of a drop hammer (monkey) of 65 kg falling vertically and freely from a height of 750 mm. The number of blows required to penetrate every 150 mm is recorded while driving the sampler. The number of blows required for the last 300 mm of penetration is added together and recorded as the N value at that particular depth of the borehole. The number of blows required to effect the first 150mm of penetration, called the seating drive, is disregarded.
- The split-spoon sampler is then withdrawn and is detached from the drill rods. The split-barrel is disconnected from the cutting shoe and the coupling. The soil

sample collected inside the split barrel is carefully collected so as to preserve the natural moisture content and transported to the laboratory for tests. Sometimes, a thin liner is inserted within the split-barrel so that at the end of the SPT, the liner containing the soil sample is sealed with molten wax at both its ends before it is taken away to the laboratory.

The SPT is carried out at every 0.75 m vertical intervals in a borehole. This can be increased to 1.50 m if the depth of borehole is large. Due to the presence of boulders or rocks, it may not be possible to drive the sampler to a distance of 450 mm. In such a case, the N value can be recorded for the first 300 mm penetration. The boring log shows refusal and the test is halted if-

- 50 blows are required for any 150mm penetration
- 100 blows are required for 300m penetration.
- 10 successive blows produce no advance.

Precautions:

- The drill rods should be of standard specification and should not be in bent condition.
- The split spoon sampler must be in good condition and the cutting shoe must be free from wear and tear.
- The drop hammer must be of the right weight and the fall should be free, frictionless and vertical.
- The height of fall must be exactly 750 mm. Any change from this will seriously affect the N value.
- The bottom of the borehole must be properly cleaned before the test is carried out. If this is not done, the test gets carried out in the loose, disturbed soil and not in the undisturbed soil. When a casing is used in borehole, it should be ensured that the casing is driven just short of the level at which the SPT is to be carried out. Otherwise, the test gets carried out in a soil plug enclosed at the bottom of the casing.
- When the test is carried out in a sandy soil below the water table, it must be ensured that the water level in the borehole is always maintained slightly above the ground water level. If the water level in the borehole is lower than the ground water level, 'quick' condition may develop in the soil and very low N values may be recorded.

In spite of all these imperfections, SPT is still extensively used because-

- The test is simple and relatively economical.

- It is the only test that provides representative soil samples both for visual inspection in the field and for natural moisture content and classification tests in the laboratory.

SPT values obtained in the field for sand have to be corrected before they are used in empirical correlations and design charts. IS: 2131-1981 recommends that the field value of N be corrected for two effects, namely

(a) effect of overburden pressure

(b) effect of dilatancy.

(a) Correction for overburden pressure

Several investigators have found that the penetration resistance or the N value in a granular soil is influenced by the overburden pressure. Of two granular soils possessing the same relative density but having different confining pressures, the one with a higher confining pressure gives a higher N value. Since the confining pressure (which is directly proportional to the overburden pressure) increases with depth, the N values at shallow depths are underestimated and the N values at larger depths are overestimated. To allow for this, N values recorded from field tests at different effective overburden pressures are corrected to a standard effective overburden pressure.

The corrected N values given by-

$$N' = C_N N$$

Where,

N' – Corrected value of observed N

C_N – Correction factor for overburden pressure

b) Correction for dilatancy

Dilatancy correction is to be applied when N' obtained after overburden correction, exceeds 15 in saturated fine sands and silts. IS: 2131-1981 incorporates the Terzaghi and Peck recommended dilatancy correction (when N'> 15) using the equation

$$N'' = 15 + 0.5\,(N' - 15)$$

Where,

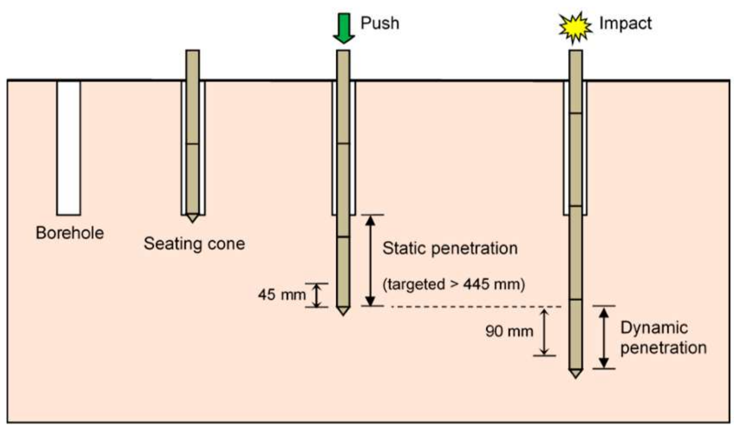
N'' - final corrected value to be used in design charts

If $N' < 5.15$, $N = N'$

N' > 15 is an indication of a dense sand. In such a soil, the fast rate of application of shear through the blows of a drop hammer, is likely to induce negative pore water pressure in a saturated fine sand under undrained condition of loading. Consequently, a transient increase in shear resistance will occur, leading to a SPT value higher than the actual one.

CONE PENETRATION TEST

The cone test was developed by the Dutch government, soil mechanics laboratory at Defit and is therefore also known as Dutch cone test. The test is conducted either by the Static method or by dynamic method.



Static Cone Penetration Test

The Dutch cone has an apex angle of 60 and an overall diagram of 35.7mm giving an overall diagram of 35.7mm giving an end area of 10cm^2. For obtaining the cone resistance, the cone is pushed downward at a steady rate of 10mm/sec through a depth of 35mm each time. The cone is pushed by applying thrust and not by driving. After the cone resistance has been determined the cone is withdrawn. The sleeve is pushed onto the cone both are driven together into the soil and the combined resistance is also determined. The resistance of the sleeve alone is obtained by subtracting the cone resistance from the combined resistance. A modification of the Dutch cone penetrometers is the refined Dutch cone. It has got a friction sleeve of limited length above the cone point. It is used for obtaining the point resistance of the cone and the frictional resistance of the soil above cone point.

For effective use of the cone penetration test, some reliable calibration is required. This consists of comparing the results with that Dutch cone obtained from conventional tests conducted on undisturbed sample in a laboratory. It is also convenient to compare the cone test results with SPT results, are related to the SPT number N, indirect correlations are obtained between the cone tests and the engineering properties of the soil.

The following relation holds approximately good between the point resistance of the cone (9c) and the standard penetration Number (N)

i) Gravels 9c = 800N to 1000N

ii) Sands 9c = 500N to 600N

iii) Silly sands 9c = 300N to 400N

iv) Sills & clayey 9c = 200N where 9c is in KN/m^2 silts

Dynamic Cone Penetration Test

The test is conducted by driving the cone by blows of a hammer. The number of blows for driving the cone through a specified distance is a measure of dynamic cone resistance. It is performed either by using a 50mm cone without bentonite slurry or by using a 65mm cone with bentonite slurry (IS 4968 – part I &II 1976) The driving energy is given by 65kg hammer falling through a height of 75cm. The number of blows required for 30cm of penetration is taken as the dynamic cone resistance (N_{cbr}). If the skin friction is to be eliminated, the test is conducted in a cased bore hole. When a 65mm cone with bentonite slurry is used, the set up should have arrangement for circulating slurry so that the friction on the driving rod is eliminated. The dynamic cone resistance (N_{cbr}) is correlated with the SPT number N. the following approximate relations may be used when a 50mm diameter cone is used.

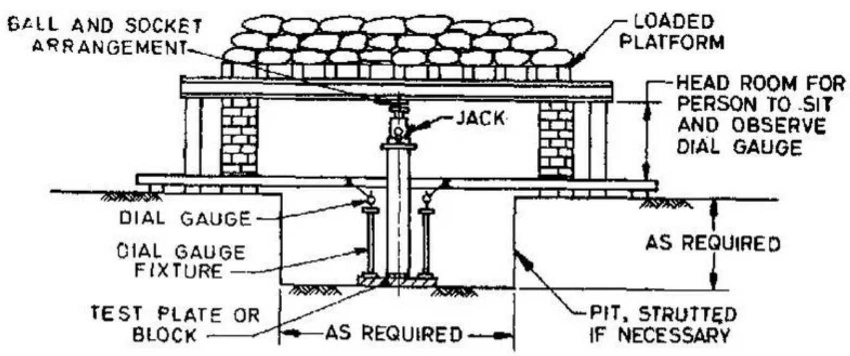
N_{cbr}	Depth
1.5N	< 3m
1.75N	3 – 6m
2.0N	> 6m

The central building research Institute, Roorkee has developed the following correlation between the dynamic cone resistance (N_{cbr}) of SPT Number N. It is applicable for medium to fine sand.

N_{cbr}	Depth
1.5N	< 4m
1.75N	4 – 9m
2.0N	> 9m

PLATE LOAD TEST

The plate load test is a semi-direct method to measure the allowable pressure of soil to induce a given amount of settlement. Plates, round or square, varying in sizes, from 30 to 60 cm and thickness of about 2.5 cm are employed for the test. The load on the plate is applied by making use of a hydraulic jack. The reaction of the jack load is taken by a cross beam or a steel truss anchored suitably at both the ends. The settlement of the plate is measured by a set of three dial gauges of sensitivity 0.02mm placed at 1200 apart. The dial gauges are fixed to independent supports which do not get disturbed during the test. Fig shows the arrangement for the plate load test.



Procedure:

- Excavate a pit of size not less than 5 times the size of the plate. The bottom of the pit coincides with level of the foundation.
- If water table is above the level of foundation, pump out the water carefully and it should be kept just at the level of the foundation.
- A suitable size of the plate is selected for the test. Normally a plate of size 30cm is used in sandy soil and bigger size in clay soils. The ground should be leveled and the plate is seated over the ground.
- A seating load of about 70g/cm2 is first placed and released after sometime. A higher load is next placed on the plate and-settlements are recorded by means of the dial gauges.
- Observations on every load increment shall be taken until the rate of settlement is less than 0.25mm per hour. Load increments shall be approximately one-fifth of the estimated safe bearing capacity of the soil. The average of the settlements recorded by 2 or 3 dial gauges taken as the settlements of the plate for each of the load increment.

- The test should continue until a total settlement of 2.5cm or the settlement at which the soil fails, whichever is earlier, is obtained. After the load is increased, the elastic rebound of the soil should be recorded.

Interpretation from test results:

The allowable pressure of the prototype foundation for an assumed settlement may be found and by making use of the following equations as suggested by Terzaghi and Peck.

For granular soils,

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

For clay soils,

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

Where,

S_f = permissible settlement of the foundation in mm

S_p = settlement of the plate in mm

B = size of plate in m

b_p = size of plate in m

The permissible settlement S_f for a prototype foundation should be known. Normally a settlement of 2.5cm is recommended. In the equation the values of S_f and b_p are known. The unknowns are S_p and B. The value of S_p for any assumed value of B may be found out from the equation. Using the plate load settlement curve, the value of the bearing pressure corresponding to the computed value of S_p is found out. This bearing pressure is the safe bearing pressure for a given permissible settlement S_f.

Limitations

- Since a plate load test is of short duration, consolidation settlements cannot be predicted. The test gives the value of immediate settlements only. If the underlying soil is sandy in nature immediate settlement can be taken as total settlement. If the soil is of clayey type, the immediate settlement is only a part of the total settlement. Load tests, therefore do not have much significance in clayey soils to determine allowable pressure on the basis of settlement criterion.

- Plate load test results should be used with caution and the present practice is not to rely too much on this test. If the soil is not homogenous to a great depth, plate load tests give very misleading results.
- Plate load tests is not at all recommended in soils which are not homogenous at least to a depth equal to 1.5 to 2 times the width of the prototype foundation.
- Plate load tests should not be relied on to determine the ultimate bearing capacity of sandy soils as the scale effects give misleading results. However, when the tests are carried on clay soils, the ultimate bearing capacity as determined by the test may be taken as equal to that of the foundation since the bearing capacity of clay is essentially independent of the footing size. The plate load test is possibly the only way of determining the allowable bearing pressures in gravelly soil deposits. For tests on such soil deposits the size of the plate should be bigger to eliminate the effect of grain size.

GEOPHYSICAL METHODS OF SOIL EXPPLORATION

SEISMIC REFRACTION METHOD

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 is 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

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

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

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

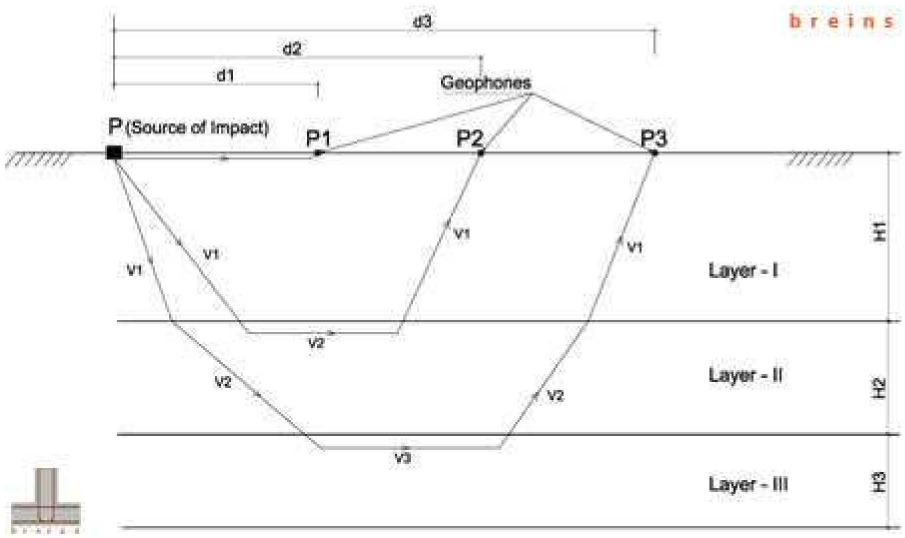
i_1 is the angle of incidence

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.



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.

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.

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.

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

where,

ρ is resistivity in ohm-cm

R is resistance in ohms

A is the cross-sectional area (cm^2)

L is the length of the conduction (cm)

Procedure

In this method the electrodes and driven approximately 20cm 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.

$$V_C = \frac{I\rho}{2\pi}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)$$

$$V_D = \frac{I\rho}{2\pi}\left(\frac{1}{r_3} - \frac{1}{r_4}\right)$$

Where,

ρ is resistivity

I is current

r_1, r_2, r_3 and r_4 are the distances between the various electrodes

Potential difference between C and D = V_{CD}

$$V_C - V_D = \frac{I\rho}{2\pi}\left[\left(\frac{1}{r^1} - \frac{1}{r^2}\right) - \left(\frac{1}{r_3} - \frac{1}{r_4}\right)\right]$$

$$P = \frac{2\pi VCD}{I}\left[\frac{1}{\left(\frac{1}{r^1}-\frac{1}{r^2}\right)-\left(\frac{1}{r^3}-\frac{1}{r^4}\right)}\right]$$

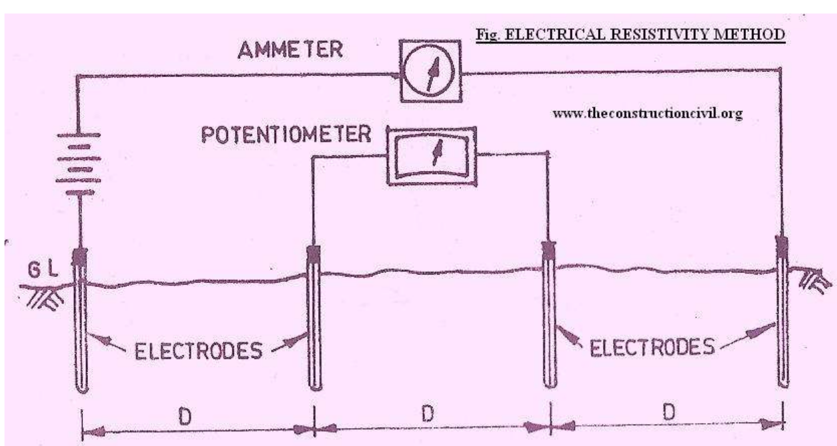
If $r_1 = r_4 = (r_2/2) = (r_3/2)$ Then the resistivity is given as

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

Where,

Resistances R= VCD/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.



In practice many several different arrays are used. For simple sounding a Wenner array is used. Then the resistivity is given as

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

Where a is the spacing between the electrodes. The Schlumberger array is used for profiling and sounding. In sounding configuration, the current electrodes separated by AB are symmetric about the potential electrodes MN. The current electrodes are then expanded and the resistivity is given as

$$\rho = \frac{\pi(s^2 - \frac{a^2}{4})}{a} R$$

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

- 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.

Foundation Engineering

ies

UNIT 2
SHALLOW FOUNDATION

INTRODUCTION

A shallow foundation is a type of foundation which transfers building loads to the earth very near the surface, rather than to a subsurface layer or a range of depths as does a deep foundation.

BASIC TERMINOLOGIES

Gross Loading Intensity

Total pressure at the level of foundation including the weight of superstructure, foundation, and the soil above foundation.

Net Loading Intensity

Pressure at the level of foundation causing actual settlement due to stress increase. This includes the weight of superstructure and foundation only.

$$q_n = q - \gamma D$$

Ultimate Bearing capacity:

Maximum gross intensity of loading that the soil can support against shear failure is called ultimate bearing capacity.

Net Ultimate Bearing Capacity

Maximum net intensity of loading that the soil can support at the level of foundation.

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

Net Safe Bearing capacity:

Maximum net intensity of loading that the soil can q safely support without the risk of shear failure.

$$q_{ns} = \frac{q_{nf}}{FOS}$$

Gross Safe Bearing capacity:

Maximum gross intensity of loading that the soil can safely support without the risk of shear failure.

$$q_s = \frac{q_{nf}}{FOS} - \gamma D$$

Safe Bearing Pressure:

Maximum net intensity of loading that can be allowed on the soil without settlement exceeding the permissible limit.

Allowable Bearing Pressure:

Maximum net intensity of loading that can be allowed on the soil with no possibility of shear failure or settlement exceeding the permissible limit.

GENERAL REQUIREMENTS OF FOUNDATION

- Location and Depth of Foundation
- Bearing Capacity of Foundation
- Settlement of Foundation

LOCATION AND DEPTH OF FOUNDATION

1.The following considerations are necessary for deciding the location and depth of foundation

- ✓ As per IS:1904-1986, minimum depth of foundation shall be 0.50 m.
- ✓ Foundation shall be placed below the zone of

  - The frost heave
  - Excessive volume change due to moisture variation (usually exists within 1.5 to 3.5 m depth of soil from the top surface)
  - Topsoil or organic material
  - Peat and Muck
  - Unconsolidated material such as waste dump

2. Foundations adjacent to flowing water (flood water, rivers, etc.) shall be protected against scouring. The following steps to be taken for design in such conditions

- Determine foundation type
- Estimate probable depth of scour, effects, etc.
- Estimate cost of foundation for normal and various scour conditions.
- Determine the scour versus risk, and revise the design accordingly.

3. IS:1904-1986 recommendations for foundations adjacent to slopes and existing structures

- When the ground surface slopes downward adjacent to footing, the sloping surface should not cut the line of distribution of the load (2H:1V).
- In granular soils, the line joining the lower adjacent edges of upper and lower footings shall not have a slope steeper than 2H:1V.
- In clayey soil, the line joining the lower adjacent edge of the upper footing and the upper adjacent edge of the lower footing should not be steeper than 2H:1V.

Other recommendations for footing adjacent to existing structures

- Minimum horizontal distance between the foundations shall not be less than the width of larger footing to avoid damage to existing structure
- If the distance is limited, the principal of 2H:1V distribution should be used so as to minimize the influence to old structure
- Proper care is needed during excavation phase of foundation construction beyond merely depending on the 2H:1V criteria for old foundations. Excavation may cause settlement to old foundation due to lateral bulging in the excavation and/or shear failure due to reduction in overburden stress in the surrounding of old foundation

4. Footings on surface rock or sloping rock faces

- For the locations with shallow rock beds, the foundation can be laid on the rock surface after chipping the top surface.

- If the rock bed has some slope, it may be advisable to provide dowel bars of minimum 16 mm diameter and 225 mm embedment into the rock at 1 m spacing.

5. A raised water table may cause damage to the foundation by

- Floating the structure
- Reducing the effective stress beneath the foundation

Water logging around the building may also cause wet basements. In such cases, proper drainage system around the foundation may be required so that water does not accumulate.

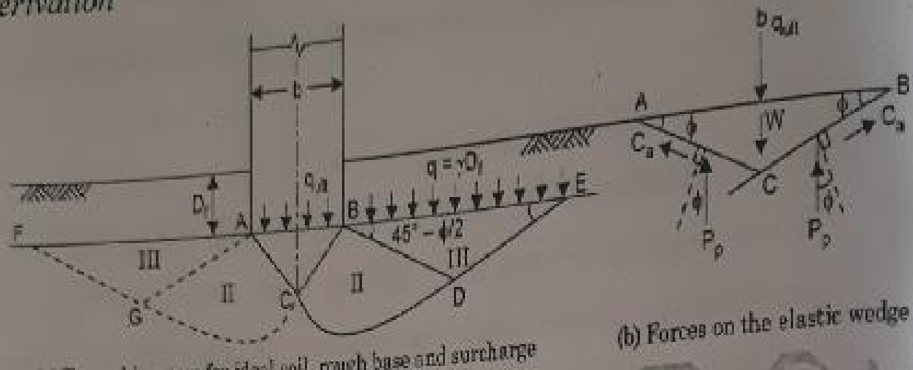
BEARING CAPACITY OF SHALLOW FOUNDATION

Terzaghi Analysis of Bearing Capacity

Assumptions

- Complete bearing capacity failure is general shear failure
- Soil behaves perfectly plastic
- Soil is homogeneous
- Soil is isotropic
- The shear strength is represented by Mohr Columb equation
- Footing is shallow
- Load on the Footing is vertical
- The soil above the base of the footing is replaced by the equivalent surcharge γD.

Derivation



(a) Terzaghi system for ideal soil, rough base and surcharge

(b) Forces on the elastic wedge

Let

B = width of footing
q_f = intensity of loading
γ = unit weight of soil

From the figure the load at the soil below the base of footing fails along the surface depth ff'. The failure surface is divided as a pair of zone iii , pair of zone ii and zone i.

1. The base of footing AB sinks into the ground due to the intensity of loading which is prevented by cohesive forces along the surface ab and ad.
2. The boundaries ab and ad is assumed as plain surface and raising angle (Φ=Ψ) to the horizontal.
3. The application of loading intensity pushes ad and db with a lateral displacement of zone ii and iii which is resisted by a passive force p_p.

