CHAPTER ONE
SOIL EXPLORATION

1.1. Purpose of Soil Exploration

The purpose of soil exploration is to find out strength characteristics of the sub-soil over which the structure has to be built.

The main purposes of soil exploration are: -

- Selection of alternative construction sites or the choice of the most economical sites.
- Selection of alternative types or depth of foundation
- Selection of alternative methods of construction.
- Evaluation of the safety of existing structure.
- Location and selection of construction materials.
- The soil exploration should provide the following data: -
- Soil parameters and properties of different layers (e.g. for classification, bearing capacity or settlement calculation)
- Thickness of soil layers and depth to bedrock (stratification of soil)
- Location of ground water level

1.2. Planning Soil Exploration Program

The planning of a program for soil exploration depends upon

- The nature of sub-soil
- The importance of structure
- The type of structure

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

a) Assembly of all available information on type and use of the structure, and also of the general topographic and geological character of the site.
b) **Reconnaissance of the area: -** This involves inspection of behavior of adjacent structures, rock outcrops, cuts, etc.
c) **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.
d) **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

1. Test pits
2. Boring and sampling
3. Field tests
4. Geophysical methods
5. Laboratory test

I. Test Pits

The simplest and cheapest method of shallow soil exploration is to sink test pit to depths of 3 to 4 m. Test pits enable the in-situ soil conditions to be examined visually. It is relatively easy to obtain disturbed or undisturbed soil samples:

II. Soil Boring and Sampling

a) 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: -

1) Auger boring
2) Wash boring
3) Percussion drilling
4) Rotary drilling

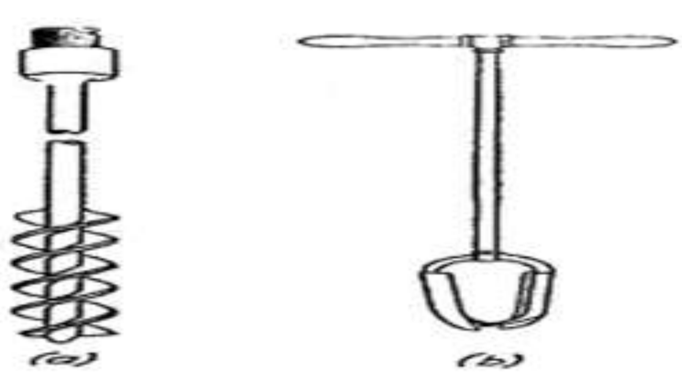
1) Auger boring: -

Operated by hand or by power. Hand operated augers, ϕ= 15 to 20cm, are of two types.

a. Post-hole and helical augers.

b. 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.



Hand Augers a) helical and b) post hole

2) 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 rise 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.

3) 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.

4) Rotary drilling: -

- Power operated.
- Hole is advanced by a rapidly rotating bit
- 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

b) Soil Sampling:- Laboratory test results are mainly dependent on the quality of soil samples. There are two main types of soil samples which can be recovered from bore holes or trial pits.

- Disturbed and
- Undisturbed samples

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. However, these samples represent the composition and the mineral content of the soil. Disturbed samples are satisfactory for performing classification tests such as, sieve analysis, Atterberg limits etc

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.

It is virtually impossible to obtain totally undisturbed samples. This is due to that:

- The process of boring, driving the coring tool, 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 ends should be sealed with wax and capped to preserve the loss of moisture content
- Core tubes should properly be labeled to indicate the number of bore holes and the depth at which they are taken and then stored away from extremes of heat or cold and vibration.

Types of tube samplers used are:

- Split Spoon Sample
- `Piston Samplers
- Thin-Walled Tube Sampler

(a)

(b)

(c)

III. 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: -

a) Penetration or sounding tests
b) Vane shear test

c) Plate loading test   d) Pile loading test

a) Penetration 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.

i) Static Penetration Tests.

1) Swedish Weight Sounding Test: -

This method of testing is widely used in Scandinavia and here in Ethiopia.

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.

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/m3)**
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/m3)**
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 (⅝" ϕ), casing pipe (ϕ ¾").

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.

- The cone is 1st 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.

