ADDIS ABABA UNIVERSITY
FACULTY OF TECHNOLOGY
DEPARTMENT OF CONSTRUCTION TECHNOLOGY AND MNAGEMENT

COURSE TITLE: - <u>COTM 442 – FOUNDATIONS</u>

COURSE OUTLINE

References

1. Foundation Engineering
   By Alemayehu Teferra
2. Foundation Analysis and Design
   By J. E. Bowles
3. Soil Mechanics and Foundations
   By Samuel Tadesse
4. Foundation Design and Construction
   By M.T. Tomlinson
5. Foundation Design
   By W.C. Teng

1. SOIL EXPLORATION

1.1 PURPOSE OF EXPLORATION

The purpose of soil exploration is to find out strength characteristics of the sub-soil over which the structure has to be built. Soil characteristics vary both with respect to depth from the ground surface and stretch in the horizontal direction. It is, therefore, the prime objective of soil exploration for a building, bridge or other civil Engineering works, to analyze the nature of soil in all respects.

The main purposes of soil exploration are: -

a. Selection of alternative construction sites or the choice of the most economical sites.
b. Selection of alternative types or depth of foundation
c. Selection of alternative methods of construction.
d. Evaluation of the safety of existing structure.
e. Location and selection of construction materials.

The soil exploration should provide the following data: -

1. Soil parameters and properties of different layers (e.g. for classification, bearing capacity or settlement calculation)
2. Thickness of soil layers and depth to bedrock (stratification of soil)
3. Location of ground water level

1.2 PLANNING AN EXPLORATION PROGRAM

The planning of a program for soil exploration depends upon

i. The nature of sub-soil
ii. The type of structure
iii. The importance of structure

The soil engineer should constantly keep in mind, when planning the exploration program, the purpose of the program and the relative costs involved. Normally, the cost involved in the soil exploration is a function of the total cost of the project. It is always advisable to spend a little more on soil investigation to understand clearly the nature of the soil so that suitable foundation can be recommended. Often an indication of the extent of an exploration of program can be estimated from the history of foundations successes and failures in an area are very helpful. Also, for planning the program, the engineer should be well acquainted with the current methods of soil boring, sampling and testing and have some idea of the limitations on both the field and laboratory equipments and methods.

The actual planning of a subsurface exploration program includes some or all of the following steps: -

I. Assembly of all available information on type and use of the structure, and also of the general topographic and geological character of the site.
II. **Reconnaissance of the area: -** This involves inspection of behavior of adjacent structures, rock outcrops, cuts, etc.
III. **A preliminary site investigation: -** This is usually in the form of a few borings or a test pit to establish the types of materials, Stratification of the soil, and possibly the location of the ground water level. For small projects this step may be sufficient to establish foundation criteria, in which case the exploration program is finished.
IV. **A detailed site investigation: -** For complex projects or where the soil is of poor quality and/or erratic, a more detailed investigation may be undertaken this may involve sinking several boreholes, taking soil samples for laboratory investigations, conducting sounding and other field tests.

1.3 METHODS OF EXPLORATION

Methods of determining the stratification and engineering characteristics of sub-surface are

- Test pits
- Boring and sampling
- Field tests
- Geophysical methods
- Laboratory tests

1.3.1 Test Pits

The simplest and cheapest method of shallow soil exploration is to sink test pit to depths of 3 to 4 m. The use of Test pits enables the in-situ soil conditions to be examined visually, thus the boundaries between strata and the nature of any macro-fabric can be accurately determined. It is relatively easy to obtain disturbed or undisturbed soil samples: in cohesive soils block samples can be cut by hand from the bottom of the pit and tube samples can be obtained from the sides of the pit.

1.3.2 Soil Boring and Sampling

1.3.2.1 Soil Boring

This is the most widely used method. It provides samples from shallow to deeper depths for visual inspection as well as laboratory tests. The most commonly used methods of boring are: -

⇒ Auger boring

⇒ Wash boring

⇒ Percussion drilling

⇒ Rotary drilling

a) Auger boring: - Operated by hand or by power. Hand operated augers, ϕ= 15 to 20cm, are of two types. Post-hole and helical augers. They are used for shallow borings depth 3 to 7.5m in soils, which possess sufficient cohesion to sand unsupported. This boring method provides highly disturbed soil samples. Power operated augers (helical) can be used to great depths, even to 30m, and used in almost all types of soils above water table.

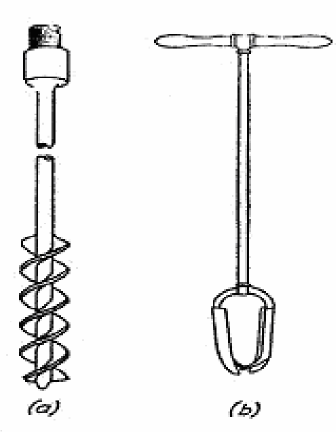


Fig.1.1 Hand Augers a) helical and b) post hole

b) Wash boring: - Power operated. Hole is advanced by chopping, twisting action of a light chopping bit and jetting action of drilling fluid, usually water, under pressure. Loosened soil particles raise as suspended particles through the annular space between casing and drill rod. This method best suits in sandy and clayey soils and not in very hard soil strata (i.e. boulders) and rocks. Depth of boring could be up to 60m or more. Changes in soil strata are indicated by changes in the rate of progress of boring, examination of out coming slurry and cutting in the slurry. Undisturbed samples whenever needed can be obtained by use of proper samplers.

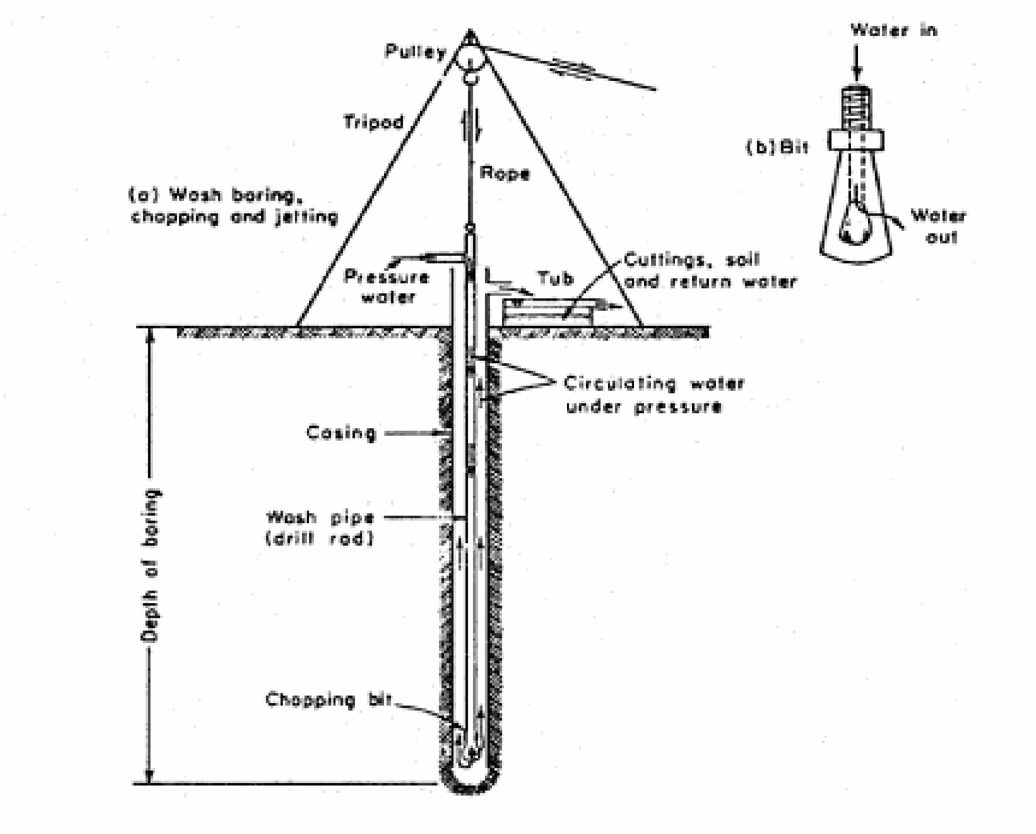


Fig.1.2 Wash boring

c) Percussion drilling: - Power operated. Hole is advanced by repeated blows of a heavy chisel into the bottom of the hole. The resulting slurry formed at bottom of borehole is removed by bailer or sand pump. Because of the deep disturbance of the soil this method of boring is not favored. Casing is generally required. Maximum depth of boring is 60m.

d) Rotary drilling: - Power operated. Hole is advanced by a rapidly rotating bit which cuts the material at the bottom of the hole into small particles which are removed by circulating fluids, which may be water, bentonite slurry or mud slurry. This is the most rapid method for penetrating highly resistant materials (e.g. bed rock). In this method undisturbed samples can be obtained at desired depths by using suitable samplers. Maximum depth of drilling is 80 to 150m.

1.3.2 Soil Sampling

There are two main types of soil samples which can be recovered from bore holes or trial pits. These are: - Disturbed and Undisturbed samples.

a) Disturbed Samples: - are samples where the structure of the natural soil has been disturbed to a considerable degree by the action of the boring tolls or excavation equipment. Disturbed samples, however, need to be truly representative of the stratum. Disturbed samples are satisfactory for performing classification tests such as, sieve analysis, Atterberg limits etc.

b) Undisturbed Samples: - are samples, which represent as closely as is practicable, the true in-situ structure and water content of the soil. Undisturbed samples are required for determining reliable information on the shearing resistance and stress-deformation characteristics of a deposit. Undisturbed samples in cohesionless deposits are extremely difficult to obtain. Because of this the above characteristics are provided by field tests.

Types of Samplers

It is virtually impossible to obtain totally undisturbed samples, especially from moderate to deep holes. The process of boring, driving the coring too, raising and withdrawing the coring tool and extruding the sample from the coring tool, all conspire to cause some disturbance. In addition, samples taken from holes may tend to swell as a result of stress relief. Samples should be taken only from a newly- drilled or newly extended hole, with care being taken to avoid contact with water. As soon as they are brought to the surface, core tubes should be labeled inside and outside, the ends sealed with wax and capped, and then stored away from extremes of heat or cold and vibration. Sample disturbance may be reduced by using an appropriate type of sample tube. The types of tube samplers in common use are described below:

a) Split Spoon Sampler: - A standard split spoon sampler has a 2" outside diameter, 1⅜" inside diameter tube, 18 to 24" long. The tube is split longitudinally in the middle. While the sample is being taken, the two halves of the spoon are held together at the ends by short pieces of threaded pipe, one of which couples, it to the drill rod and the other serves as the cutting edge. The sampler is forced or driven into the soil to obtain a sample and is then removed from the hole. With these sampler-disturbed samples of soft rock, cohesive and cohesionless soils are obtained. This sampler is used for making standard penetration test.

b) Thin-Walled Tube Sampler: - It is a thin walled seamless brass or steel tubing, with common out side diameter of 2 to 3" and length of 30 to 36". The lower end is beveled to form a cutting edge and it can be slightly tapered to reduce the wall friction and the upper end fitted for attachment to the drill rod. In order to take sample the sampler is pushed downward into the soil by static force instead of being driven by a hammer. This sampler is used to take undisturbed samples from cohesive soils.

c) Piston Samplers: - They are very thin tube samplers with pistons fitted at their cutting ends. While taking sample, the piston is held in positions and the tube pushed down. The piston aids the retention of the soil in the tube during withdrawal. Piston samples provide best-undisturbed samples of cohesive soils.

1.4 FIELD [IN-SITU] TESTS

These tests are valuable means of determining the relative densities; shear strengths and bearing capacities of soils directly without disturbing effects of boring and sampling.

The most commonly used field tests are; -

- Penetration or sounding tests
- Vane shear test
- Plate loading test
- Pile loading test

1.4.1 Penetration Tests

Penetration tests are the most useful tests. They are conducted mainly to get information on the relative density of soils with little or no cohesion. The tests are based on the fact that the relative density of a soil stratum is directly proportional to the resistance of the soil against the penetration of the drive point. From this, correlations between values of penetration resistance versus angle of internal friction (ϕ), bearing pressure, density and modulus of compressibility have been developed. Penetration tests are classified as: Static and dynamic penetration tests.

a) Static Penetration Tests.

1) Swedish Weight Sounding Test: -This method of testing is widely used in Scandinavia and here in Ethiopia. The test consists of weights: 5,10,10.25,25, and 25kgs($\sum$=100 kg), screw point, driving rod (ϕ 20 to 22 mm), made up of 100cm parts, and a rotating handle. The depth of penetration is measured for each loading after which the number of half-turns is counted by 100Kg.load; the penetration depth is then measured after 25 half-turns. If the penetration after 25 half-turns is less than 5cm the rod is unloaded and driven down by a 5 to 6kg hammer.

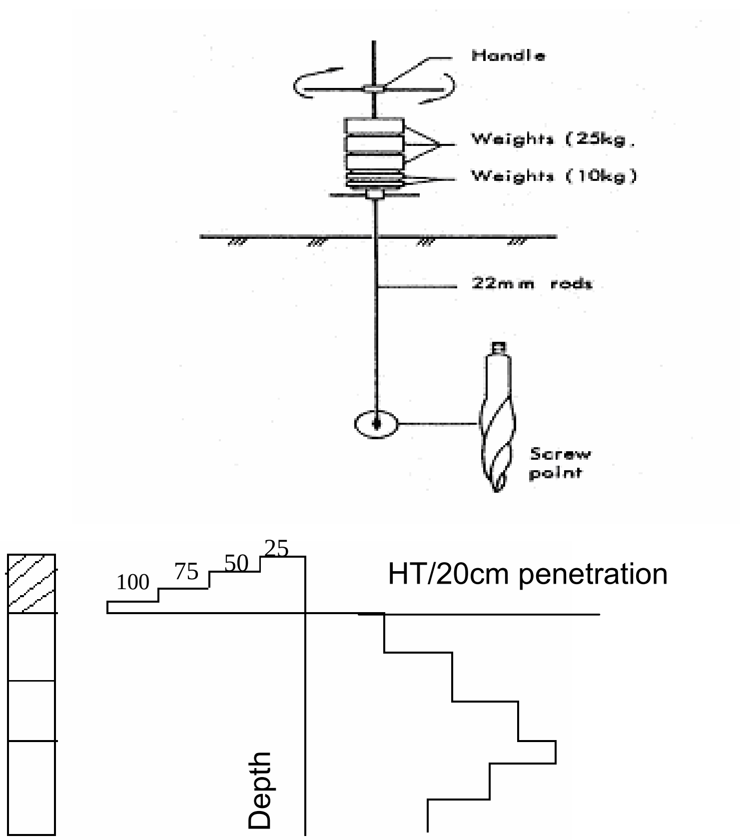


Fig. 1.3 Swedish weight sounding equipment, penetration diagram

The correlation between density of frictional soils and consistency of cohesive soils and ht/m (half-turns per meter) are as given below.

Frictional Soils		**Density (kN/m^3)**
Very loose	<50ht/m	11-16
Loose	50 -150ht/m	14.5 - 18.5
Medium	150 - 300ht/m	17.5 - 21
Dense	300 - 500ht/m	17.5 - 22.5
Very dense	> 500ht/m	21 - 24

Cohesive Soils		**Density (kN/m^3)**
Soft	0 ht/m	16 –19
Firm	0 – 100 ht/m	17.5 – 21
Stiff	100-200 ht/m	19 – 22.5
Very stiff	200 - 500 ht/m	
Hard	>500 ht/m	

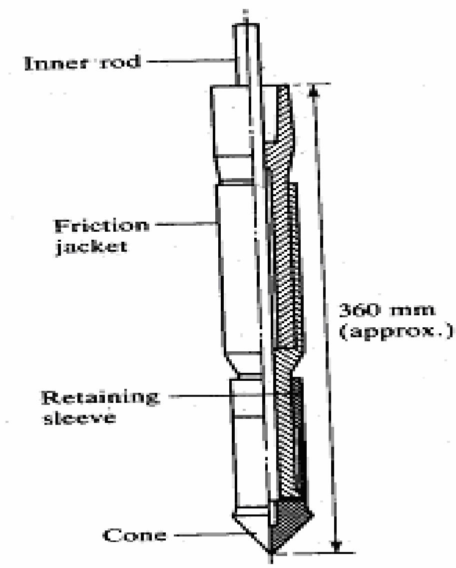
2) Static Cone Penetration Test (Dutch Cone Penetrometer Test): -This method is widely used in Europe. The test consists of a cone (apex angle 60^0, overall diameter 35.7mm, end area $10cm^2$, rods ($5/8''$ ϕ), casing pipe (ϕ $3/4''$). The rod is pushed hydraulically into the ground at a rate of 10mm/sec. The pressure exerted on the rod is measured with a proving ring, manometer or a strain gauge. Readings are usually taken every 20cm. From this test point resistance and skin frictional resistance can be determined separately.

- The cone is 1^{st} pushed into the ground. The force required to push the cone 20cm into the soil is recorded.
- The casing pipe is then advanced to join the cone. The force required to push the pipe is also recorded.
- The readings thus taken are plotted against depth.

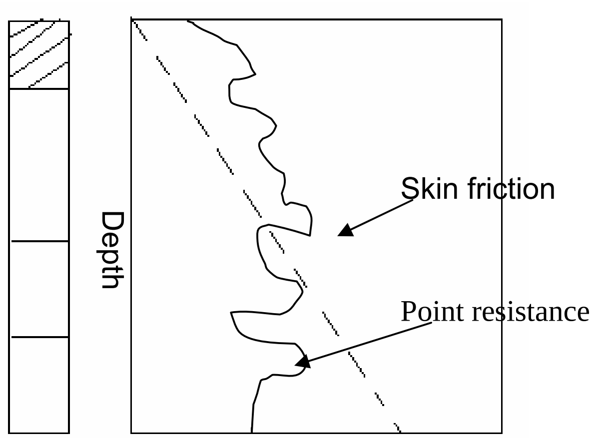
The correlation between the cone (point) resistance and relative density of frictional soils are given in Table 1.1

Table 1.1 Correlations between Cone (Point) Resistance and Relative Density of Frictional Soils

Relative Density	**Point Resistance (kN/m^2)**
Very loose soil	< 2500
Loose soil	2500 – 5000
Medium dense	5000 – 10,000
Dense	10,000 – 15,000
Very dense	> 15,000



Cone resistance (point resistance) in kN/m^2



Casing (skin) resistance in kN/m^2

Fig. 1.4 Static cone penetration testing equipment, penetration diagram

- According to Meyerhof:

$$N = \tfrac{1}{4}\,(C_{kd}) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1.1)$$

where N = Standard penetration number

C_{kd} = Static Cone resistance (kg/cm^2)

For sand, modulus of compressibility (E_s) can be estimated from cone resistance from the following relationship.

$$E_S = 3/2(\,C_{kd}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1.2)$$

b) Dynamic Penetration Tests

1) Standard Penetration Test (SPT): -This is the most common of the field tests and measures the resistance of the soil to dynamic penetration by a 50mm diameter split spoon sampler which is driven into the soil at the bottom of a borehole (sometimes cased). The sampler is attached to drill rods and the dynamic driving force is a 63.5kg mass falling through a height of 76cm onto the top of the rods as shown in Fig.11.5. The sampler is initially driven 15cm below the bottom of the borehole. It is then further driven 30cm. The number of blows required to drive the last 30cm is termed as the standard penetration value denoted by N. The standard penetration number has been correlated to soil characteristics such as: density, angle of shearing resistance, ϕ, unconfined compressive strength, as given in Tables 1.2 and 1.3.

Table 1.2 Correlation between Number of blows (N), Angle of Internal Friction and Relative Density of Frictional Soils(Terzaghi and Peck).

N	0 - 4	4 -10	10-30	30 - 50	> 50
ϕ	<28^0	28 -30^0	30-36^0	35 - 40^0	>42^0
Relative Density	Very loose	Loose	Medium	Dense	Very dense

Table 1.3 Correlation between Number of blows (N), Unconfined Compressive Strength and Consistency of Cohesive Soils. (Terzaghi and Peck).