4. The passive force p_p makes an angle Φ with the surface normal to ab and ad.
5. The downward forces are
   - Intensity of loading ($D=q_f.B$)
   - Weight of the wedge

Total downward force = $q_f.B$ + weight of the wedge

Weight of the wedge

Weight of the soil = area of the triangle x unit weight of the soil

Area of triangle = 0.5 bh

$h = (B/2) \times \tan\Phi$ and $b=B$

Therefore,

Area= $(B^2 \tan\Phi \times \gamma)/4$

Weight of the wedge = $[(B^2 \tan\Phi \times \gamma)/4] \times \gamma$

6. The upward forces are
   - Passive force along the surface ad and ab
   - Cohesive force along the surface ab and ad

ad or ab = $B/(2 \cos\Phi)$

cohesive forces along the surface = $c \sin\Phi + [B/(2 \cos\Phi)] = (B\ c \tan\Phi)/2$

Total upward force = $2 P_p + [(B\ c \tan\Phi)/2]$

Based on the equilibrium conditions

Total upward force = Total downward force

$2 P_p + [(B\ c \tan\Phi)/2] = q_f.B + [(B^2 \tan\Phi \times \gamma)/4] \times \gamma$

Assume that P_p can be divided into 3 components

1. $P_p\gamma$ – force produced by the wedge
2. $P_p c$ – force produced by the soil cohesion
3. $P_p q$ – force produced by the surcharge

$$q_f.B = 2\,[P_p\gamma + P_p c + P_p q] + [\,(B\,c\tan\Phi)/2] - \{[(B^2 \tan\Phi \times \gamma)/4] \times \gamma\}$$

Let the terms

$2\,P_p\gamma - \{[(B^2 \tan\Phi \times \gamma)/4] \times \gamma\} = 0.5\,B\,\gamma\,N_\gamma$

$2\,P_p c + [\,(B\,c\tan\Phi)/2] = B\,c\,N_c$

$2\,P_p q = B\,\gamma\,D\,N_q$

According to the theory of Terzaghi

$q_f.B = 0.5\,B\,\gamma\,N_\gamma + B\,c\,N_c + B\,\gamma\,D\,N_q$

$q_f = 0.5\,\gamma\,N_\gamma + c\,N_c + \gamma\,D\,N_q$

$q_f = c\,N_c + \gamma\,D\,N_q + 0.5\,B\,\gamma\,N_\gamma$ [for general shear failure]

$q_f = 2/3[\,c\,N_c] + \gamma\,D\,N_q + 0.5\,B\,\gamma\,N_\gamma$ [for local shear failure]

For c-Φ soil

Circular, $q_f = 1.3\,c\,N_c + \gamma\,D\,N_q + 0.5\,B\,\gamma\,N_\gamma$

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

Rectangular, $q_f = [1 + 0.3B/L]\,c\,N_c + \gamma\,D\,N_q + 0.5\,B\,\gamma\,N_\gamma$

For Cohesive Soil

Circular, $q_f = 1.3\,c\,N_c + \gamma\,D\,N_q$

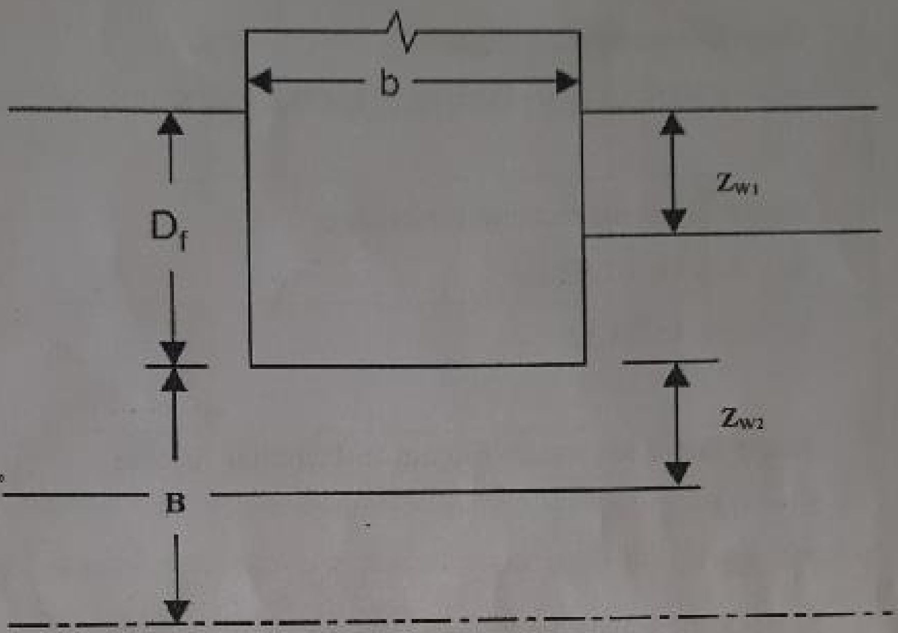
Rectangular and square, $q_f = [1 + 0.3B/L]\,c\,N_c + \gamma\,D\,N_q$

For Cohesionless soil

Circular, $q_f = \gamma\,D\,N_q + 0.4\,B\,\gamma\,N_\gamma$

Rectangular and square, , $q_f = \gamma\,D\,N_q + 0.4\,B\,\gamma\,N_\gamma$

Correction for water table



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

where,

$R_{w1} = 0.5[\,1 + (z_{w1}/D)]$

$R_{w2} = 0.5[\,1 + (z_{w2}/B)]$

Limitations

1. As the soil compresses, pi changes slight down ward movement of footing may not develop fully the plastic zones
2. Error due to assumption that the resultant passive pressure consists of three components is small.

BIS Formula for Bearing capacity

Net ultimate bearing capacity

$q_{nf} = c\,N_c\,S_c\,d_c\,i_c + \gamma\,D\,N_q\,S_q\,d_q\,i_q + 0.5\,\gamma\,N_\gamma\,S_\gamma\,d_\gamma\,i_\gamma\,w'$

Shape factor for rectangular footing

$S_c = S_q = 1 + 0.2\,(B/L)$

$S_\gamma = 1 - 0.4\,(B/L)$

Shape factor for square footing and circular footing

$S_c = 1.3$

$S_q = 1.2$

$S_\gamma = 0.8$ for square footing and 0.6 for circular footing

Depth factor

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

$d_q = d_\gamma = 1 + 0.1\,(D_f/B)\tan(45 + \Phi/2)$ for $\Phi \geq 10$

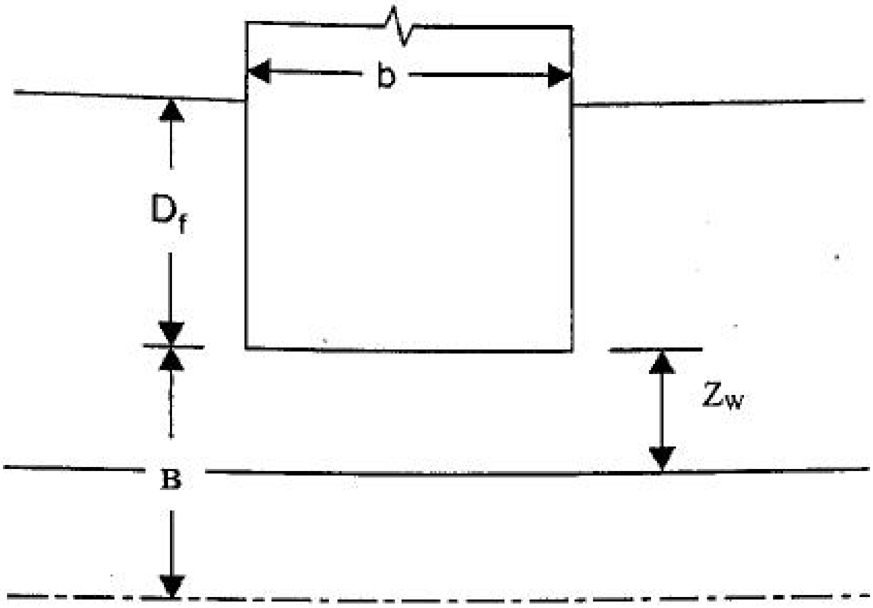
$d_q = d_\gamma = 1$ for $\Phi < 10$

Inclination Factor

$i_c = i_q = [1 - (\beta/90)]^2$

$i_\gamma = [1 - (\beta/\Phi)]^2$

Water table Correction



w′=1 when water table is below the zone of D+B

w′=0.5 when water table is at ground level or till the base of footing

w′= 0.5 [1+ (Z_w/B)] when D < D_w < D+B

FOUNDATION SETTLEMENT

Foundations of all structures have to be placed on soil. The structure may undergo settlement depending upon the characteristics such as compressibility of the strata of soil on which it is founded. Thus the term 'settlement' indicates the sinking of a structure due to the compression and deformation of the underlying soil. Clay strata often need a very long time—a number of years—to get fully consolidated under the loads from the structure. The settlement of any loose strata of cohesion less soil occurs relatively fast. Thus, there are two aspects—the total settlement and the time-rate of settlement—which need consideration. If it can be assumed that the expulsion of water necessary for the consolidation of the compressible clay strata takes place only in the vertical direction, Terzaghi's theory of one dimensional consolidation may be used for the determination of total settlement and also the time-rate of settlement.

Depending upon the location of the compressible strata in the soil profile relative to the ground surface, only a part of the stress transmitted to the soil at foundation level may be transmitted to these strata as stress increments causing consolidation. The theories of stress distribution in soil have to be applied appropriately for this purpose.

The vertical stress due to applied loading gets dissipated fast with respect to depth and becomes negligible below a certain depth. If the compressible strata lie below such depth, their compression or consolidation does not contribute to the settlement of the structure in any significant manner. There is the other aspect of whether a structure is likely to undergo 'uniform settlement' or 'differential settlement'. Uniform settlement or equal settlement under different points of the structure does not cause much harm to the structural stability of the structure. However, differential settlement or different magnitudes of settlement at different points underneath a structure—especially a rigid structure is likely to cause supplementary stress and thereby cause harmful effects such as cracking, permanent and irreparable damage, and ultimate yield and failure of the structure. As such, differential settlement must be guarded against.

The total settlement may be considered to consist of the following contributions:

(*a*) Immediate settlement or elastic compression.

(*b*) Primary Consolidation settlement or primary compression.

(*c*) Secondary Consolidation settlement or secondary compression.

Immediate Settlement

This is also referred to as the 'distortion settlement' or 'contact settlement' and is usually taken to occur immediately on application of the foundation load. Such immediate settlement in the case of partially saturated soils is primarily due to the expulsion of gases and to the elastic compression and rearrangement of particles. In the case of saturated soils immediate settlement is considered to be the result of vertical soil compression, before any change in volume occurs.

Immediate settlement in cohesion less soils

The elastic as well as the primary compression effects occur more or less together in the case of cohesion less soils because of their high permeability. The resulting settlement is termed 'immediate settlement'. The methods available for predicting this settlement are far from perfect; either the standard penetration test or the use of charts is resorted to.

$S_i = C_1\, C_2\, (\bar{q} - q) \sum_{z=0}^{2B} \frac{I_z}{E_s} \Delta z$

Where,

C_1 = Correction factor for depth of environment

$C_1 = 1 - 0.5 \left[\frac{q}{\bar{q} - q}\right]$

C_2 = Correction factor for creep in soil

$C_2 = 1 + 0.2 \log_{10}$ {time in years / 0.1}

$E_s = 2q_c$ or 766 N

q_c = Cone resistance

N= SPT N value

$\overline{q}$ = Presssure at the level of foundation

q = Surcharge = γ D

Immediate Settlement in Cohesive Soils

If saturated clay is loaded rapidly, excess hydrostatic pore pressures are induced; the soil gets deformed with virtually no volume change and due to low permeability of the clay little water is squeezed out of the voids. The vertical deformation due to the change in shape is the immediate settlement. The immediate settlement of a flexible foundation, according to Terzaghi (1943), is given by

$S_i = q\ B\ [(1-\mu^2)/E_s]\ I_f$

Where

S_i = Immediate settlement (mm)

B = width of the footing (m)

q = uniform pressure acting on the footing (KN/m^2)

E_s = Modulus of elasticity of the soil beneath the foundation (KN/m^2)

μ = Poissons ratio of soil

I_f = Influence value

For flexible footing

L/B	1	2	3	4	5
I_f	0.56	0.76	0.88	0.96	1

For rigid footing

L/B	1	2	5	10
I_f	0.82	1.0	1.22	1.26

Foundations are commonly more rigid than flexible and tend to cause a uniform settlement which is nearly the same as the mean value of settlement under a flexible foundation.

Primary Consolidation Settlement

The phenomenon of consolidation occurs in clays because the initial excess pore water pressures cannot be dissipated immediately owing to the low permeability. The theory of one-dimensional consolidation, advanced by Terzaghi, can be applied to determine the total compression or settlement of a clay layer as well as the time-rate of dissipation of excess pore pressures and hence the time-rate of settlement. The settlement computed by this procedure is known as that due to primary compression since the process of consolidation as being the dissipation of excess pore pressures alone is considered. The total consolidation settlement, *Sc*, may be obtained from one of the following equations:

$$S_c = \frac{HC_c}{1+e_0}\log_{10}\left(\frac{\overline{\sigma_0}+\Delta\bar{\sigma}}{\overline{\sigma_0}}\right)$$

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

$$S_c = \frac{\Delta e}{(1+e_0)}H$$

Secondary Consolidation Settlement

- Secondary compression is the compression of soil that takes place after primary consolidation. Even after the reduction of hydrostatic pressure some compression of soil takes place at slow rate. This is known as secondary compression. Secondary compression is caused by creep, viscous behavior of the clay-water system, compression of organic matter, and other processes. In sand, settlement caused by secondary compression is negligible, but in peat, it is very significant. Due to secondary compression some of the highly viscous water between the points of contact is forced out.

TOTAL SETTLEMENT

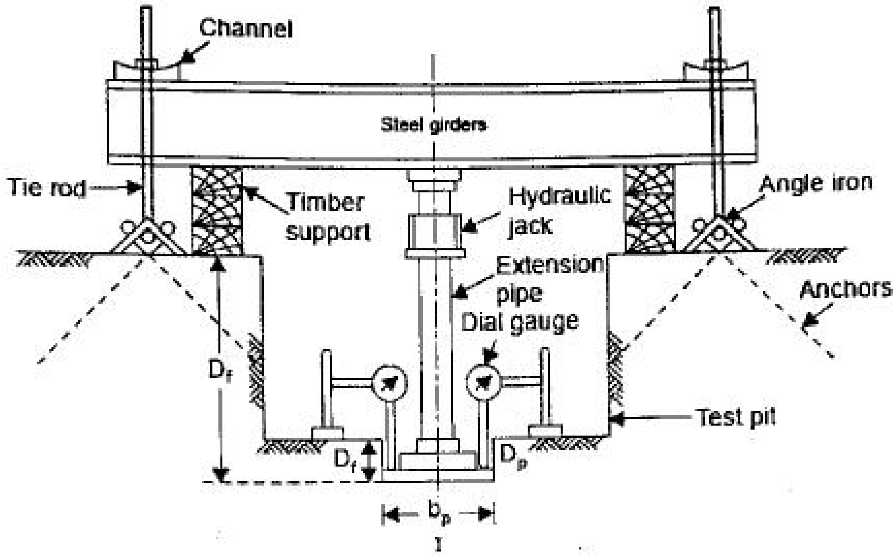
Total settlement = immediate settlement + primary consolidation settlement + secondary consolidation settlement

INSITU BEARING CAPACITY TESTS

PLATE LOAD TESTS

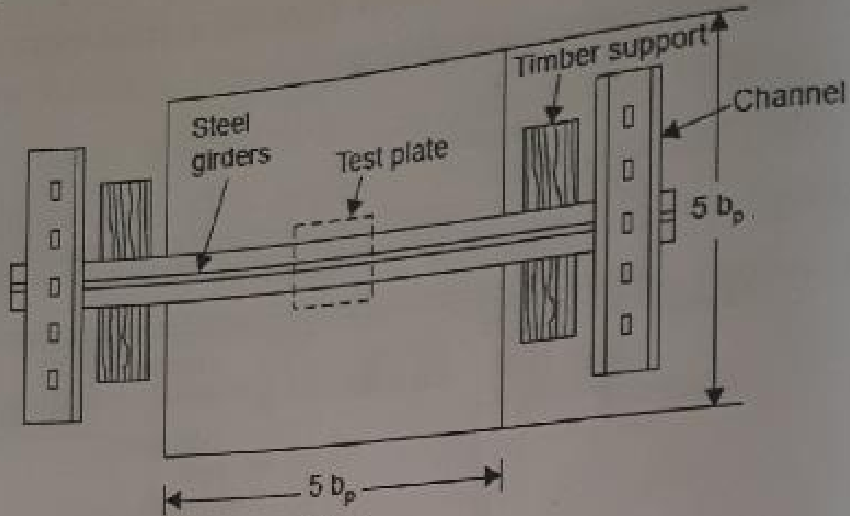
The most direct approach to obtain information on the bearing capacity and the settlement characteristics at a site is to conduct a load test. As tests on prototype foundation are not practicable in view of the large loading required, the time factor involved and the high cost of a full-scale test, a short-term model loading test, called the 'plate load test' or 'plate bearing test', is usually conducted. This is a semi-direct method since the differences in size between the test and the structure are to be properly accounted for in arriving at meaningful interpretation of the test results.

The test essentially consists in loading a rigid plate at the foundation level, increasing the load in arbitrary increments, and determining the settlements corresponding to each load after the settlement has nearly ceased each time a load increment is applied. The nature of the load applied may be gravity loading or dead weights on an improvised platform or reaction loading by using a hydraulic jack. The reaction of the jack load is taken by a cross beam or a steel truss anchored suitably at both ends.



Section

The test set-up with a jack is shown in Fig. Test plates are usually square or circular, the size ranging from 300 to 750 mm (side or diameter); the minimum thickness recommended is 25 mm for providing sufficient rigidity. If the loading set-up is a platform with dead weights, the kentledge may be in the form of sand bage, scrap iron or ingots or any other convenient heavy material. Jack-loading is superior in terms of accuracy and uniformity of loading. Settlement of the test plate is measured by means of at least two or three dial gauges with a least count of 0.02 mm.



Plan

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

$D_p/B_p = D_f/B_f$

where D_f and B_f are the depth and width of the proposed foundation. Bigger size plates are preferred in cohesive soils. The test procedure is given in IS: 1888–1982. The procedure, in brief, is as follows:

- After excavating the pit of required size and levelling the base, the test plate is seated over the ground. A little sand may be spread below the plate for even support. If ground water is encountered, it should be lowered slightly below the base by means of pumping.
- A seating pressure of 7.0 KN/m2 (70 g/cm2) is applied and released before actual loading is commenced.
- The first increment of load, say about one-tenth of the anticipated ultimate bearing capacity, is applied. Settlements are recorded with the aid of the dial gauges after 1 min., 4 min., 10 min., 20

min., 40 min., and 60 min., and later on at hourly intervals until the rate of settlement is less than 0.02 mm/hour, or at least for 24 hours.

- The test is continued until a load of about 1.5 times the anticipated ultimate load is applied. According to another school of thought, a settlement at which failure occurs or at least 2.5 cm should be reached.
- From the results of the test, a plot should be made between pressure and settlement, which is usually referred to as the "load-settlement curve", rather loosely. The bearing capacity is determined from this plot, which is dealt with in the next subsection.

Limitations of Plate Load Tests

Although the plate load test is considered to be an excellent approach to the problem of determining the bearing capacity by some engineers, it suffers from the following limitations:

(i) Size effects are very important. Since the size of the test plate and the size of the prototype foundation are very different, the results of a plate load test do not directly reflect the bearing capacity of the foundation. The bearing capacity of footings in sands varies with the size of footing; thus, the scale effect gives rather misleading results in this case. However, this effect is not pronounced in cohesive soils as the bearing capacity is essentially independent of the size of footing in such soils. The settlement versus size relationship is rather complex in the case of cohesionless soils (Terzaghi and Peck, 1948); however, in the case of

cohesive soils, this relation is rather simple, the settlement being proportional to the size. This should be considered appropriately in arriving at the bearing capacity based on the settlement criterion.

(ii) Consolidation settlements in cohesive soils, which may take years, cannot be predicted, as the plate load test is essentially a short-term test. Thus, load tests do not have much significance in the determination of allowable bearing pressure based on settlement criterion with respect to cohesive soils.

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

(iv) The load test results reflect the characteristics of the soil located only within a depth of about twice the width of the plate. This zone of influence in the case of a prototype footing will be much larger and unless the soil is essentially homogeneous for such a depth and more, the results could be terribly misleading. For example, if a weak or compressible stratum exists below the zone of influence of the test plate, but within the zone of influence of the prototype foundation, the plate test may not record settlements which are sure to occur in the case of the prototype foundation. This aspect has also been explained in Chapter 11 on "Settlement Analysis", with the aid of the pressure bulb concept. Perhaps the plate load test is the only good method for the determination of bearing capacity of gravel deposits; in such cases, bigger size plates are used to minimize the effect of grain size. Thus, it may be seen that interpretation and use of the plate load test results requires great care and judgment, on the part of the foundation engineer.

BEARING CAPACITY FROM SPT TEST

Ultimate bearing capacity can be determined from the SPT based on the N values.Net ultimate bearing capacity for 25 mm settlement can be determined from the formula.

$q_{nf} = 34.3\ (N-3)\ [(B+0.3)/2B]^2\ R_{w2}\ R_d$

Allowable gross safe pressure,

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

R_{w2} = Water table reduction factor = 0.5 $[(1+Z_{w2})/B]$

Z_{w2} = Depth of water table below the face of footing

R_d = Depth factor = $[1+(0.2\ D/B]$

SEISMIC CONSIDERATIONS IN BEARING CAPACITY OF SHALLOW FOUNDATION

Seismic loading could reduce the soil bearing capacity and cause large settlement or failure of the supported foundation. Mathematical analysis was developed in literature on calculating the seismic bearing capacity. The developed analysis used horizontal and vertical acceleration coefficients that applied by earthquake shaking in evaluating the seismic bearing capacity of dry soils. The intention of this contribution is to present a graphical method for checking the stability of designed shallow footings when subjected to seismic loads and to suggest the required minimum safety factor for new design.

DEPTH OF BEARING CAPACITY FAILURE

For static bearing capacity analyses, it is often assumed that the soil involved in the bearing capacity failure can extend to a depth equal to B (footing width) below the bottom of the footing. However, for cases involving earthquake-induced liquefaction failures or punching shear failures, the depth of soil involvement could exceed the footing width. For buildings with numerous spread footings that occupy a large portion of the building area, the individual pressure bulbs from each footing may combine, and thus the entire width of the building could be involved in a bearing capacity failure. Either a total stress analysis or an effective stress analysis must be used to determine the bearing capacity of a foundation.

It is commonly stated: "For the analysis of earthquake loading, the allowable bearing pressure and passive resistance may be increased by a factor of one-third." The rational behind this recommendation is that the allowable bearing pressure has an ample factor of safety, and thus for seismic analyses, a lower factor of safety would be acceptable. Usually the above recommendation is appropriate for the following materials:

(i) Massive crystalline bedrock and sedimentary rock that remains intact during the earthquake.

(ii) Dense to very dense granular soil

(iii) Heavily over consolidated cohesive soil, such as very stiff to hard clays

These materials do not lose shear strength during the seismic shaking, and therefore an increase in bearing pressure is appropriate.

A one-third increase in allowable bearing pressure should not be recommended for the following materials:

(i) Foliated or friable rock that fractures apart during the earthquake

(ii) Loose soil subjected to liquefaction or a substantial increase in excess pore water pressure

(iii) Sensitive clays that lose shear strength during the earthquake

(iv) Soft clays and organic soils that are overloaded and subjected to plastic flow

These materials have a reduction in shear strength during the earthquake. Since the materials are weakened by the seismic shaking, the static values of allowable bearing pressure should not be increased for the earthquake analyses. In fact, the allowable bearing pressure may actually have to be reduced to account for the weakening of the soil during the earthquake.

Seismic bearing capacity analysis is performed in two different ways:

1. Considering shear failure where the footing punches into the liquefied soil layer.

2. Using Terzaghi's bearing capacity equation, with a reduction in the bearing capacity factors to account for the loss of shear strength of the underlying liquefied soil layer

SETTLEMENT OF FOUNDATION

Uniform Settlement - The magnitude of the settlement that should occur, when foundation loads are applied to the ground, depend on the rigidity of substructure and compressibility of the underlying strata. In silts and clays the settlement may continue for a long period after the construction of structure. Due allowance shall, therefore, need be made for this slow consolidation settlement. In sand and gravels, the settlement is likely to be complete to a great extent by the end of the construction activities. In strata of organic soils, settlement may continue almost indefinitely. For the safety of foundations, the engineer-in-charge should be well familiar with all causes of settlement. Foundations may settle due to some combination of the following reasons:

1. Elastic compression of the foundation material and the underlying soil
2. Consolidation including secondary compression
3. Ground water Lowering-specially repeated lowering and raising of water level in loose granular soils tend to compact the soil and cause settlement of the foundations. Prolonged lowering of the water table in fine grained soils may introduce settlements because of the extrusion of water from the voids. Pumping water or draining water by wells or pipes from granular soils without adequate filter material as protection may, in a period of time,

carry a sufficient amount of fine particles away from the soil and cause settlement;

4. Seasonal swelling and shrinkage of expansive clays
5. Ground movement on earth slopes, for example, surface erosion, slow creep or landslides
6. Other causes, such as adjacent excavation, mining subsidence and underground erosion by streams or floods and
7. The effects of vegetation leading to shrinking and swelling of clay soils.

Differential Settlements -- The foundations of different elements of a structure may have unequal settlements and the difference between such settlements will cause differential settlement. Some of the causes for differential settlements are as follows:

1. Geologic and physical non-uniformity or anomalies in type, structure, thickness, and density of the soil medium (pockets of sand in clay, clay lenses in sand, wedge like soil strata, that is, lenses in soil), an admixture of organic matter, peat, mud, etc.
2. Non-uniform pressure distribution from foundation to the soil due to non-uniform loading and incomplete loading of the foundations.
3. Varying water regime at the construction site
4. Over stressing of soil at adjacent site by heavy structures built next to light ones.
5. Overlap of stress distribution in soil from adjoining structures
6. Unequal expansion of the soil due to excavation for footings
7. Non-uniform development of extrusion settlements and
8. Non-uniform structural disruptions or disturbance of soil due to freezing and thawing, swelling, softening and drying of soils.

Types of Settlement in Different soils

Principal soil type	Types of Movement			
Rock	Yes	No	No	Some
Gravel	Yes	No	No	No
Sand	Yes	No	No	No
Silt	Yes	Minor	No	Yes
Clay	Yes	Yes	Yes	Yes
Organic	Yes	Minor	Yes	Yes

Allowable Settlement

Permissible uniform and differential settlement and tilt for shallow foundations

Type of Footing		Sand & Hard Clay			Plastic Clay		
		Max Settlement	Differential Settlem-ent	Angular Distortion	Max Settlement	Differe-ntial Settle-ment	Angular Distortion
Isolated Foundation	Steel	50 mm	0.0033 L	1/300	50 mm	0.0033L	1/300
	RCC	50 mm	0.0015 L	1/666	75 mm	0.0015L	1/666
Raft Foundation	Steel	75 mm	0.0033 L	1/300	100 mm	0.0033L	1/300
	RCC	75 mm	0.002L	1/500	100 mm	0.002L	1/500

FACTORS INFLUENCING FOUNDATION SETTLEMENT

- Type of soil
- Frost & Heave
- Change in water table
- Underground corrosion
- Mining Subsidence
- Vibration
- Landslide
- Creep
- Thermal changes

METHODSS TO IMPROVE BEARING CAPACITY OF SOILS AND TO REDUCE SETTLEMENTS

The centuries-old problem of land scarcity in the vicinity of existing urban areas often necessitates the use of sites with soils of marginal quality. In many cases these sites can be utilized for the proposed project by using some kind of soil improvement. This chapter will focus on several of the more widely used methods of improving soils for bearing capacity. An extremely large number of methods have been used and/or reported in the literature—many of which have been patented—and at an individual site one may use a mix of several methods to achieve the desired result. For a given site a first step is to make a literature review of at least some of the methods reported. This together with a reasonable knowledge of geotechnical fundamentals allows the engineer to use either an existing method, a mix of methods, or some method coupled with modest ingenuity (unless limited by a governmental agency) to produce an adequate solution for almost any site.