Correlation 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**

According to Meyerhof:

$$N = \frac{1}{4}(Ckd)$$

where N = Standard penetration number

Ckd = Static Cone resistance (kg/cm2)

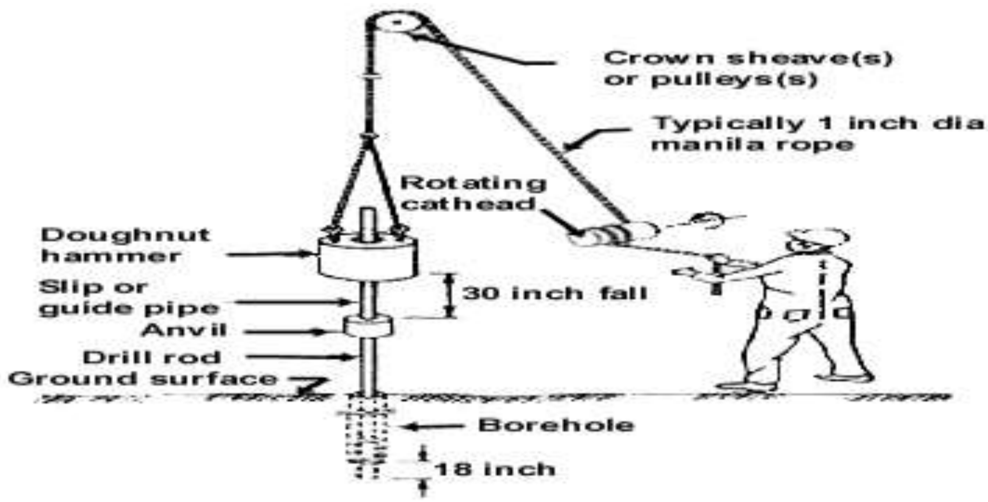
For sand, modulus of compressibility (Es) can be estimated from cone resistance from the following relationship.

$$E_S = 3/2(Ckd)$$

ii) 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. 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.



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

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**
q_u(kN/m^2)	0 -25	25 -50	50 -100	100 -200	200-400
Consistency	Very soft	Soft	Medium	Stiff	Very stiff

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

$$\phi^0=25+0.15Dr$$

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

$$\phi^0=30+0.15Dr$$

For granular soil, containing less than 5 percent fine sand and silt. In the equations Dr 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.

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

$N = 15 + ½ (N' – 15)$ Suggested by Terzaghi and peck

Where N = corrected value

N' = Recorded value

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

$$N = \leq 2N', \text{ for } \sigma_o' \leq 280 \text{ kN/m}^2$$

σ_o' = effective overburden pressure in 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 Nc 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.

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.

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

$$N_c = N/C$$

Where

N = blow count for SPT

Nc = blow count for dynamic cone

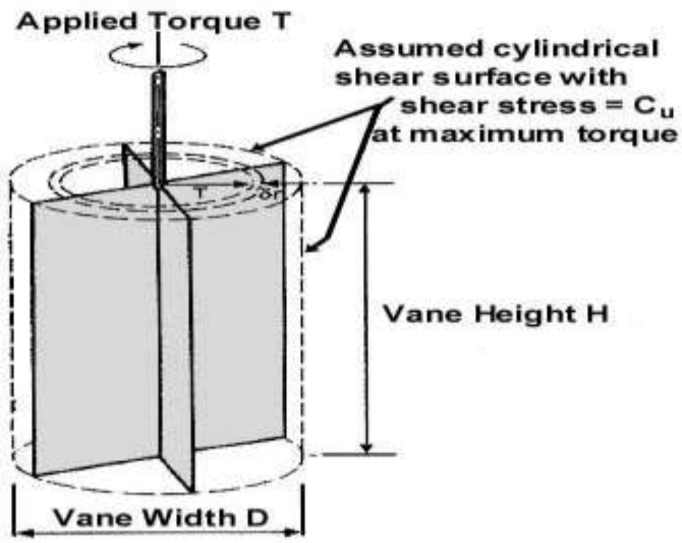
C = Constant, lies between 0.8 and 1.2 when bentonite is used.