N	0 -2	2 - 4	4 - 8	8 -15	15-30	>30
q_u(kN/m^2)	0 -25	25 -50	50 -100	100 -200	200-400	>400
Consistency	Very soft	Soft	Medium	Stiff	Very stiff	Hard

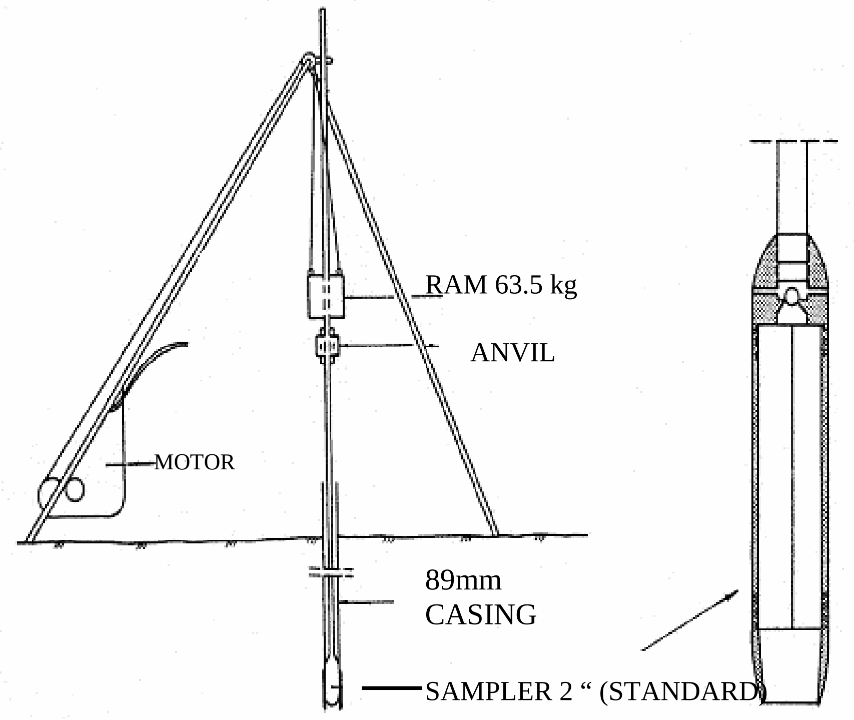


Fig. 1.5 Standard penetration test (SPT) equipment.

The relationship between ϕ and D_r may be expressed approximately by the following equation (Meyerhof).

$$\phi^0=30+0.15D_r \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1.3)$$

For granular soil, containing more than 5 percent fine sand and silt.

$$\phi^0=30+0.15D_r \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1.4)$$

For granular soil, containing less than 5 percent fine sand and silt. In the equations D_r is expressed in percent.

Correction to be applied to measured values of SPT

The N. values of SPT as measured in the field may need to be corrected.

i. When SPT is made in fine saturated sands, saturated silty sands, or saturated silts, correction is usually made for possible build up of pore water pressure. The SPT values, greater than 15 are modified as follows

$$N = 15 + \tfrac{1}{2}\ (N' - 15)$$ Suggested by Terzaghi and peck

where N= corrected value

N'= actual value

ii. The other type of correction is known as correction for overburden pressure. This correction is applied only to cohesionless soils (dry, moist or wet). The correction suggested by Gibbs and Holtz and widely used is as follows.

$$N = \frac{345' N}{(\sigma_o'+69)} \leq 2N', \text{ for } \sigma_o' \leq 276 \text{ kN/m}^2$$

σ_o' = effective overburden pressure in kN/m^2 $N = \dfrac{35N'}{(\sigma_o'+7)} \leq 2N'$, for $\sigma_o' \leq 28\text{kN/m}^2$

2) Dynamic Cone Penetration Test: - This is another useful test, which is normally used to determine the relative resistance offered by the different soil layers. The cone is fixed to the bottom of a rod by pushed fit. The cone is driven into the ground in the same way as a SPT is performed. The number of blows required to penetrate 30 cms depth is called as Nc value. In the case of dynamic cone penetration test no borehole is used. Experiments carried out indicate that beyond about 6m depth, frictional resistance on the rod increases which gives erroneous results for N_c value. The maximum depth suggested for this test is about 6 m. If the test has to be conducted beyond 6 m depth, one has to use drilling mud (bentonite slurry) under pressure forced through the pipe and the cone as shown in Fig 11.6. The mud solution coming out of the cone rises above along the drill rod eliminating thereby the frictional resistance offered by the soil for penetration. The former method is called as dry method and the latter wet method.

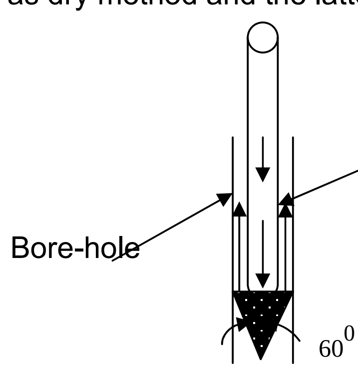


Fig. 1.6 Dynamic cone penetration testing equipment.

To judge the consistency of soil from N_c values, the general practice is to convert N_c to N values of SPT

Nc = N/C .... (1.5)

where

N = blow count for SPT

Nc = blow count for dynamic cone

C = Constant, lies between 0.8 and 1.2 when bentonite is used.

Nc= 1.5N for depths up to 3m

Nc= 1.75N for depths between 3m and 6m

Nc Values need to be corrected for overburden pressure in cohesionless soils like SPT

1.4.2 Vane Shear Test

This test is useful in determining the in-place shear strength of very soft and sensitive clays, which lose a large part of their strength when even slightly disturbed by the sampling operation. The strength parameter obtained is consolidated- undrained shear strength, Cu.

In most cases a hole is drilled to the desired depth, where the vane shear test is planned to be performed and the vane is carefully pushed into the soil. A torque necessary to shear the cylinder of soil defined by the blades of the vane is applied by rotating the arm of the apparatus with a constant speed of 0.5 degree/sec. The maximum torque is then measured from which the shearing strength is determined.

From the measured maximum torque one may estimate the shearing resistance of the tested clay from the following formula

$$\tau = \frac{T}{\pi\left[D^2\frac{H}{2} + \frac{D^3}{12}\right]} \qquad (1.6)$$

where T = Torque

D = Diameter of Vane

H = Height

Since for quick condition $\tau = C_u$, one ultimately arrived the in-situ value of cohesion

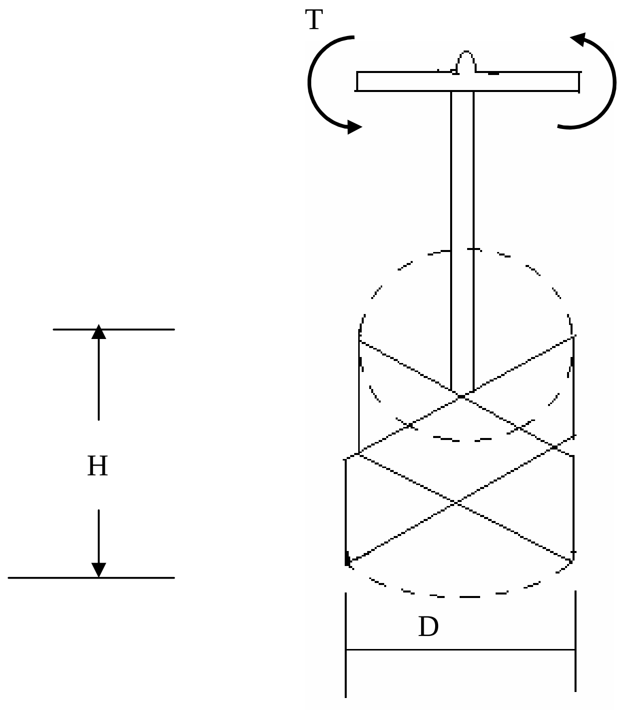


Fig.1.7 Vane shear test

1.4.3 Plate Loading Test

In this test a gradually increasing static load is applied to the soil through a steel plate, and readings of the settlement and applied load are recorded, from which a relationship between bearing pressure and settlement for the soil can be obtained.

Fig. 1.8 shows the arrangement and typical load settlement curve for a plate load test. The test procedure used for performing the test is as follows:

1. Pit for the test must be at least 5 times the size of the plate.
2. The plate should be properly placed in the soil. In the case of cohesionless soil (to prevent early displacement of soil under the edges of the plate), the plate must be positioned in cast in-situ concrete.
3. Loading platform should be properly erected.
4. Loading of the soil is conducted in steps (loading increment is kept constant).
5. Once completion of the test, the plate is unloaded in the same incremental steps (to draw the expansion curve).

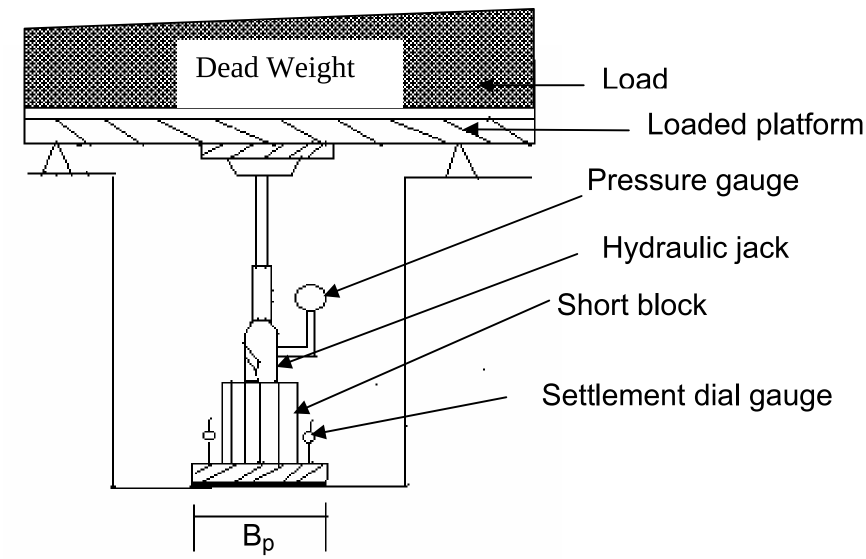
Bearing capacity of non-cohesive soil is determined from settlement consideration. If the maximum permissible settlement, S, of a footing of width B_f is given, the settlement, S_p, of a plate of width B_p under the same intensity of loading is given by

$$S = \frac{Sp\,(2Bf)^2}{(B_f + B_p)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \tag{1.7}$$

Using the value S_p, computed from the above equation, the loading intensity under the footing could be read from the load settlement curve.

The settlement of footing in clay is normally determined from principles of consolidation. However from plate load test, the approximate settlement of footing of width B can be determined using the following expression

$$S = Sp\,\frac{B_t}{B_p} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \tag{1.8}$$



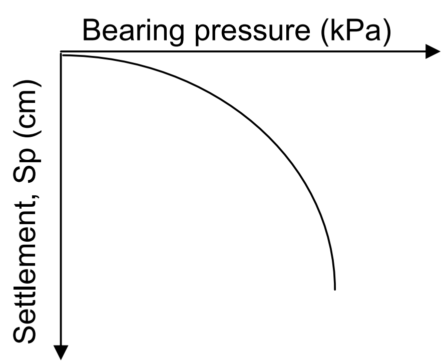


Fig. 1.8 Plate loading test, test result

Limitation of Plate Loading Test

- Plate loading test is of short duration. Hence consolidation settlement does not fully occur during the test.
- For settlement consideration, its use is restricted to sandy soils, and to partially saturated or rather unsaturated clayey soils.
- Plate loading test can give very misleading information of the soil is not homogeneous within the effective depth (depth of stress influence) of the prototype foundation.
- Plate loading test should not recommended in soils which are not homogeneous at least to depth of 1½ to 2 times the width of the prototype foundation

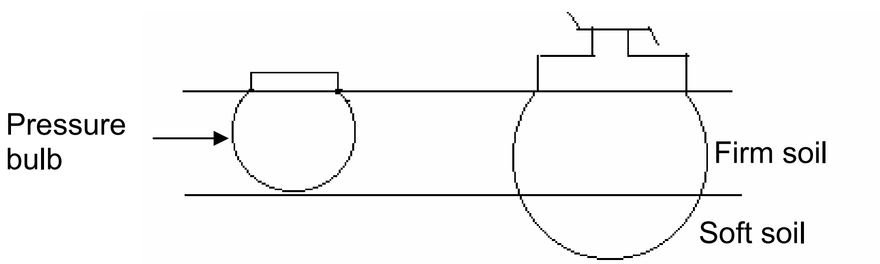


Fig. 1.9 Pressure bulbs for the plate and the actual foundation.

1.4.4 Pile Loading Test

This is the most reliable means for determining the load carrying capacity of a pile. The load arrangement and testing procedure are more or less similar to the plate-loading test. From the results of this test the allowable bearing capacity and load- settlement relationship of a group of friction piles can be estimated.

1.5 GEOPHYSICAL METHODS

These comprise the seismic and resistivity methods. These methods are usually limited to establishing location of bedrock underlying softer material (by seismic method) or locating gravel or sand deposits (by resistivity method). The seismic method is based on the fact that sound waves travel faster through rocks than through soils. The resistivity method makes use of the fact some soils (e.g. soft clays) have low electrical resistivity than others (e.g. sand or gravel). These methods are normally employed as preliminary or supplementary to other methods of exploration.

1.6 LABORATORY TESTS

Laboratory tests are useful in providing reliable data for calculating ultimate bearing capacity of soils, stability and settlement behavior of foundation, and for determining physical characteristics of soils. Results of laboratory tests should be used in conjunction with borehole records and results of field test.

The common laboratory tests that concern the foundation engineers are

- Grain size analysis
- Atterberg limits
- Natural moisture content
- Unit weight
- Unconfined compression test
- Direct shear test
- Triaxial compression test
- Consolidation test
- Compaction test
- Chemical analysis

1.7 GROUND WATER MEASUREMENT

Ground water affects many elements of foundation design and construction. Because of this its location should be determined in each job with reasonable accuracy.

Water table level can be determined by measuring the depth to the water surface in a borehole. Water levels in bore holes may take a considerable time to stabilize, this time, known as the response time, depending on the permeability of the soil. Measurements, therefore, should be taken at regular intervals until the water level becomes constant.

The depth of water table is measured by lowering a chalk-coated steel tape in the borehole. The depth can also be measured by lowering the leads of an electrical circuit. As soon as the open ends of the leads touch the water in the borehole, the circuit is completed. It is indicated by glow of the indicator lamp.

1.8 DEPTH AND NUMBER OF BORINGS.

1.8.1 Depth of Boring

The depth to which boreholes should be sunk is governed by the depth of soil affected by foundation bearing pressures. According to Tomlinson the following depths of boreholes for various foundation conditions may be used.

i. For widely spaced strip of pad foundations, boring depth should be deeper than 1.5 times the width of the foundation.
ii. For raft foundations, boring depth deeper than 1.5 times width of raft should be used.
iii. For closely spaced strip or pad foundations where there is overlapping of the zones of pressure, boring depth deeper than 1.5 times width of building should be used.
iv. For group of piled foundation on soil, boring depth should be deeper than 1.5 times width of pile group, the depth being measured from a depth of two- thirds of the length of the piles.
v. For piled foundation on rock, boring depth should be deeper than 3.0m inside bedrock.

According to Teng, for high ways and airfields minimum depth of boring is 1.5m, but should be extended below organic soil, fill or compressible layers such as soft clays and silts.

1.8.2 Number of Borings

Boring is an expensive undertaking. One should therefore minimize the number of borings for a construction in a given site. From experience Teng has suggested the following guideline for preliminary exploration.

Project	Distance between boring (m)			Minimum number of boring for each structure
	Horizontal stratification of soil			
	Uniform	Average	Erratic	
Multi-story building	45	30	15	4
One or two story building	60	30	15	3
Bridge piers, abutments, television towers, etc	-	30	75	1-2 for each foundation unit
Highways	300	150	30	

1.9 DATA PRESENTATION

The results of borings, samplings, penetration tests and laboratory tests of a site are usually plotted graphically on a sheet of drawing paper. The graphical presentation should include.

a. A plot plan, showing the location of all boreholes, test pits, etc and their identification number.
b. A separate plot, showing the soil profile as established from the drillings or test pits records.
c. Soil profiles along given lines in the ground surface, showing the boundaries between identifiable soil layers, variation of thickness of firm bottom layer, thickness of soft clay layers etc.
d. The penetration number, the unconfined compression strength, Atterberg limits, natural moisture content, and other appropriate laboratory data may be shown on each boring on the soil profile.
e. The location of ground water table should also be shown on the soil profile.

1.10 SOIL EXPLORATION REPORT

A soil exploration report should contain all available data from bore holes, test pits, field and laboratory tests and site observation. Most reports have the following contents.

1. Introduction: - Purpose of investigation, type of investigation carried out.
2. General description of the site: - general configuration and surface features of the site.
3. General geology of the area.
4. Description of soil conditions found in bore holes (and test pits)
5. Laboratory test results.
6. Discussion of results of investigation in relation to foundation design and constructions.
7. Conclusion: - recommendations on the type and depth of foundations, allowable bearing pressure and methods of construction.

2. BEARING CAPACITY OF SHALLOW FOUNDATIONS

2.1 INTRODUCTION

The lowest part of a structure is generally called a foundation and its function is to transfer the load of the structure to the soil on which it is resting. If the soil near the surface is capable of adequately supporting the structural loads it is possible to use either a footing or a raft. A footing is a relatively small slab giving separate support to part of the structure. A footing supporting a single column is referred to as an individual (isolating) footing, one supporting a group of columns as a combined footing and one supporting a load-bearing wall as strip footing. A raft is a relatively large single slab, usually stiffened, supporting the structure as a whole. If the soil near the surface is incapable of adequately supporting the structural loads, piles or piers are used to transmit the loads to suitable soil at greater depth.

A foundation must satisfy two fundamental requirements: -

I. The factor of safety against shear failure of the supporting soil must be adequate,
II. The settlement of the foundation should be tolerable and, in particular, differential settlement should not cause any unacceptable damage nor interfere with the function of the structure.

Foundations may be broadly classified into two categories: i) Shallow foundations, ii) Deep foundations. The distinction between a shallow foundation and a deep foundation is generally made according to Terzaghi's criterion. According to which, a foundation is termed as shallow if it is laid at a depth (the depth of foundation, D, is the vertical distance between the base of the foundation and the ground surface) equal to or less than its width. The bearing capacity of shallow foundation will be presented here.

2.2 BASIC DEFINITIONS

1. Ultimate Bearing Capacity: - The ultimate bearing capacity is defined as the least pressure, which would cause shear failure of the supporting soil immediately below and adjacent to a foundation.

2. Net Ultimate Bearing Capacity: - It is the net increase in pressure at the base of foundation that causes shear failure of the soil. It is equal to the gross pressure minus overburden pressure.

Thus $q_{nult} = q_{ult} - \gamma D$ ……………………… (2.1)

where q_{ult} = ultimate bearing capacity (gross)

γ = effective unit weight of soil above foundation base, and

D = depth of foundation

It may be noted that the overburden pressure equal to γD existed even before the construction of foundation.

3. Allowable Bearing Capacity: - The allowable bearing capacity or safe bearing capacity, of a soil is defined as the maximum pressure which provides an adequate safety factory against soil rupture and also insures that settlement due to static loading will not exceed the tolerable value

4. Net Allowable Bearing Capacity: - It is the net allowable bearing pressure that can be used for the design of foundation. It is equal to the allowable bearing capacity minus overburden pressure.

2.3 FAILURE ZONES BENEATH A SHALLOW CONTINUOUS FOOTING.

Let us consider a footing placed on the surface of the ground. If the base of the footing is perfectly smooth, theoretical investigations have shown that if overloaded, the soil below the footing fails as shown in Fig.9.1. The failure wedge may be divided into five zones. Zone I represents active Rankine zone and the two zones III on either side represent passive Rankine zones. The inclination of the active Rankine zone with the principal plane (horizontal) is $\left(45+\frac{\phi}{2}\right)$ while that of passive Rankine zone is $\left(45-\frac{\phi}{2}\right)$. The two zones II located between the zones I and III are zones of radial shear. One set of lines of the shear patterns in these zones radiates from the outer edges of the base of the footing. The curved surfaces of sliding de and de_1 in Fig.2.1 are logarithmic spirals. The Dotted lines on the right hand side of the central line indicate the boundaries of the zones I and III at the instant of failure and the solid lines represent these boundaries when the foundation sinks into the soil. The soil in zone I spreads in the horizontal direction, while the soil in zone III is compressed laterally.