Mechanical stabilization- In this method the grain size gradation of the site soil is altered. Where the site soil is predominantly gravel (say, from 75 mm down to 1 mm) binder material is added. *Binder* is defined as material passing either the No. 40 (0.425 mm) or No. 100(0.150 mm) sieve. The binder is used to fill the voids and usually adds mass cohesion. Where the soil is predominantly cohesive (No. 40 and smaller sieve size) granular soil is imported and blended with the site soil.In either case the amount of improvement is usually determined by trial, and experience shows that the best improvement results when the binder (or filler) occupies between 75 and 90 percent of the voids of the coarse material. It usually requires much more granular materials to stabilize cohesive deposits than binder for cohesion less deposits and as a result other stabilizing methods are usually used for clayey soils.

Compaction- This method is usually the most economical means to achieve particle packing for both cohesion less and cohesive soils and usually uses some kind of rolling equipment. *Dynamic compaction* is a special type of compaction consisting of dropping heavy weights on the soil.

Preloading- This step is taken primarily to reduce future settlement but may also be used to increase shear strength. It is usually used in combination with drainage.

Drainage- This method is undertaken to remove soil water and to speed up settlements under preloading. It may also increase shear strength since *su,* in particular, depends on water content. For example, consolidation without drainage may take several years to occur whereas with drainage facilities installed the consolidation may occur in 6 to 12 months.

Densification using vibratory equipment- Densification is particularly useful in sand, silty sand, and gravelly sand deposits with *Dr* less than about 50 to 60 percent. This method uses some type of vibrating probe, which is inserted into the soil mass and withdrawn. Quality fill is added to the site to bring the soil surface to the required grade since the site soil usually settles around and in the vicinity of the vibrating probe.

Use of in situ reinforcement- This approach is used with stone, sand, cement, or lime columns. This treatment produces what is sometimes called *composite* ground. Sometimes small amounts of short lengths of plastic fibers or fiberglass can be mixed with the soil for strength improvement. The major precaution is to use a fiber material that has an adequate durability in the hostile soil environment.

Grouting- Initially this was the name for injection of a viscous fluid to reduce the void ratio (and *k)* or to cement rock cracks. Currently this term is loosely used to describe a number of processes to improve certain soil properties by injection of a viscous fluid, sometimes mixed with a volume of soil. Most commonly, the viscous fluid is a mix of water and cement or water and lime, and/or with additives such as fine sand, bentonite clay, or fly ash.1 Bitumen and certain chemicals are also sometimes used. Additives are used either to reduce costs or to enhance certain desired effects. Since the term *grout* is so loosely used in construction, the context of usage is important to define the process.

Use of geotextiles- These function primarily as reinforcement but sometimes in other beneficial modes.

Chemical stabilization- This means of stiffening soil is seldom employed because of cost. The use of chemical stabilizers is also termed *chemical grouting*. The more commonly used chemical agents are phosphoric acid, calcium chloride, and sodium silicate soil-cement and lime-soil treatment (often together with fly ash and/or sand) is a *chemical stabilization* treatment, but it is usually classified separately.

PROBLEMS

Example 1: A strip footing 2 m wide carries the load intensity of 400 KN/m^2 at the depth of 1.2 m sand. The saturated unit weight of sand is 19.5 KN/m^3 and the unit weight above the water table is 16.8 KN/m^3, angle of internal friction is 36°. Determine the factor of safety with respect to shear failure for the following provision of water table using terzaghi analysis.[N_c = 57.8, N_q = 41.4, N_γ = 42.4]

i. Water table is 4 m below the ground level
ii. Water table is 2.5 m below the ground level
iii. Water table is 1.2 m below the ground level
iv. Water table is 0.5 m below the ground level
v. Water table is at ground level

Solution

Given Data:

B = 2 m

D = 1.2 m

Φ = 36°

γ = 16.8 KN/m^3

$\gamma_{sat} = 19.5\ KN/m^3$

Formula for Strip footing resting on sand

$q_f = \gamma\ D\ N_q\ R_{w1} + 0.5\ \gamma\ N_\gamma R_{w2}\ [c = 0]$

i. Water table is 4 m below the ground level

$q_f = (16.8 \times 1.2 \times 41.4) + (0.5 \times 16.8 \times 2 \times 42.4) = 1546.94\ KN/m^2$

$FOS = q_f / q_a = 1546.94/ 400 = 3.87$

ii. Water table is 1.2 m below the ground level

$R_{w1} = 0.5[\ 1 + (z_{w1}/D)]$

$= 0.5\ [\ 1 + (1.2/1.2)]$

$= 1$

$R_{w2} = 0.5[\ 1 + (z_{w2}/B)]$

$= 0.5\ [\ 1 + 0]$

$= 0.5$

$q_f = (16.8 \times 1.2 \times 41.4 \times 1) + (0.5 \times 16.8 \times 2 \times 42.4 \times 0.5) = 1248.024\ KN/m^2$

$FOS = q_f / q_a = 1248.024/ 400 = 3.12$

iii. Water table is 2.5 m below the ground level

$q_f = \gamma\ D\ N_q\ R_{w1} + 0.5\ \gamma_{avg}\ N_\gamma R_{w2}$

$R_{w1} = 0.5[\ 1 + (z_{w1}/D)]$

$= 0.5\ [\ 1 + (0)]$

$= 1$

$R_{w2} = 0.5[\ 1 + (z_{w2}/B)]$

$= 0.5\ [\ 1 + (1.3/2)]$

$= 0.5$

$\gamma_{avg} = [(16.8 \times 1.3) + (\ 0.7 \times 19.5)]\ /\ (1.3+0.7) = 17.75$

$q_f = (16.8 \times 1.2 \times 41.4 \times 0.5) + (0.5 \times 17.75 \times 2 \times 42.4 \times 0.76) =$ 1038.032 KN/m^2

$FOS = q_f / q_a = 1038.032/\ 400 = 2.6$

iv. Water table is 0.5 m below the ground level

$q_f = \gamma\ D\ N_q\ R_{w1} + 0.5\ \gamma_{avg}\ N_\gamma\ R_{w2}$

$R_{w1} = 0.5[\ 1 + (z_{w1}/D)]$

$= 0.5\ [\ 1 + (0.5/1.2)]$

$=1$

$R_{w2} = 0.5[\ 1 + (z_{w2}/B)]$

$= 0.5\ [\ 1 + (0)]$

$= 0.5$

$\gamma_{avg} = [(0.5 \times 16.8) + (\ 0.7 \times 19.5)]\ /\ (0.5+0.7) = 17.745$

$q_f = (16.8 \times 1.2 \times 41.4 \times 0.5) + (0.5 \times 17.75 \times 2 \times 42.4 \times 0.76) = 1037.55$ KN/m^2

$FOS = q_f / q_a = 1037.55/\ 400 = 2.6$

v. Water table is 2.5 m below the ground level

$q_f = \gamma_{sub}\ D\ N_q\ R_{w1} + 0.5\ \gamma_{sub}\ N_\gamma\ R_{w2}$

$R_{w1} = 0.5[\ 1 + (z_{w1}/D)]$

$= 0.5\ [\ 1 + (0)]$

$=0.5$

$R_{w2} = 0.5[\ 1 + (z_{w2}/B)]$

$= 0.5 [1 + (0)]$

$= 0.5$

$q_f = (19.5 \times 1.2 \times 41.4 \times 0.5) + (0.5 \times 19.5 \times 2 \times 42.4 \times 0.5) = 783.55$ KN/m^2

$FOS = q_f / q_a = 783.55/ 400 = 1.95$

Example 2: A rectangle footing has a size of footing of 1.8 m x 3 m has to transmit a load of a column at a depth of 1.5 m. Calculate the safe load which the footing can carry wit FOS of 3 against the shear failure. Use BIS method. The soil has the following properties Φ=32.5°, n=40%, G=2.67, Moisture content=15%, c=8 KN/m^3.

Solution

$\gamma_d = (G\,\gamma_w)/ (1+e) = (2.67 \times 9.81)/ (1+0.67) = 15.68$

$e = n/ (1-n) = 0.4 / (1-0.4) = 0.67$

$\gamma = \gamma_d (1 + w) = 15.68 (1+0.15) = 18.03$ KN/m^3

For,

$\Phi = 30°$, $N_c = 30.14$

$\Phi = 35°$, $N_c = 46.12$

By interpolation $N_c = 38.13$, $N_q = 25.85$, $N_\gamma = 35.21$

$q_{nf} = c\,N_c\,S_c\,d_c\,i_c + \gamma\,D\,N_q\,S_q\,d_q\,i_q + 0.5\,\gamma\,N_\gamma\,S_\gamma\,d_\gamma\,i_\gamma\,w'$

$w'=1$, $i_c = i_q = i_\gamma = 1$

Shape factor for rectangular footing

$S_c = S_q = 1+ 0.2\,(B/L) = 1 + 0.2\,(1.8/3) = 1.12$

$S_\gamma = 1- 0.4\,(B/L) = 1- 0.4\,(1.8/3) = 0.76$

Depth factor

$d_c = 1 + 0.2\,(D_f/L)\tan(45 + \Phi/2) = 1.78$

$d_q = d_\gamma = 1 + 0.1\,(D_f/L)\tan(45 + \Phi/2) = 1.09$

$q_{nf} = 1699.78$ KN/m^2

safe load = 1699.78/3 = 3205.9 KN/m^2

Example 3: A RCC foundation of dimensions 18 m x 36 m exist a unit pressure of 180 KN/m^2 on a soil mass with a modulus of elasticity 45 MN/m^2. Determine the immediate settlement under the foundation assume μ=0.5.

Given Data

Type of Foundation= RCC rigid footing

Length of footing= L= 36 m

Breadth of footing= B = 18 m

q = 180 KN/m^2

E_s=45 MN/m^2

Solution

L/B = 36/18 = 2

I_f= 1 from the table

Therefore

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

$$S_i = 180 \times 18\left[\frac{1-0.5^2}{45 \times 10^3}\right] 1$$

= 0.054 m = 54 mm

Example 4: A layer of soft clay is 6 m thick clay layer underlying a newly constructed building. The unit weight of sand overlying the clay layer produces a pressure of 2.6 Kg/cm^2 and the new construction increases the pressure by 1 Kg/cm^2. If the C_c is 0.5. Compute the settlement for the soil having water content 40 % and specific gravity 2.65.

Given Data

$C_c = 0.5$

$\sigma_0 = 2.6$ Kg/cm^2

$\Delta\sigma = 1$ Kg/cm^2

$w = 40\ \% = 0.4$

$G = 2.65$

$$e = \frac{w\,G}{S} = 0.4 \times 2.65\ (s = 1\ for\ saturated\ clay) = 1.06$$

$$S_c = \frac{C_c H_0}{1+e}\, log_{10}\left[\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right]$$

$$S_c = \frac{0.5 \times 600}{1+1.06}\, log_{10}\left[\frac{2.6+1}{2.6}\right] = 20.58 \text{ cm}$$

UNIT – 3

FOOTINGS AND RAFTS

Types of isolated footing, combined footing, mat foundation – contact pressure and settlement distribution – proportioning of foundations for conventional rigid behavior – minimum thickness for rigid behavior – applications – compensated foundation – Codal provision

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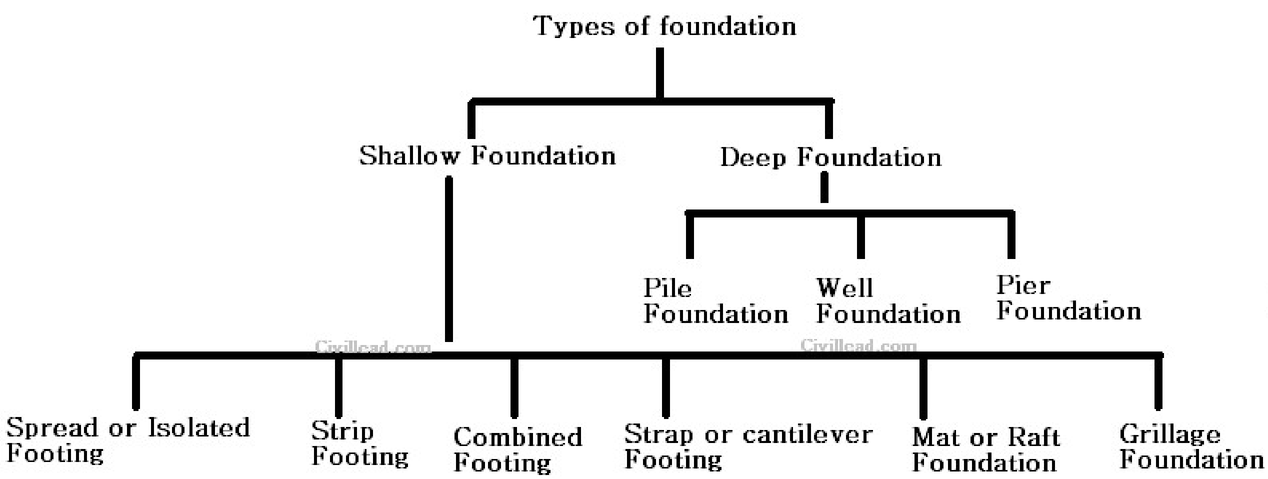
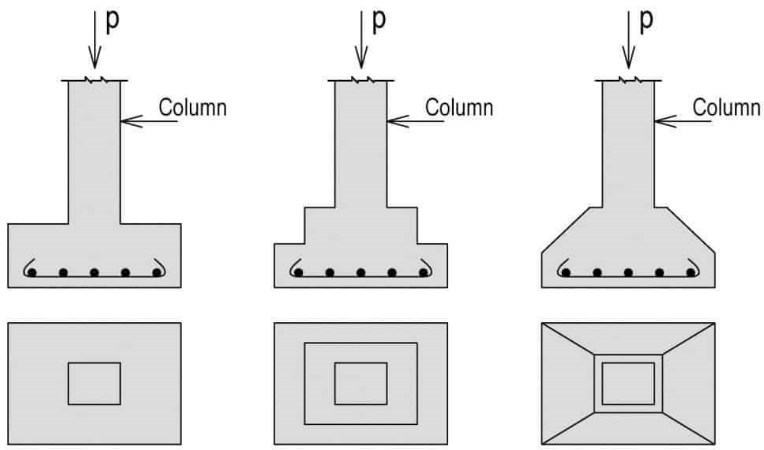


SPREAD FOOTING

- It is basically a pad used to spread out loads from walls or columns over a sufficiently large area of foundation soil.
- These are constructed as close to the ground surface as possible.
- This type of foundation is suitable for walls and masonry columns.
- It is economical for a maximum depth of 3 m.
- Then a plain concrete mix 1:4:8 is placed. Its thickness differs from 150 to 200 mm.
- The stone-masonry footing is constructed over this bed.
- It is constructed in courses and each course is projected 50 to 75 mm from the top course and the height of each course is 150 to 200 mm.
- The projection of bed concrete from the lowest course of foundation masonry is usually 150 mm.



(a) Pad Footing (b) Stepped Footing (c) Sloped Footing

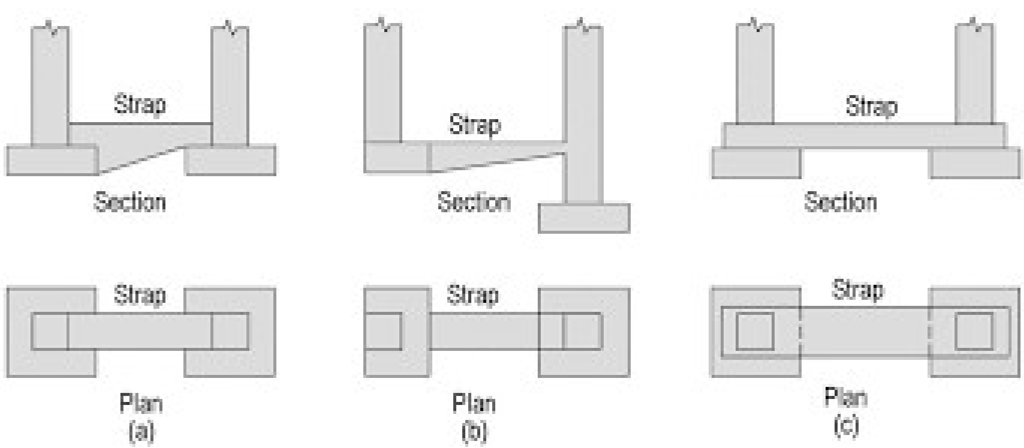
Advantages

- It decreases cracking bring out by settlement.
- It balances soil around the base of the structure.
- Easy construction of basements.
- It has continuous contact with the whole foundation which decreases the risk of foundation collapse.
- Cost savings in construction, design, and quality control.

Disadvantages

- It is restricted to certain soil structures only and cannot be utilized for all forms of soil.
- This type of foundation is regularly put through torsion, moment, and pullout.
- Settlement is a huge problem in this type of foundation.

STRAP FOOTING



- A strap footing comprises two or more footings connected by a beam called strap.
- This is also called as cantilever footing or pump-handle foundation.
- This may be required when the footing of an exterior column cannot extend into an adjoining private property.
- The base regions of the footings are proportioned so the bearing pressing factors are uniform and equivalent under the two bases.

Advantages

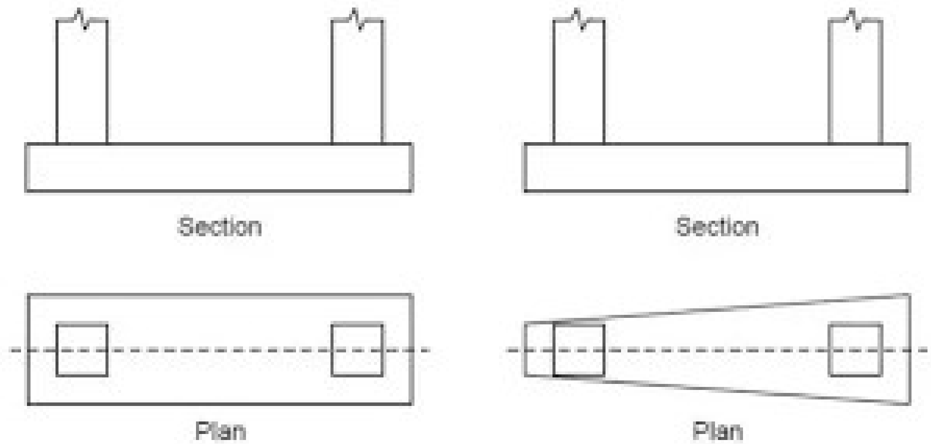
- It helps distribute load uniformly and transfer moment to the adjacent footing.
- It prevents the column from tilting.
- The employment of a strap footing may be excusable where the gap between columns is long and a regular combined footing is impractical due to the required massive excavation.

Disadvantages

- Strap footing requires more concentration while it has been constructed because it has a difficult design.
- It can become more time-consuming comparatively as the calculation of pressure bearing, pressure distribution, moment, forces, and shears can become complex.
- It will demand expert workers for its construction.

COMBINED FOOTING

- If two or more columns are supported in a row and a common foundation is provided them then the footing is called combined footing.
- If the individual footing has to accommodate two or in exceptional cases more than two columns are known as combined footing.
- Usually, the combined footing is made also of reinforcement concrete. In the combined footing the footing assumes to rigid and resting on homogeneous soil.
- When the load-bearing capacity of the soil is low and when under the individual footing required more area then combined footing is construed.
- The combined footing should be rectangular combined footing, trapezoidal combined footing, strap combined footing, and raft combined footing.
- In this combined footing the maximum bending moment takes as the design value for the reinforced concrete footing.
- The development length check for the longitudinal steel. The longitudinal bars curtail the economy. To get uniform pressure distribution under the footing the combined footing is widely used.



(a) Rectangular combined footing (b) Trapezoidal combined footing

Advantages

- If there is requirement of two or more columns to be construed in a small area then we can use combined footing.
- Where the load-bearing capacity of the soil is low, we construed combined footing.
- Where the soil capacity of the Construction area is unequal to distribute the load evenly to the sub-soil combined footing is constructed.
- The combined footing can be used in the difficult construction sides.
- when a column closes the property line and footing is not spread the front of the property line then, combined footing help to construct the column.
- To get uniform pressure distribution under the footing the combined footing is widely used.
- This type of combined footing is more profitable than another footing.
- The combined footing is provided to equally distribute the load to the super-structure to the sub-soil.

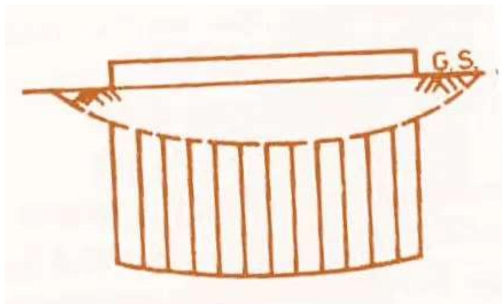
CONTACT PRESSURE

- 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.
- This upward pressure is assumed to be uniform in deriving different relationship for soil-structure interaction problem.
- But actually, a footing is not flexible as well as contact pressure is not uniform, necessitating more investigation for actual pressure distribution.

Contact pressure distribution under flexible footing

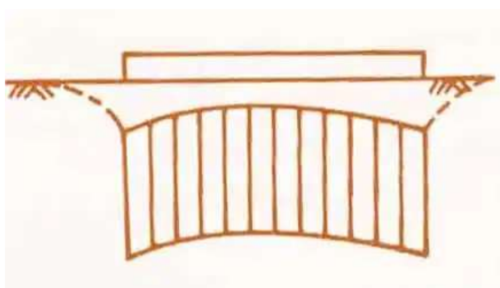
Cohesive soil

- The settlement is maximum at center of footing and minimum at the edges which forms bowl like shape.
- But the contact pressure is distributed uniformly along the settlement line or deflected line.



Cohesionless soil

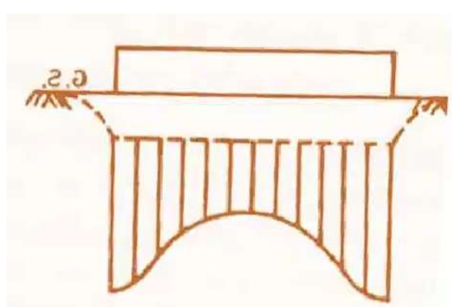
- The settlement at center becomes minimum while at edges it is maximum which exact opposite case of the settlement of flexible footing over cohesive soil.
- But in this case also contact pressure is uniform along the settlement line.



Contact pressure distribution under rigid footing

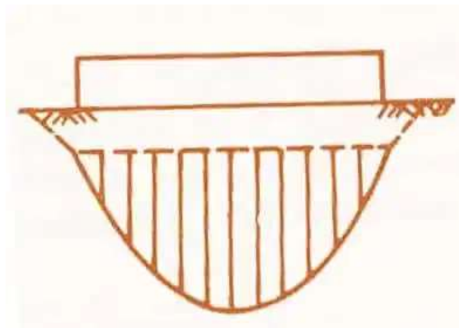
Cohesive soil

- The settlement is uniform but contact pressure varies.
- At edges, contact pressure is maximum and at center it is minimum which forms inverted bowl.
- The values of stresses at edges becomes finite when plastic flow occurs in real soils.



Cohesionless soil

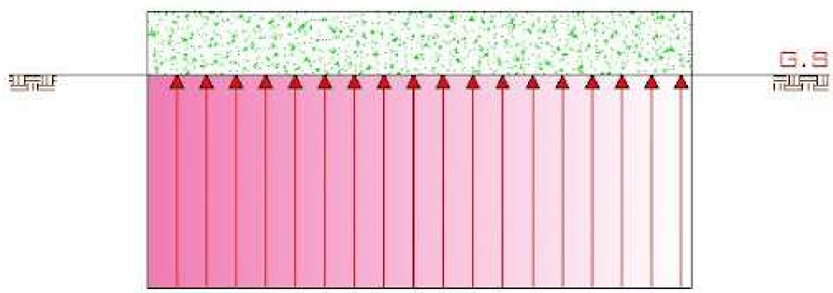
- The contact pressure is maximum at center and gradually reduces to zero towards edges.
- Settlement is uniform.
- If the footing is embedded, then there may be some amount of contact pressure at the edges of rigid footing.