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 cohesion less soils like SPT

b) Vane Shear Test: It is used to determine the un drained shear strength of soft clays soils. The apparatus consists of a vertical steel rod having four thin stainless steel blades (vanes) fixed at its bottom ends. Vane head (torsion head), complete with pointer, stop pin, circumferential graduated scale, calibrated torsion spring



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

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

where T = Torque

D = Diameter of Vane

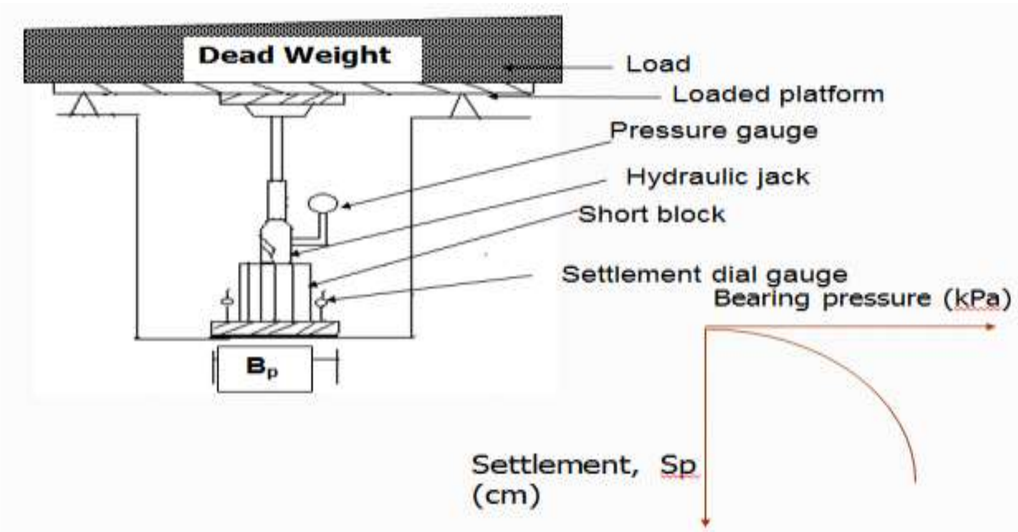
H = Height

c) 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.

The test procedure

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 cohesion less 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 Bf is given, the settlement, Sp, of a plate of width Bp under the same intensity of loading is given by

$$\boxed{S = \frac{Sp(2Bf)^2}{(B_f + B_p)^2}}$$

Using the value Sp, 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 fo

$$\boxed{S = S_p \frac{B_f}{B_p}}$$

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 be recommended in soils which are not homogeneous at least to depth of 1½ to 2 times the width of the prototype foundation

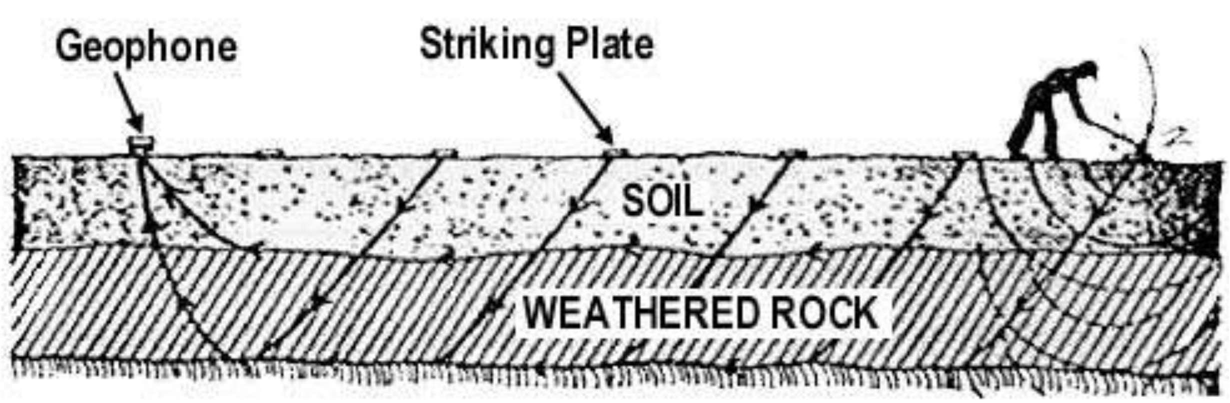
d) Pile Loading Test

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.