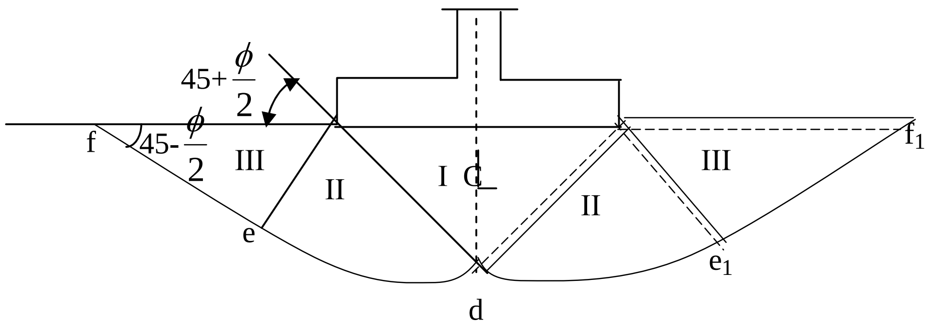


Fig. 2.1 Failure zones below a footing with smooth base

If the base of the footing is rough, which is the usual case in practice, the failure zones are as sketched in Fig. 2.2. Because of the friction and adhesion between base of the footing and the soil, the soil in zone I cannot expand laterally and essentially remains in elastic state and acts as a part of the footing. The inclination of this wedge with the horizontal is equal to the angle of internal friction of the soil. Zone II and III are similar to the corresponding zones in case of a footing with smooth base.

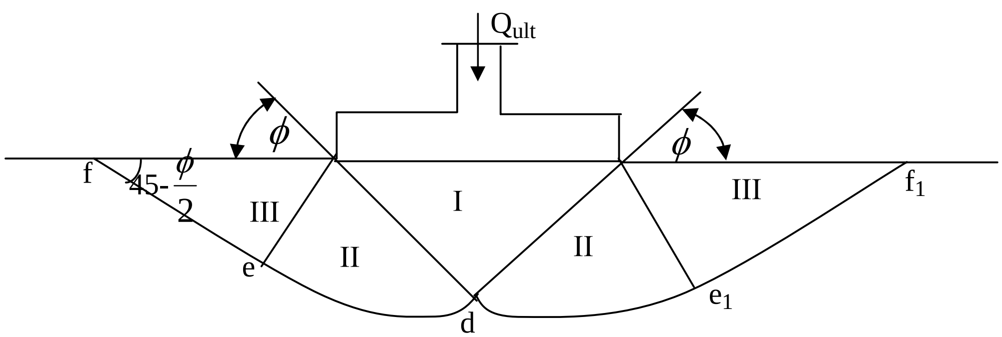


Fig. 2.2 Failure zones below a footing with rough base.

For a cohesive soil, ϕ=0, the logarithmic spiral becomes a circle and zone I in Fig. 9.2 vanishes. Model tests indicate that in dense sands, well-defined failure zones occur when the footing is subjected to gradually increasing loads.

2.4 BEARING CAPACITY EQUATIONS.

2.4.1 Terzaghi Bearing Capacity Equation

The following assumptions were made by Terzaghi (1943) to develop an ultimate bearing capacity equation for soils under a strip footing;

a. The base of the footing is rough
b. The footing is laid at a shallow depth, i.e. $D \leq B$
c. The shear strength of the soil above the base of footing is neglected. The soil above the base is replaced by a uniform surcharge, γD.
d. The load on the footing is vertical and is uniformly distributed.
e. The footing is long, i.e. L/B ratio is infinite, where B is the width and L is the length of the footing.
f. The shear strength of the soil is governed by the Mohr-Coulomb equation.

The failure surface assumed by Terzaghi for the determination of ultimate bearing capacity is shown in Fig. 2.3. As the footing sinks into the ground, the faces ac and bc of the wedge abc push the soil to the sides. When the soil mass below the footing is in a state of plastic equilibrium, the analysis of forces acting on the wedge abc gives the ultimate bearing capacity. The forces acting on the faces of this wedge at the instant of failure are shown in the same figure.

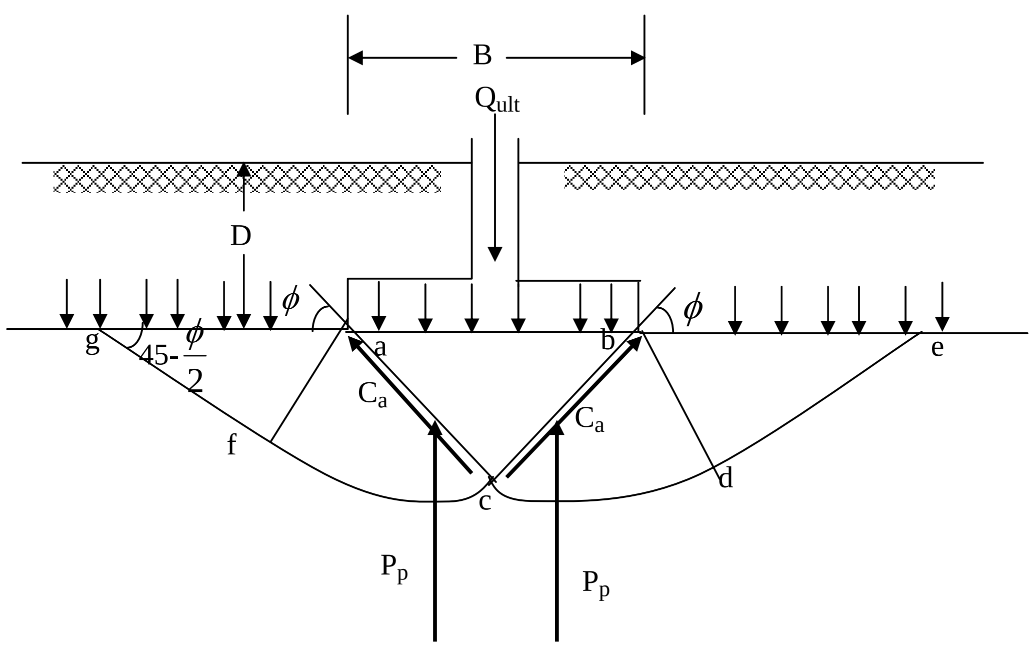


Fig 2.3 Failure surfaces as assumed by Terzaghi

The equilibrium of the mass of soil located within the zone I of elastic equilibrium requires that the sum of all vertical forces should be equal to zero.

That is

$$Q_{ult} - 2\,P_p - 2\,C_a \sin\phi = 0$$

$$Q_{ult} = 2\,P_p + 2\,C_a \sin\phi$$

$$ac = bc = \frac{B}{2\cos\phi}$$

$$C_a = \frac{C}{2}\frac{B}{\cos\phi}$$

where C_a = Cohesive force

C = Unit cohesion

$$Q_{ult} = 2\,P_p + 2\frac{C}{2}\frac{B}{\cos\phi}\sin\phi$$

$$= 2\,Pp + B.C.\tan\phi$$

In the above equation total passive earth pressure Pp is composed of the following: -

i. P'_p, passive earth pressure because of weights of the wedges acfg and bcde. These are computed by considering the stability of the wedges and assuming C=0, D = 0, P'_p acts at one-third distance from c along ac and bc.

ii. P_c, passive earth pressure because of cohesive force only. This is computed on the assumption that γ = 0, D = 0, It acts at middle of ac and bc.

iii. P_q, passive earth pressure because of surcharge; it is computed by considering γ = 0, c=0 and also acts at middle of ac and bc.

Therefore, the above equation becomes,

$$Qult = 2\,(P'_p + P_c + P_q + \tfrac{1}{2}\,Bc\tan\phi)$$

$$= B\left[\frac{2P'_P}{B} + \frac{2P_C}{B} + \frac{2P_q}{B} + C\tan\phi\right]$$

Let
$$\frac{2P_C}{B} + C\tan\phi = N_C.C$$

$$\frac{2P'_P}{B} = N_\gamma \cdot \frac{\gamma B}{2}$$

$$\frac{2P_Q}{B} = N_q \cdot \gamma D$$

Then

$$Q_{ult} = B\,(CN_c + \tfrac{1}{2}\gamma BN_\gamma + \gamma D\,N_q)$$

$$q_{ult} = \frac{Q_{ult}}{A}$$

For strip footing, A = Bx1

$$q_{ult} = CN_c + \tfrac{1}{2}\gamma BN_\gamma + \gamma D\,N_q \quad \ldots\ldots\ldots\ldots\ldots \quad (2.2)$$

Where q_{ult} is ultimate bearing capacity per unit area.

Terzaghi equations establish ultimate bearing value as a function of resistance due to three factors, namely, cohesion, internal friction, and surcharge effect.

The general expression for ultimate bearing capacity is given by,

$$q_{ult} = \underbrace{K_1 N_c C}_{\text{Cohesion}} + \underbrace{K_2 \gamma_1 N_\gamma B}_{\text{Friction}} + \underbrace{N_q \gamma_2 D}_{\text{Surcharge}} \quad \ldots\ldots\ldots\ldots \quad (2.3)$$

Where K_1 & K_2 = Coefficient dependent on the type of footing

For continuous footing, $K_1 = 1.0$ and $K_2 = 0.5$

For square footing, $K_1 = 1.30$ and $K_2 = 0.4$

For round footing, $K_1 = 1.30$ and $K_2 = 0.6$, B = R

N_c, N_γ, N_q = Non- dimensional bearing capacity factors and they are depend only on the angle of internal friction of a soil, see Table 2.1

C = Unit Cohesion

γ_1 = Effective unit weight of soil below footing grade

γ_2= Effective unit weight of soil above footing grade

B = Breadth of footing

D = Depth of footing

Table 2.1 Terzaghi's Bearing Capacity Factors

ϕ^0	N_C	N_γ	N_q
0	5.7	0.0	1.0
5	7.3	1.0	1.6
10	9.6	1.2	2.7
15	12.9	2.5	4.4
20	17.7	5.0	7.4
25	25.1	9.7	12.5
30	37.2	19.7	22.5
34	52.6	35.0	36.5
35	57.8	42.4	41.4
40	95.7	100.4	81.3
45	172.3	297.5	173.3
50	347.5	1153.0	415.1

2.4.2 HANSEN'S ULTIMATE BEARING CAPACITY EQUATION

Hansen (1961) has proposed a general equation for determining ultimate bearing capacity of soil as follows

$$q_{ult} = CN_c\, S_c d_c i_c + q\, N_q\, S_q d_q i_q + \tfrac{1}{2} B \gamma N_\gamma S_\gamma d_\gamma i_\gamma \quad \text{………. ……} \qquad (2.4)$$

Where

q_{ult} = Ultimate bearing capacity of footing

C = Cohesion

B = Width of footing

q = Effective surcharge at the base level of the footing.

γ = effective unit weight of soil

N_c, N_q, N_γ = Bearing capacity factor

S_c, S_q, S_γ = Shape factors

d_c, d_q, d_γ = Depth factors

i_c, i_q, i_γ, = Inclination factors

Hansen's Bearing capacity factors

$N_q = \tan^2(45+\phi/2)\, e^{\pi \tan\phi}$

$N_c = (N_q - 1) \operatorname{Cot} \phi$

$N_\gamma = 1.8\ ((N_q - 1) \tan \phi$

Hansen's Shape Factors

Factors	Types of foundation			
Shape factor	Continuous (strip)	Rectangular	Square	Circular
S_c	1.0	1+0.2B/L	1.3	1.3
S_q	1.0	1+0.2B/L	1.2	1.2
S_γ	1.0	1-0.4B/L	0.8	0.6

Hansen's Depth Factors

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

$d_q = d_\gamma = 1$ for $\phi \leq 10^0$

$d_q = d_\gamma = 1+0.1(D_f / B) \tan(45+\phi/2)$ for $\phi > 10^0$

Where ϕ = Angle of shearing resistance of soil in degree

Hansen's Inclination Factors

$i_c = i_q = (1-\alpha / 90^0)^2$

$i_\gamma = (1-\alpha / \phi)^2$

Where α = Inclination of the load to the vertical in degree

2.4.3 Effect of Water Table on Bearing Capacity

The unit weight of soil gets reduced when submerged. The unit weight γ, used in the bearing capacity equations, should be the effective unit weight. The effect of submergence on the bearing capacity may be explained with reference to the equation.

$$q_{ult} = CN_c + \tfrac{1}{2}\gamma BN_\gamma + \gamma D\, N_q$$

The first term CN_c in the above equation is not affected by the position of the water table. However, the shear parameter C used in the term should be found out in the laboratory for the soil under saturated conditions since this state gives the minimum value for C. The second term $\frac{1}{2}\gamma BN_\gamma$ is not affected if the water table level is at a depth D_w equal to or greater than B, the width of the footing, from the base of the footing as shown in Fig. 2.4 (a)

Since the depth of the shear failure zone below the base of the footing is assumed to be about equal to the width of the footing the unit weight γ in the term ½ γBN_γ will not be affected if $D_w \geq B$. When the water table level is at the base of the footing, i.e., when $D_w=0$, the submerged unit weight γ_b should be used in the term. For all practical purposes the submerged unit weight, γ_b of the soil may be taken as equal, to half its saturated unit weight, γ_{sat}. In such a case the term ½ γBN_γ gets reduced by 50 percent when the water table is at the base of the footing. When $0<D_w<B$ the soil below the base is partly submerged and partly moist.In such a case, a linear interpolation for the reduction in the bearing capacity is proposed as follows. The bearing capacity term for the soil below the base of the footing may be written as

$$\tfrac{1}{2}\gamma BN_\gamma R_w \text{ ……………………..} \qquad (2.5)$$

Where R_w is designated as the correction factor. The unit weight γ is taken as the saturated weight. We may write

$$R_w = 1, \text{ when } \frac{D_w}{B} \geq 1$$

$$R_w = 0.5, \text{ when } \frac{D_w}{B} = 0$$

When $0 < D_w / B < 1$ the equation for R_w may be written as

$$R_w = \tfrac{1}{2}\left(1 + \frac{D_w}{B}\right)$$

The variation of R_w with the ratio D_w/B is shown in Table 2.2.

When the water table is at the base or below the base of the footing, the third term γDN_q is not affected. When the water table is at ground level, the term γDN_q gets reduced by 50 percent in the same way. For any position of water table at or above the base of the foundation, we may write the bearing capacity expression as $\gamma DN_q\ R'_w$. The equation for R'_w may be written as

$$R'_w = \tfrac{1}{2}\left(1 + \frac{D'_w}{B}\right) \text{ ………………………………} \qquad (2.6)$$

where, R'_w = The correction factor for the surcharge soil. The maximum value is one.

D'_w = Depth of water table below the ground level limited to the depth equal to D.

The variation of R'_w with $\frac{D_w'}{D}$ is shown in Table 2.2

The bearing capacity equation, which takes the water table effect, may therefore be written as,

$$q_{ult} = CN_c + \tfrac{1}{2}\gamma BN_\gamma R_w + \gamma DN_q R'_w \ldots\ldots\ldots\ldots \quad (2.7)$$

$$q_{ult} = CN_c S_c d_c i_c + q N_q S_q d_q i_q + \tfrac{1}{2} B\gamma N_\gamma S_\gamma d_\gamma i_\gamma R_W \ldots\ldots\ldots \quad (2.8)$$

The factor R'_w in Eqn (2.6) is indirectly accounted for by taking q as the effective surcharge.

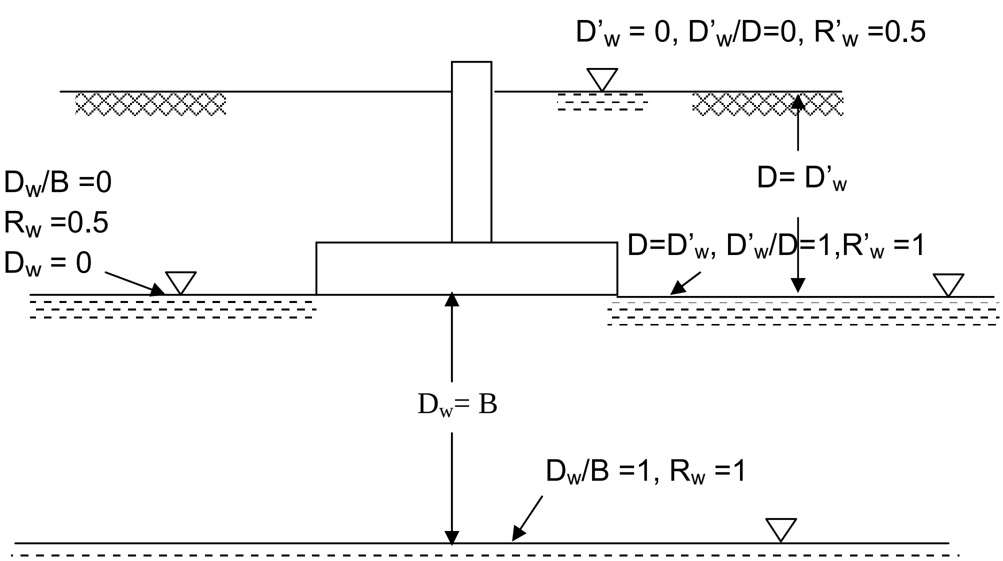


Fig. 2.4. Effect of water table on bearing capacity.

Table 2.2 Water Table Correction Factors for the Second and Third Terms of Bearing Capacity Equation.

D_W/B	R_W	D'_W/D	R'_W
0	0.5	0	0.5
0.2	0.6	0.2	0.6
0.4	0.7	0.4	0.7
0.6	0.8	0.6	0.8
0.8	0.9	0.8	0.9
1.0	1.0	1.0	1.0

2.4.4 Factors on Which the Ultimate Bearing Capacity of a Soil Depends

Factors, which have influence on the ultimate bearing capacity of a soil, will be discussed in the light of weather the soil is cohesionless or cohesive.

a) Cohesionless Soil

If the soil on which the footing is to rest is cohesionless, then the cohesion C is zero. Substituting this in Eqn. 2.3, the ultimate bearing capacity equation for cohesionless soils reduce to

$$q_{ult} = k_2 \gamma_1 N_\gamma B + N_q \gamma_2 D \quad \ldots\ldots\ldots\ldots\ldots \qquad (2.9)$$

From the above equation it can be seen that ultimate bearing capacity of sands depends upon: -

i. **Unit weight: -** As it was explained in the previous section, if the water table is close to the ground surface, the effective unit weight is reduced and hence the ultimate bearing capacity is affected.
ii. **Width of footing: -** Ultimate bearing capacity increases with width of footing.
iii. **Depth of footing: -** Ultimate bearing capacity increase with depth of footing.
iv. **Relative density: -** a sand with greater relative density exhibits larger angle of internal friction. It will be observed from Table 2.1 that the bearing capacity factors N_γ and Nq increase with ϕ. Hence dense sands have greater bearing capacity.

b) Cohesive Soil

If the soil on which the footing is to rest is cohesive and $\phi = 0$,

$$\text{For } \phi = 0,\ N_c = 5.7,\ N_\gamma = 0, Nq = 1$$

Therefore the bearing capacity equations (Eqn.2.3) reduce to

$$q_{ult} = K_1\, 5.7\, C + \gamma_2 D \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \qquad (2.10)$$

The ultimate bearing capacity of clays depends on the value of cohesion, and depth of footing. The former is considerably more important than the later.

2.4.5 Bearing Capacity Factors for Local Shear Failure

The soil which supports a footing begins to yield when the full shear resistance has been mobilized directly underneath the footing, but does not reach the ultimate bearing capacity until full resistance is reached all along the boundary of the failure wedge. For dense sand full resistance is mobilized almost simultaneously along the entire boundary. However, for loose sand considerable footing movement is necessary before full resistance is reached along the outermost portions of the boundary. Terzaghi has recommended that an approximate value of ultimate bearing capacity q'_{ult} of shallow foundations on such soils may be computed based on the soil parameters C' and ϕ' where

$$C' = \tfrac{2}{3}\, C$$

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

Where C and ϕ are the actual values. The ultimate bearing capacity is such a case is given by

$$q'_{ult} = K_1 N_c' C + K_2\, \gamma_1\, N'_\gamma\, B + N_q'\, \gamma_2 D \quad \ldots\ldots\ldots \qquad (2.11)$$

where N'_c, N'_γ and N'_q are the bearing capacity factors for local shear failure.

Example 2.1

What will be the gross and net safe bearing capacity of sand having $\phi = 36^0$ and effective unit weight 18 kN/m^3 under the following cases.

a. 1m wide strip footing
b. 1m x 1 m square footing
c. Circular footing of 1 m diameter

Consider the footings are placed at depth of 1 m from ground surface and no problem of water table rising. Take a factor of safety of three.