Factors influencing contact pressure

1. Elastic properties of footing.
2. Elastic properties of soil.
3. Thickness of footing.

Consequences of assuming uniformity in pressure



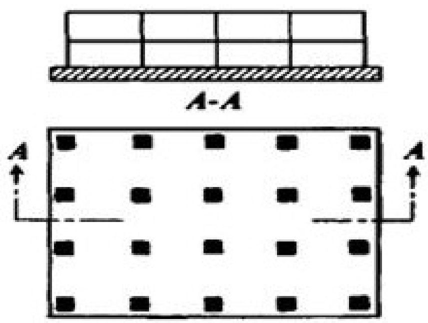
- For convenience, the contact pressure is assumed to be uniform for all types of footings and all types of soils if load is symmetric.
- The above assumption of uniform pressure distribution will result in a slightly unsafe design for rigid footing on clays, as the maximum bending moment at center is underestimated.
- It will give a conservative design for rigid footings on sandy (cohesionless) soils, as the maximum bending moment is overestimated.
- However, at the ultimate stage just before failure, the soil behaves as an elasto-plastic material (and not an elastic material) and the contact pressure is uniform and the assumption is justified at the ultimate stage.

RAFT FOUNDATION

- A raft or mat foundation is a combined footing which covers the entire area beneath of a structure and supports all the walls and columns.
- This type of foundation is most appropriate and suitable when the allowable soil pressure is low, or the loading heavy and spread footings would cover more than half the plan areas.
- Also, when the soil contains lenses of compressible strata which are likely to cause considerable differential settlement, a raft foundation is well-suited, since it would tend to bridge over the erratic spots, by virtue of its rigidity.

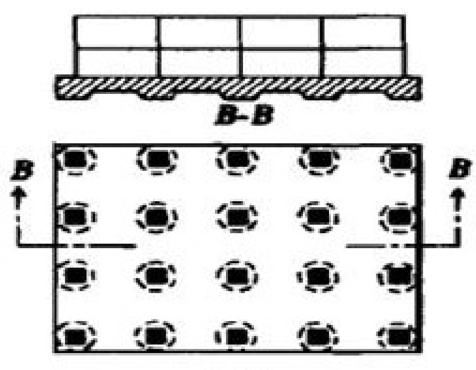
Types of mat foundation

1. **Flat slab type**



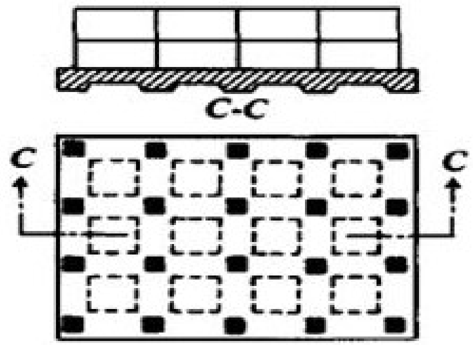
- It represents a true raft which is flat concrete slab of uniform thickness throughout the entire area.
- It is suitable for closely spaced columns carrying small loads.

2. **Flat slab thickened under columns**



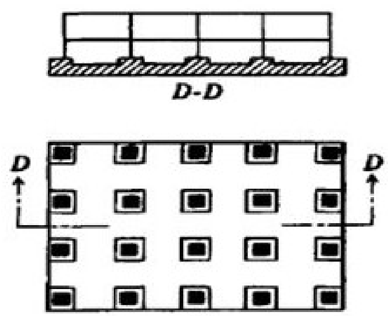
- It represents a raft with a portion of the slab under the thickened column.
- It provides sufficient strength for relatively large column loads.

3. **Two-way beam and slab type**



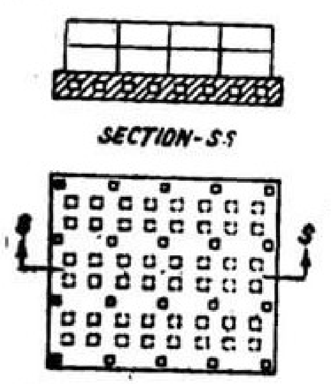
- It represents raft with thickened bands provided along column lines in both directions.
- This provides sufficient strength when column spacing is large and column load is unequal.

4. **Flat slab with pedestals**



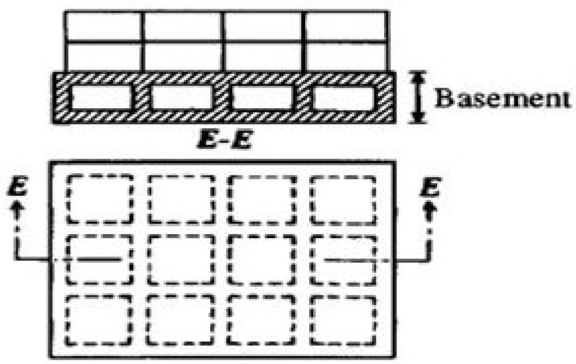
- It represents a raft in which pedestals are provided under each column.
- It provides sufficient strength for relatively large column loads.

5. **Cellular type**



- It represents a two-way grid structure made of cellular construction and of intersecting structural steel construction.

6. **Basement walls as rigid frame**



- It represents wherein basement walls have been used as ribs or deep beams.

Advantages of Raft Foundation

- Raft foundations are constructed for shallow depths hence, it requires less excavation
- Well suitable in soils of low bearing capacity.
- Loads coming from superstructure are distributed over a larger area.
- Differential settlement of soil can be reduced.

Disadvantages of Raft Foundation

- In some cases, large amount of reinforcement is required for raft foundation which increases the cost of project.
- Special attention on raft foundations is required in case of concentrated loads.
- If they are not treated properly, there is a chance of edge erosion.
- Skilled workers are required to construct the raft foundations.

SEISMIC CONSIDERATIONS FOR FOOTING DESIGN

Foundations in seismic zones are subjected to additional forces due to seismic waves. If the structure is not able to withstand these forces, it may collapse leading to serious catastrophes. A foundation is the lowest part of a structure, which is normally below the ground level. Foundations are provided to transmit the load of the superstructure to the underlying soil. It is necessary that the subsoil can withstand these loads. For this, the superimposed load should be lower than the safe bearing capacity of the soil. In case of foundations in seismic zones, additional loads are created due to the seismic vibrations. The design should consider the additional forces.

General requirements in the seismic design of foundations

- Site investigations and determination of soil properties.
- Details of geological and geotechnical environment.
- Identification of loads - static and dynamic.
- Type of foundation.
- Safety verification as per building codes.

Soil Investigations for Seismic Designs

The important soil parameters to be analyzed are:

- Particle size distribution
- Relative density
- Shear modulus
- Damping factors

The important field tests to be conducted are:

- SPT Hammer Energy
- Pressure Meter Testing
- Shear wave velocity Measurement
- Cone Penetrometer Test
- Seismic Piezo cone Penetrometer

The purpose of these laboratory and field tests are to define soil deposit details, hydraulic conditions, soil index properties, static and dynamic stress-strain soil behavior.

Main Factors That Influence Site Effects

Seismological:

- Intensity and frequency characteristics of bed rocks in seismological environment.
- Duration of bed rock motions.

Geotechnical:

- Nonlinear behavior of soils.
- Elastic vibration characteristics of soils.

Geometrical:

- Topography of underlying bedrock.
- Non-horizontal soil deposit layering.

Geological:

- Soil deposit thickness.
- Type of under lying rock.

Earthquake Characteristics

The main effect of an earthquake is the horizontal forces that are generated in the structure. The horizontal ground accelerations give a measure of this force. The granular soils get compacted due to the vibrations. This in turn causes the settlement of the ground surface. Another effect of the vibrations is liquefaction. The degree of liquefaction depends on the relative density of the soil, percentage of fines, depth of water table and the ground acceleration.

Effects of Earthquake

- The dynamic stress and induced pore water pressure may reduce the bearing capacity of the soil
- Loose granular soils are compacted by ground motion. This causes large subsidence of the ground surface.
- Compaction of loose granular soil may induce excess pore water pressure, which causes liquefaction of soil.
- The vibration due to earthquake may cause structural damage.

Measures to prevent Liquefaction

1. Compaction of loose soil
   - with vibratory rollers
   - compaction piles
   - vibro floatation
   - blasting
2. Grouting and chemical stabilization.
3. Application of surcharge.

CODAL PROVISIONS

For the design of foundation, the provisions of IS:1904-1986 in conjunctions with IS:1893-1984 shall generally be followed.

1. The subgrade below the entire area of the building shall preferably be of the same type of the soil. Whenever this isn't possible, a suitably located separation or crumple section shall be provided.
2. Loose fine sand, soft silt and expansive soils should be avoided. If unavoidable, the building shall rest either on a rigid raft foundation or piles taken to a firm stratum. However, for light constructions the following measures may be taken to improve the soil on which the foundation of the building may rest:
   (a) Sand Piling
   (b) Soil Stabilization
3. Isolated footings for columns
   All the individual footings or pile caps used in type III soft soils (Table 3 of IS:1893-1984), shall be connected by reinforced concrete ties at least in two directions approximately at right angles to each other. For buildings with no basement the ties may be placed at or below the plinth level and

for buildings with basement they may be placed at the level of basement floor. They may be designed to carry the load of the panel walls also.

4. Where ties are used, their sections shall be designed to carry in tension as well as in compression, an axial load not less than the earthquake force acting on the heavier of the columns connected, but the sections shall not be less than 200mm × 200mm with M15 concrete reinforced with 4 bars of 12mm dia plain mild steel bars or 10mm dia high strength deformed bars, one at each corner, bound by 6mm dia mild steel stirrups not more than 150mm apart.
5. In the case of reinforced concrete slab, the thickness shall not be less than $1/50^{th}$ of the clear distance between the footings, but not less than 100mm in any case. It shall be reinforced with not less than 0.15% mild steel bars or 0.12% high strength deformed bars in each direction placed symmetrically at top and bottom.

UNIT IV
PILE FOUNDATION

INTRODUCTION

The shallow foundations are used in case of small buildings or structures, which carry lesser loads, and hence the loads are dissipated into the soil mass at much lower depth. However when we are considering large structures, which carry heavy loads, the loads are dissipated at greater depths where usually the soil bearing capacity is quite high. One guideline of differentiating between the shallow and deep foundations is that in case of the deep foundations the depth of foundations is more than the dimension of the structure (usually the width is considered as the dimension).

Deep foundations are of the following types:

- Deep footings.
- Piles.
- Piers.
- Caissons /Well foundations.

Requirements for Deep Foundations

Generally for structures with load > 10 $t/m^2$10, we go for deep foundations. Deep foundations are used in the following cases:

- Huge vertical load with respect to soil capacity.
- Very weak soil or problematic soil.

- Huge lateral loads eg. Tower, chimneys.
- Scour depth criteria.
- For fills having very large depth.
- Uplift situations (expansive zones)
- Urban areas for future large and huge construction near the existing building.

TYPES OF PILES AND THEIR FUNCTION

Piles may be classified in a number of ways based on different criteria: (i) function or action, (ii) composition and material and (iii) installation.

(I) FUNCTION OR ACTION

End-bearing piles: The load is transferred through the pile tip to a suitable bearing stratum.

Friction piles: The load is transferred through a depth by skin friction along the surface area of the pile.

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

Compaction piles: Used to compact loose granular soils in order to increase the bearing capacity; a sand pile is used to form this type as it need not carry any load.

Anchor piles: Used to provide anchorage against horizontal pull.

Fender piles: Used to protect waterfront structures against impact from ships or other floating objects.

Sheet piles: Commonly used as bulkheads or cut-off to reduce seepage and uplift in hydraulic structures.

Batter piles: Used to resist horizontal and inclined forces, especially in waterfront structures.

Laterally loaded piles: Used to support retaining walls, bridges and dams, and as fenders in docks and harbors.

(II) COMPOSITION AND MATERIAL

Timber piles: Timber of sound quality is used. Timber piles perform well in either fully dry condition or submerged condition.

Steel piles: These are usually H-piles, pipe piles or sheet piles (rolled sections of regular shapes).

Concrete piles: These may be precast or cast-in-situ. Precast piles are invariably reinforced. Cast-in-situ piles are installed by pre-excavation; the common types are Raymond pile, MacArthur pile and Franki pile.

Composite piles: These are made of either concrete and timber or concrete and steel. They are used when part of pile is submerged under water.

(III) INSTALLATION

Driven piles: Timber, steel or precast concrete piles are driven into position by using pile-driving equipment.

Cast-in-situ piles: Only concrete piles can be cast-in-situ. Piles are drilled and filled with concrete. Reinforcements can be added according to requirements.

Driven and cast-in-situ piles: These are combination of both driven and cast-in-situ piles. Casing or shell mat be used. The Franki pile falls in this category.

FUNCTIONS OF PILE:

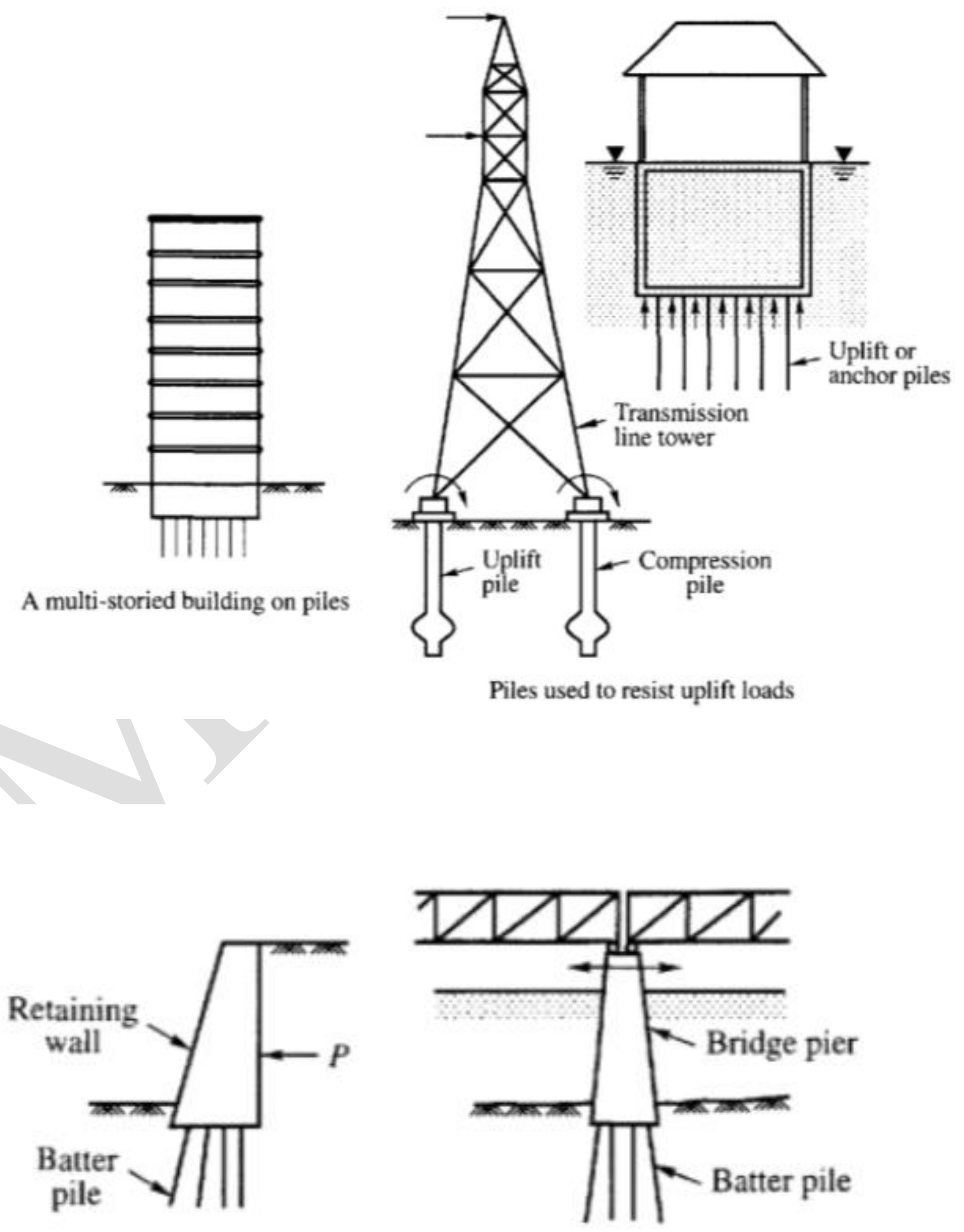
The major uses of piles are:

(i) to carry vertical compressive loads,

(ii) to resist uplift loads

(iii) toresist horizontal or inclined loads.

Normally vertical piles are used to carry vertical compression loads coming from superstructures such as buildings, bridges etc. The piles are used in groups joined together by pile caps. The loads carried by the piles are transferred to the adjacent soil. If all the loads coming on the tops of piles are transferred to the tips, such piles are called end-bearing or point-bearing piles. However, if all the load is transferred to the soil along the length of the pile such piles are called friction piles. If, in the course of driving a pile into granular soils, the soil around the pile gets compacted, such piles are called compaction piles. Figure below shows piles used for the foundation of a multistoried building to carry loads from the superstructure. Piles are also used to resist uplift loads. Piles used for this purpose are called tension piles or uplift piles or anchor piles. Uplift loads are developed due to hydrostatic pressure or overturning movement as shown in figure. Piles are also used to resist horizontal or inclined forces. Batter piles are normally used to resist large horizontal loads.



Uses of Piles

FACTORS INFLUENCING THE SELECTION OF PILE

The selection of the type, length and capacity is usually made from estimation based on the soil conditions and the magnitude of the load. In large cities, where the soil conditions are well known and where a large number of pile foundations have been constructed, the experience gained in the past is extremely useful. Generally the foundation design is made on the preliminary estimated values. Before the actual construction begins, pile load tests must be conducted to verify the design values. The foundation design must be revised according to the test results. The factors that govern the selection of piles are:

1. Length of pile in relation to the load and type of soil
2. Character of structure
3. Availability of materials
4. Type of loading
5. Factors causing deterioration
6. Ease of maintenance
7. Estimated costs of types of piles, taking into account the initial cost, life expectancy and cost of maintenance
8. Availability of funds

All the above factors have to be largely analyzed before deciding up on a particular type.

CARRYING CAPACITY OF SINGLE PILE:

The ultimate bearing capacity, Q_u, of a single vertical pile may be determined by any of the following methods.

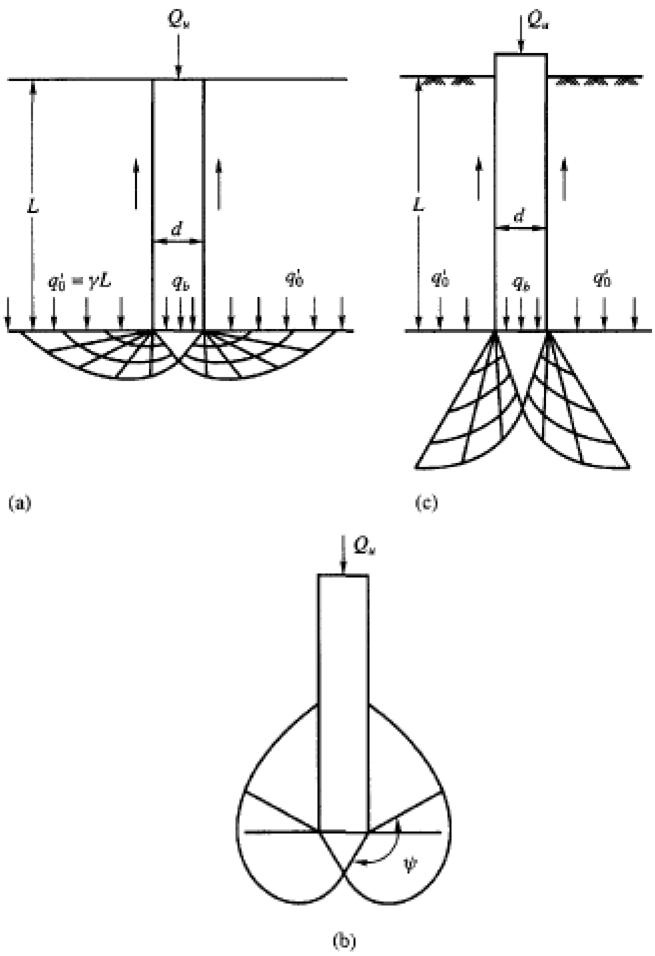
1. By the use of static bearing capacity equations

2. By the use of SPT and CPT values
3. By field load tests
4. By dynamic method

The determination of the ultimate point bearing capacity, q_b, of a deep foundation on the basis of theory is a very complex one since there are many factors which cannot be accounted for in the theory. The theory assumes that the soil is homogeneous and isotropic which is normally not the case. All the theoretical equations are obtained based on plane strain conditions. Only shape factors are applied to take care of the three-dimensional nature of the problem. Compressibility characteristics of the soil complicated the problem further. Experience and judgement are therefore very essential in applying any theory to a specific problem. The skin load Q, depends on the nature of the surface of the pile, the method of installation of the pile and the type of soil. An exact evaluation of QAs a difficult job even if the soil is homogeneous over the whole length of the pile. The problem becomes all the more complicated if the pile passes through soils of variable characteristics.

GENERAL THEORY FOR ULTIMATE BEARING CAPACITY

According to Vesic (1967), only punching shear failure occurs in deep foundations irrespective of the density of the soil so long as the depth-width ratio *Lid* is greater than 4 where *L* = length of pile and *d* = diameter (or width of pile). The types of failure surfaces assumed by different investigators are shown in the figure for the general shear failure condition. The detailed experimental study of Vesic indicates that the failure surfaces do not revert back to the shaft.



The shapes of failure surfaces at the tips of piles as assumed by (a)Terzaghi, (b) Meyerhof, and (c) Vesic

The total failure load $\overline{Q}_u$ may be written as follows

$$\overline{Q}_u = Q_u + W_p = Q_b + Q_f + W_p \qquad - (1)$$

where, Q_u = load at failure applied to the pile

Q_b = base resistance

Q_f = shaft resistance

W_p = weight of the pile.

The general equation for the base resistance may be written as

$$Q_b = cN_c + q'_o N_q + \frac{1}{2}\gamma d N_\gamma A_b \qquad - (2)$$

where, *d* = width or diameter of the shaft at base level

q'_o = effective overburden pressure at the base level of the pile

A_b = base area of pile

c = cohesion of soil

γ = effective unit weight of soil

Nc, N ,N = bearing capacity factors which take into account the shape factor.

LOAD CARRYING CAPACITY OF PILES

The load carrying capacity of a single pile can be estimated using,

1. Static formulae
2. Dynamic formulae
3. From insitu tests - correlations with penetration test data
4. Load tests

1. STATIC FORMULAE

The static formulae for ultimate load carrying capacity of pile based on soil properties and pile geometry are as given in Eq(3) and Eq(4) for piles in granular soils and cohesive soils respectively.

Piles in granular soils:

The ultimate load Qu is given by

Qu = End bearing resistance Qp+ Skin resistance Qs

$$Q\text{u} = A\text{p}(0.5\gamma D N\gamma) + A\text{p}(\sigma' N\text{q}) + \Sigma_{i=1}^{n} kA\text{si}(\sigma_{\text{i}}' tan\delta) \qquad - (3)$$

where, Ap = Cross section area of pile.

D = Stem diameter of pile.

Nγ = Bearing capacity factor taken for general shear.

Nq = Bearing capacity factor.

σ'= Effective overburden pressure (Critical depth taken as 15D for ø ≤30◦ and 20D for ϕ ≥40◦)

k= Co-efficient of earth pressure.

σ_{i}' = Effective over burden pressure at middle of corresponding layer.

δ = Angle of wall friction usually taken as ¾ ø of soil.

Asi = Surface area of pile.

Piles in cohesive soils

The ultimate bearing Q_u of piles in cohesive soils is given by the following formula

Q_u = End bearing resistance,Q_p + Skin resistance, Q_s

$$Qu = Ap\ NcCp + \Sigma_{i=1}^{n}\ \alpha\ A_{si}\ C_{si} \qquad - (4)$$

where, N_C = Bearing capacity factor in clays which is taken as 9 (See Skempton's curve)

c_p = Average cohesion at pile toe.

$\acute{\alpha}_i$ = Adhesion factor.

c_i = Average cohesion of the ith layer on the side of the pile.

A_{si} = Surface area of pile stem in the ith layer.

$\acute{\alpha}_i\ c_i$ = Adhesion between shaft of pile and clay.

Piles in C-Ø soils

Where the soil has large values of both c and ø (as for a true c- ø), we should use the conservative Terzaghi's bearing capacity factors to determine the load carrying capacity.

$$Q_u = A_p\ CN_c + \sigma_{vb}N_q + 0.5\gamma DN_\gamma + \Sigma_{i=1}^{n} A_s\alpha_c + k\ (\sigma_v \tan\phi) \qquad - (5)$$

where, Nc, Nq, Nγ = Terzaghi's bearing capacity factors

σvb, σv = Effective overburden pressure at base and pile shaft, irrespective of the critical depth.