IV. GEOPHYSICAL METHODS

These comprise the seismic and resistivity methods.

a. **Seismic Method:-** In this method shock or seismic are created by detonating small charges or by striking a rod or a plate near the surface. The radiating waves are picked up and time of travel from source recorded by detectors known as geophones or seismometers. Seismic method is based on the fact that sound waves travel faster through rocks than through soils.



b. **Electrical Resistivity: -** In this method four metallic spikes to serve as electrodes are driven in to the ground at equal intervals along a line. A known potential is then applied between the outermost electrodes and potential drop is measured between the innermost electrodes.

c. 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). Seismic and resistivity methods are normally employed as preliminary or supplementary to other methods of exploration.

V. LABORATORY TESTS

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

1.4. 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. 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.

1.5. Depth And Number Of Borings.

1.5.1. 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.

1. For widely spaced strip of pad foundations, boring depth should be deeper than 1.5 times the width of the foundation.
2. For raft foundations, boring depth deeper than 1.5 times width of raft should be used.
3. 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.
4. 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.
5. For piled foundation on rock, boring depth should be deeper than 3.0m inside bedrock.
6. 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.

1.5.2. Number of Borings

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	

1.6. 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.

1.7. Soil Exploration Report

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.

CHAPTER TWO

2. FOUNDATION TYPES AND THEIR SELECTIONS

Definition: Foundation is the lowest artificially built part of a structure which transmits the load of the structure to the ground. The foundation of a structure is always constructed below ground level to increase the lateral stability of the structure.

2.1. Purposes of Foundations

Foundations are used for the following purposes:

- 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.
- To load the bearing surface at a uniform rate so as to prevent unequal settlement.
- To prevent the lateral movement of the supporting material.
- To secure a level and firm bed for building operations.
- To increase both the overturning and sliding stability of the structure as a whole.

2.2.Types of Foundations

Foundations can be broadly classified into the following two categories based on depth:

- Shallow foundations
- Deep foundations

2.2.1. Shallow Foundations

Shallow foundations are provided immediately beneath the lowest part of the structure, near to the ground level. Shallow foundations are further classified into the following types:

- Spread or Isolated footings
- Continuous (strip) or wall footing
- Combined footing
- Raft (Mat) foundation
- Cantilever footing

A) Spread or Isolated Footings (simple, stepped or slope type

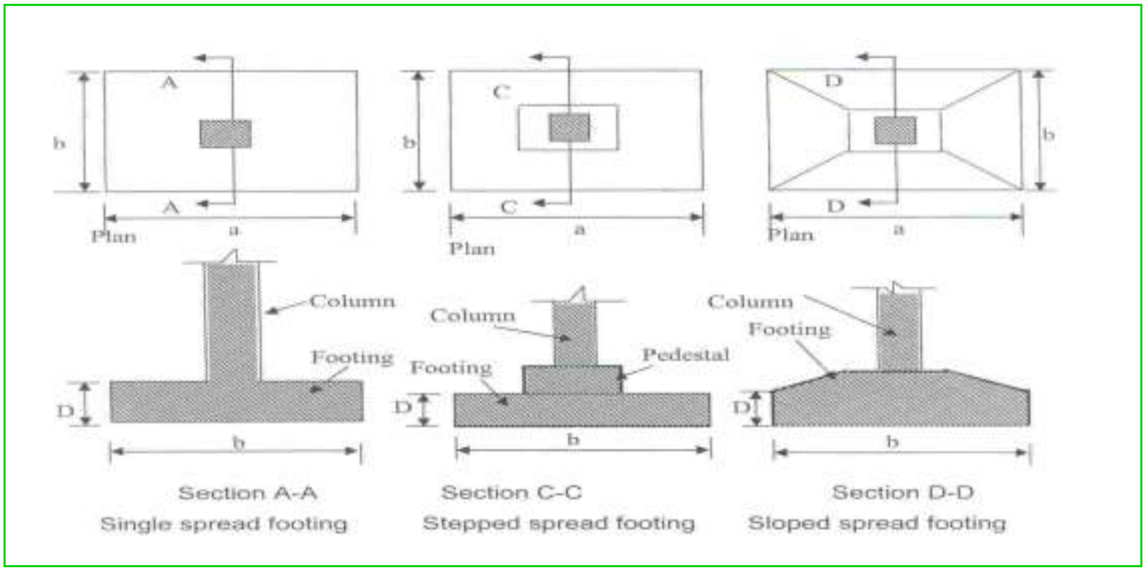


Figure: Types of isolated footing

Isolated footing is used when:

- The soil has sufficient strength within a short depth below the ground level.
- The soil has adequate bearing capacity.
- The super structural load is very small

The major advantages of selecting the isolated footings are:

- Economical when columns are placed at longer distances.
- Ease of Constructability- Excavation, Form-work, Reinforcement placement and placing of Concrete is at ease.
- Workmen with little or no knowledge can easily construct an Isolated Footing.
- Combined Footing

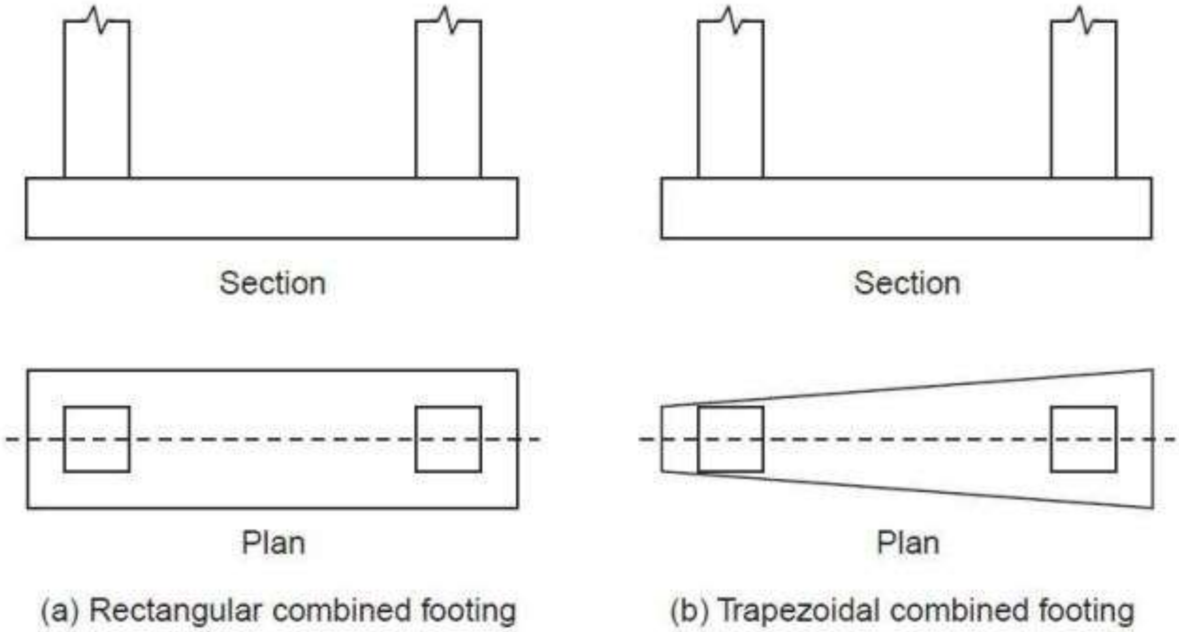
Combined footing is required:

- Whenever two columns are nearby together, inducing overlap of adjacent isolated footings
- Where soil bearing capacity is low, inducing overlap of adjacent isolated footings
- When column end is situated near the property line and the footing cannot be extended
- When the super structural load to be transferred to the foundation is very high

Types of Combined Footing:

- Rectangular: is provided when the load of the two or more columns combined together is the same in magnitude.

- Trapezoidal: Trapezoidal footing is provided when the load on one of the columns is larger than the other column.