Solution

For $\phi = 36^0$, $N_\gamma = 43$ and $N_q = 47$

a) Strip footing

$$q_{ult} = k_2\gamma B N_\gamma + \gamma D N_q$$

$$q_{all} = \frac{1}{F.S}(K_2\gamma B N_\gamma + \gamma D N_q)$$

$$= \tfrac{1}{3}(0.5x18x1x43+18x1x47)$$

$$= 411 \text{ kN/m}^2$$

$$q_{nall} = q_{all} - \gamma D = 411-18x1 = 393 \text{ kN/m}^2$$

b) Square footing

$$q_{all} = \tfrac{1}{3}(0.4x18x1x43+18x1x47)$$

$$= 385.2 \text{ kN/m}^2$$

$$q_{nall} = 385.2 - 18 = 367.\ 2 \text{ kN/m}^2$$

c) Circular footing

$$q_{all} = \tfrac{1}{3}(0.6x18x0.5x43+18x1x47)$$

$$= 359.4 \text{ kN/m}^2$$

$$q_{nall} = 359.4 - 18 = 341.4 \text{ kN/m}^2$$

Example 2.2

For the soil of example 2.1 and a strip footing having width and depth of 1m, what will be the net allowable bearing capacity under the following conditions of ground water table?

a. Water table touching the base of the foundation
b. Water table touching the ground surface

Saturated unit weight of the soil is 21.3 kN/m^3. The soil is assumed to remain saturated below ground surface.

Solution

$$q_{all} = \frac{1}{F.S}\left(0.5\gamma_1 BN_\gamma + \gamma_2 DN_q\right)$$

a) When water table touching the foundation base

$$\gamma_b = \gamma_{sat} - \gamma_\omega = 21.3 - 10 = 11.3 \text{ kN/m}^3$$

$$q_{all} = ⅓\,(0.5 \times 11.3 \times 43 \times 1 + 47 \times 21.3 \times 1)$$

$$= 414.68 \text{ kN/m}^2$$

$$q_{nall} = 414.68 - (21.3 \times 1) = 393.38 \text{ kN/m}^2$$

b) When the water table is at the ground level

$$q_{all} = ⅓\,(0.5 \times 11.3 \times 43 \times 1 + 47 \times 11.3 \times 1)$$

$$= 258.01 \text{ kN/m}^2$$

$$q_{nall} = 258.01 - 11.3 = 246.72 \text{ kN/m}^2$$

Example 2.3

A square column rests 1.5m below the ground surface. The total load transmitted by the footing is 2000 kN. The water table is located at the base of the footing. Assuming a saturated unit weight of sand as 22 kN/m^3 and angle of internal friction of 33^0, find a suitable size of the footing for the above condition. Take F.S = 3

Solution

$$q_{all} = \frac{1}{F.S}\left(k_2\gamma BN_\gamma + \gamma DN_q\right)$$

For square footing, $k_2 = 0.4$

For $\phi = 33^0$, $N_\gamma = 32$, $Nq = 32$

$$\gamma_1 = \gamma_{sat} - \gamma_\omega = 22\text{-}10 = 11\ \text{kN/m}^3,$$

$$\gamma_2 = 22\ \text{kN/m}^3$$

$$q_{all} = \tfrac{1}{3}\,[\,0.4\text{x}11\text{x}32\text{x}B + 22\text{x}1.5\text{x}32]$$

$$q_{all} = \tfrac{1}{3}\,[\,140.8B + 1056]$$

$$\frac{Q}{B^2} \le q_{all}$$

$$\frac{2000}{B^2} \le \frac{1}{3}\,[140.8B + 1056]$$

$$6000 = 140.8B^3 + 1056\,B^2$$

By trial and error, B = 2.15 m

Use footing 2.15 x 2.15 m

3. FOUNDATION

3.1 INTRODUCTION

The lowest artificially built part of a structure which transmits the load of the structure to the ground is called foundation.

The foundation of a structure is always constructed below ground level so as to increase the lateral stability of the structure. It includes the portion of the structure below ground level and other artificial arrangements in the form of concrete block, grillage, raft, piles etc. at its base so as to provide a firm and level surface for transmitting the load of the structure on a large area of the soil lying underneath.

3.2 PURPOSES OF FOUNDATIONS

Foundations are used for the following purposes.

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

3.3 TYPES OF FOUNDATIONS.

Foundations can be broadly classified into the following two categories

- Shallow foundations
- Deep foundations

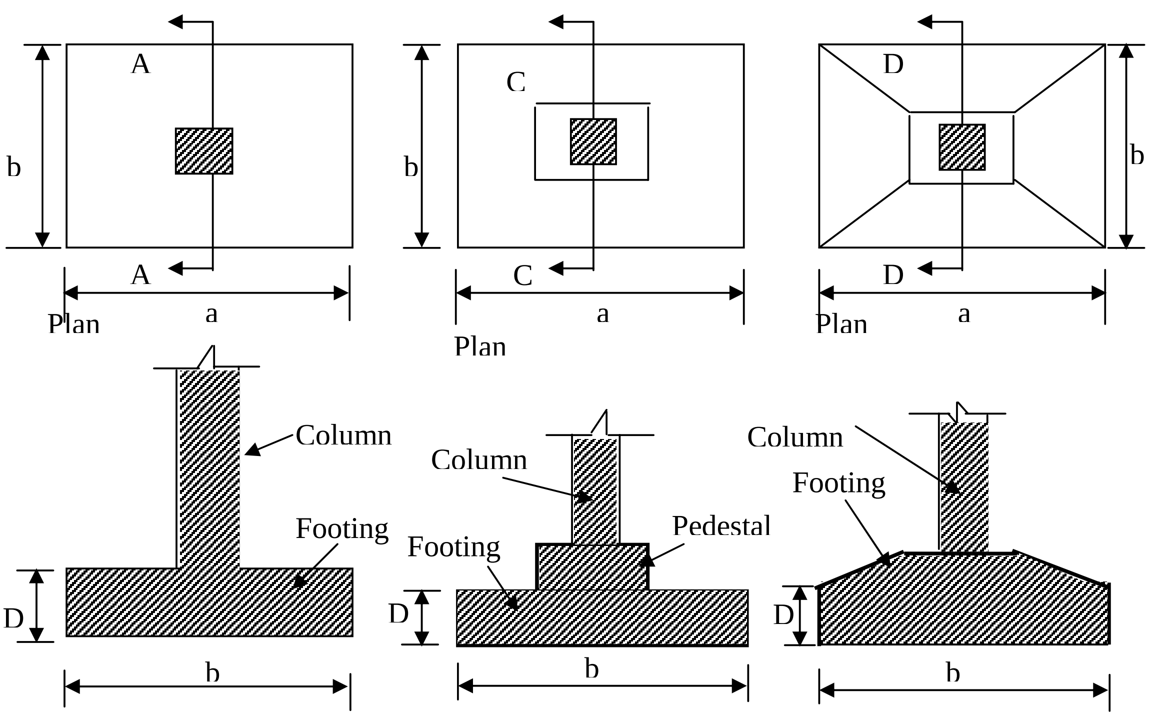
3.3.1 <u>Shallow Foundations</u>

The foundations provided immediately beneath the lowest part of the structure, near to the ground level are known as shallow foundations. The purpose of this type of foundations is to distribute the structural loads over a considerable base area at the foundation bed. Since spread foundations (shallow foundations) are constructed in open excavations, therefore, they are termed as open foundations

Shallow foundations are further classified into the following types: -

a. Spread or Isolated footings
b. Combined footing
c. Cantilever footing
d. Continuous or wall footing
e. Raft foundation

Spread or Isolated Footings:- They are used to support individual column. Isolated footings are stepped type, simple type or slope type, having projections in the base concrete. To support heavy loads, reinforcement is also provided at the base. The reinforcement provided is in the form of steel bars and is placed in both directions.

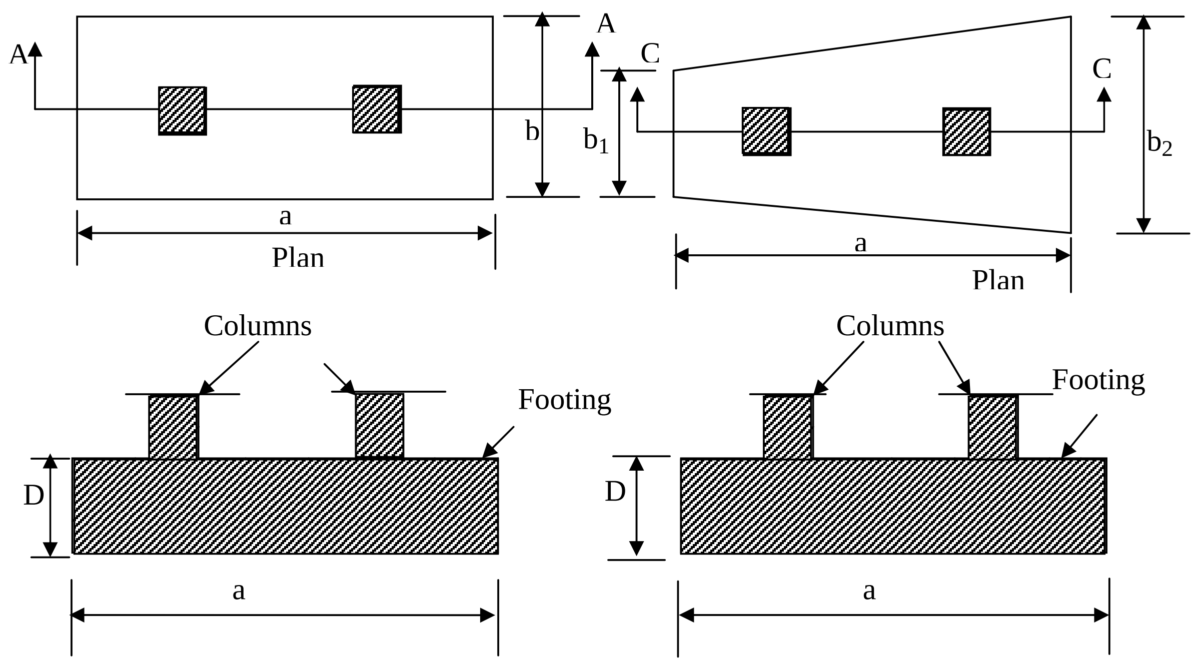


Section A-A Section C-C Section D-D

Single spread footing Stepped spread footing Sloped spread footing

Fig. 3. 1 Spread or isolated footings

Combined Footing:- A combined footing supports two or sometimes three column in a row. Combined footing is used when property lines, equipment locations, column spacing or other considerations limit the footing clearance at the column locations. The combined footing can be rectangular in shape if both the columns carry equal loads, or can be trapezoidal if there is a space limitation and they carry unequal loads. Generally they are constructed of reinforced concrete.



Section A.A
Combined footing (rectangular)

Section C.C
Combined footing (trapezoidal)

Fig. 3. 2 Combined footing

Cantilever or Strap Footing: - Cantilever footing consists of two individual footings connected by a beam called a strap. It is also sometimes called as strap footing. Cantilever footing may be used where the distance between the columns is so great that a trapezoidal combined footing becomes quite narrow, with resulting high bending moments. The strap beam does not remain in contact with soil so a strap doesn't transfer any pressure to the soil.

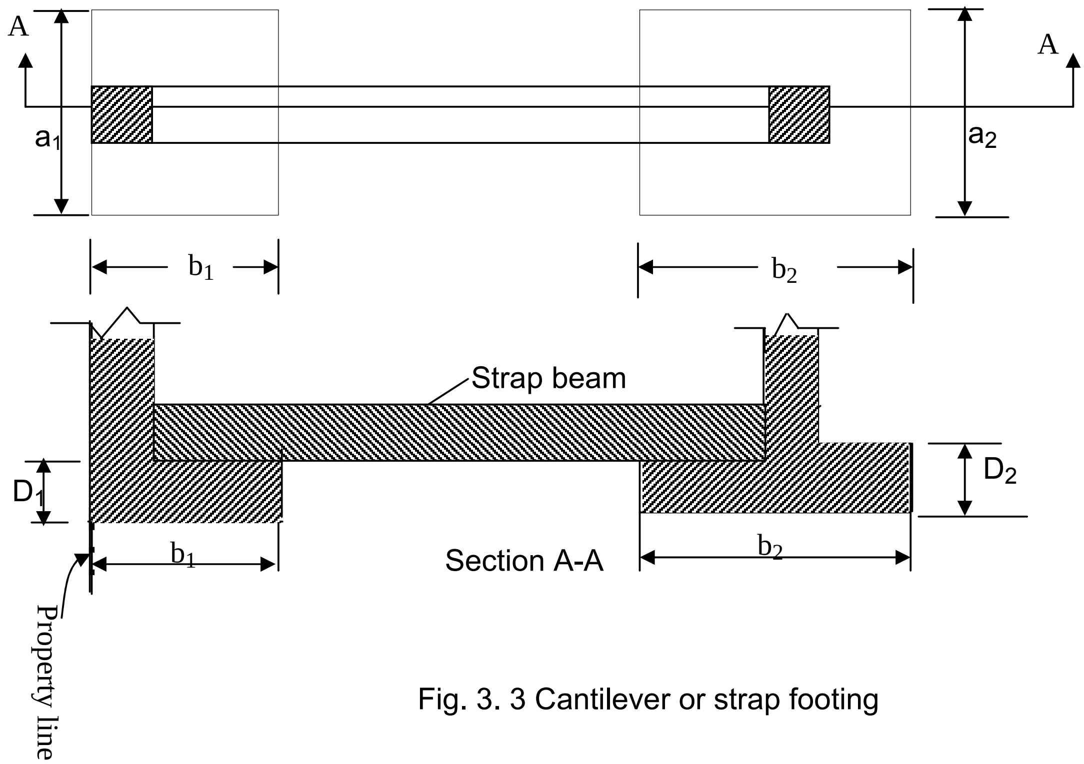


Fig. 3. 3 Cantilever or strap footing

Continuous or Wall Footing:- In this type of footing a single continuous reinforced concrete slab is provided as foundation of wall and three or more columns in a row. This type of footing is suitable at locations liable to earthquake activities. This also prevents differential settlement in the structure.

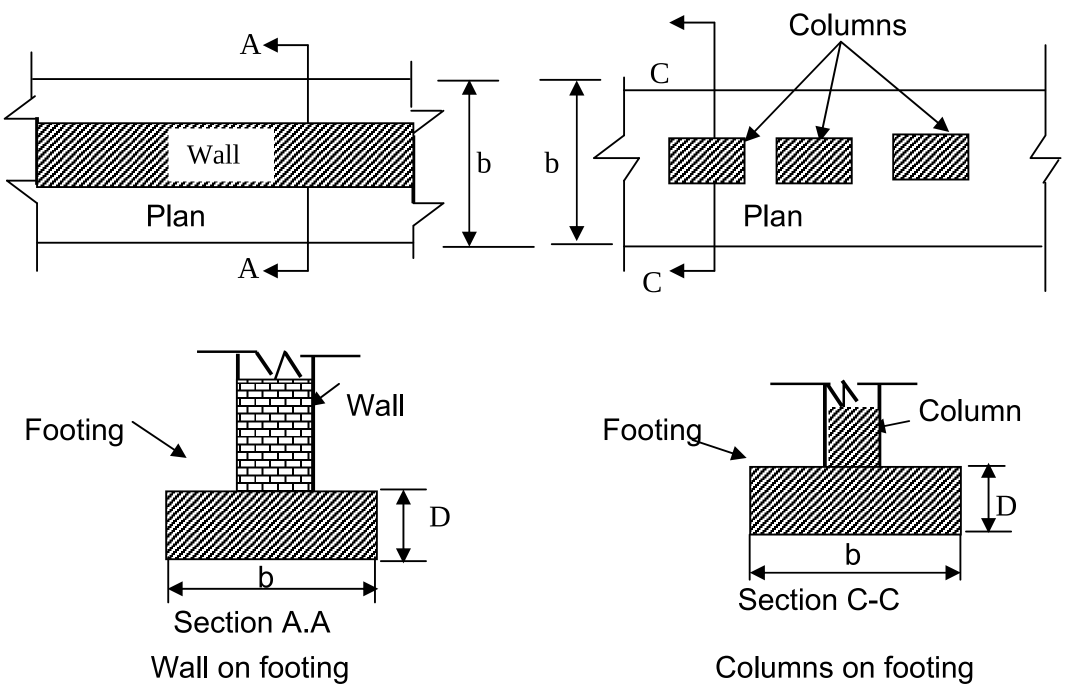
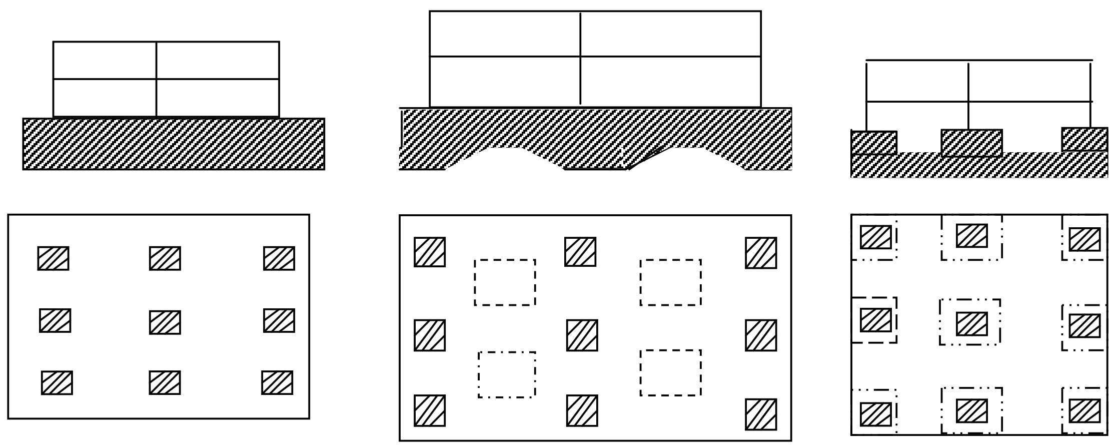


Fig. 3. 4 Continuous or wall footing

Raft Foundation:- A raft or mat is a combined footing that covers the entire area beneath a structure and supports all the columns. When the allowable soil pressure is low or the structure loads are heavy, the use of spread footings would cover more than one-half of the area, and it may prove more economical to use raft foundation. It is also used where the soil mass contains compressible layers so that the differential settlement would be difficult to control the raft tends to bridge over the erratic deposits and eliminates the differential settlement.



Flat plate mat foundation

Two-way beam and slab
(Ribbed mat)

Fig. 3. 5 Raft footing

3.3.2 Deep Foundations.

When the upper ground stratum at a site is weak and unable to carry the load even by a raft foundation, then eventually shallow foundation has to be ruled out, and a deep foundation, taken to an available firm stratum, is adopted. Deep foundation may be in the form of Piles or Well (i.e., Caissons).

A pile is relatively a small diameter shaft, which is used to transmit the loads to deeper soil layers capable of supporting the loads. A well on the other hand is a large diameter circular body, usually, sunk into the ground, by removing the ground soil and it is usually adopted for structures across rivers streams, where heavy scouring is involved, such as for supporting the piers of a road or a railway bridge, or some monumental building.

Pile Foundations

A foundation supported on piles is called a pile foundation

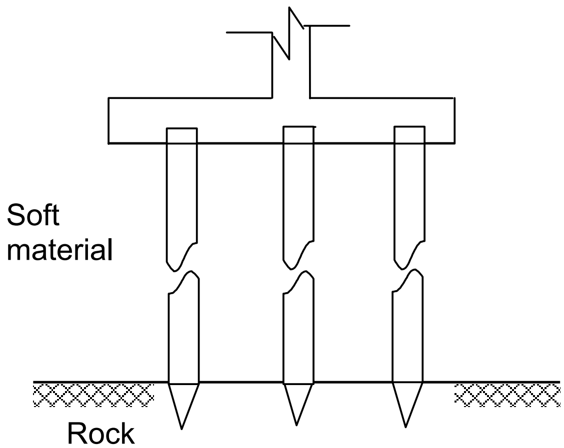


Fig. 3.6 Pile foundation.

Pile foundation is suitable under the following situations

a) When the soil is very soft and solid base is not available at a reasonable depth to keep the bearing power within safe limits.

b) When shallow foundations are very expensive

c) When the building is a very high and carrying heavy concentrated load.