2. DYNAMIC FORMULAE

Engineering News Formula:

For Piles driven in soils there are a set of formulae based on the so-called Engineering News (1888) formula.

$$Q_u = \frac{WHn}{s+c} \qquad - (6)$$

where, Qu= Ultimate load capacity of the driven pile.

W= Hammer weight (tons)

H= Height of fall of hammer (cm)

S= Final set (cm/blow)

C= a constant depending on type of hammer (2.54 for drop hammer, 0.254 for steam hammer)

η_h= efficiency of hammer(0.65 for steam hammer, 1.0 for drop hammer)

For double-acting steam hammer

The hammer weight W is replaced by W+ap,
where, 'a' is the area of the piston (cm2) and p is the steam pressure (kg/cm2).

$$Q_a = \frac{Qu}{F}$$

where, Q_a= allowable load

F is taken as 6.

Hiley's Modification of Wellington's formula

$$Q_u = \frac{WH\eta_h n_b}{s+\frac{C}{2}} \qquad - (7)$$

where, the terms W, H, η_h and S are the same as before.

η_b is the efficiency of the hammer blow.

$$\eta_h = \frac{W+P_e^2}{W+P_e} \quad \text{if } W > ep$$

$$\eta_b = \frac{W+P_e^2}{W+P_e} \quad \text{if } W < ep$$

Here P is the pile weight and e is the co-efficient of restitution, whose value is 0.4 for concrete and 0.5 for steel.

The quantity C in Hiley's formula is total elastic compression given by

$$C = C_1 + C_2 + C_3$$

where, C1, C2, C3 are the compression of pile cap, pile shaft and soil respectively.

Dynamic formulae are generally found to be less reliable than static formulae.

CAPACITY FROM INSITU TESTS

BEARING CAPACITY OF PILES IN GRANULAR SOILS BASED ON SPT VALUE

Meyerhof (1956) suggests the following equations for single piles in granular soils based on SPT values.

For displacement piles:

$$Q_u = Q_b + Q_f = 40N_{cor}(L/d)A_b + 2\overline{N}_{cor}A_s \qquad - (8)$$

For H-piles:

$$Qu = 40N_{cor}(L/d)A_b + \overline{N}_{cor}A_s \qquad - (9)$$

where, $q_b = 40\ N_{cor}(L/d) < 400\ N_{cor}$ - (10)

For bored piles:

$$Q_u = 133N_{cor}A_b + 0.67\overline{N}_{cor}A_s \qquad - (11)$$

where, Q_u = ultimate total load in kN

N_{cor} = average corrected SPT value below pile tip

$\overline{N}_{cor}$ = corrected average SPT value along the pile shaft

A_b = base area of pile in m2 (for H-piles including the soil between the flanges)

A_s = shaft surface area in m2

In English units Qu for a displacement pile is

$$Q_u(\text{kip}) = Q_b + Q_f = 0.8N_{cor}(L/d)A_b + 0.4\overline{N}_{cor}A_s \qquad - (12)$$

where, A_b = base area in ft^2 and As = surface area in ft^2

and 0.80Ncor $(\frac{L}{d})$Ab ≤ $8N_{cor}A_b$(kip)

A minimum factor of safety of 4 is recommended. The allowable load Q_a is

$$Q_a = \frac{Q_u}{4} \qquad - (13)$$

BEARING CAPACITY OF PILES BASED ON STATIC CONE PENETRATION TESTS (CPT)

The cone penetration test may be considered as a small scale pile load test. As such the results of
this test yield the necessary parameters for the design of piles subjected to vertical load. Various methods for using CPT results to predict vertical pile capacity have been proposed. The following methods will be discussed:

1. Vander Veen's method.
2. Schmertmann's method.

In the Vander Veen et al., (1957) method, the ultimate end-bearing resistance of a pile is taken, equal to the point resistance of the cone. To allow for the variation of cone resistance which normally occurs, the method considers average cone resistance over a depth equal to three times the diameter of the pile above the pile point level and one pile diameter below point level. Experience has shown that if a safety factor of 2.5 is applied to the ultimate end resistance as determined from cone resistance, the pile is unlikely to settle more than 15 mm under the working load (Tomlinson, 1986). The equations for ultimate bearing capacity and allowable load may be written as,

pile base resistance, $q_b = q$ (cone)

ultimate base capacity, $Q_b = A_b q_p$

allowable base load, $Q_a = \frac{A_b q_p}{F_s}$

where, q_p = average cone resistance over a depth 4d and Fs = factor of safety.

The skin friction on the pile shaft in cohesionless soils is obtained from the relationships established by Meyerhof (1956) as follows.

For displacement piles, the ultimate skin friction, f_s, is given by

$$f_s = \frac{\bar{q}_c}{2} \text{ (kPa)}$$

and for H-section piles, the ultimate limiting skin friction is given by

$$f_s = \frac{\bar{q}_c}{4} \text{ (kPa)}$$

where, $\bar{q}_c$ = average cone resistance in kg/cm2 over the length of the pile shaft under consideration.

Meyerhof states that for straight sided displacement piles, the ultimate unit skin friction, f_s, has a maximum value of 107 kPa and for H-sections, a maximum of 54 kPa (calculated on all faces of flanges and web).

The ultimate skin load is

$$Q_f = A_s f_s \qquad - (14)$$

The ultimate load capacity of a pile is

$$Q_u = Q_b + Q_f \qquad - (15)$$

The allowable load is,

$$Q_a = \frac{Q_b + Q_f}{2.5} \qquad - (16)$$

If the working load, Qa, obtained for a particular position of pile, is less than that required for the structural designer's loading conditions, then the pile must be taken to a greater depth to increase the skin friction f_s or the base resistance q_b.

NEGATIVE SKIN FRICTION

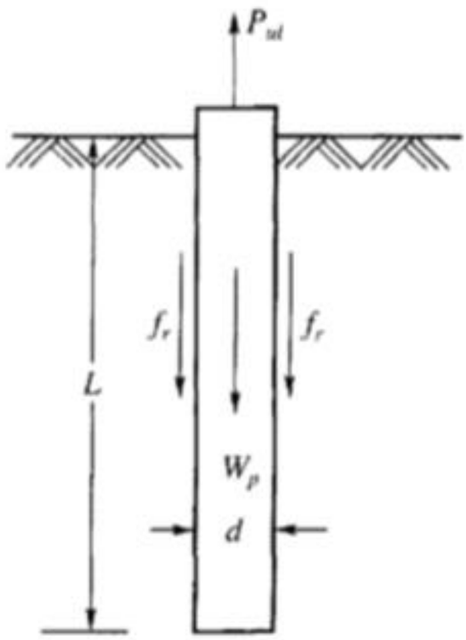
When a weak, compressible soil layer is sandwiched between hard layers, a pile passing through such a stratum may be subjected to an additional load due to compression of the weak layer. This compression may be caused by consolidation, fill placing, remolding during driving, or lowering of the water table. The portion of the pile within this layer is subjected to draw down force in addition to the structural loads. This force should be taken into account when designing the pile foundation.

An approximate estimate of the force can be made by empirical formulae such as following

F_d= force due to negative skin friction
F_d= (perimeter*soil depth)*C_u [for clays]
F_d= 0.5(perimeter*(soil depth)^2*$\gamma K\tan\delta$) [for sands]
C_u= undrained shear strength
γ= unit weight of soil
K= coefficient of earth pressure
δ = angle of internal friction.

UPLIFT CAPACITY

Piles are also used to resist uplift loads. Piles used for this purpose are called tension piles, uplift piles or anchor piles. Uplift forces are developed due to hydrostatic pressure or overturning moments as shown in figure.



Single pile subjected to uplift

The figure shows a straight edged pile subjected to uplift force. The equation for the uplift force P_{ul} may be written as

$$P_{ul} = W_p + A_s f_s \qquad - (17)$$

where, P_{ul} = uplift capacity of pile,

W = weight of pile,

f_s = unit resisting force

A_s = effective area of the embedded length of pile.

Uplift Resistance of Pile in Clay

For piles embedded in clay, Eq. (17) may written as

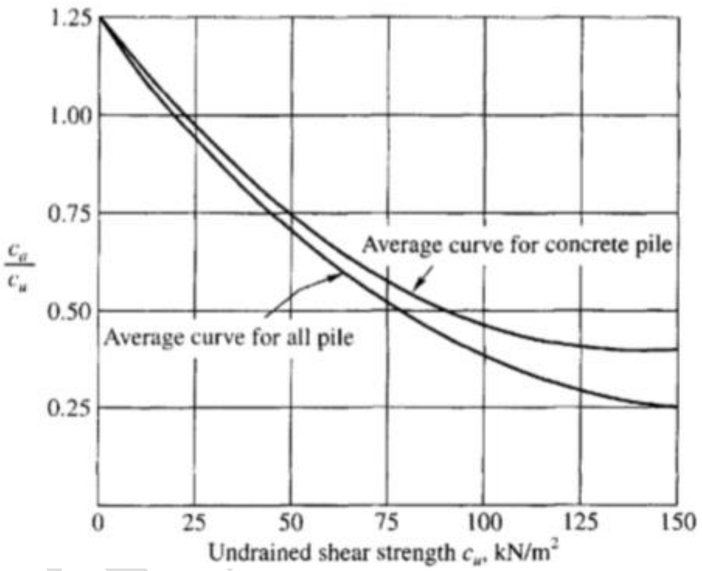
$$P_{ul} = W_p + A_s \alpha \bar{c}_u \qquad - (18)$$

where, $\bar{c}_u$= average undrained shear strength of clay along the pile shaft,

α = adhesion factor (= cfl/cM),

c_a = average adhesion.

Figure below gives the relationship between α and c_u based on pull out test results as collected by Sowa (1970). As per Sowa, the values of c_a agree reasonably well with the values for piles subjected to compression loadings.



Relationship between adhesion factor α and undrained shear strength c_u (Source: Poulos and Davis, 1980)

Uplift Resistance of Pile in Sand

Adequate confirmatory data are not available for evaluating the uplift resistance of piles embedded in cohesionless soils. Ireland (1957) reports that the average skin friction for piles under compression loading and uplift loading are equal, but data collected by Sowa (1970) and

Downs and Chieurzzi (1966) indicate lower values for upward loading as compared to downward loading especially for *cast-in-situ* piles. Poulos and Davis (1980) suggest that the skin friction of upward loading may be taken as two-thirds of the calculated shaft resistance for downward loading.

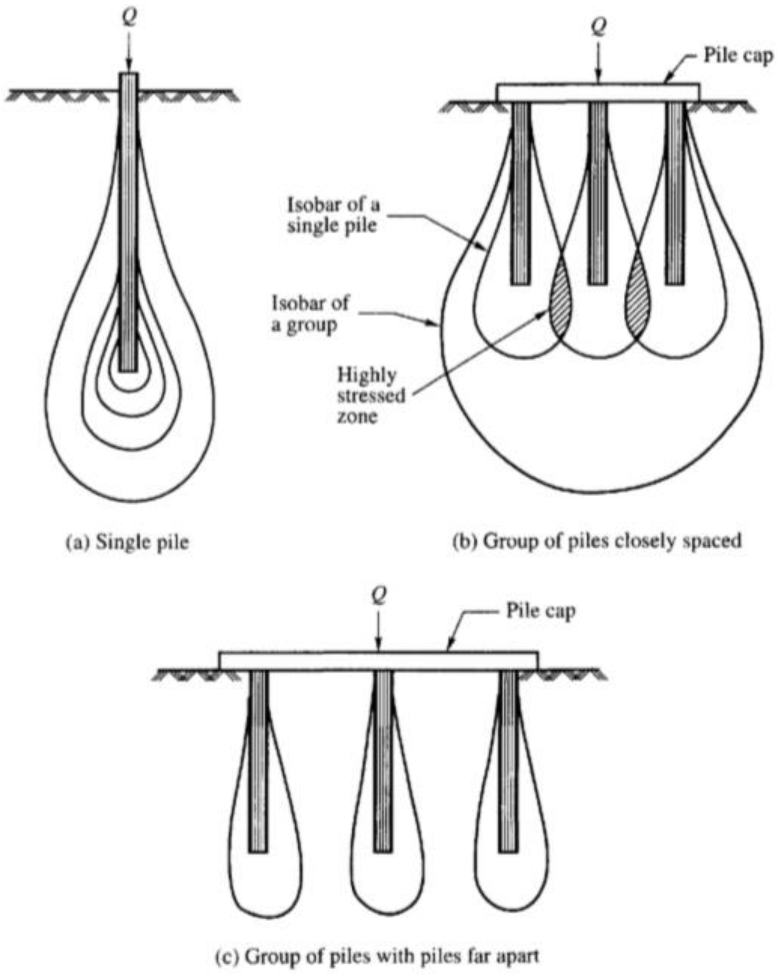
A safety factor of 3 is normally assumed for calculating the safe uplift load for both piles in clay and sand.

PILE GROUP

NUMBER AND SPACING OF PILES IN A GROUP

Very rarely are structures founded on single piles. Normally, there will be a minimum of three piles under a column or a foundation element because of alignment problems and inadvertent eccentricities. The spacing of piles in a group depends upon many factors such as,

1. overlapping of stresses of adjacent piles,
2. cost of foundation,
3. efficiency of the pile group



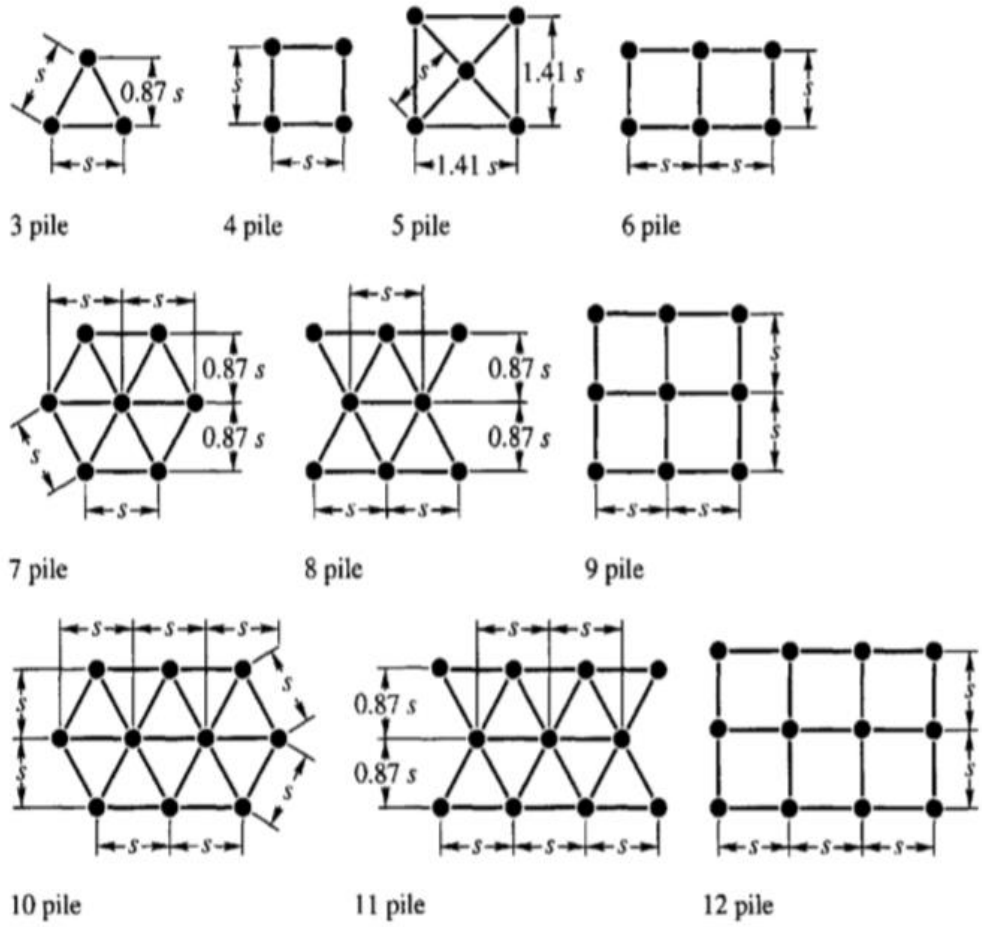
Pressure isobars of piles

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

The spacing of piles depends upon the method of installing the piles and the type of soil. The piles can be driven piles or cast-in-situ piles. When the piles are driven there will be greater overlapping of stresses due to the displacement of soil. If the displacement of soil compacts the soil in between the piles as in the case of loose sandy soils, the piles may be placed at closer intervals. But if the piles are driven into saturated clay or silty soils, the displaced soil will not compact the soil between the piles. As a result the soil between the piles may move upwards and in this process lift the pile cap. Greater spacing between piles is required in soils of this type to avoid lifting of piles. When piles are cast-in-situ, the soils adjacent to the piles are not stressed to that extent and as such smaller spacings are permitted.

Generally, the spacing for point bearing piles, such as piles founded on rock, can be much less than for friction piles since the high-point-bearing stresses and the superposition effect of overlap of the point stresses will most likely not overstress the underlying material nor cause excessive settlements.

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



Typical arrangement of piles in group

PILE GROUP EFFICIENCY

The spacing of piles is usually predetermined by practical and economical considerations. The design of a pile foundation subjected to vertical loads consists of

1. The determination of the ultimate load bearing capacity of the group, Q_{gu}.
2. Determination of the settlement of the group, S_g, under an allowable load Q_{ga}.

The ultimate load of the group is generally different from the sum of the ultimate loads of individual piles Q_u.

The factor,

$$E_g = \frac{Q_{gu}}{\Sigma Q_u} \quad - (19)$$

is called group efficiency which depends on parameters such as type of soil in which the piles are embedded, method of installation of piles i.e. either driven or cast-in-situ piles, and spacing of piles.

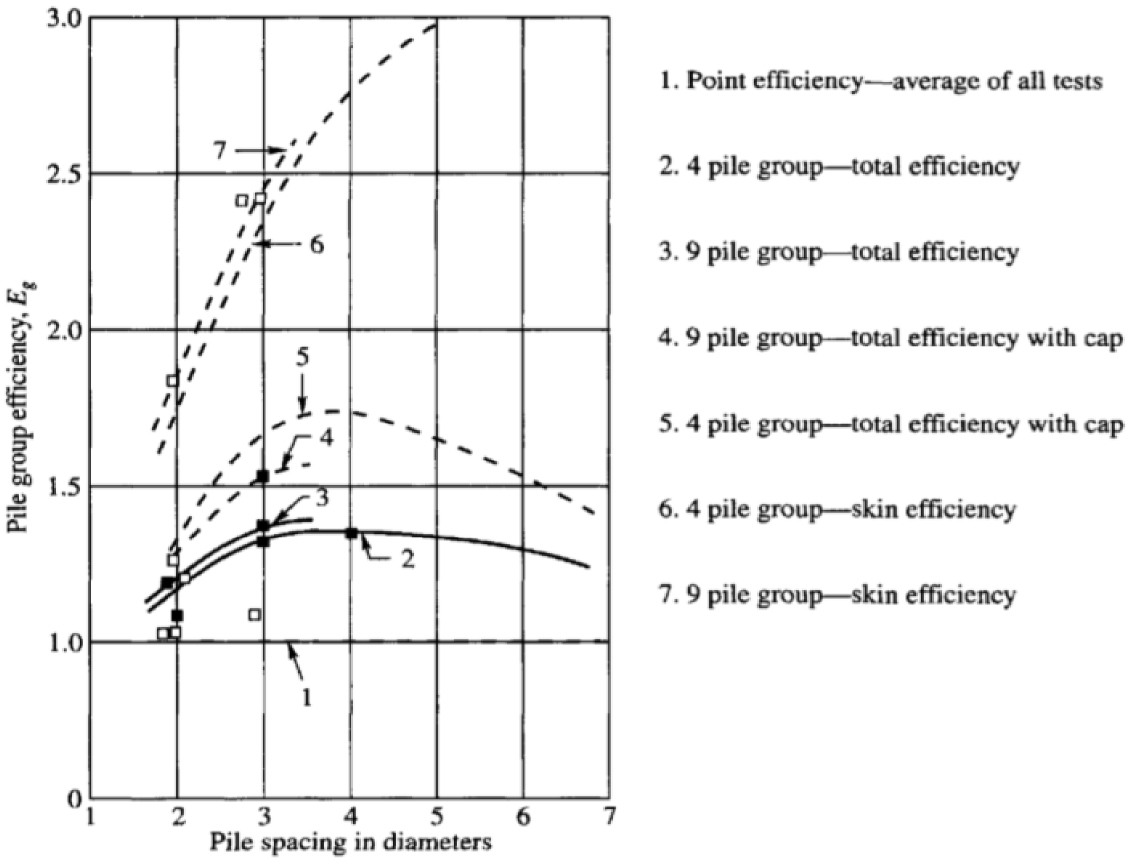
There is no acceptable "efficiency formula" for group bearing capacity. There are a few formulae such as the Converse-Labarre formula that are sometimes used by engineers. These formulae are empirical and give efficiency factors less than unity. But when piles are installed in sand, efficiency factors greater than unity can be obtained as shown by Vesic (1967) by his experimental investigation on groups of piles in sand. There is not sufficient experimental evidence to determine group efficiency for piles embedded in clay soils.

Efficiency of Pile Groups in Sand

Vesic (1967) carried out tests on 4 and 9 pile groups driven into sand under controlled conditions. Piles with spacings 2, 3,4, and 6 times the diameter were used in the tests. The tests were conducted in homogeneous, medium dense sand. His findings are given in figure. The figure gives the following:

1. The efficiencies of 4 and 9 pile groups when the pile caps do not rest on the surface.
2. The efficiencies of 4 and 9 pile groups when the pile caps rest on the surface.
3. The skin efficiency of 4 and 9 pile groups.
4. The average point efficiency of all the pile groups.

It may be mentioned here that a pile group with the pile cap resting on the surface takes more load than one with free standing piles above the surface. In the former case, a part of the load is taken by the soil directly under the cap and the rest is taken by the piles. The pile cap behaves the same way as a shallow foundation of the same size. Though the percentage of load taken by the group is quite considerable, building codes have not so far considered the contribution made by the cap.



Efficiency of pile groups in sand (Vesic, 1967)

It may be seen from the figure that the overall efficiency of a four pile group with a cap resting on the surface increases to a maximum of about 1.7 at pile spacings of 3 to 4 pile diameters, becoming somewhat lower with a further increase in spacing. A sizable part of the increased bearing capacity comes from the caps. If the loads transmitted by the caps are reduced, the group efficiency drops to a maximum of about 1.3.

Very similar results are indicated from tests with 9 pile groups. Since the tests in this case were carried out only up to a spacing of 3 pile diameters, the full picture of the curve is not available. However, it may be seen that the contribution of the cap for the bearing capacity is relatively smaller.

Vesic measured the skin loads of all the piles. The skin efficiencies for both the 4 and 9-pile groups indicate an increasing trend. For the 4-pile group the efficiency increases from about 1.8 at 2 pile diameters to a maximum of about 3 at 5 pile diameters and beyond. In contrast to this, the average point load efficiency for the groups is about 1.01. Vesic showed for the first time that the increasing bearing capacity of a pile group for piles driven in sand comes primarily from an increase in skin loads. The point loads seem to be virtually unaffected by group action.

Pile Group Efficiency Equation

There are many pile group equations. These equations are to be used very cautiously, and may in many cases be no better than a good guess. The Converse-Labarre Formula is one of the most widely used group-efficiency equations which is expressed as

$$E_g = 1 - \frac{\theta(n-1)m+(m-1)n}{90\,mn} \qquad - (20)$$

where,

m = number of columns of piles in a group,

n = number of rows,

$\theta = \tan^{-1}(d/s)$ in degrees,

d = diameter of pile,

s = spacing of piles center to center.

VERTICAL BEARING CAPACITY OF PILE GROUPS EMBEDDED IN SANDS AND GRAVELS

Driven piles:

If piles are driven into loose sands and gravel, the soil around the piles to a radius of at least three times the pile diameter is compacted. When piles are driven in a group at close spacing, the soil around and between them becomes highly compacted. When the group is loaded, the piles and the soil between them move together as a unit. Thus, the pile group acts as a pier foundation having a base area equal to the gross plan area contained by the piles. The efficiency of the pile group will be greater than unity as explained earlier. It is normally assumed that the efficiency falls to unity when the spacing is increased to five or six diameters. Since present knowledge is not sufficient to evaluate the efficiency for different spacing of piles, it is conservative to assume an efficiency factor of unity for all practical purposes. We may, therefore, write

$Q_{gu} = nQ_u$

where n - the number of piles in the group.

The procedure explained above is not applicable if the pile tips rest on compressible soil such as silts or clays. When the pile tips rest on compressible soils, the stresses transferred to the compressible soils from the pile group might result in over-stressing or extensive consolidation. The carrying capacity of pile groups under these conditions is governed by the shear strength and compressibility of the soil, rather than by the 'efficiency" of the group within the sand or gravel stratum.

Bored Pile Groups In Sand And Gravel:

Bored piles are cast-in-situ concrete piles. The method of installation involves

1. Boring a hole of the required diameter and depth,
2. Pouring in concrete.

There will always be a general loosening of the soil during boring and then too when the boring has to be done below the water table. Though bentonite slurry (sometimes called as drilling mud) is used for stabilizing the sides and bottom of the bores, loosening of the soil cannot be avoided. Cleaning of the bottom of the bore hole prior to concreting is always a problem which will never be achieved quite satisfactorily. Since bored piles do not compact the soil between the piles, the efficiency factor will never be greater than unity. However, for all practical purposes, the efficiency may be taken as unity.