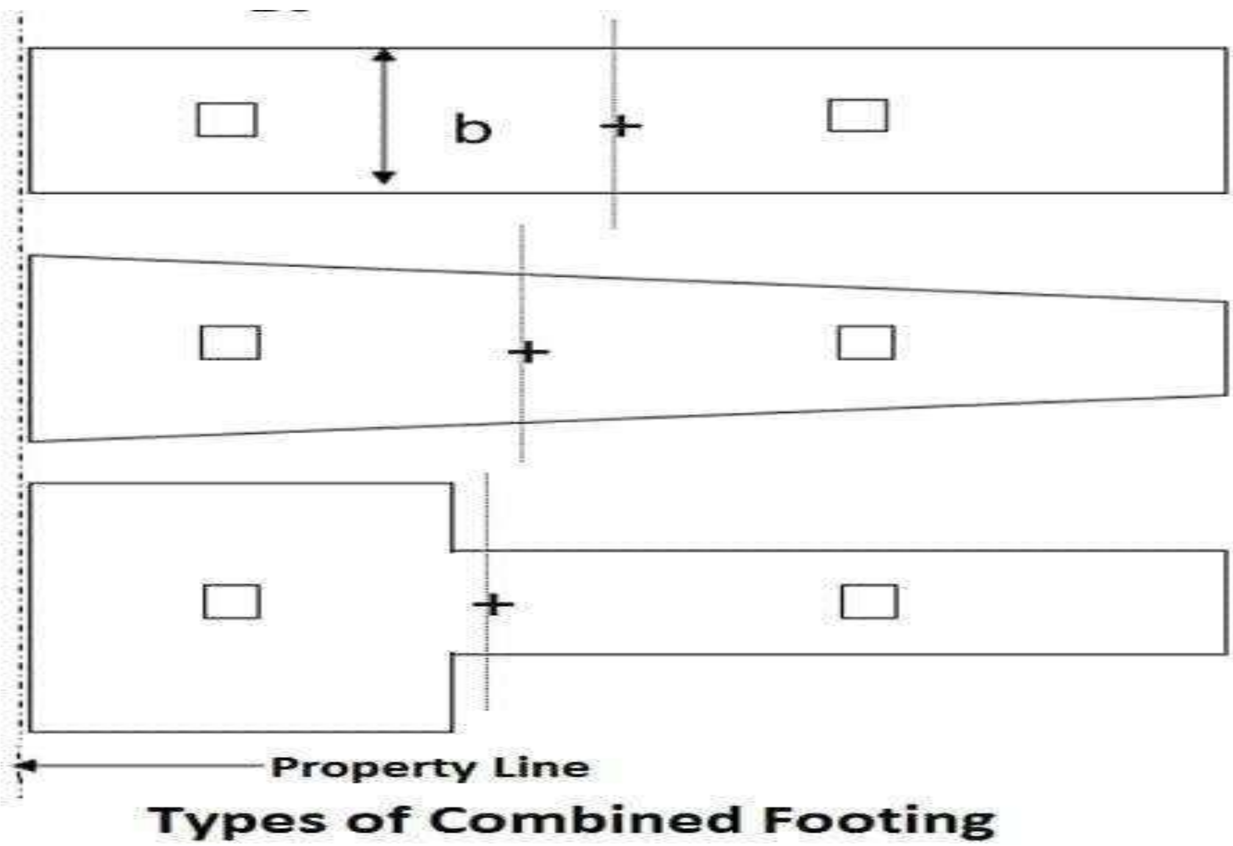


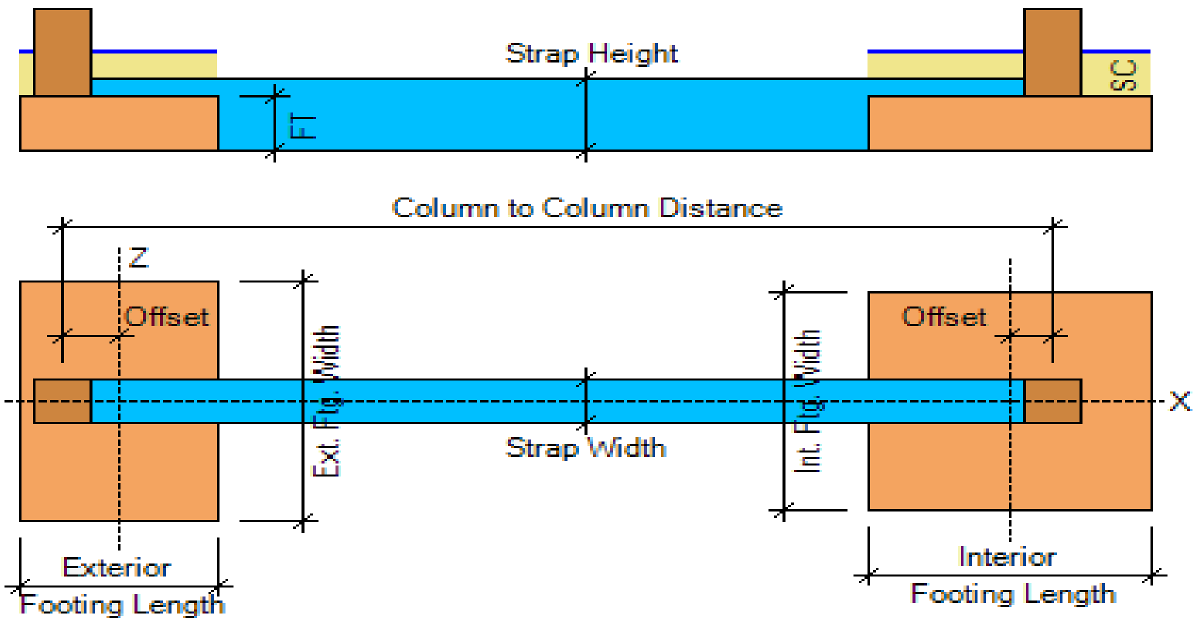
Figure: Combined footing types

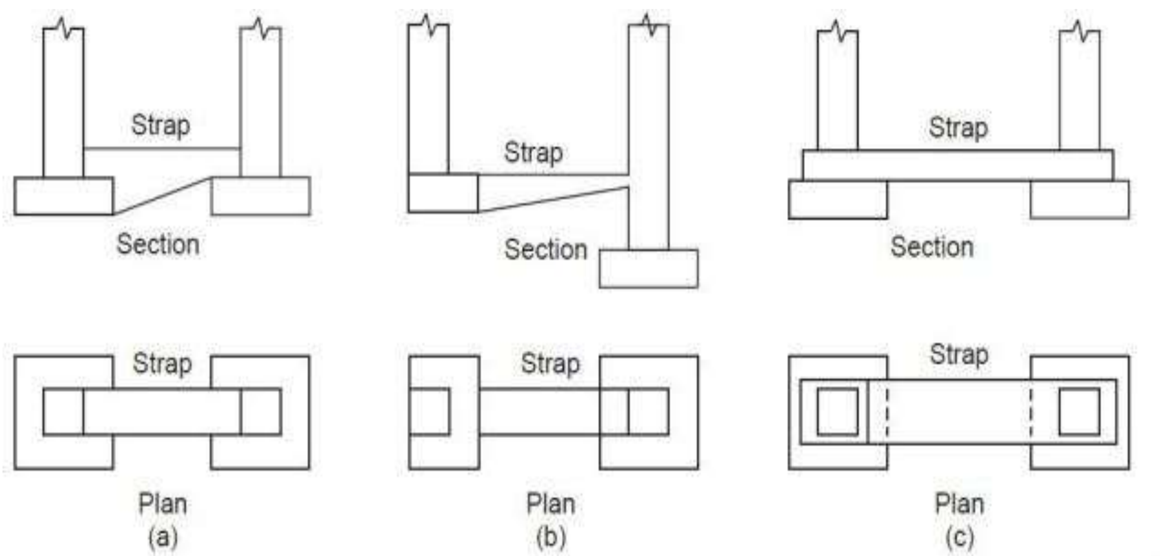
B) Cantilever or Strap Footing

Strap footing consists of two isolated footings connected with a structural strap. The strap connects the footing such that they behave as one unit.