Classification of Piles

Piles are generally classified into the following categories according to

1. the mode of transfer of load
2. the use
3. composition or material of construction.
4. the method of construction

i. Classification of Piles According to the Mode of Transfer of Loads

a. **End-Bearing Piles:-** these piles penetrate through the soft soil or water and their bottoms rest on a hard stratum and transmits the load to it.

b. **Friction Piles:-** When loose soil extends to a great depth, piles are driven up to such a depth that frictional resistance developed at the sides of the piles equals the load coming on the piles.

c. **Combined End-Bearing and Friction Piles:-** the piles which rest on hard strata and resist the loads partly by bearing and partly by their skin friction are known as friction -cum- Bearing piles

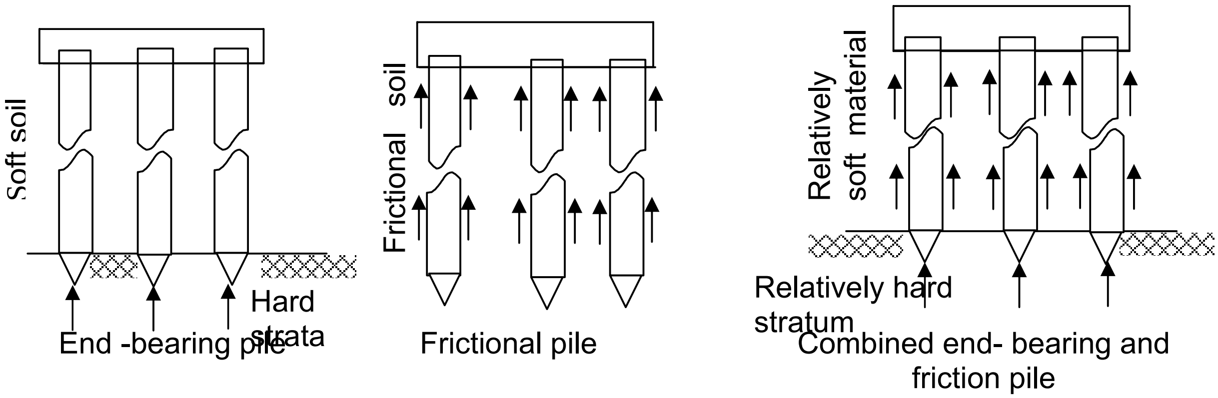


Fig 3.7 Types of piles

ii. Classification of Piles Based on Use

a. **Uplift Piles:-** These piles anchor down the structure subjected to uplift or overturning movement.
b. **Batter Piles:-** The piles driven at an inclination to resist inclined loads are known as batter piles. These piles are used generally to resist lateral forces in case of retaining walls, abutments etc.
c. **Compaction Piles:-** they are used to compact loose granular soils in order to increase their bearing capacity. These piles themselves do not carry any load.
d. **Sheet Piles:-** The piles which consist of thin steel sheets driven in the ground to enclose an area are known as sheet piles. These piles are used to enclose soil so as to prevent the leakage of water and to enclose soft material.

iii. Classification of Piles According to Their Composition or Material of Construction.

a. Timber Piles:-Timber piles are made of tree trunks with the branches trimmed off. The timber to be used for the construction should be free from defects; decay etc. and it should be well seasoned. These piles are circular or square in cross section. Top of these piles is provided with an iron ring to prevent it from splitting under bellows of the hammer. The bottom is fitted with an iron shoe to facilitate sinking of the piles. Piles entirely submerged in water last long without decay. When a pile is subjected to alternate wetting and drying the useful life is relatively short.

b. Concrete Piles:- Cement concrete is used in the construction of concrete piles. These piles are strong and durable and can bear more load than timber piles.

Concrete piles may be classified into the following two types

i) Pre-cast concrete piles

ii) Cast in situ piles

Pre-Cast Concrete Piles:- These piles are manufactured in factory. These are R.C piles, which are usually square, circular or octagonal in cross-section. Normally these are made to resist the stresses caused by driving and handling as well as those produced by loads they are supposed to carry.

Cast-in situ Concrete Piles:- These type of piles are constructed in their locations in the bore holes prepared for these purpose. The operation consists of boring a hole, filling it with only concrete or with steel reinforcement and concrete. Straight bored piles or piles with one or more bulbs at intervals may be cast at site the latter type is call as under reamed pile.

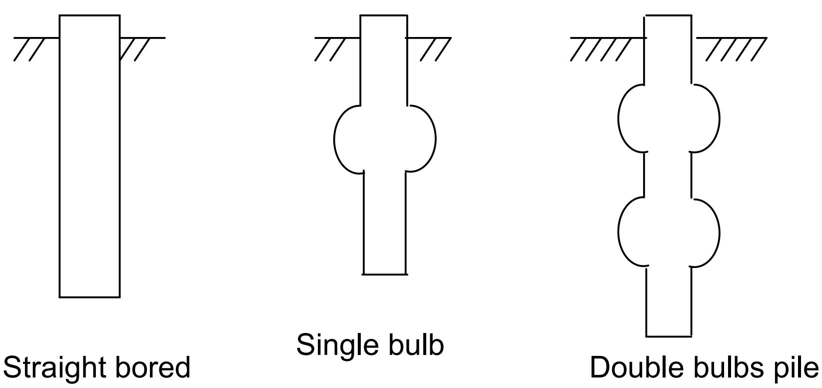


Fig 3.8 Cast-in situ piles

Cast in-situ concrete piles are easy to handle. They do not require any extra reinforcement to resist the stresses developed during handling or driving operations. There is no wastage of material as the pile of required length is only constructed. The extra cost of transporting pile is eliminated.

The disadvantages of these piles are

i. It is difficult to maintain the reinforcement in correct position during construction of pile

ii. The piles can not be constructed under water, and

iii. It is not possible to have a proper control over the composition and design of these piles.

c. **Sand Piles**: **-** The piles consisting of sand filled in boreholes are called sand piles. These piles are formed by digging holes. The holes are then filled sand and compacted. Top of the sand pile is covered with concrete to prevent the sand to come upwards due to lateral pressure. Sand piles are used occasionally for taking light loads. They are not suitable in regions subjected to frequent earthquakes.

d. **Steel Piles**: **-** The piles consisting of a steel section are called steel piles. These piles are useful where driving conditions are difficult and other types of piles are not suitable. Steel piles are usually H shapes or pipe piles. H-piles are proportioned to withstand large impact stresses during hard driving. Pipe piles fitted with conical cost iron shoes are driven in the ground and then hollow space is filled with concrete.

e. **Composite Piles**: **-** A composite pile is formed when it is a combination either of a bored pile and a drive pile or of driven piles of two different materials. They are suitable where the upper part of a pile to project above the water table. They are economical and easy to construct.

iv. Classification Based on Method of Construction (Installation)

a. **Driven Piles: -** These piles are driven into the soil by applying blows of a heavy hammer on their tops.
b. **Driven and Cast In-Situ Piles: -** These piles are formed by driving a casing with a closed bottom end into the soil. The casing is later filled with concrete. The casing may or may not be with drawn.
c. **Bored and Cast In-Situ Piles: -** These piles are formed by excavating a hole into the ground and then filling it with concrete.
d. **Screw Piles: -** The piles are screwed into the soil.
e. **Jacked Piles: -** These piles are jacked into the soils by applying a downward force with the help of a hydraulic jack.

Selection of Type and Length of Piles

The choice of a pile is governed largely by the site and soil conditions. Based on the soil conditions the following types of piles are recommended.

Soil conditions	Choice of pile	Remarks
Coarse sand or Gravel	Driven pile or cast in situ pile	Develops point bearing and friction resistance
Firm stratum with soft material below	Cast in-situ Pile with enlarged base	Improves bearing capacity
Hard stratum at reasonable depth (15-30m). With no soft material below	Point bearing pile	Embed pile about 1m inside hard stratum
Where hard stratum is very deep (>30m)	Friction pile	Cheaper than long point bearing pile
Expansive and poor soils overlying firm soil stratum	Short cast in-situ pile with enlarged base	Base shall rest on stable zone

Based on site conditions the following type of piles are recommended

Site conditions	Choice of piles	Remarks
Close to existing building	Cast in-situ pile	Cause less damage to existing building
Under water construction	Driven pile	Easier to install and also cheaper

Pile Cap

The main function of pile cap is to transfer loads from a column or wall to an underlying group of piles. To ensure stability against lateral forces, a pile cap must include at least three piles; otherwise it should be connected by the beams to adjacent caps.

In general, pile caps should be arranged in such a manner that the centroid of pile group coincides with the line of action of load, to ensure that all piles carry an equal load and avoid tilting of group in compressible bearing stratum.

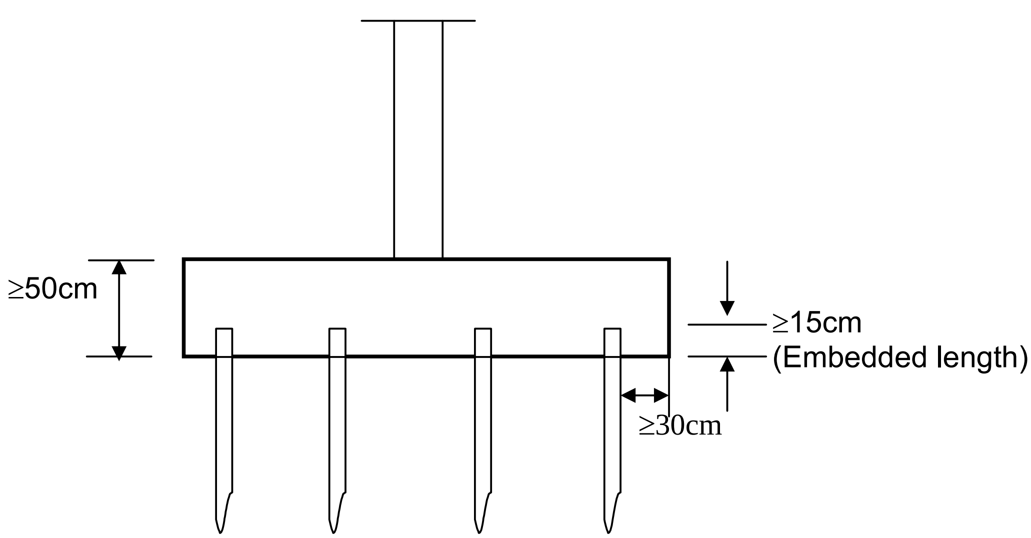


Fig. 3. 9 Pile Cap

3.4 GENERAL PRINCIPLES OF FOUNDATION DESIGN

3.4.1 General

The usual approach to a normal foundation-engineering problem is

1. To prepare a plan of the base of the structure showing the various columns, load-bearing walls with estimated loads, including dead load, live load, moments and torques coming into the foundation units.
2. To study the tentative allowable bearing pressures allocated for the various strata below the ground level, as given by the soil investigation report.
3. To determine the required foundation depth. This may be the minimum depth based on soil strength or structural requirement considerations.
4. To compute the dimensions of the foundation based on the given loading and allowable bearing pressure.
5. To estimate the total and differential settlements of the structure. If these are excessive the bearing pressure will have to be reduced or the foundation taken to a deeper and less compressible stratum or the structure will have to be founded on piles or other special measures taken

3.4.2 Loads on Foundation

An accurate estimation of all loads acting on the foundation should be made before it can be properly designed. A foundation may be subjected to two or more of the following loads.

a) Dead load: - - Weight of structure
- All material permanently attached to structure
- Static earth pressure acting permanently against the structure below ground surface.
- Water pressure acting laterally against basement walls and vertically against slab.

b) Live load:- temporary loads expected to superimpose on the structure during its useful life.

c) Wind load:- lateral load coming from the action of wind. Local building codes provide magnitude of design wind pressure.

d) Earth-quake load:- lateral load coming from earth quake motion. The total lateral force (base shear) at the base of a structure is evaluated in accordance with local building code.

e) Dynamic load:- load coming from a vibrating object (machinery). In such case, separate foundation should be provided. The impact effect of such loads should be considered in design.

3.4.3 <u>Selection of Foundation Type</u>

In selecting the foundation type the following points must be considered

a. Function of the structure
b. Loads it must carry
c. Subsurface conditions
d. Cost of foundation in comparison with the cost of the superstructure.

Having these points in mind one should apply the following steps in order to arrive at a decision.

i. Obtain at least approximate information concerning the nature of the superstructure and the loads to be transmitted to the foundation
ii. Determine the subsurface condition in a general way.
iii. Consider each of the usual types of foundations in order to judge whether or not

   a. They could be constructed under existing conditions.
   b. They are capable of carrying the required load.
   c. They experience serious differential settlements.

The types that are found to be unsuitable should then be eliminated.

iv. Undertake a detailed study of the most promising types. Such a study may require additional information on loads and subsurface conditions.

Determine the approximate size of footing or the approximate length and number of piles required

v. Prepare an estimate for the cost of each promising type of foundation.
vi. Select the type that represents the most acceptable compromise between performance and cost.

4. Design of shallow Foundations

4.1 INTRODUCTION

This chapter deals with the economical and safe design of the common types of shallow foundations. The main foundation types that are considered here are: isolated or spread footings, combined footings, strap or cantilever footings and mat or raft foundations.

Shallow foundations are structural members that are used to transfer safely to the ground the dead load of the superstructure and all external forces acting upon it. The type and magnitude of the loading will usually be furnished by the engineer design the superstructure. It is up to the foundation engineer to collect all the information regarding the purpose of the superstructure, the material that will be used in its construction, its sensitivity to settlements in general and to differential settlement in particular and all other pertinent information that may influence the successful selection and execution of the foundation design. The foundation engineer should also select the soil stratum that most suitable for the support of the superstructure.

The design of shallow foundations is based on the assumption that they are rigid so that the variation of pressure under the foundations will be linear. The distribution of pressure will be uniform if the centroid of the foundation coincides with the resultant of the applied loads. The requirements in design of foundations are:

1. The pressure on the soil should not exceed the bearing capacity of the soil.
2. The settlement of the structure should be within the permissible limits. Further there should be no differential settlement.

In order to proportion shallow foundations one should either know the presumptive allowable soil pressure as dictated by prevalent code or know the appropriate strength parameters of the soil, i.e., the angle of internal friction, ϕ , and cohesion, C.

4.1.1 Proportioning of shallow foundations using presumptive allowable soil pressure.

Through many years of practice, it has been possible to estimate the allowable soil pressure for different types of soils for uncomplicated soil conditions. Accordingly different Building codes give allowable average soil pressure. Here EBCS 7 is presented.

Table 4,1 Presumed Design Bearing resistance * under static loading(EBCS 7)

Supporting Ground Type	Description	Compactness** or Consistency***	Presumed Design Bearing Resistance (kPa)	Remarks
Rocks	Massively crystalline igneous and metamorphic rock (granite, basalt, gneiss)	Hard and sound	5600	These values are based on the assumptions that the foundations are carried down to unweathered rock
	Foliated metamorphic rock (slate, schist)	Medium hard and sound		
	Sedimentary rock (hard shale, siltstone, sandstone, limestone)	Medium hard and sound	2800	
	Weathered or broken-rock (soft limestone)	Soft	2800	
	Soft shale	Soft	1400	
	Decomposed rock to be assessed as soil below.		850	
Non-cohesive soils	Gravel, sand and gravel	Dense Medium dense Loose	560 420 280	Width of foundation (B) not less than 1m

	Sand	Dense Medium dense Loose	420 280 140	Ground water level assumed to be depth not less than B below the base of the foundation.
Cohesive soils	Silt Clay	Hard Stiff Medium stiff Soft Hard Stiff Medium stiff Soft Very soft	280 200 140 70 420 280 140 70 Not applicable	

* The given design bearing values do not include the effect of the depth of embedment of the foundation.

** Compactness: dense: N> 30
medium dense: N is 10 to 30
loose: N< 10, where N is standard penetration value

*** Consistency: hard: q_u > 400kPa
stiff: q_u = 100 to 200kPa
medium stiff q_u = 50 to 100kPa
soft: q_u = 25 to 50 kPa, where q_u is unconfined compressive strength

4.1.2 <u>Proportioning of shallow foundations using the soil strength parameters ϕ and C</u>.

For cases where presumptive allowable soil pressures can not be used, one should determine the soil strength parameters ϕ and C. These parameters may be approximated or determined from laboratory tests. Using the value of ϕ and C thus obtained, one can easily determine the area of the foundation in question using bearing capacity equations.

In applying the bearing capacity equations one should differentiate two states of loading, namely, *the initial or instantaneous loading condition* and *the final or long- term loading condition.*

In the initial loading condition, the load is assumed to act instantaneously. At this stage the pore water pressure in the soil does not have time to dissipate. This situation corresponds to the quick or undrained test condition of the triaxial test. The soil parameters are designated by ϕ_u and C_u - in most cases $\phi_u = 0$.

In the final or long term loading condition, the load is assumed to act gradually as construction progresses thus giving the pore water pressure in the soil ample time to dissipate. Here the situation corresponds to the slow or drained test condition of the triaxial test. The soil parameters in this case are designated by ϕ' and C'.

The ultimate load that may be applied on a foundation with sides **a** and **b** may be determined from the following equation

$$V_f = A'\sigma_f \quad \text{--} \qquad (4.1)$$

Where $A' = a' b' =$ effective area (Fig. 4.1)

$a' = a-2e_a$ = effective length

$b' = b-2e_b$ = effective width

σ_f = ultimate bearing capacity of the footing

e_a and e_b = Eccentricities in the long and short directions, respectively.

The actual sustained load on the footing may be related to the ultimate load

$$V_f = \eta p \qquad (4.2)$$

Where η = factor of safety

P = actual sustained load on the foundation

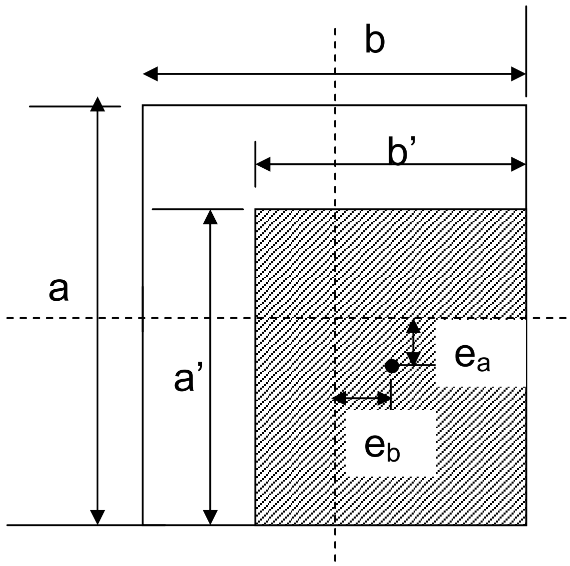


Fig.4.1 Effective width and length of a foundation.

One may then express Eqn. (4.1) as

$$\eta p = A'\sigma_f \qquad (4.3)$$

From which it follows

$$A' = \frac{\eta P}{\sigma_f} \qquad (4.4)$$

From Eqn. (4.4) one easily determines the required area since all the quantities on the right hand side of the equation are known.

The ultimate bearing capacity, σ_f , may be determined from the following equation

$$\sigma_f = CN_c S_c d_c i_c + \tfrac{1}{2} b' \gamma N_\gamma S_\gamma d_\gamma i_\gamma + q N_q S_q d_q i_q \qquad (4.5)$$

Where

q_{ult} = Ultimate bearing capacity of footing,

C = Cohesion,

q = Effective surcharge at the base level of the footing.

γ = effective unit weight of soil

N_{c}, N_{q}, N_{γ} = Bearing capacity factor

S_{c}, S_{q}, S_{γ} = Shape factors

d_{c}, d_{q}, d_{γ} = Depth factors

i_{c}, i_{q}, i_{γ} , = Inclination factors

For initial loading conditions, where ϕ_u = 0, the failure surface of the soil consist of straight lines and an arc of a circle. The bearing capacity coefficient would have the values Nc =5.1, Nq= 1.0, N_{γ} = 0. Eqn. (4.5) may be written as

$$\sigma_f = 5.1 C_u S_c d_c i_c + q S_q d_q i_q \text{ --------------------------------} \qquad (4.6)$$

4.1.3 Structural Considerations.

Before going into the structural design, one should check if the settlement of the selected foundation is within the prescribed safe limits. If the settlement exceeds the safe limits, one should increase the dimensions of the foundations until the danger of settlement is eliminated.

The last stage in the design of foundations is the structural design. One should check the adequacy of the thickness of the footing and provide the necessary reinforcement to withstand punching shear, diagonal tension (wide beam shear), bending moment and bond stress.