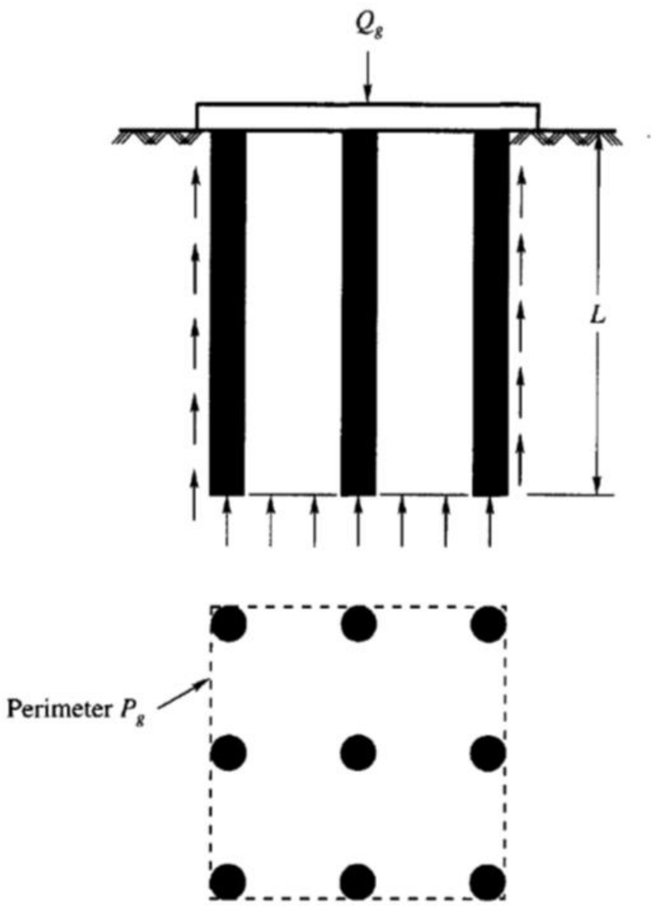
Pile Groups In Cohesive Soils:

The effect of driving piles into cohesive soils (clays and silts) is very different from that of cohesionless soils. It has already been explained that when piles are driven into clay soils, particularly when the soil is soft and sensitive, there will be considerable remolding of the soil. Besides there will be heaving of the soil between the piles since compaction during driving cannot be achieved in soils of such low permeability. There is every possibility of lifting of the pile during this process of heaving of the soil. Bored piles are, therefore, preferred to driven piles in cohesive soils. In case driven piles are to be used, the following steps should be favored:

1. Piles should be spaced at greater distances apart.
2. Piles should be driven from the center of the group towards the edges, and
3. The rate of driving of each pile should be adjusted as to minimize the development of pore water pressure.

Experimental results have indicated that when a pile group installed in cohesive soils is loaded, it may fail by any one of the following ways:

1. May fail as a block (called block failure).
2. Individual piles in the group may fail.



Block failure of a pile group in clay soil

When piles are spaced at closer intervals, the soil contained between the piles move downward with the piles and at failure, piles and soil move together to give the typical 'block failure'. Normally this type of failure occurs when piles are placed within 2 to 3 pile diameters. For wider spacings, the piles fail individually. The efficiency ratio is less than unity at closer spacings and may reach unity at a spacing of about 8 diameters.

The equation for block failure may be written as,

$$Q_{gu} = cN_cA_g + P_gL\bar{c} \qquad - (21)$$

where,

c = cohesive strength of clay beneath the pile group,

$\bar{c}$ = average cohesive strength of clay around the group,

L = length of pile,

P_g = perimeter of pile group,

A_g = sectional area of group,

Nc = bearing capacity factor which may be assumed as 9 for deep foundations.

The bearing capacity of a pile group on the basis of individual pile failure may be written as

$$Q_{gu} = nQ_u \qquad - (22)$$

where,

n = number of piles in the group,

Q_u = bearing capacity of an individual pile.

The bearing capacity of a pile group is normally taken as the smaller of the two given by Eqs.(21) and (22).

SETTLEMENT OF PILE GROUP

In Cohesionless Soil: The relation between the settlement of a group and a single pile at corresponding working loads may be expressed as

$$F_g = S_g / S \qquad - (23)$$

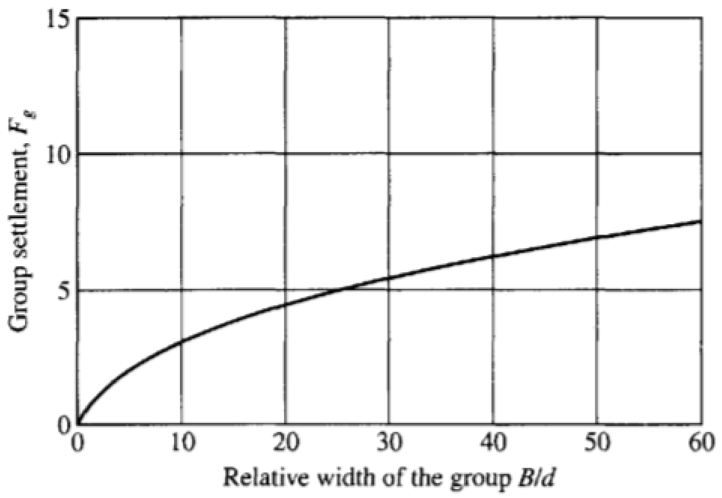
where,

F = group settlement factor,

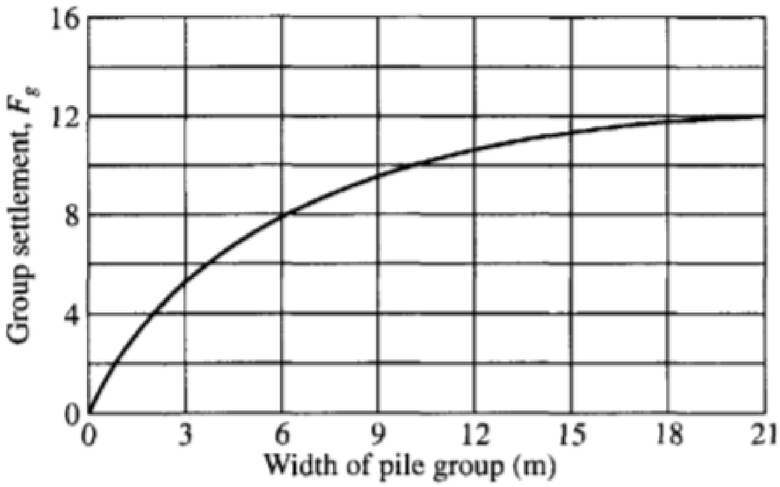
S_g = settlement of group,

S = settlement of a single pile.

Vesic (1967) obtained the curve given in figure by plotting F against Bid where d is the diameter of the pile and B, the distance between the center to center of the outer piles in the group (only square pile groups are considered). It should be remembered here that the curve is based on the results obtained from tests on groups of piles embedded in medium dense sand. It is possible that groups in much looser or much denser deposits might give somewhat different behavior. The group settlement ratio is very likely be affected by the ratio of the pile point settlement S to total pile settlement.



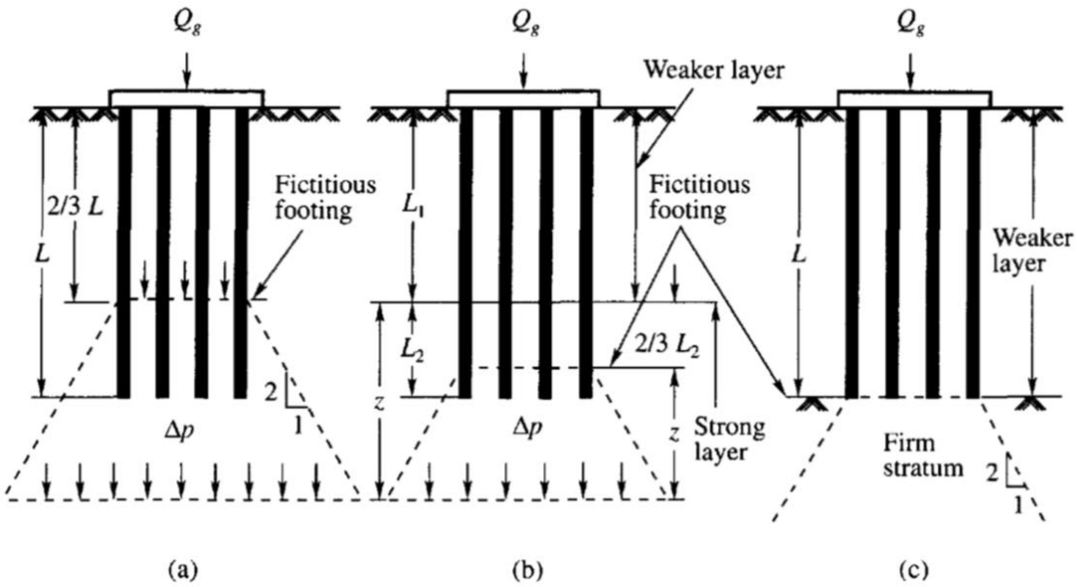
Skempton et al., (1953) published curves relating F with the width of pile groups as shown in figure. These curves can be taken as applying to driven or bored piles. Since the abscissa for the curve in figure is not expressed as a ratio, this curve cannot directly be compared with Vesic's curve given in Fig. 15.28. According to the figure a pile group 3 m wide would settle 5 times that of a single test pile.



In Cohesive Soil: The total settlements of pile groups may be calculated by making use of consolidation settlement equations. The problem involves evaluating the increase in stress Δp beneath a pile group when the group is subjected to a vertical load Q_g. The computation of stresses depends on the type of soil through which the pile passes. The methods of computing the stresses are explained below:

1. The soil in the first group given in Fig. (a) is homogeneous clay. The load Qg is assumed to act on a fictitious footing at a depth 2/3L from the surface and distributed over the sectional area of the group. The load on the pile group acting at this level is assumed to spread out at a 2 Vert : 1 Horiz slope.
2. In the second group given in (b) of the figure, the pile passes through a very weak layer of depth L_1 and the lower portion of length L_2 is embedded in a strong layer. In this case, the load Q is assumed to act at a depth equal to 2/3 L_2 below the surface of the strong layer and spreads at a 2 : 1 slope as before.

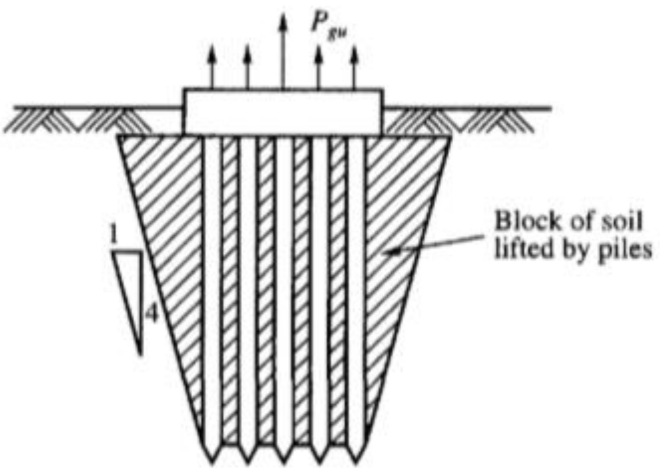
3. In the third case shown in (c) of the figure, the piles are point bearing piles. The load in this case is assumed to act at the level of the firm stratum and spreads out at a 2 : 1 slope.



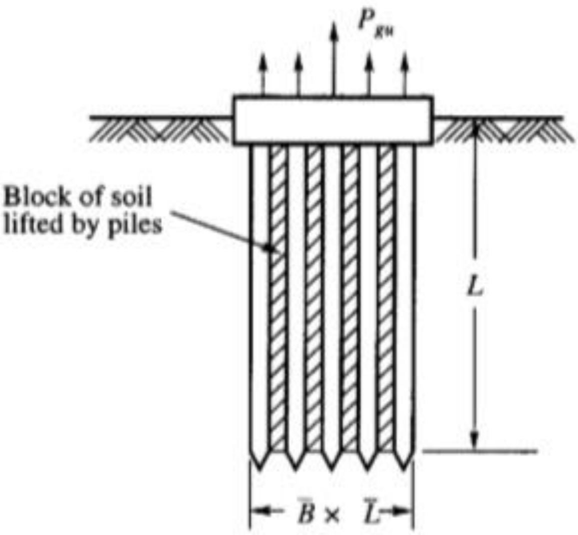
Settlement of pile groups in clay soil

UPLIFT CAPACITY OF A PILE GROUP

The uplift capacity of a pile group, when the vertical piles are arranged in a closely spaced groups may not be equal to the sum of the uplift resistances of the individual piles. This is because, at ultimate load conditions, the block of soil enclosed by the pile group gets lifted. The manner in which the load is transferred from the pile to the soil is quite complex. A simplified way of calculating the uplift capacity of a pile group embedded in cohesionless soil is shown in Fig. (a). A spread of load of 1 Horiz : 4 Vert from the pile group base to the ground surface may be taken as the volume of the soil to be lifted by the pile group (Tomlinson, 1977). For simplicity in calculation, the weight of the pile embedded in the ground is assumed to be equal to that of the volume of soil it displaces. If the pile group is partly or fully submerged, the submerged weight of soil below the water table has to be taken.



(a) Uplift of a group of closely-spaced piles in cohesionless soils



(b) Uplift of a group of piles in cohesive soils

In the case of cohesive soil, the uplift resistance of the block of soil in undrained shear enclosed by the pile group given in Fig. (b) has to be considered. The equation for the total uplift capacity P u of the group may be expressed by

$$P_{gu}=2L(\bar{L} + \bar{B})\ \bar{c}_u + W \qquad - (24)$$

where

L = depth of the pile block

$\bar{L}$ and $\bar{B}$= overall length and width of the pile group

$\bar{c}_u$ = average undrained shear strength of soil around the sides of the group

W = combined weight of the block of soil enclosed by the pile group plus the weight of the piles and the pile cap.

A factor of safety of 2 may be used in both cases of piles in sand and clay.

The uplift efficiency E of a group of piles may be expressed as

$$E_{gu} = \frac{P_{gu}}{nP_{us}}$$

where

P_{us} = uplift capacity of a single pile

n = number of piles in the group

The efficiency E_{gu} varies with the method of installation of the piles, length and spacing and the type of soil. The available data indicate that E_{gu} increases with the spacing of piles. Meyerhof and Adams (1968) presented some data on uplift efficiency of groups of two and four model circular footings in clay. The results indicate that the uplift efficiency increases with the spacing of the footings or bases and as the depth of embedment decreases, but decreases as the number of footings or bases in the group increases. How far the footings would represent the piles is a debatable point. For uplift loading on pile groups in sand, there appears to be little data from full scale field tests.

PILE LOAD TEST:

Before finalizing the design, load tests are carried out on piles installed for the purpose on the site. These are called initial load tests. They are useful in determining the general suitability of the proposed pile foundation, comparing the load capacity obtained from formulae, and for a general check on the piling equipment to be used as well as on soil properties.

Procedure for pile load test

The pile head is chipped off to natural horizontal plane till sound concrete is met. The projecting reinforcement is cut off suitably and the top is finished smooth & level with plaster of Paris.Loading platform of 6.2m x 6.2m is constructed by using 2nos. of ISMB 500 as main girders and 21nos of ISMB 300 as secondary girders.

The CG of platform is made to coincide with centre of pile. Platform thus constructed is loaded with sand bags for required weight. A 20mm thick mild steel plate is placed on the top of pile head, Hydraulic jack of 250T Capacity is placed centrally on top of the plate. The gap between the top of jack and bottom of main girders is filled with steel packing materials. The Hydraulic pump is connected to jack by flexible pressure hose. Calibrated pressure gauge is connected to hydraulic pump. Datum bars of heavy sections were placed very near to pile head and are supported on ends at a distance of 2m on either side from face of the pile. Two numbers of settlement gauges are placed on pile head at diametrical opposite locations with the help of magnetic bases fixed on datum bars.

The pump is operated till the ram of jack touches the bottom of main girders. At this stage the pressure gauge reading is zero and dial gauge reading are adjusted for zero loading. The loads are then applied in increments of 20% of safe load. For each increment of load the dial gauge reading are taken at intervals of 15 minutes, till the rate of settlement is less than 0.1 mm in the first half hour or 0.2 mm in one hour of for a maximum period of 2hrs. Then the next increment of load is applied and the procedure repeated till the test load is reached. This load is maintained for 24 hours and hourly settlement readings are noted. At the end of 24 hours, unloading is done gradually till the entire load is released.

INTERPRETATION OF PILE LOAD TEST:

There are different methods for determining the allowable loads on a single pile which can be determined by making use of load test data. If the ultimate load can be determined from load-settlement curves, allowable loads are found by dividing the ultimate load carried by a pile by suitable factor of safety which varies from 2 to 3. Normally a factor safety is 2.5 is recommended.

Determination of Ultimate load from load-settlement curve

1. The ultimate load, Q_u can be determined as the abscissa of the point where the curved part of the load-settlement curve changes to falling straight line, Fig.a)
2. Q_u is the abscissa of the point of intersection of the initial and final tangents of the load settlement curve, Fig.(b)
3. The allowable load Q_a is 50 percent of the ultimate load at which the total settlement amounts to one-tenth of the diameter of the pile.
4. The allowable load Q_a is sometimes taken as equal to two-thirds of the load which causes a total settlement of 12mm.
5. The allowable load Q_a is sometimes taken as equal to two-thirds of the load which causes a net settlement of 6mm.

UNDER-REAMED PILES

These are bored, cast in-situ, concrete piles with one or more bulbs formed by enlarging the pile stem. They are suitable for loose and filled up sites, or where soils are weak or expansive like black cotton soil.

The bulbs are located at depths where good bearing strata are available but they should not be placed too near the ground level. Bulb size is usually 2 to 3 times the pile stem diameter. The bulb provides a large bearing area, increasing the pile load capacity. They are also effective in resisting the downward drag due to the negative skin friction that arises in loose or expansive soils. Bulb spacing should not exceed 1.5 times the bulb diameter.

Procedure for Construction of Under-Reamed Piles

The hole is drilled to the full required depth using augers. The under reaming tool consists of a link mechanism attached to a vertical rod with a handle at the top and connected to a bucket at the bottom. The link mechanism incorporates cutting blades. The under reaming tool is inserted into the hole. When the central rod is pressed by the handle the mechanism actuates the cutting blades to open out. The mechanism is now made to rotate keeping the handle under pressure. The blades now scrap the soil from the sides of the hole which falls into the bucket below. The rotation under pressure is continued until the full amount of soil forming the bulb is removed which is identified by the free rotation of the mechanism. The volume of the bucket is such that it gets filled when the bulb is fully formed. The handle is now tightened which makes the link mechanism to collapse back into the position. The under reamed tool is now withdrawn, the reinforcement cage inserted and the hole concreted.

CAPACITY UNDER COMPRESSION

The load carrying of a single under-reamed pile under compression maybe obtained as follows,

$$Q_u = Q_b + Q_f = A_b q_b + A_s f_s$$

where,

A_b = sectional area of bulb

q_b = base resistance per unit area of the bulb

A_s = surface area of the embedded shaft of the pile

f_s = unit skin friction

CAPACITY UNDER HYDROSTATIC UPLIFT

Under-reamed piles are also used as anchor piles to take up hydrostatic uplift pressure exerted on submerdged structures and as tension piles to take up uplift loads under tall towers subjected to moments. The equation for uplift load foe a double under-reamed pile is

$$Qup = A_s f_s + A_s f_s + W_p$$

where,

W_p = weight of the pile

f_s = 50% of fs for compression piles

UNIT V
RETAINING WALLS

BASIC TERMINOLOGY

Introduction

Soil is neither a solid nor a liquid, but it exhibits some of the characteristics of both. One of the characteristics similar to that of a liquid is its tendency to exert a lateral pressure against any object in contact. This important property influences the design of retaining walls, abutments, bulkheads, sheet pile walls, basement walls and underground conduits which retain or support soil, and, as such, is of very great significance. Retaining walls are constructed in various fields of civil engineering, such as hydraulics and irrigation structures, highways, railways, tunnels, mining and military engineering.

Retaining structures:

The structure used to retain or support the material/soil is called retaining structure. e.g. retaining walls, which may be of RCC, brick or stone masonry or sheet piling etc.

Retaining walls:

A retaining wall is a structure designed to sustain the material pressure of earth or other materials as grains, ores, etc.

Surcharge:

The material which lies above the horizontal level of the retaining structure is known as surcharge. The angle which this material makes with the retaining wall is called surcharge angle.

LATERAL EARTH PRESSURE

In 1929 Terzaghi (The Father of Soil Mechanics) conducted experiments on the retaining wall and showed the relation of pressure on the wall if wall changes its position i.e. to move inwards to the backfill, outwards of it or remain at its place. **There are three types of earth pressures on the basis of the movement of the wall.**

1. Earth Pressure at rest
2. Active Earth Pressure
3. Passive Earth Pressure

These are explained below

Pressure at rest:

When the wall is at rest and the material is in its natural state then the pressure applied by material is known as Earth Pressure at Rest. It is represented by Po.

Active earth pressure

When the wall moves away from the backfill, there is a decrease in the pressure on the wall and this decrease continues until a minimum value is reach after which there is no reduction in the pressure and the value will become constant. This kind of pressure is known as active earth pressure.

Passive earth pressure

When the wall moves towards the backfill, there is an increase in the pressure on the wall and this increase continues until a maximum value is reach after which there is no increase in the pressure and the value will become constant. This kind of pressure is known as passive earth pressure.

This means that when the wall is about to slip due to lateral thrust from the backfill, a resistive force is applied by the soil in front of the wall.

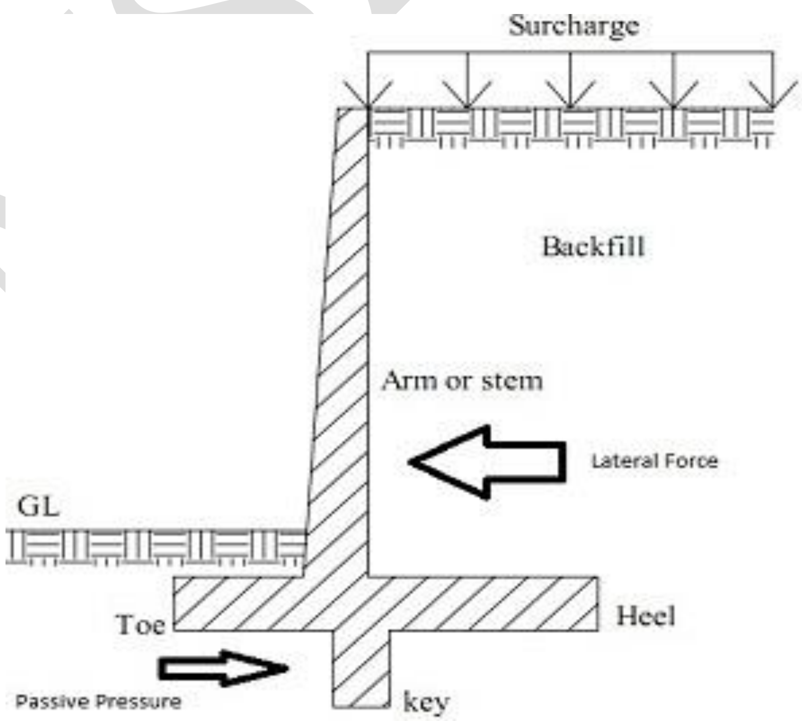


Figure 1: Retaining Wall

The magnitude of lateral earth pressure depends on:

1. Shear strength characteristics of soil
2. Lateral strain condition
3. Pore water pressure
4. State of Equilibrium of soil
5. Wall and ground surface shape

Soil state of Equilibrium

The state of Equilibrium of soil can be divided into two states:

a) State of Elastic Equilibrium

When a small change in stress produces a corresponding small change in strain.

b) State of Plastic Equilibrium

When irreversible strain takes place at a constant stress.