Allowable stresses according to EBCS-2

I. Punching Shear Resistance

$$V_p = 0.5f_{ctd}\,(1+50\rho_e) \quad \text{(MPa)}$$

$$\rho_e = \sqrt{\rho_{ex}\cdot\rho_{ey}} \le 0.008$$

$$V_{up} = 0.5f_{ctd}\,(1+50\rho_e)\,u\,d \quad \text{(MN)}$$

Ii . Diagonal Tension

$$V_d = 0.3f_{ctd}\,(1+50\rho) \quad \text{(MPa)}$$

$$\rho = \frac{a_s}{b_w d} \le 0.02$$

$$V_{ud} = 0.3f_{ctd}\,(1+50\rho)\,b_w d \quad \text{(MN)}$$

iii. Development length

$$l_d = \frac{\phi\, f_{yd}}{4 f_{bd}} \quad \text{(cm)}$$

$$f_{yd} = f_{yk}/\gamma_s \quad ; f_{bd} = f_{ctd}$$

$$f_{ctd} = \frac{0.35\sqrt{f_{ck}}}{\gamma_c}$$

Where a_s= area of tension reinforcement (mm^2)

b_w= width of web (mm)

d = effective depth (mm)

f_{bd}= design bond strength (MPa)

f_{ck} = characteristics compressive strength of concrete (MPa)

f_{ctd} = design tensile strength of concrete (MPa)

f_{yd} = design yield strength of reinforcement (MPa)

f_{yk} = characteristics yield strength of concrete (MPa)

u = periphery of critical section (mm)

γ_c = partial safety factor for concrete = 1.5

γ_s = partial safety factor for steel = 1.15

ρ = geometrical ratio of reinforcement

ρ_e =effective geometrical ratio of reinforcement

ρ_{ex} = geometrical ratio of reinforcement in the x-direction

ρ_{ey}= geometrical ratio of reinforcement in the y-direction

4.2 Isolated or Spread Footings

I. Depth of footing

The depth of embedment must be at least large enough to accommodate the required footing thickness. This depth is measured from the lowest adjacent ground surface to the bottom of the footing.

Footings should be carried below

a) zone of high volume change due to moisture fluctuation
b) top (organic) soil
c) peat and muck
d) unconsolidated (or fill) material

According to EBCS-7

- minimum depth of footing should be 50cm
- for footings on sloping sites, minimum depth of footing should be 60cm and 90cm below ground surface on rocky and soil formations, respectively.

Footing at different elevations: - When adjacent footings are to be placed at different levels, the distance between the edges of footings shall be such as to prevent undesirable overlapping of stresses in soils and disturbance of the soil under the higher footing due to excavation for the lower footing. A minimum clear distance of half the width of the footing is recommended.

II. Proportioning of footing

The required area of the footing and subsequently the proportions will be determined using presumptive allowable soil pressure and/or the soil strength parameters ϕ and C as discussed previously.

III. Structural Design

i) **Punching shear**:- This factor generally controls the depth of footings. It is the normal practice to provide adequate depth to sustain the shear stress developed without reinforcement. The critical section that is to be considered is indicated in Fig. 4.2

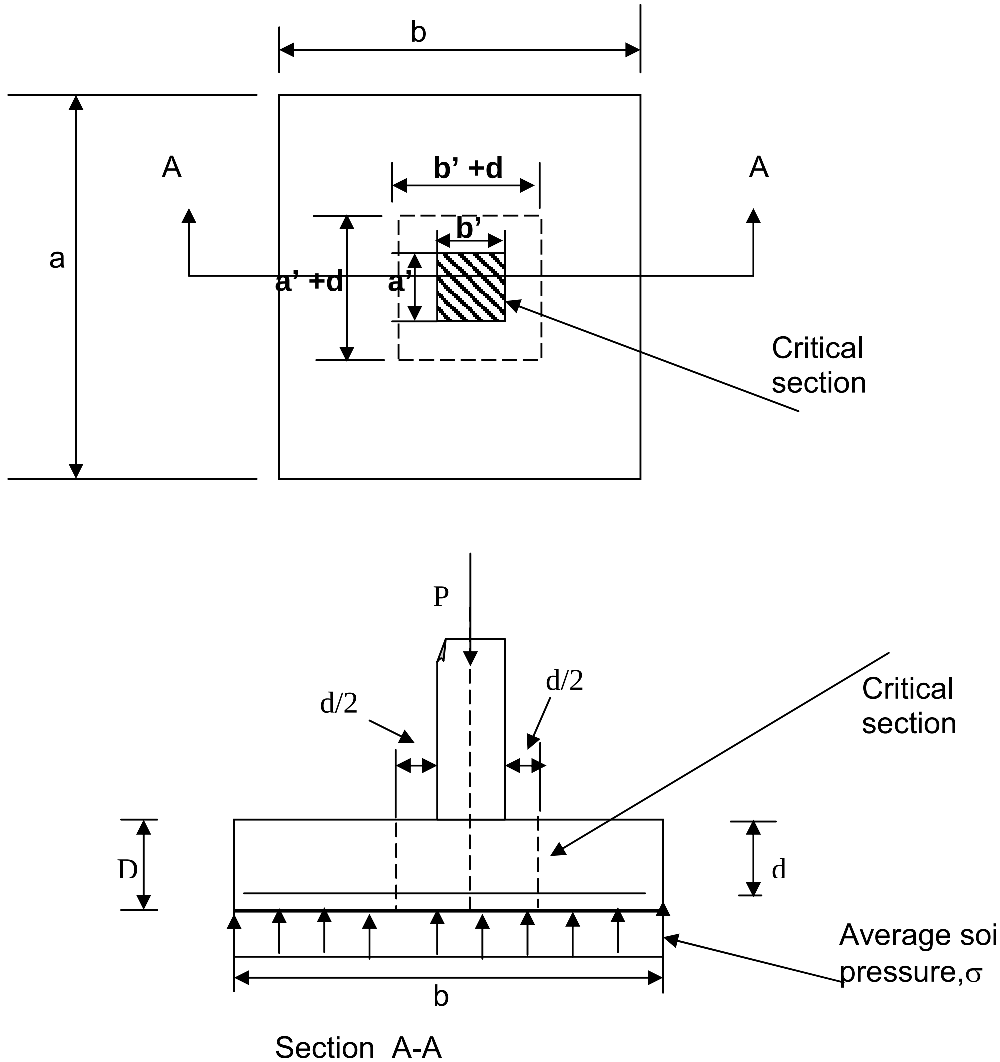


Fig. 4.2 Critical section for punching shear

From the figure it is apparent the concrete shear resistance along the perimeter would be

2(a' +d + b'+ d) dV_{up} ………………………………………………………… (4.7)

Where V_{up} = allowable soil pressure

The net force on the perimeter due to the soil pressure would be

$$\{a*b-[(a'+d)(b'+d)]\}\sigma \quad \text{……………………………………………} \quad (4.8)$$

From equilibrium consideration, Eqn. (4.7) and Eqn. (4.8) should be equal

$$2(a'+d+b'+d)\,dV_{up} = \{a*b-[(a'+d)(b'+d)]\}\sigma$$

$$2a'dV_{up} + 2d^2V_{up} + 2b'dV_{up} + 2d^2V_{up} = \left(ab - a'b' - a'd - b'd - d^2\right)\sigma$$

$$2a'dV_{up} + 2d^2V_{up} + 2b'dV_{up} + 2d^2V_{up} + a'd\sigma + b'd\sigma + d^2\sigma = \left(ab - a'b'\right)\sigma$$

$$2a'dV_{up} + 2b'dV_{up} + 4d^2V_{up} + d^2\sigma + a'd\sigma + b'd\sigma = \left(ab - a'b'\right)\sigma$$

$$d(2a'V_{up} + 2b'V_{up} + a'\sigma + b'\sigma) + d^2(4V_{up} + \sigma) = \left(ab - a'b'\right)\sigma$$

$$d^2(4V_{up} + \sigma) + d(2V_{up}(a'+b') + \sigma(a'+b')) = \left(ab - a'b'\right)\sigma$$

$$d^2(4V_{up} + \sigma) + d(2V_{up} + \sigma)(a'+b') = \left(ab - a'b'\right)\sigma$$

$$d^2(4V_{up} + \sigma) + d(2V_{up} + \sigma)(a'+b') = \left(A_{footing} - A_{column}\right)\sigma \quad \text{..............} \quad (4.9)$$

For square columns a’ = b’ and round columns with diameter a’, Eqn. (4.9) would be

$$d^2(4V_{up} + \sigma) + d(2V_{up} + \sigma)(2a') = \left(A_{footing} - A_{column}\right)\sigma \quad \text{.........} \quad (4.10)$$

In the above equations, all quantities with the exception of d are known. By solving one of the equations the effective depth necessary to sustain the punching shear may be determined.

ii) **Diagonal Tension (wide beam shear)**

The selected depth using the punching shear criterion may not be adequate to withstand the diagonal tension developed. Hence one should also check the safety against diagonal tension. The critical sections that should be considered are given in Fig. 4.3 .

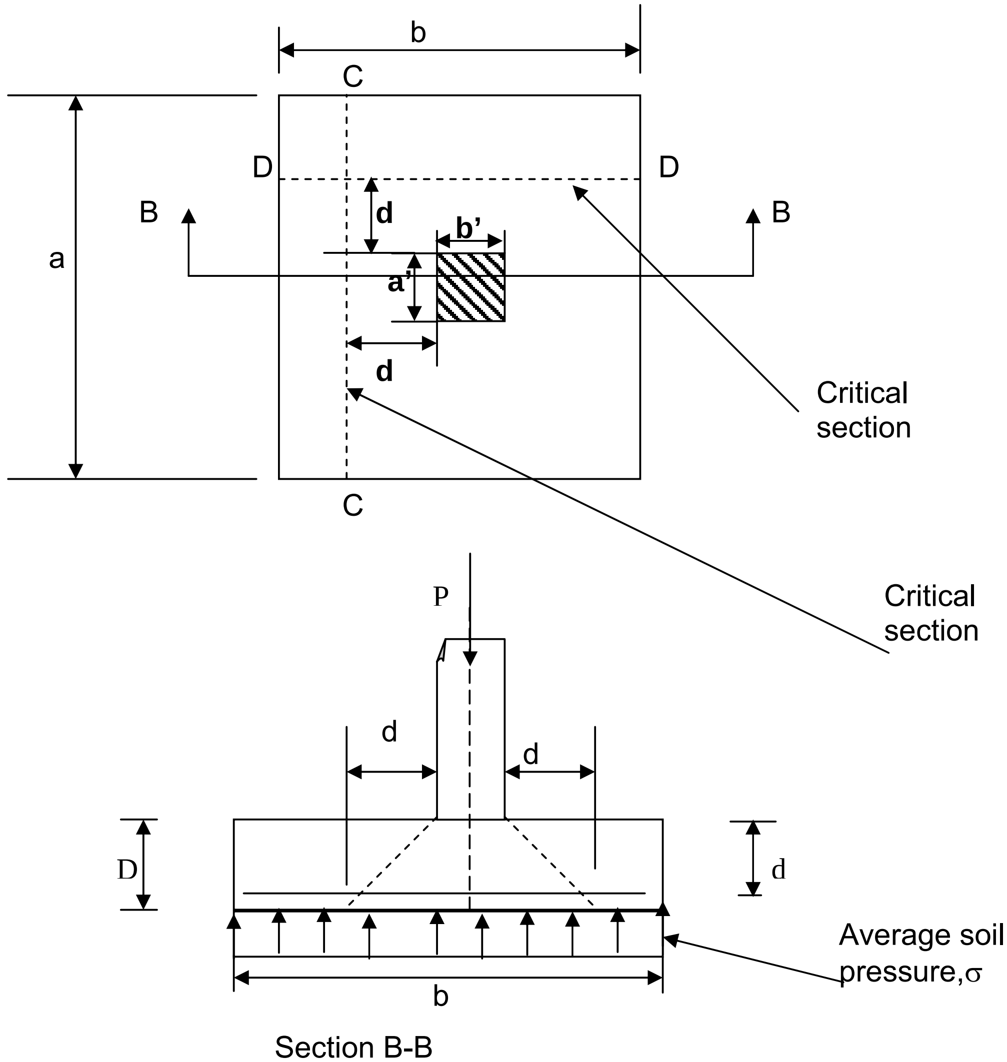


Fig. 4.3 Critical section for diagonal tension

The shear forces are calculated along the plane C-C and D-D

$$V_{C\text{-}C} = (b/2 - d - b'/2)\sigma a \qquad (4.11)$$

$$V_{D\text{-}D} = (a/2 - d - a'/2)\sigma b \qquad (4.12)$$

The actual shear stress is then calculated from

$$v_{C\text{-}C} = \frac{V_{C-C}}{ad} \qquad (4.13)$$

$$v_{D\text{-}D} = \frac{V_{D-D}}{bd} \qquad (4.14)$$

These calculated actual shear stresses should be compared with allowable stress.

iii) <u>Bending Moment</u>

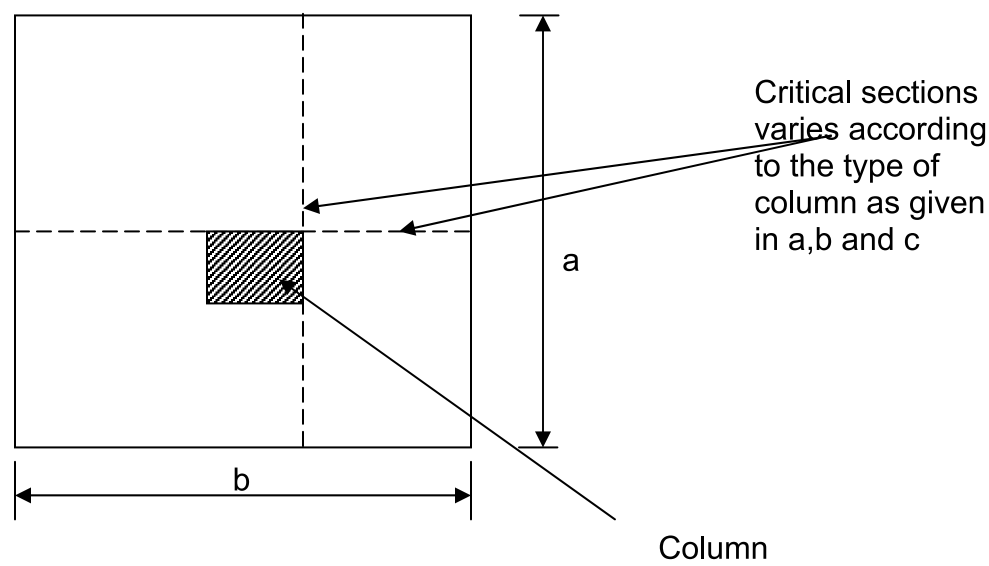
The external moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over the entire area of the footing on one side of that vertical plane. The critical sections for the bending moment vary according to the type of columns.

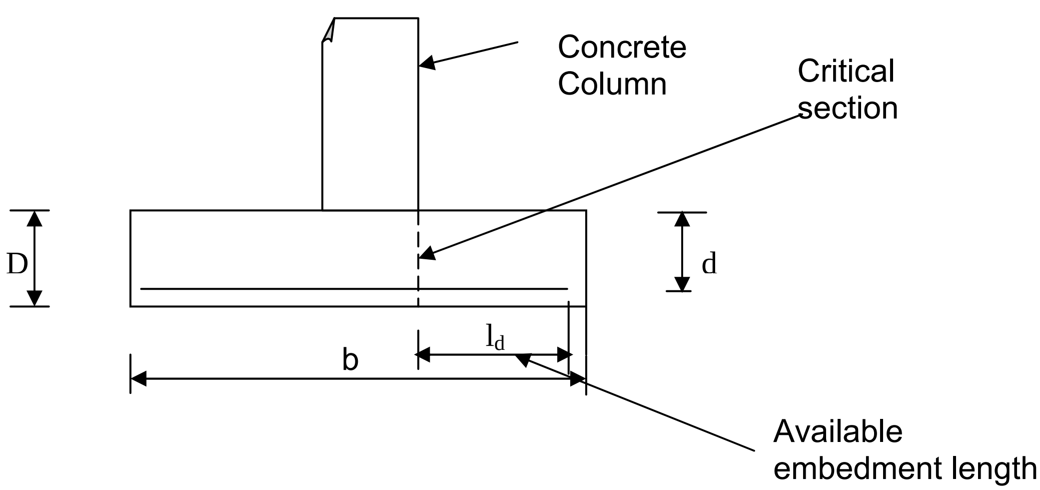
According to EBCS 2-1995, the critical section for moment shall be taken as follows:

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

b) Halfway between middle and edge of wall, for footings supporting a masonry wall

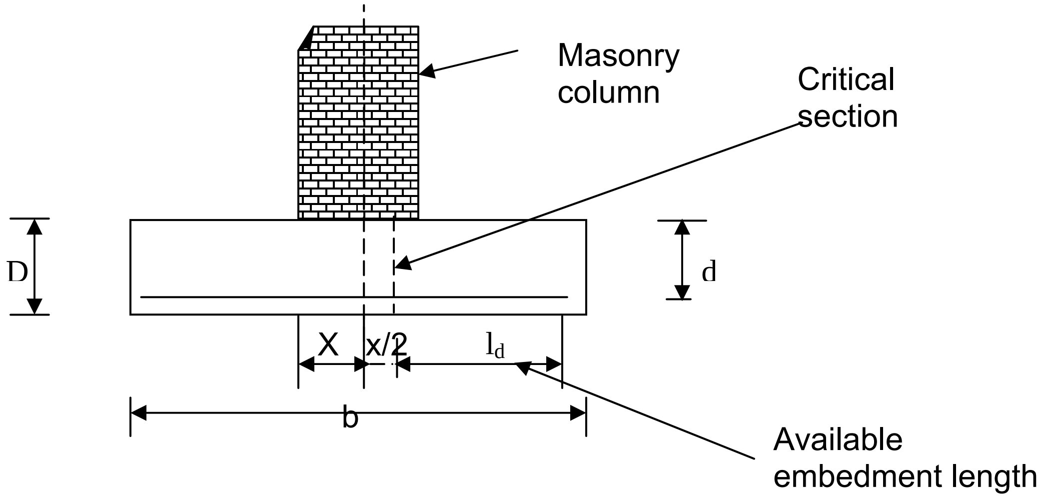
c) Halfway between face of column and edge of steel base for footings supporting a column with base plates.





a)

b)



c)

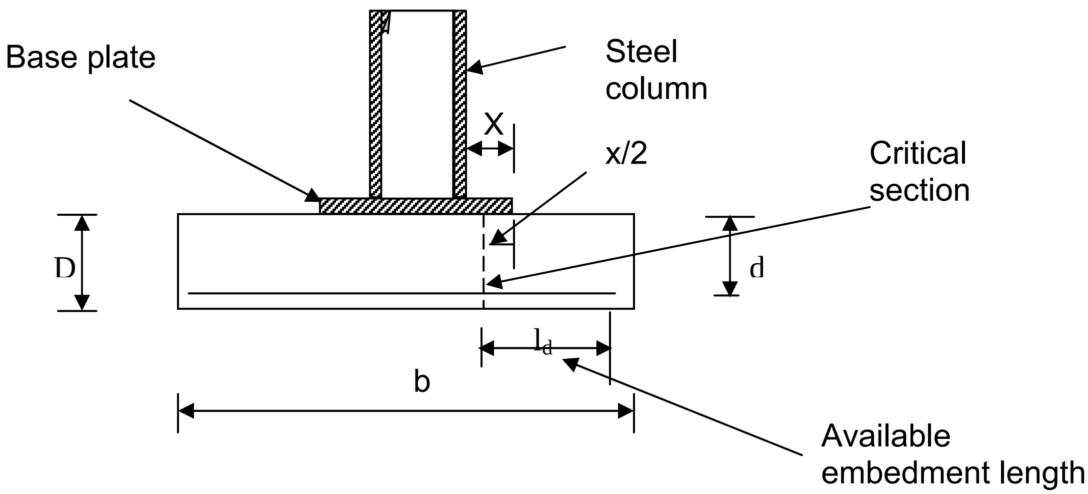


Fig. 4.4 Critical sections for moments

Flexural Reinforcement

1. Distribution: In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across the entire width of footing.
2. In two-way rectangular footings, reinforcement shall be distributed as follows:

a) Reinforcement in long direction shall be distributed uniformly across the entire width of footing

b) For reinforcement in the short direction, a portion of the total reinforcement given by Eqn.(4.15) shall be distributed uniformly over a band width (centered on center line of column or pedestal) equal to the length of the short side of footing. The reminder of the reinforcement required in the short direction shall be distributed uniformly out side the center band width of the footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{\beta + 1} \ldots\ldots\ldots\ldots\ldots \qquad (4.15)$$

Where β is the ratio of long side to short side of footing (a/b).

IV. Development length

The reinforcement bars must extend a sufficient distance into the concrete to develop proper anchorage. This distance is called the development length.

The necessary development length may be calculated using the following equation.

$$l_d = \frac{\phi \ f_{yd}}{4 \, f_{bd}}$$

Concrete cover to reinforcement (According to EBCS2-1995)

- Concrete cast directly against the earth, the minimum cover should be greater than 75mm
- Concrete cast against prepared ground (including blinding) the minimum cover should be greater than 40mm.

Spacing of reinforcement

The clear horizontal and vertical distance between bars shall be at least equal to the largest of the following values: (EBCS2-1995)

a) 20mm
b) the diameter of the largest bar
c) the maximum size of the aggregate plus 5mm

The spacing between main bars for slabs shall not exceed the smaller of 2h or 350mm

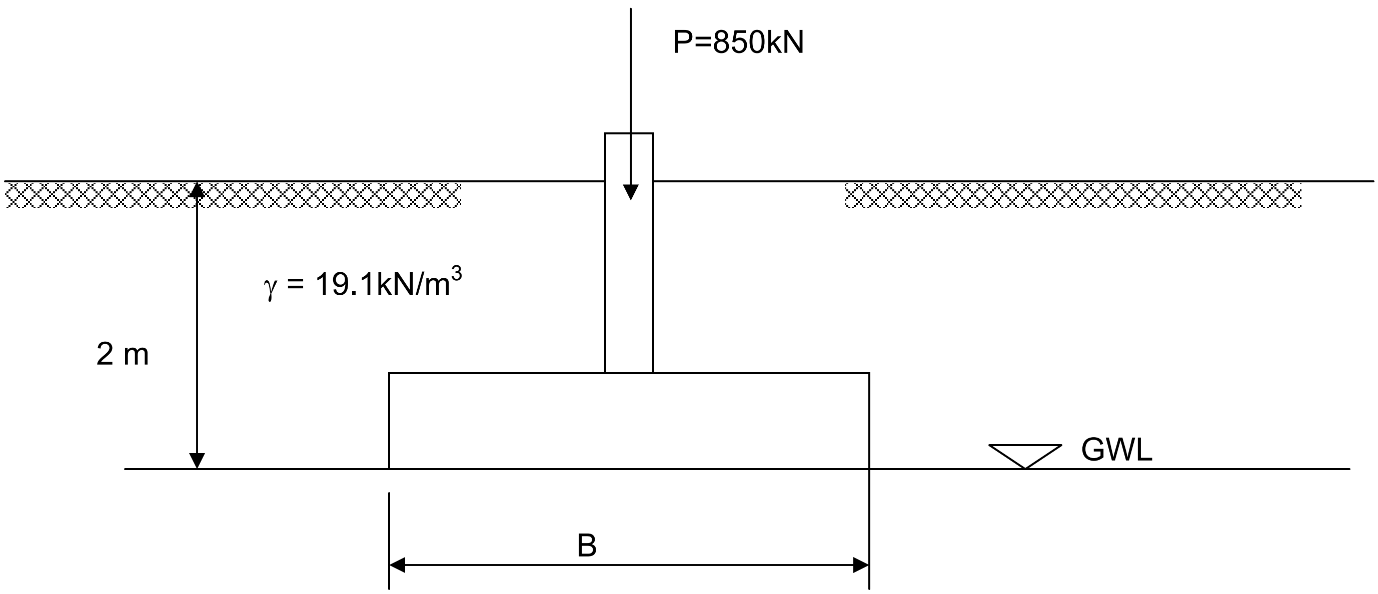
The spacing between secondary bars shall not exceed 400mm

Examples

4.1 Determine the dimensions of a square footing necessary to sustain an axial column load of 850kN as shown in Fig. below, if

a) an allowable presumptive bearing pressure of 150kN/m^2 is used.

b) Cu = 40 kN/m^2 ; C' = 7.5 kN/m^2 ; ϕ' =22.5^0



Solution

a) Using presumptive value

$$A = \frac{P}{\sigma_{as}} = \frac{850}{150} = 5.67m^2 = B^2$$

The dimension of the footing would be 2.40m X 2.40m

b) Using the bearing capacity formula

i) Initial loading condition

$$\sigma_f = 5.1Cu\ S_c\, d_c\, i_c + q\ S_q\, d_q\, i_q$$

Shape factors

Sc = 1.2, Sq = 1

Depth factors

dc = (1+0.4(2/B)) , dq = 1

Load inclination factors

ic = 1 , iq = 1

Hence

σ_f = 5.1*40 *1.2*(1+0.8/B)*1+ 19.1*2*1**1*1 = (244.8+195.84/B +38.2)

A σ_f = Pη

$$A = \frac{P\eta}{\sigma_f} = \frac{(850)2}{253 + 195.84/B} = B^2$$

$253B^2 + 195.84B - 1700 = 0$

The dimension of the footing would be 2.25m X 2.25m

ii) Final or long term loading condition

$$\sigma_f = CN_c\, S_c d_c i_c + ½\, B'\, \gamma\, N_\gamma\, S_\gamma d_\gamma\, i_\gamma + q\, N_q\, S_q d_q i_q$$

Bearing capacity factors

N_c= 17.45, N_γ = 6.82, N_q = 8.23

Shape factors

$S_c = 1+(N_q/N_c)$=1.47, S_γ = 0.6 , Sq = 1+ tan ϕ= 1.41

Depth factors

d_c = 1+ 0.4 (2 / B)=1+0.8/B

d_γ = 1

d_q = 1+2 tan 22.5(1-sin22.5)^2(D_f / B) =1+0.63/B

Load inclination factors

ic = 1, i_γ= 1 , iq = 1

Hence

σ_f = 7.5*17.45*1.47*(1+0.8/B')*1+ ½ B' *9.1* 6.82* 0.6*1*1 + 19.1*2*8.23* 1.41*(1+0.63/B)*1 = 192.39 +153.91/B +18.62B +443.28 + 279.27/B

A σ_f = Pη

$$B^2 = \frac{P\eta}{\sigma_f} = \frac{850 * 2}{635.67 + \left(433.18/B\right) + 18.62B}$$

$$18.62*B^3 + 635.67*B^2 + 433.18*B = 1700$$

From the above the dimension of the footing would be 1.35m X1.35m

4.2 Given R.C. column size 30X50 cm with 4ϕ22.

P = 1000kN

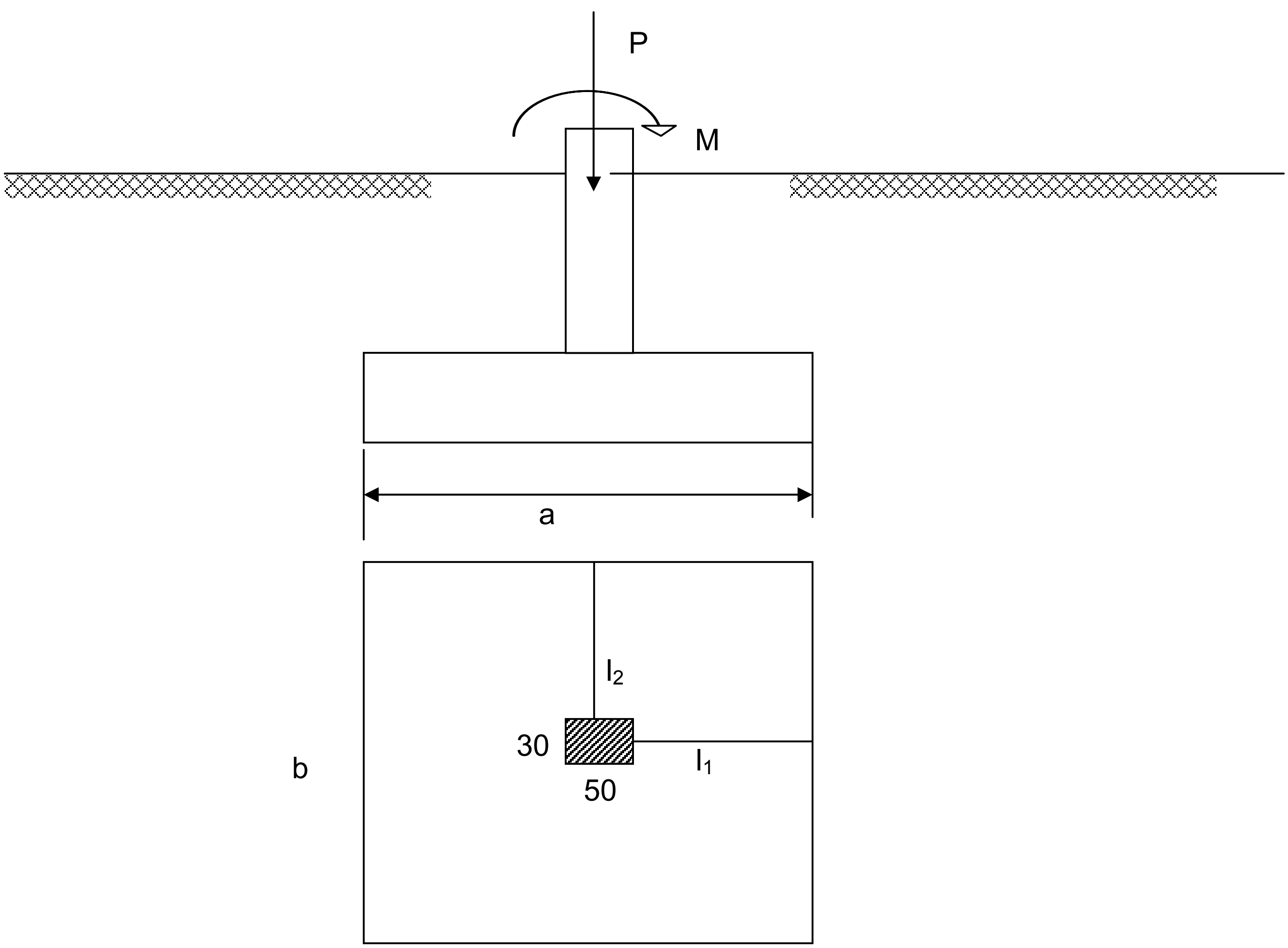
M = 250 kN-m

Net allowable soil bearing pressure = 400kPa

f_{yk} = 300MPa$\Rightarrow$ fyd = 300/1.15 = 260.87 MPa

C25 $\Rightarrow$fck= 20MPa$\Rightarrow$$f_{ctk}$ = 1.5 MPa,

Required:- Design of rectangular R.C. footing



Solution

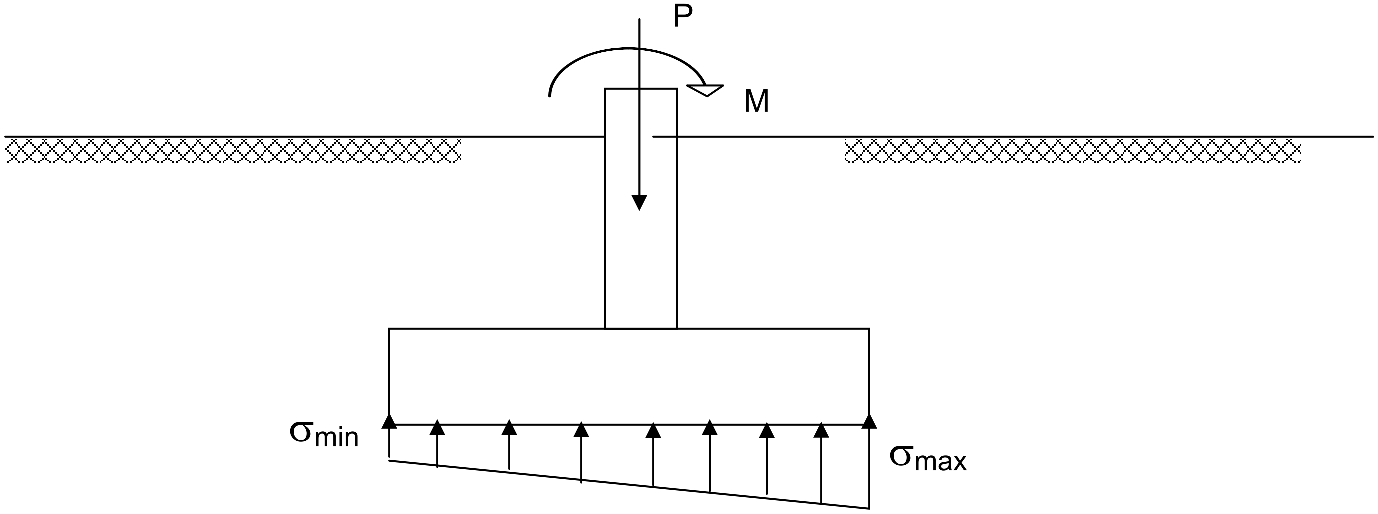
Size of footing

Let $l_1 = l_2$

Then $\frac{a-50}{2} = \frac{b-30}{2} \Rightarrow a-b = 50-30 = 20cm = 0.2m$

Eccentricity, $e_a = \frac{M}{P} = \frac{250}{1000} = 0.25m$

Contact pressure



$$\sigma_{\max} = \frac{P}{A}\left(1+\frac{6e_a}{a}\right) = \frac{P}{ab}\left(1+\frac{6e_a}{a}\right)$$

$$400 = \frac{1000}{(0.2+b)b}\left(1+\frac{6*0.25}{(0.2+b)}\right)$$

$$400(0.20b+b^2) = 1000+\frac{1500}{(0.2+b)}$$

$$400b^3+160b^2-984b-1700=0$$

by trial and error b= 1.96 m

Take b= 2m

Then a = b+0.20m = 2.20m

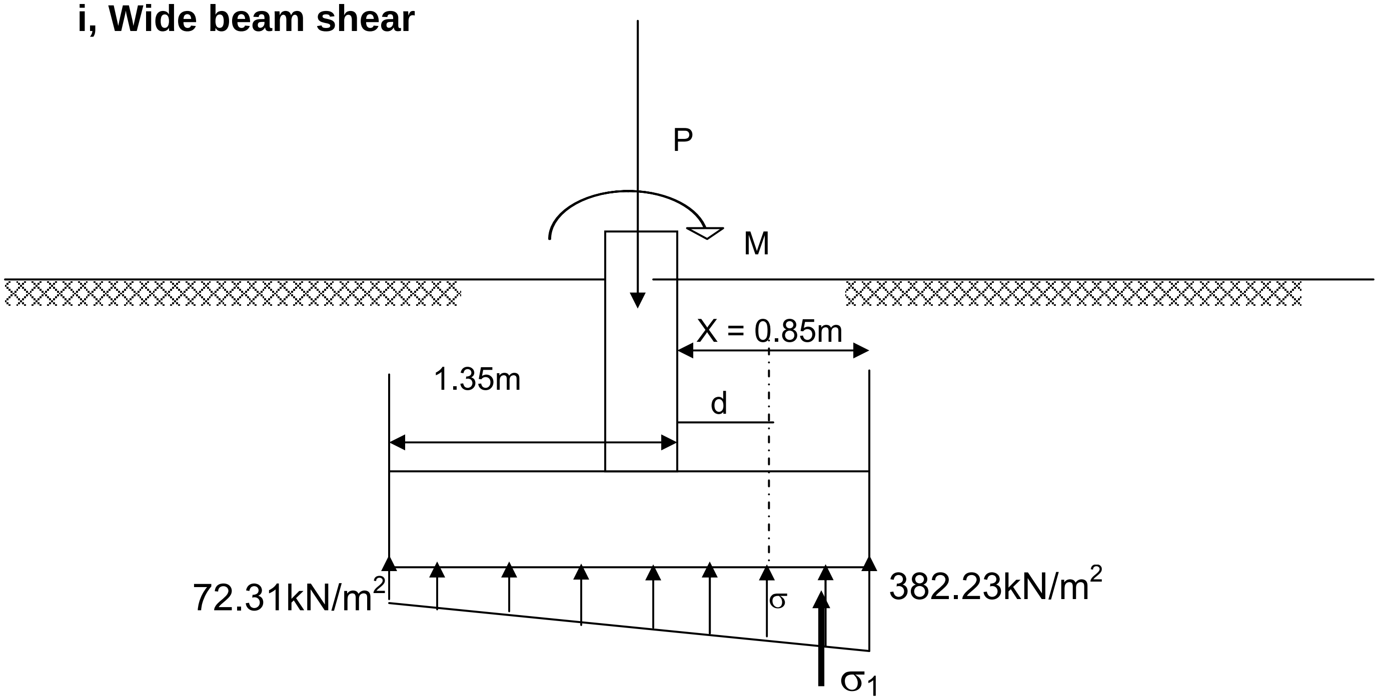
Actual contact pressure

$$\sigma_{\max} = \frac{1000}{(2.2)(2)}\left(1+\frac{6*0.25}{(2.2)}\right) = 382.23kN/m^2 < \sigma_{all} \quad ok$$

$$\sigma_{\min} = \frac{1000}{(2.2)(2)}\left(1-\frac{6*0.25}{(2.2)}\right) = 72.31kN/m^2 > 0 \quad ok$$

Thickness of footing

i, Wide beam shear



Contact stress at distance d from the face of the column, σ

$$\sigma = 72.31 + \frac{(382.23-72.31)(1.35+d)}{2.20}$$

$$\sigma = 262.48 + 140.87d$$

$$\sigma_1 = \left(\frac{\sigma_{\max} + \sigma}{2}\right)(0.85 - d) = \left(\frac{382.23 + 262.48 + 140.87d}{2}\right)(0.85 - d)$$

$$= 274 - 262.49d - 70.44d^2$$

$$V_d = \frac{V}{bd}\ , \qquad V = \left[247 - 262.49d - 70.44d^2\right]b$$

$V_d = 0.3f_{ctd}\,(1+50\rho) \leq 0.02$

For C25 $\Rightarrow$fck= 20MPa$\Rightarrow$ f_{ctk} = 1.5 MPa,

fcd = 0.85*fck /γc =0.85 *20/1.5 = 11.33 MPa

f_{ctd} = f_{ctk} /γc = 1.5/1.5 =1MPa

Assume ρ=0.02

Then

V_d = 0.3*1.0 (1+50*0.02)= 0.6 MPa =600kPa

$$600 = \frac{\left(274 - 262.49d - 70.44d^2\right)2}{2 * d}$$

$$140.88d^2 + 1462.49d - 548 = 0$$

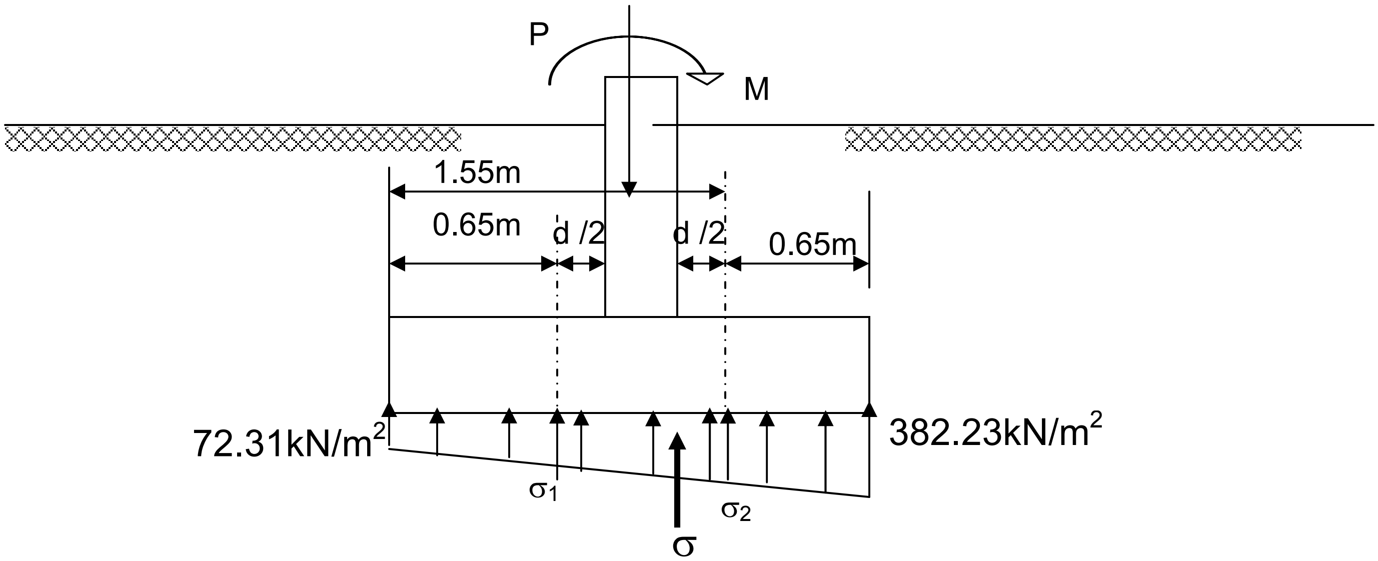
d = 0.36m

take d = 0.40m

ii, Punching shear

Allowable punching shear $V_p = 0.5f_{ctd}\,(1+50\rho_e)$, assume ρ_e = 0.008

V_p = 0.5*1(1+50*0.008)=0.7MPa = 700kPa



$$\sigma_1 = 72.31 + \frac{0.65*(382.23-72.31)}{2.20} = 163.88 kN/m^2$$

$$\sigma_2 = 72.31 + \frac{1.55*(382.23-72.31)}{2.20} = 290.66 kN/m^2$$

$$\sigma = \frac{\sigma_1+\sigma_2}{2}*0.90 = \frac{163.88+290.66}{2}*0.90 = 204.54 kN/m$$