The strain state relating to earth pressure calculation falls into three categories:

a) *At Rest State:*

It is the case when state of elastic Equilibrium with no lateral displacement occurs (Figure 1).

b) *Active state:*

It is the case when plastic equilibrium with lateral expansion takes place (wall moves outward from the soil). (Figure 2).

c) *Passive state:*

Plastic equilibrium with lateral contraction takes place (wall moves toward the soil). (Figure 2)

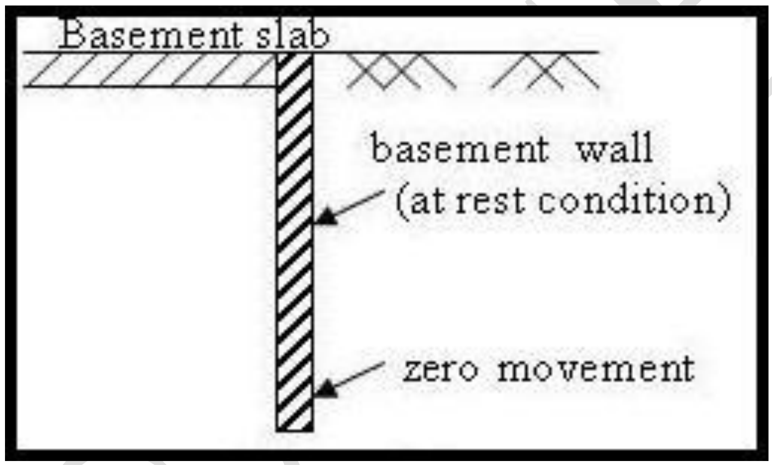


Figure 1

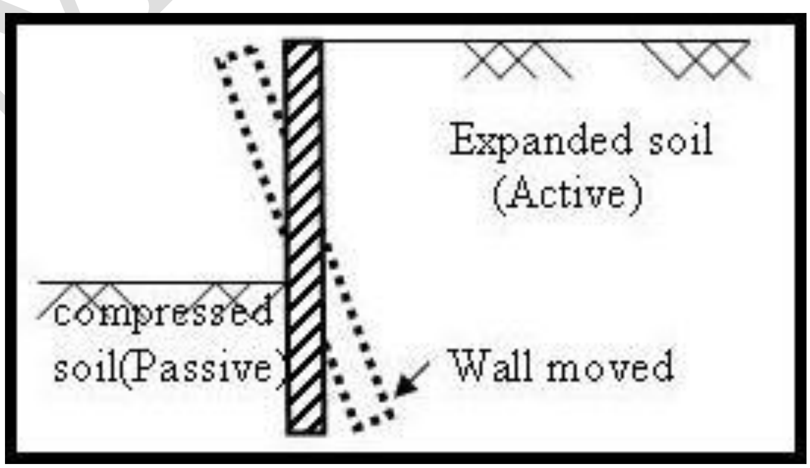


Figure 2

Earth Pressure at rest (ko condition)

lateral strain , $\varepsilon_h = 0$

$$\sigma_h = k_0 \sigma_v$$

Where,

k_o = coefficient of lateral earth presser at rest. k_o depends on :

- Soil type (sand, silt, clay)
- Loading –Unloading history
- Relative density of soil

For normally consolidated soil (N.C.) and sand

$$k_0 = 1 - \sin\emptyset$$

For over consolidated soil (O.C.)

$$k_0 = k_{0\,(NC)}\sqrt{OCR}$$

where

OCR is the over consolidated ratio

$$OCR = \frac{pre\ consolidation\ pressure}{present\ overburden\ pressure}$$

$$k_0 = \frac{\mu}{1-\mu}$$

Where

μ is Poisson ratio

In this case ,the mohr circle is below the shear envelope of this soil (Figure 3).

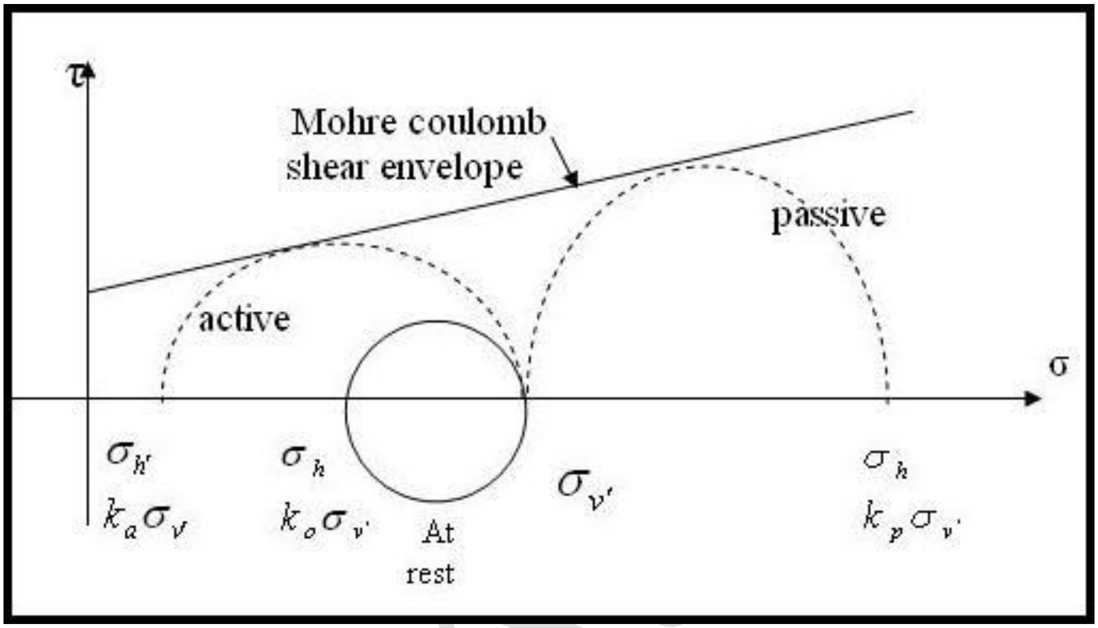


Figure 3

Active Earth pressure (Rankine's Active state)

- Rankine's theory neglects friction between soil and wall .when the wall moves outward from the soil, the lateral earth pressure starts to be reduced until it reaches its minimum value.

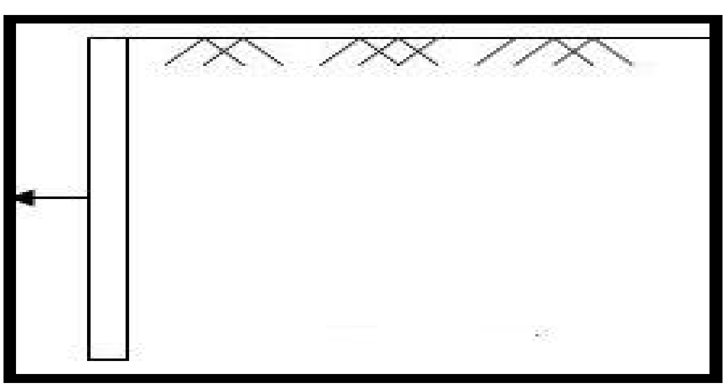


Figure 4

$\sigma_h = k_a \sigma'_v$

$k_a = \dfrac{1 - \sin \emptyset}{1 + \sin \emptyset} = \tan^2\left(45 - \dfrac{\emptyset}{2}\right)$

where ,

k_a = coefficient of active earth pressure. (Figure 4)

And in this case , the failure plane makes (45 – Ø/2) with the direction of the major principal plane (usually horizontal axes) .

Rankine's Passive State

If the wall moved toward the soil , the passive condition will take place.The passive pressure will increase until it reaches its maximum value of:

$$\sigma'_h = k_p \sigma'_v$$

Where

k_p= coefficient of passive earth pressure

$$k_p = \frac{1 + sin\emptyset}{1 - sin\emptyset} = \frac{1}{k_a} = \tan^2\left(45 + \frac{\emptyset}{2}\right)$$

and in this case , the failure place makes (45 + Ø/2) with the direction of the major principal plane.

Earth pressure Distribution for Sand

In general, the active earth pressure distribution in sand will be:

$$e_a = \gamma H k_a$$

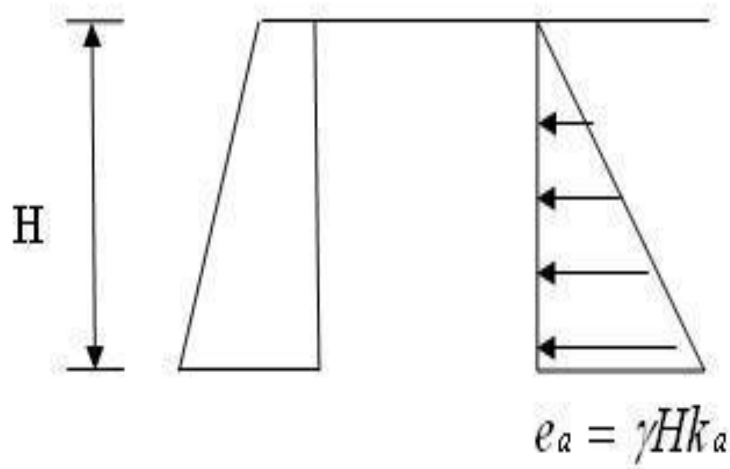
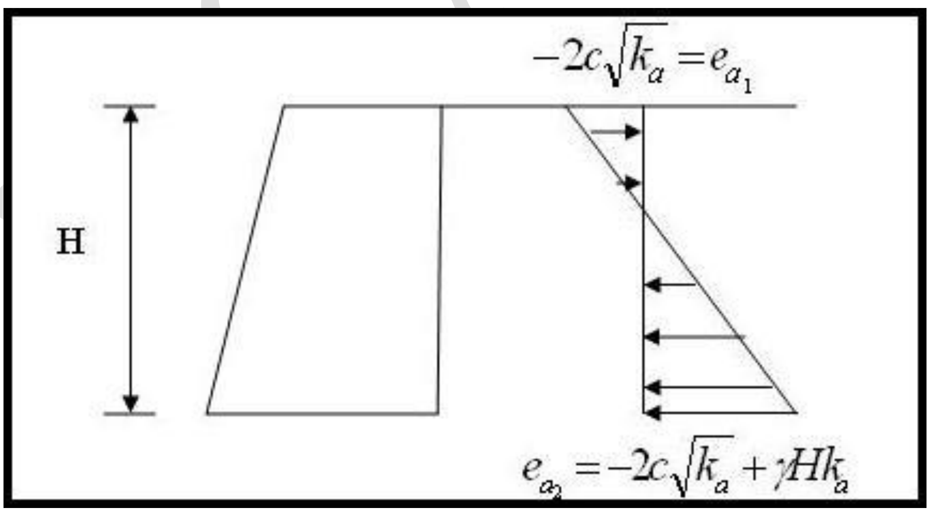


Figure (5)

However the distribution and value of passive earth pressure in sand is, Figure(5).

(b) For clay -

The value of active earth pressure is (Figure 6)



Figure(6)

$$e_{a_1} = -2c\sqrt{k_a}$$

$$e_{a_2} = -2c\sqrt{k_a} + \gamma H k_a$$

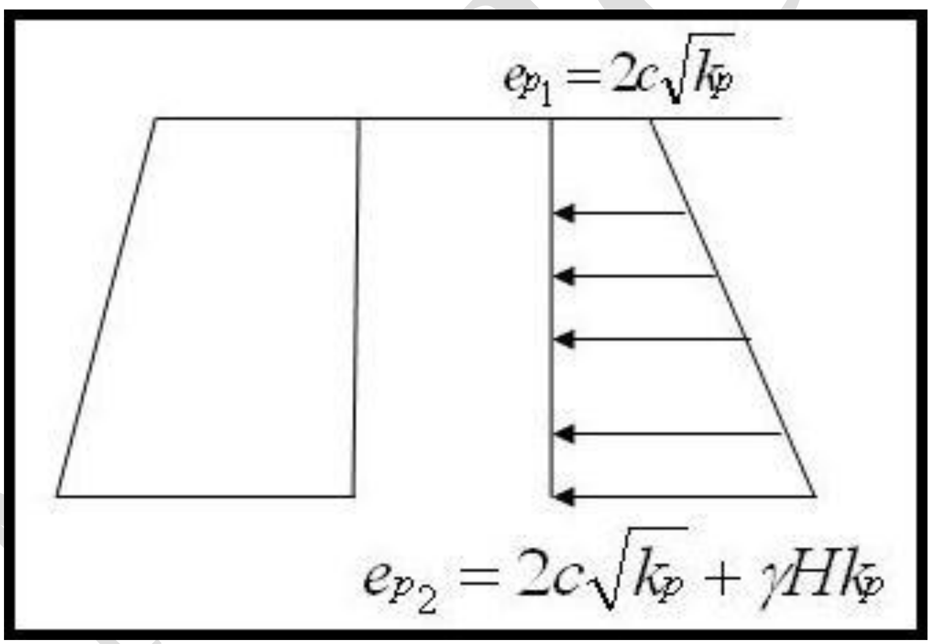
where ,

c = soil cohesion

The passive pressure values are (Figure 7)

$$e_{p1} = 2c\sqrt{k_p}$$

$$e_{p1} = 2c\sqrt{k_p} + \gamma H k_p$$



Figure(7)

Effect of Ground Water Table

Static water table

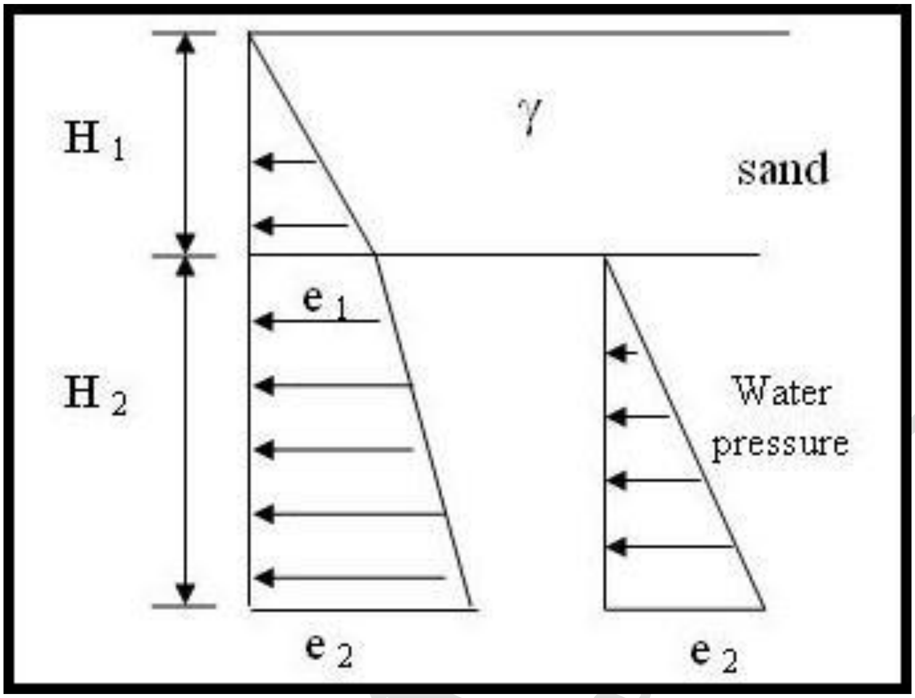


Figure 8

For static water table the water pressure should be treated separately from soil lateral pressure as shown in Figure(8)

$e_1 = \gamma H_1 k_a$

$e_2 = (\gamma H_1 + \gamma_{sub} H_2) k_a$

$e_3 = H_2 \gamma_w$

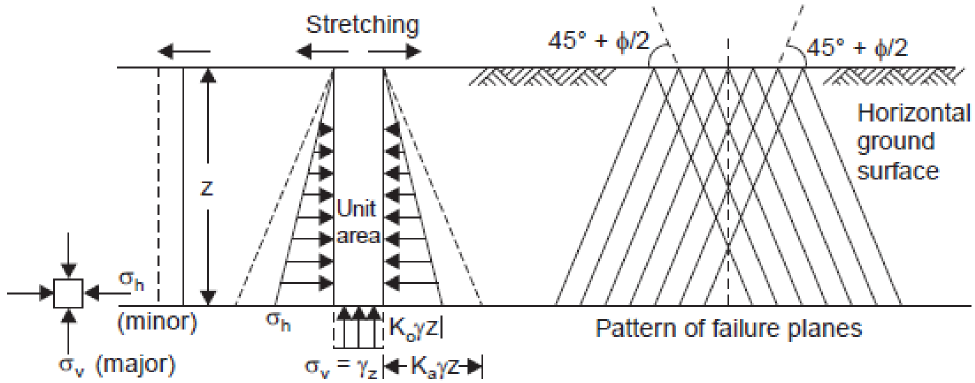
where

γ_{sub} = submerged unit weight of soil = γ_{sat} - γ_w

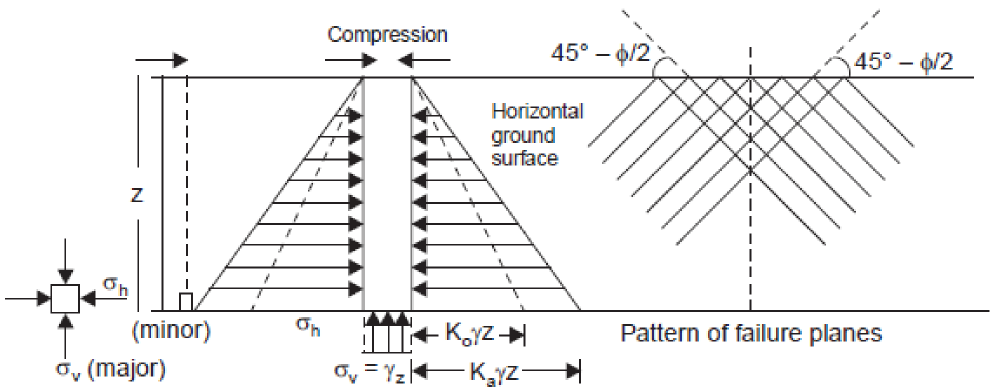
RANKINE'S THEORY

The following are the important assumptions in Rankine's theory:

1. The soil mass is semi-infinite, homogeneous, dry and cohesion less.
2. The ground surface is a plane which may be horizontal or inclined.
3. The face of the wall in contact with the backfill is vertical and smooth. In other words, the friction between the wall and the backfill is neglected (these amounts.
4. The wall yields about the base sufficiently for the active pressure conditions to develop; if it is the passive case that is under consideration, the wall is taken to be pushed sufficiently towards the fill for the passive resistance to be fully mobilized. (Alternatively, it is taken that the soil mass is stretched or gets compressed adequately for attaining these states, respectively. Friction between the wall and fill is supposed to reduce the active earth pressure on the wall and increase the passive resistance of the soil. Similar is the effect of cohesion of the fill soil). Thus it is seen that, by neglecting wall friction as also cohesion of the backfill, the geotechnical engineer errs on the safe side in the computation of both the active pressure and passive resistance. Also, the fill is usually of cohesionless soil, wherever possible, from the point of view of providing proper drainage.



(a) Active Rankine State



(b) Passive Rankine State

EARTH PRESSURE ON RETAINING WALLS: COHESIONLESS BACKFILL

Active Earth Pressure

a) Dry backfill with no surcharge

$$P_A = K_a \gamma z$$

Where, γ= bulk unit weight of soil above the water table

At the base of the wall, where z= H

$$p_A = K_a \gamma H$$

The total active thrust is given by the following equation, $P_A = \frac{1}{2} K_a \gamma H \times H = \frac{1}{2} K_a \gamma H^2$ acting at a height of H/3 above the base of the wall.

b) Submerged backfill

If the backfill is submerged by the presence of the natural water table at a depth of H1 from the top, the active earth pressure up to the depth H1 is determined according to the above equation. For the submerged backfill, the lateral earth pressure at any depth is the sum of

1. Active earth pressure due to submerged unit weight of the soil mass γ' and
2. Hydrostatic pressure

$$p_A = K_a \gamma H + K_a \gamma' (H - H_1) + \gamma_w (H - H_1)$$

c) Effect of uniform surcharge

If a uniformly distributed surcharge load of intensity q per unit area is acting over the surface of the backfill, it is assumed that the effective vertical pressure p_v is increased by q. Hence the increase in active pressure is uniform throughout the back of the wall and is equal to magnitude K_aq.

$$p_A = K_a p_V$$

Where,

p_V = vertical pressure at a given depth

$p_V = \gamma z + q \ (q \ is \ the \ surcharge)$

$p_A = K_a \gamma z + K_a q$

d) Effect of Sloping ground surface

The active earth pressure at a depth z acting parallel to slope is given by

$$p_A = K_a \gamma z \cos\beta$$

$$K_a = \frac{\cos\beta - \sqrt{cos^2\beta - cos^2\emptyset}}{\cos\beta + \sqrt{cos^2\beta - cos^2\emptyset}}$$

Total active thrust acting on the wall is given by

$$P_A = \frac{1}{2} K_a \gamma H^2 \cos\beta$$

e) Case of inclined wall

When the back of the wall has batter, the following procedure can be adopted to determine the earth pressure. The height of the vertical plane AB is used in calculating P_A.

$$P_A = \sqrt{P_{A1}^2 + W^2}$$

Passive Earth Pressure

When the backfill is horizontal and the soil is dry sand, passive earth pressure at a depth z is given by,

$$p_{pz} = K_p \gamma z$$

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

Hence the total passive resistance P_p for the full height H of the retaining wall is

$$P_P = \frac{1}{2} K_p \gamma H^2$$

$$P_P \; acts \; as \; height \frac{H}{3} \; from \; the \; base \; of \; the \; wall$$

If the uniform surcharge load q is applied over the surface, passive earth pressure is increased by $K_p q$ at every depth.When the backfill has sloping surface inclined at β to the horizontal, it can be shown that the passive earth pressure p_p at a depth z is given by,

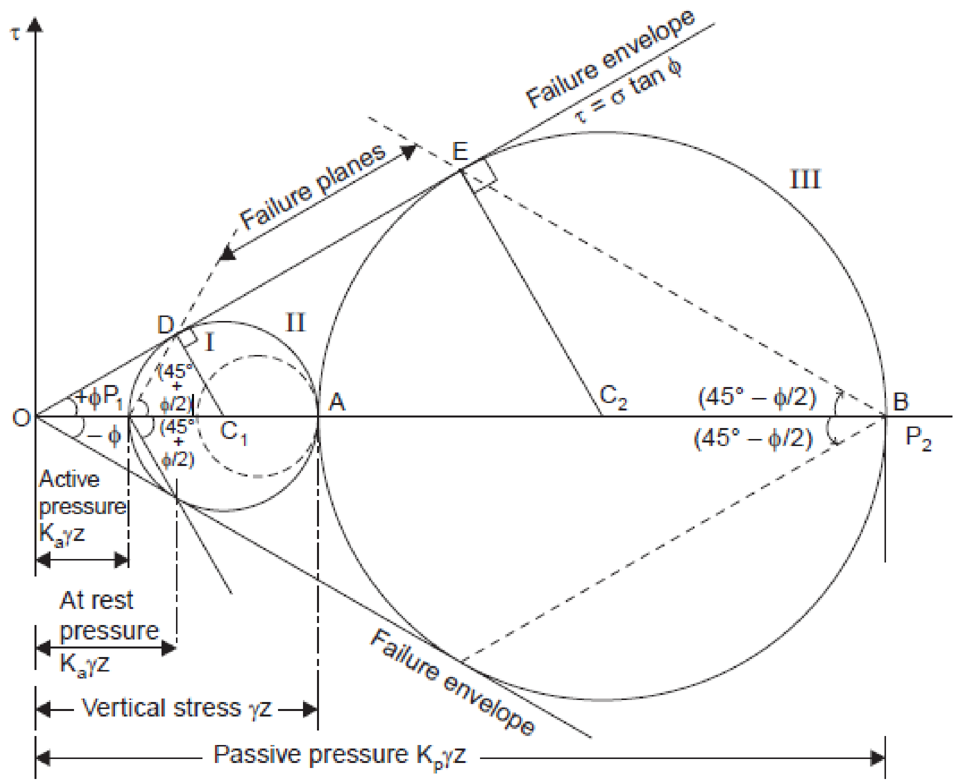
$$p_p = K_p \gamma z \cos\beta$$

Where

$$K_p = \frac{\cos\beta + \sqrt{cos^2\beta - cos^2\emptyset}}{\cos\beta - \sqrt{cos^2\beta - cos^2\emptyset}}$$

The total passive resistance P_p for a wall height H is given by

$$P_p = \frac{1}{2} K_p \gamma H^2 \cos\beta$$



1. Mohr's circle for 'At rest' condition
2. Mohr's circle for 'Active' condition
3. Mohr's circle for 'Passive' condition

(c) Mohr's stress circles and failure envelopes for active and passive states

EARTH PRESSURE ON RETAINING WALLS: COHESIVE BACKFILL

Active Earth Pressure

$$p_A = \gamma z K_a - 2c\sqrt{K_a}$$

At z=0 and $p_A = -2c\sqrt{K_a}$

And $p_A = 0$ at $z = z_0 = \frac{2c}{\gamma\sqrt{K_a}}$

In calculating the total active thrust on the wall, the tension zone is usually ignored and only the area of the pressure distribution between depths z_0 and H.

$$P_A = \frac{1}{2} K_a \gamma H^2 - 2c\, H\sqrt{K_a} + \frac{2\, c^2}{\gamma}$$

The net total active thrust is zero for a depth equal to $2z_0$. In cohesive soil, a vertical cut can be made upto a depth of $2z_0$ without having to provide any lateral support. Thus, the critical depth of vertical cut $H_c$nin a cohesive soil is given by

$$H_c = 2z_0 = \frac{4c}{\gamma\sqrt{K_a}}$$

Passive Earth Pressure

$$p_p = \gamma z K_p + 2c\sqrt{K_p}$$

The total passive earth pressure for the full height H of the wall is given by

$$P_p = \frac{1}{2}\gamma H^2 K_p + 2c\, H\sqrt{K_p}$$

COULOMB'S WEDGE THEORY AND CRITICAL FAILURE PLANE

The primary assumptions in Coulomb's wedge theory are as follows:

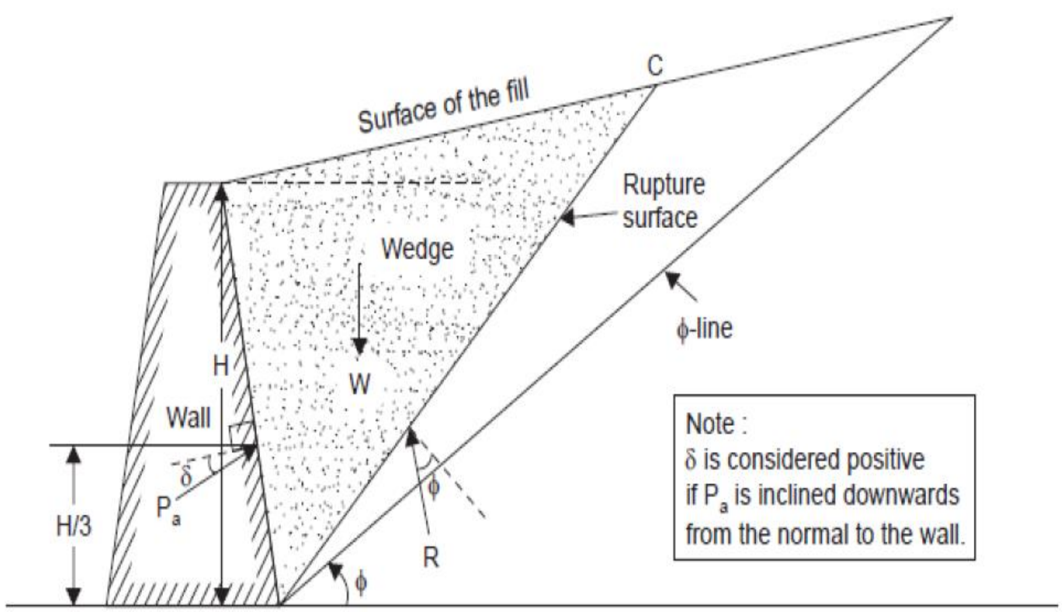
1. The backfill soil is considered to be dry, homogeneous and isotropic; it is elastically undeformable but breakable, granular material, possessing internal friction but no cohesion.
2. The rupture surface is assumed to be a plane for the sake of convenience in analysis. It passes through the heel of the wall. It is not actually a plane, but is curved and this is known to Coulomb.
3. The sliding wedge acts as a rigid body and the value of the earth thrust is obtained by considering its equilibrium.
4. The position and direction of the earth thrust are assumed to be known. The thrust acts on the back of the wall at a point one-third of the height of the wall above the base of the wall and makes an angle δ, with the normal to the back face of the wall. This is an angle of friction between the wall and backfill soil and is usually called 'wall friction'.
5. The problem of determining the earth thrust is solved, on the basis of two-dimensional case of 'plane strain'. This is to say that, the retaining wall is assumed to be of great length and all conditions of the wall and fill remain constant along the length of the wall. Thus, a unit length of the wall perpendicular to the plane of the paper is considered.
6. When the soil wedge is at incipient failure or the sliding of the wedge is impending, the theory gives two limiting values of earth

pressure, the least and the greatest (active and passive), compatible with equilibrium. The additional inherent assumptions relevant to the theory are as follows:

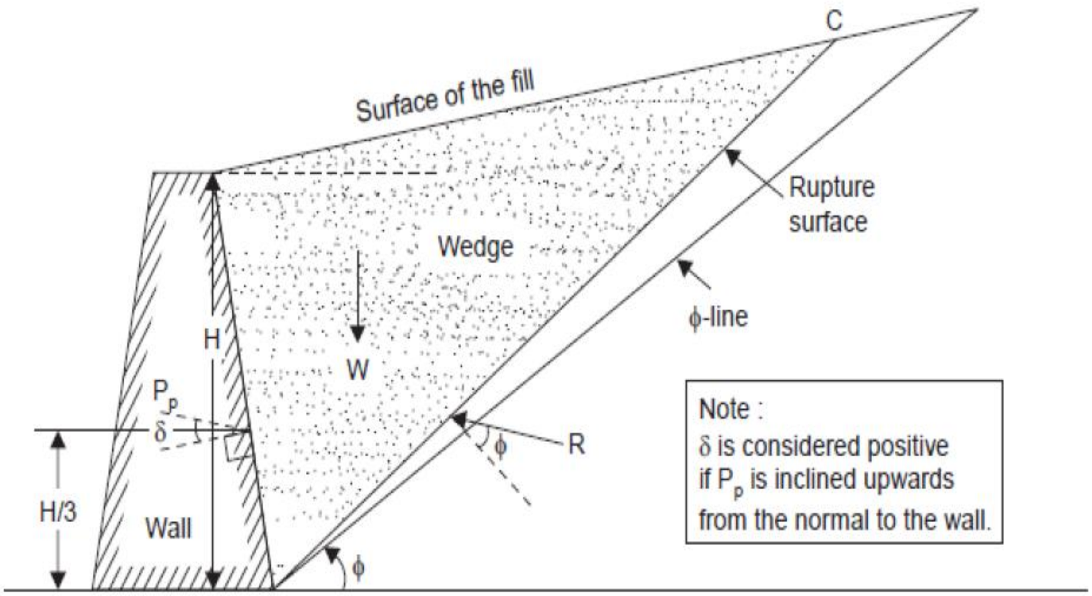
7. The soil forms a natural slope angle, φ, with the horizontal, without rupture and sliding. This is called the angle of repose and in the case of dry cohesion less soil; it is nothing but the angle of internal friction. The concept of friction was understood by Coulomb.
8. If the wall yields and the rupture of the backfill soil takes place, a soil wedge is torn off from the rest of the soil mass. In the active case, the soil wedge slides sideways and downward over the rupture surface, thus exerting a lateral pressure on the wall. In the case of passive earth resistance, the soil wedge slides sideways and upward on the rupture surface due to the forcing of the wall against the fill.
9. For a rupture plane within the soil mass, as well as between the back of the wall and the soil, Newton's law of friction is valid (that is to say, the shear force developed due to friction is the coefficient of friction times the normal force acting on the plane). This angle of friction, whose tangent is the coefficient of friction, is dependent upon the physical properties of the materials involved.
10. The friction is distributed uniformly on the rupture surface.
11. The back face of the wall is a plane.

Limitations of coulomb's theory

Also note that Coulomb's theory treats the soil mass in the sliding wedge in its entirety. The assumptions permit one to treat the problem as a statically determinate one. Coulomb's theory is applicable to inclined wall faces, to a wall with a broken face, to a sloping backfill curved backfill surface, broken backfill surface and to concentrated or distributed surcharge loads. One of the main deficiencies in Coulomb's theory is that, in general, it does not satisfy the static equilibrium condition occurring in nature. The three forces (weight of the sliding wedge, earth pressure and soil reaction on the rupture surface) acting on the sliding wedge do not meet at a common point, when the sliding surface is assumed to be planar. Even the wall friction was not originally considered but was introduced only some time later. Regardless of this deficiency and other assumptions, the theory gives useful results in practice; however, the soil constants should be determined as accurately as possible.



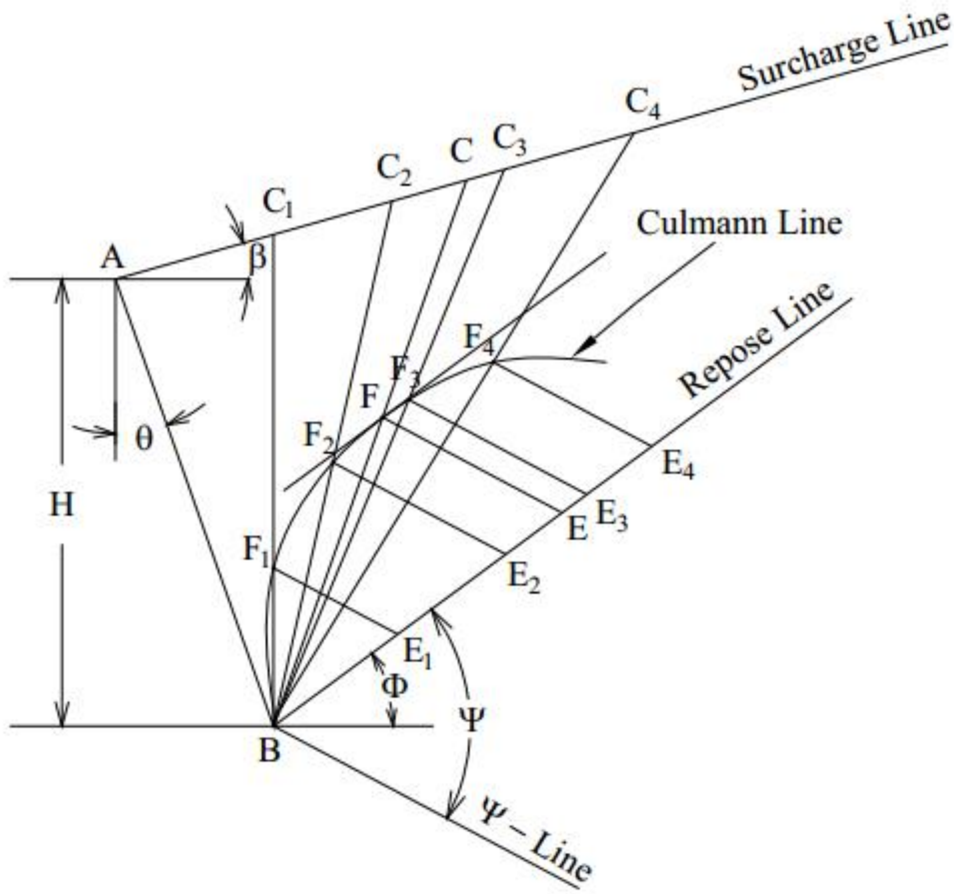
(a) Active earth pressure



(b) Passive earth resistance

CULMANN'S GRAPHICAL METHOD

This graphical method given by Culmann (1886) is more general than Rebhann's method and is very convenient to use in the case of layered backfill, backfill with breaks at surface and different types of surcharge load.



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

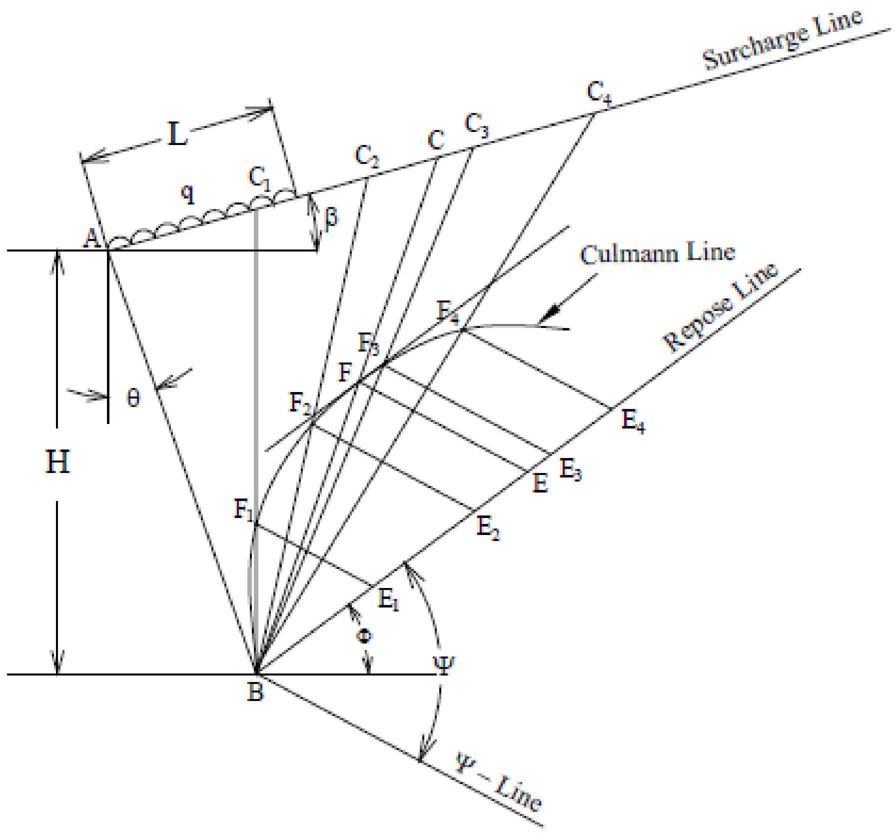
1. Given height H and batter angle θ, the back AB of the wall is constructed.
2. Through A, the surcharge line (β-line) is drawn inclined at angle β to the horizontal.
3. Through B, the repose line (∅-line) is drawn inclined at an angle ∅ to the horizontal.
4. Again through B, the Ψ-line is drawn inclined at an angle Ψ to the ∅-line ($\Psi = 90° - \theta - \delta$).

5. Trial slip planes BC1, BC2…… are drawn. The weights of the wedges ABC1, ABC2…… are calculated and plotted to scale as BE1,BE2,…… on the Ø-line.
6. Through E1, E2,…. lines are drawn parallel to Ψ-line, intersecting BC1, BC2,……. at F1, F2..... respectively.
7. A smooth curve is drawn through points B, F1, F2…… This curve is called Culmann line.
8. A line is drawn parallel to Ø-line and tangential to Culmann line. Let it touch Culmann line at F. BF is joined and produced to intersect the β-line at C. Then BC is the critical slip plane.
9. Through F, line FE is drawn parallel to Ψ-line, intersecting Ø-line at E.
10. The weight W of the wedge ABC is calculated. The resultant active earth pressure Pa is given by

$$P_A = w\left(\frac{FE}{BE}\right)$$

SPECIAL CASES:

(i) Backfill with uniform surcharge load.



As an illustration consider in which uniformly distributed surcharge load of intensity q is shown acting over a length L. The procedure is similar to the previous case but for the following changes.

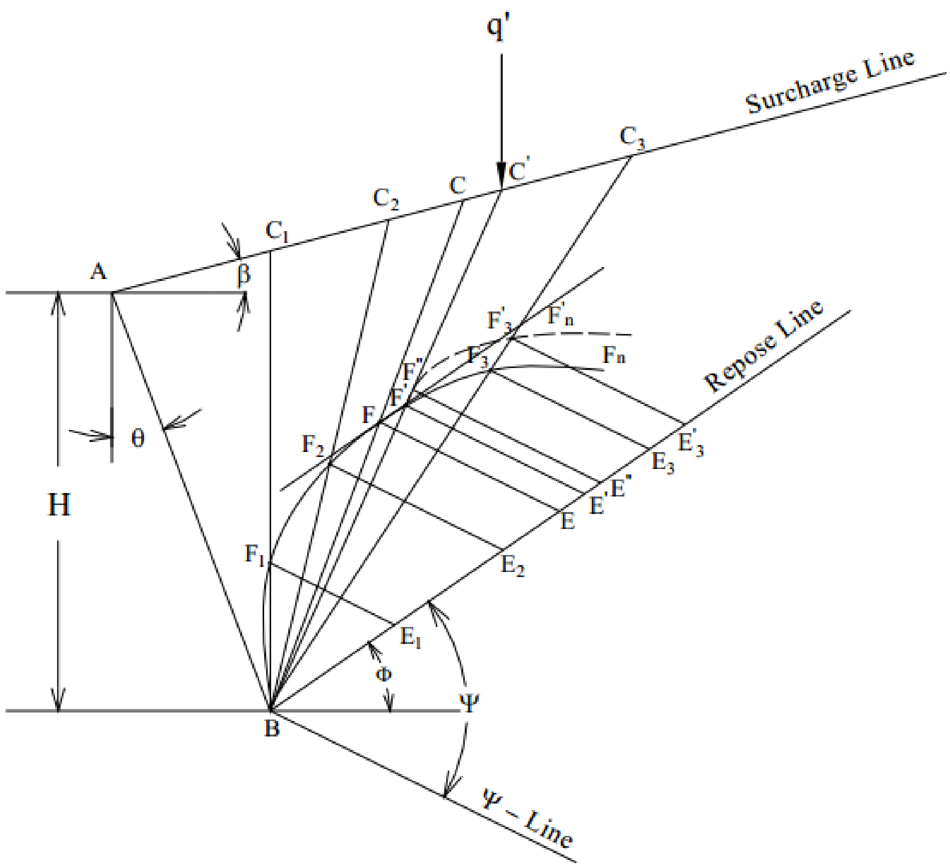
1. BE1 represent the sum of weight of wedge ABC1 and surcharge load q(AC1).
2. BE2 represents the sum of weight of wedge ABC2 and surcharge load qL. Similarly BE3, BE4 represent the sum of respective sliding wedges and surcharge load Lq.
3. The resultant active earth pressure is given by

$$P_A = w\left(\frac{FE}{BE}\right)$$

Where,

w = weight of wedge ABC + qL

ii) Backfill with line load



As an illustration, consider Fig. in which a line load of intensity q1 (per unit run) acts at distance d from top of wall. Example of line load is load due to any wall or a railway track running parallel to retaining wall. In the Fig. B, F1, F2…… Fn is Culmann line obtained without considering line load. BC then represents the critical slip plane and the resultant active pressure is given by,

$$P_A = w\left(\frac{FE}{BE}\right) \ldots\ldots\ldots\ldots\ldots\ldots \text{(1)}$$

Where,

w = weight of wedge ABC

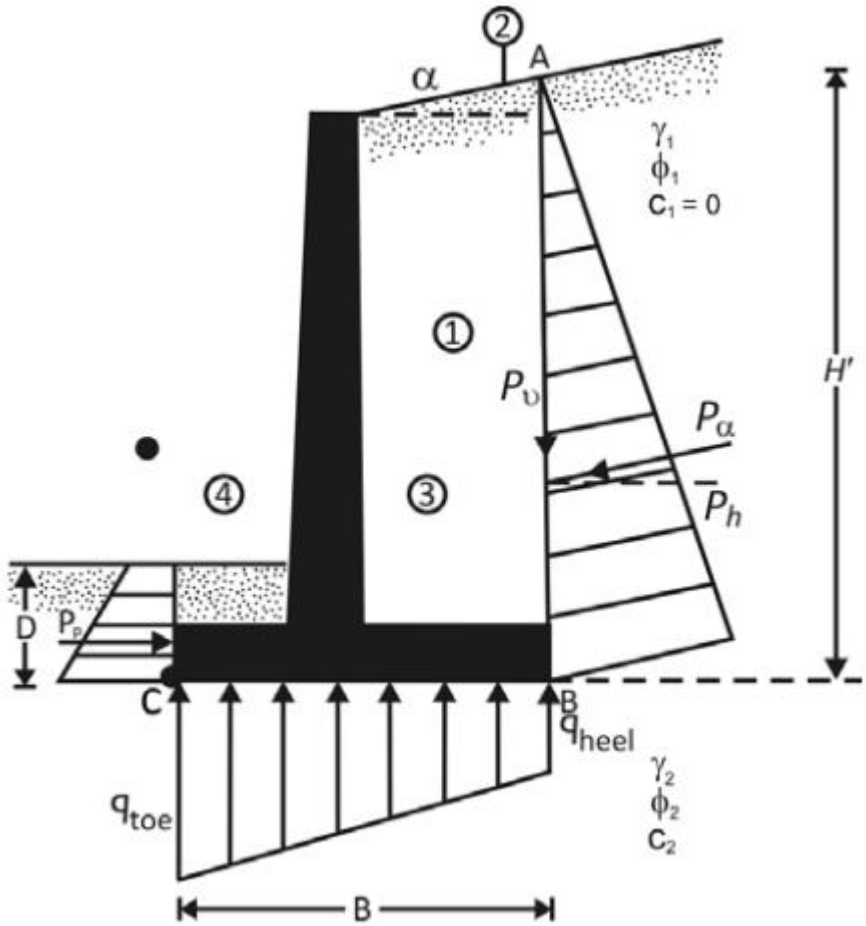
If we consider the line load then E′, E3….. will get shifted to E″, E_3'… with E′ E″ =E_3 E_3' = ….. q. There is an abrupt change in the Culmann line from F1 to F′ and BF1F2F′F″…..F_n represents Culmann line obtained considering the line load. If E″F″ is greater than EF, slip occurs along BC′ and the resultant active earth pressure is given by

$$P_A = w'\left(\frac{FE}{BE}\right) \ldots\ldots\ldots\ldots\ldots\ldots \text{(2)}$$

where ,

W′ = (weight of wedge ABC′) + q. On the other hand if E″F″ is less than EF, slip occurs along BC and Pa is given by Eqn (1).

STABILITY OF RETAINING WALLS



Stability against overturning

$$\text{Factor of safety} = \frac{\sum M_R}{\sum M_o}$$

$$\sum M_R = Restraining\ moment$$

$$\sum M_o = Overturning\ moment$$

$$\sum M_o = P_h\left(\frac{H'}{3}\right)$$

Calculation for overturning moment

Section	Area	Weight per unit length	Moment arm from C	Moment about C
1	A1	$\gamma_1 \times A1$	X1	$M = \gamma_1 \times A1 \times X1$
2	A2	$\gamma_2 \times A2$	X2	$M = \gamma_2 \times A2 \times X2$
3	A3	$\gamma_c \times A3$	X3	$M = \gamma_C \times A3 \times X3$
4	A4	$\gamma_c \times A4$	X4	$M = \gamma_C \times A4 \times X4$
5	A5	$\gamma_c \times A5$	X5	$M = \gamma_C \times A5 \times X5$
6	A6	$\gamma_c \times A6$	X6	$M = \gamma_C \times A6 \times X6$
		P_V	V	$M_V = R \times V$
		$\sum V=$		$\sum M_R=$

Stability against Sliding

Factor of safety $= \frac{\sum F_R}{\sum F_D}$

$$\sum F_R = Restraining\ Force$$

$$\sum F_R = \frac{\sum V \tan \propto + C_a + P_p}{P_a + \cos \propto}$$

$$\sum F_D = Driving\ Force$$

Stability against Bearing capacity

$$q_{max\ toe} = \frac{\sum V}{B}\left[1 + \frac{6e}{B}\right]$$

$$q_{min\ heal} = \frac{\sum V}{B}\left[1 - \frac{6e}{B}\right]$$

Factor of safety = $\frac{q_u}{q_{max}}$

PROBLEMS

Example 1:A retaining wall with a smooth vertical back is 10 m high and retains a two layer of sand backfill with the following properties:

0-5 m depth : Φ=30°, γ = 18 KN/m^3

Below 5 m : Φ=34°, γ = 20 KN/m^3

Show the active earth pressure distribution assuming the water table is well below the base of wall

Solution

When the backfill consists of more than one soil layer, the lateral earth pressure distribution for each of the layer is worked out and a combined diagram should be drawn. At the interface of two layers, there will be a break in the pressure distribution diagram, since there will be two value of pressure the upper layer will act as an overburden for the bottom layer.

K_{a1} for upper layer = $\frac{1-\sin\emptyset}{1+\sin\emptyset} = \frac{1-\sin 30}{1+\sin 30}$ = 0.333

K_{a2} for lower layer = $\frac{1-\sin\emptyset}{1+\sin\emptyset} = \frac{1-\sin 34}{1+\sin 34}$ = 0.283

Active pressure distribution for the upper layer:

Z= 0 m ; vertical pressure P_v=0 and P_A= 0

Z= 5 m ; vertical pressure P_v=18 x 5 =18 KN/m^2

And

$P_A = K_{a1} P_v$ = 0.333 x 90 = 30 KN/m^2

Active pressure distribution for the lower layer

Z = 5 m; Vertical pressure P_v = 90 KN/m^2

$P_A = K_{a2} P_v$ = 0.283 x 90 = 25.47 KN/m^2

Z = 10 m; Vertical pressure P_v = 90 + (20 x 5) = 190 KN/m^2

$P_A = K_{a2} P_v$ = 0.283 x 190 = 53.77 KN/m^2

Example 2: A retaining wall 8 m high, with a smooth vertical back, retains a clay backfill with c'= 15 KN/m^2, Φ'= 15°, γ = 18 KN/m^3. Calculate the total active thrust on the wall assuming that tension cracks may develop to the full theoretical depth.

Solution

The active pressure at a depth z in a c-Φ soil is given by

$$p_A = K_a p_v - 2c\sqrt{K_a}$$

$K_a = \frac{1-sin\emptyset}{1+\sin\emptyset}$ = 0.558

At

z =0, p_v=0 ;

p_A= -2 x 15$\sqrt{0.558}$ = -23 KN/m^2

At z = 8 m

p_v = 18 x 8 = 144 KN/m^2

p_A= 0.588 x 18 x 8 – 23 = 61.67 KN/m^2

p_A=0 and

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

$$z_0 = \frac{2 \times 15}{18\sqrt{0.588}}$$

$$z_0 = 2.17\ m$$

$P_A = \frac{1}{2} \times 61.67 \times 5.83 = 179.8\ {}^{KN}/_{m}$