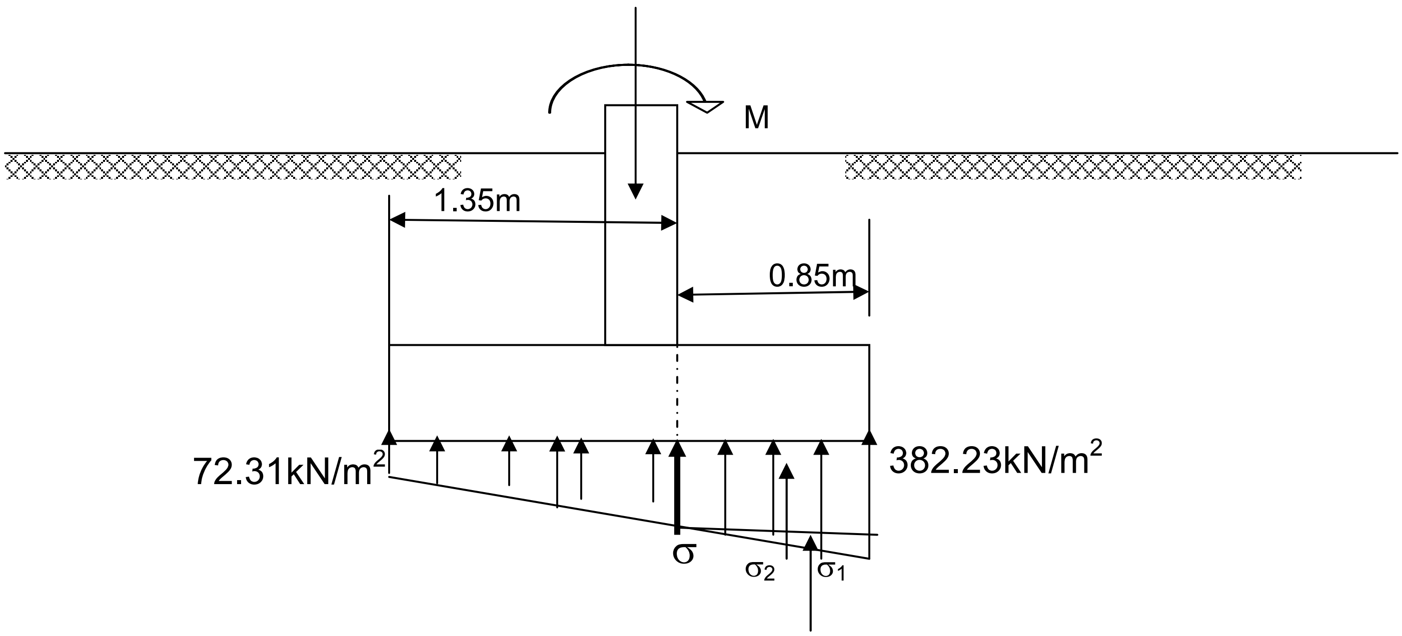
$$V = 204.54*2.00 = 409.08 kN$$

Net shear force developed = 1000 – 409.08 = 590.92kN

Punching stress,

$$\frac{V}{b_p d} = \frac{590.92}{2(0.90+0.7)0.40} = 461.66 kN/m^2 < 700 kPa \quad ok$$

Bending Moment



$$\sigma = 72.31 + \frac{1.35*(382.23-72.31)}{2.20} = 262.49kN/m^2$$

$$\sigma_1 = \frac{1}{2}(0.85)(382.23-262.49) = 50.93kN/m$$

$$\sigma_2 = \frac{1}{2}(0.85)(262.49) = 223.12kN/m$$

$$M = \left[\sigma_1 \frac{2}{3}(0.85) + \sigma_2 (\frac{0.85}{2})\right] b$$

$$M = \left[(50.93)\frac{2}{3}(0.85) + (223.12)(\frac{0.85}{2})\right] 1 = 123.69kN-m/m$$

Moment capacity of concrete

$$M = 0.32 * f_{cd} * bd^2$$
$$= 0.32 * 11.33 \times 10^3 * 1.0 * (0.4)^2 = 1450.24kN-m/m$$

Calculation of reinforcement

Long direction

$$\rho = \frac{f_{cd}}{f_{yd}}\left[1-\sqrt{1-\frac{2M}{f_{cd}bd^2}}\right]$$

$$= \frac{11.33}{260.87}\left[1-\sqrt{1-\frac{2*123.69}{11.33\times10^3*1.0*(0.4)^2}}\right] = 0.0031 > \rho_{\min}$$

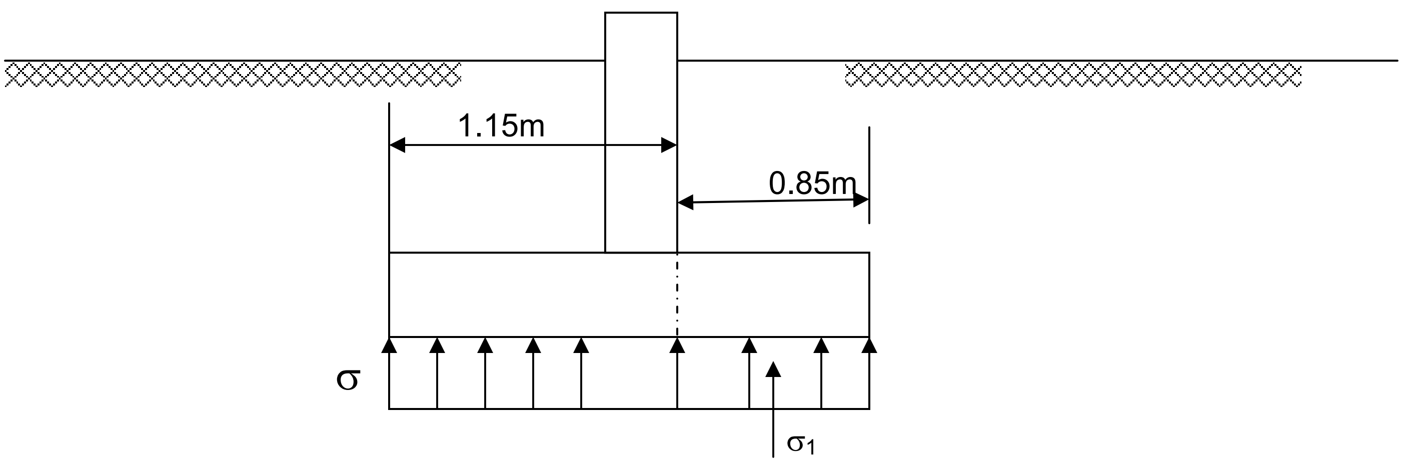
$$As = \rho bd = 0.0031*100*40 = 12.4cm^2/m$$

use ϕ16

$$spacing = \frac{as*100}{As} = \frac{2.01*100}{12.4} = 16.2cm$$

Use ϕ16c/c16cm

Short direction



average contact pressure ,σ

$$\sigma_{avg} = \frac{\sigma_{\max}+\sigma_{\min}}{2}$$

$$\sigma_{avg} = \frac{382.23+72.31}{2} = 227.27kN/m^2$$

$$M = \left[\sigma_1 \left(\frac{0.85}{2}\right) \right] a$$

$$M = \left[227.27 \left(\frac{0.85}{2}\right) \right] 1 = 96.59\, kN - m/m$$

$$\rho = \frac{f_{cd}}{f_{yd}} \left[1 - \sqrt{1 - \frac{2M}{f_{cd} b d^2}} \right]$$

$$= \frac{11.33}{260.87} \left[1 - \sqrt{1 - \frac{2 * 96.59}{11.33 \times 10^3 * 1.0 * (0.384)^2}} \right] = 0.0026 > \rho_{\min}$$

$$As = \rho bd = 0.0026 * 100 * 38.4 = 9.98 cm^2/m$$

$$spacing = \frac{as * 100}{As} = \frac{2.01 * 100}{9.98} = 20.1 cm$$

Use ϕ16c/c20cm

Since there is no much difference between a and b, distribute these reinforcement uniformly.

Development length

$$l_d = \frac{\phi\, f_{yd}}{4 f_{bd}}$$

$$f_{yd} = f_{yk} / \gamma_s = 260.87 MPa \quad ; f_{bd} = f_{ctd}$$

$$f_{ctd} = \frac{0.35 \sqrt{f_{ck}}}{\gamma_c} = \frac{0.35 \sqrt{20}}{1.5} = 1 MPa$$

$$l_d = \frac{\phi\, f_{yd}}{4 f_{bd}} = \frac{1.6 * 260.87}{4 * 1} = 104.35 cm$$

$l_{davailable}$ = 85cm < ld , bend the bars upward with a length of 30cm

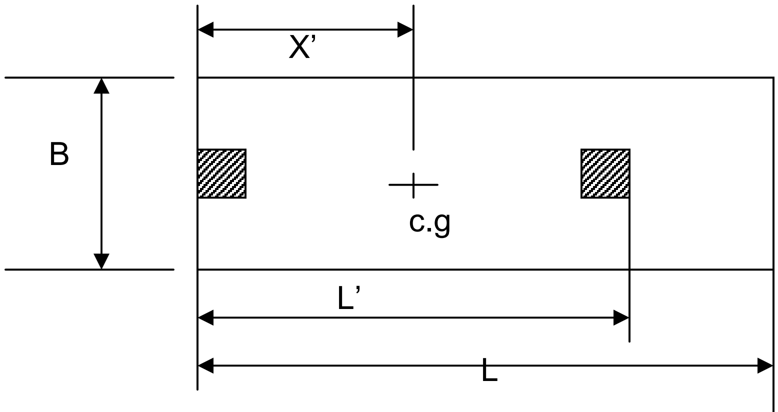
4.3 Combined Footing

A) Rectangular Combined footing

a) Area of use :- Used to carry two or more columns in one row

-used to carry two columns when X' = L'/2,

X'= distance to center of gravity of column load

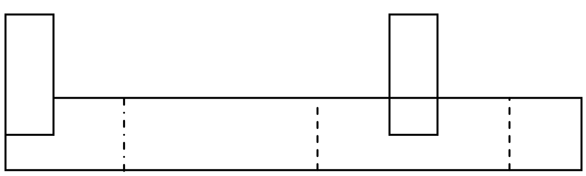


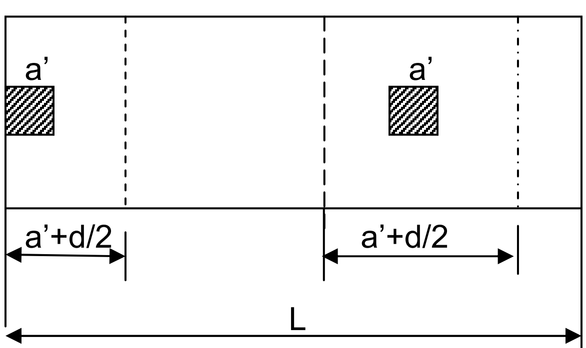
b) Design Assumptions :- footing is infinitely rigid

Linear soil pressure distribution under footing

c) Analysis: - In the long direction, it is analyzed as a continuous beam

In the short direction, it is analyzed as spread footing with effective widths at exterior and interior columns being a' +d/2 and a' +d respectively





d) Design procedure

i) determine length of footing (L) in such a way that the center of gravity(c.g.)of footing area coincides that of the c.g. of loads

i.e., L = 2x'

ii) determine the width of footing(B) such that the allowable soil pressure is not exceeded

i.e., $$B = \frac{\sum P}{L\sigma_{all}}$$

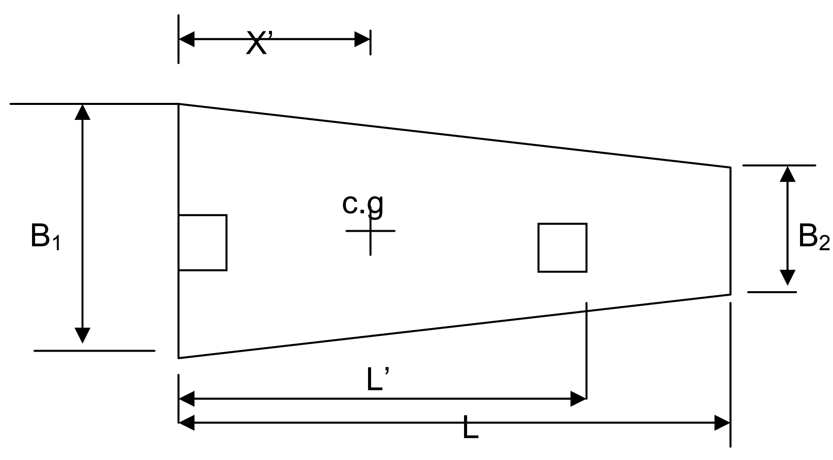
iii) determine and draw shear force and bending moment diagrams along the length of the footing

iv) calculate depth of footing

v) calculate steel reinforcement for bending moment requirement

B) Trapezoidal combined footing

Area of use:- used in case where exterior column carries largest load and X' < L'/2 but X' > L'/3



a) Design Assumptions :- footing is infinitely rigid

Linear soil pressure distribution under footing

b) Analysis: - In the long direction, it is analyzed as a continuous beam

In the short direction, it is analyzed as spread footing similar to that of rectangular combined footing.

c) Design procedure

1) determine the sizesof footing (L,B_1,B_2) from conditions that

i) the minimum required are

$$A=\frac{\sum P}{\sigma_{all}}$$

$$A=\left(\frac{B_1+B_2}{2}\right)L$$

ii) the c.g. of footing are coincides that of column loads. The distance to the c.g. of trapezoidal footing x' is calculated from

$$X'=\frac{L}{3}\left(\frac{2B_2+B_1}{B_2+B_1}\right)$$

2) determine and draw shear force and bending moment diagrams along the length of the footing. In this case, the shear force and bending moment diagrams are 2nd degree and 3rd degree curves, respectively.

3) calculate depth of footing

4) calculate steel reinforcement for bending moment requirement

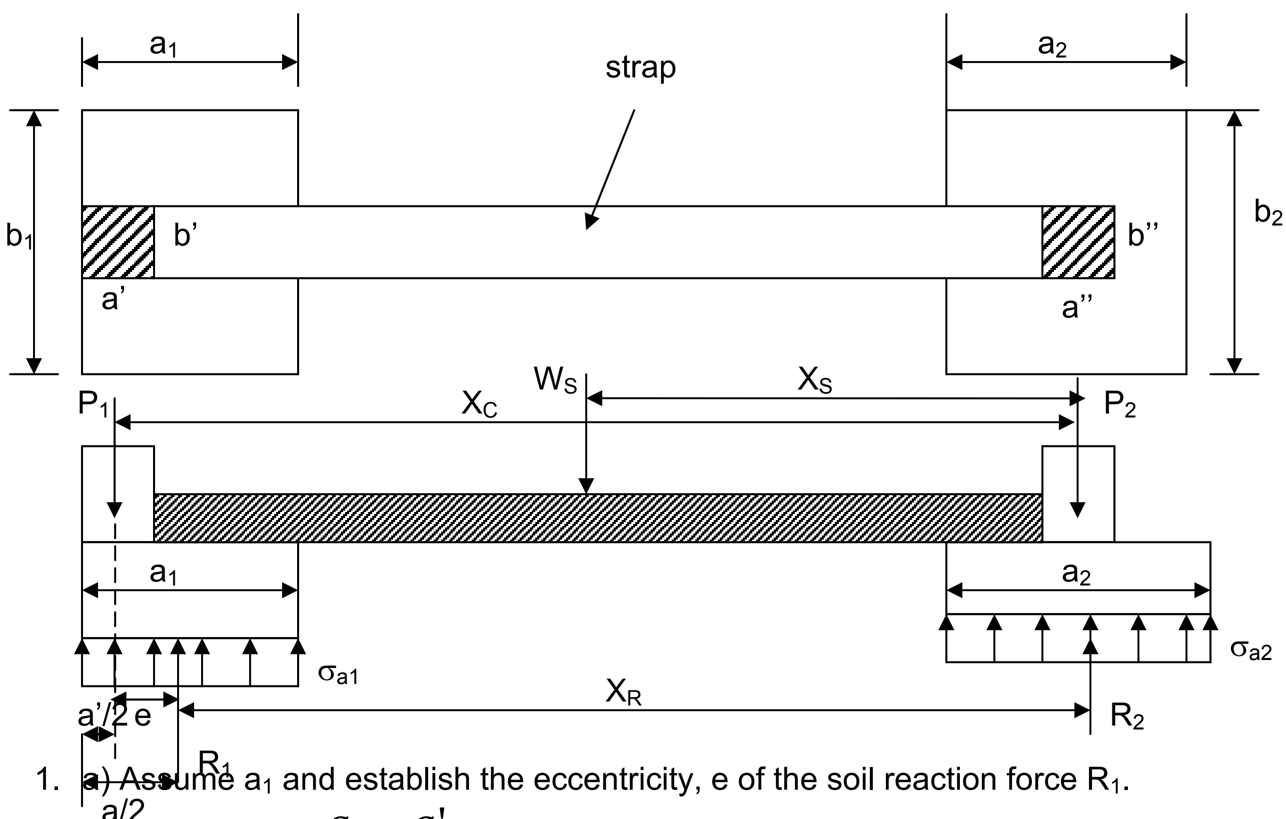
4.4 Strap or Cantilever Footing

Strap footings are used as alternatives to combined footings when the cost of combined footings is relatively high.

Essentially a strap footing consists of a rigid beam connecting two pads (footings) to transmit unbalanced shear and moment from the statically unbalanced footing to the second fotting.

Design Assumptions

- strap is infinitely rigid
- strap is a pure flexural member and does not take soil reaction. (To confirm with this, strap is constructed slightly above soil or soil under strap is loosened).



1. a) Assume a_1 and establish the eccentricity, e of the soil reaction force R_1.

$$e = \frac{a_1 - a'}{2}$$

$$e = X_C - X_R$$

b) Determine the magnitude of the soil reaction force by taking moments about R_2.

$$R_1 = P1\frac{X_c}{X_R} + Ws\frac{X_s}{X_R}$$

In this equation the weight of the strap, Ws, may be neglected if the strap is relatively short.

c) Determine the reaction R_2 from equilibrium consideration

$$R_2 = P_1 + P_2 + W_s - R_1$$

2. Determine sizes of footings using known values of R_1, R_2 and σ_{all}.

$$b_1 = \frac{R_1}{\sigma_{a1} * a_1}$$

$$b_2 = \frac{R_2}{\sigma_{a2} * a_2}$$

(For square footing $b_2 = a_2 = \sqrt{\dfrac{R_2}{\sigma_{a2}}}$. For rectangular footing assume some value of a_2 and determine b_2).

It should be noted that the actual bearing pressures under the footings should not very different from each other in order to minimize differential settlement.

3. Determine and draw shear force and bending moment diagrams along the length of the footings.
4. Select depths of footings for shear requirement.
5. Select steel reinforcement for bending requirement.
6. In short direction, the footings analyzed as spread footing subject to uniform soil pressure.

7. Design strap as flexural member for the shear and moment obtained above.