A strap footing is more economical than a combined footing when:

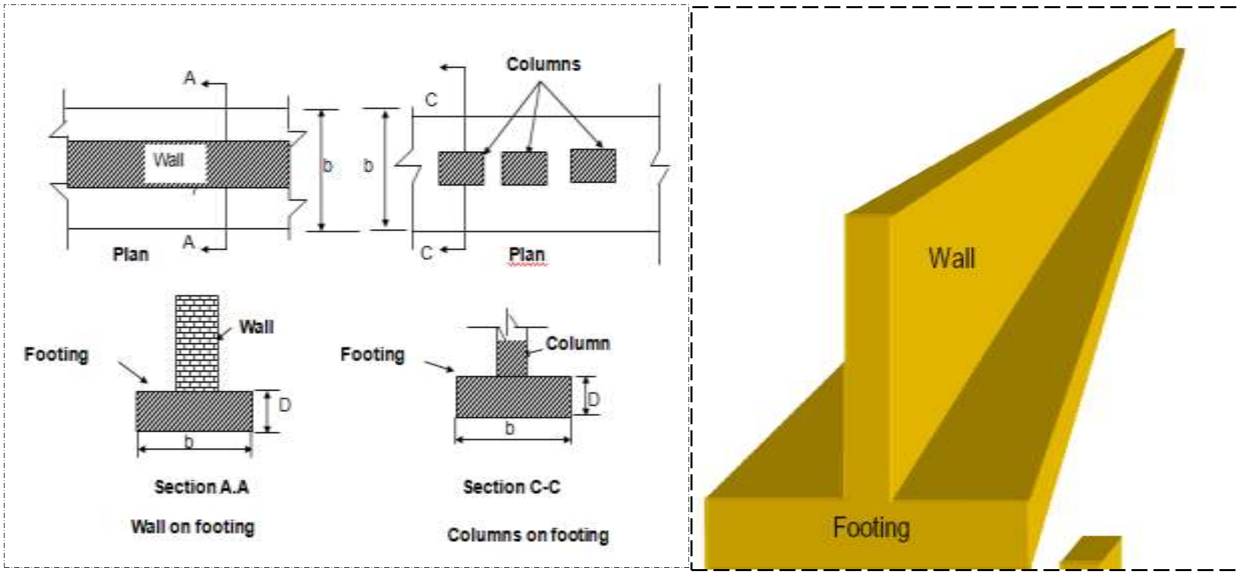
- The allowable soil pressure is relatively high and distance between the columns is large.
- A normal combined footing is impractical due to the required large excavation.





C) Continuous or wall footing or strip footing

A strip footing is another type of spread footing which is provided for a load bearing wall. A strip footing can also be provided for a row of columns which are so closely spaced that their spread footings overlap or nearly touch each other. In such cases, it is more economical to provide a strip footing than to provide a number of spread footings in one line. A strip footing is also known as continuous footing.



D) Mat (Raft) Foundation

A raft foundation is a solid reinforced concrete slab covering entire area beneath the structure and supporting all the columns. Such foundation due to its own rigidity minimizes differential settlements.

Raft Foundation is preferably used:

- When the column loads are heavy or when the safe bearing capacity of soil is very low
- In a places like seashore area, coastal area where the water table is very high
- In swampy areas dominated by soft soil
- When the columns and walls are so close that individual footings would overlap or nearly touch each other (when the area covered by the footing is greater than 50% of the total plan area)

The drawbacks of raft foundation:

- Sometimes, mat foundations need heavy reinforcement in certain areas, which can add up to the price of the manufacture.
- The edges of the mat foundation, if not properly take care of, may erode with time. However, when maintained well, these edges can last as long as the building it supports.
- Sometimes, the design can become very complex and thus, requires really skillful and experienced engineers as well as workers.
- Frost can have an adverse effect on the mat formation.

Figure: Typical View of Mat Foundation

2.2.2. Deep Foundations

Deep foundation is required basically;

- When the soil at or near the ground surface is not capable of supporting a structure, deep foundations are required to transfer the loads to deeper strata.
- Deep foundations are therefore, used:
- When surface soil is unsuitable for shallow foundation and a firm stratum is so deep that it cannot be reached economically by shallow foundations.

The most common types of deep foundations are piles, piers and caissons.

i) Pile: is a long (slender) vertical load transferring member made of timber, steel or concrete.

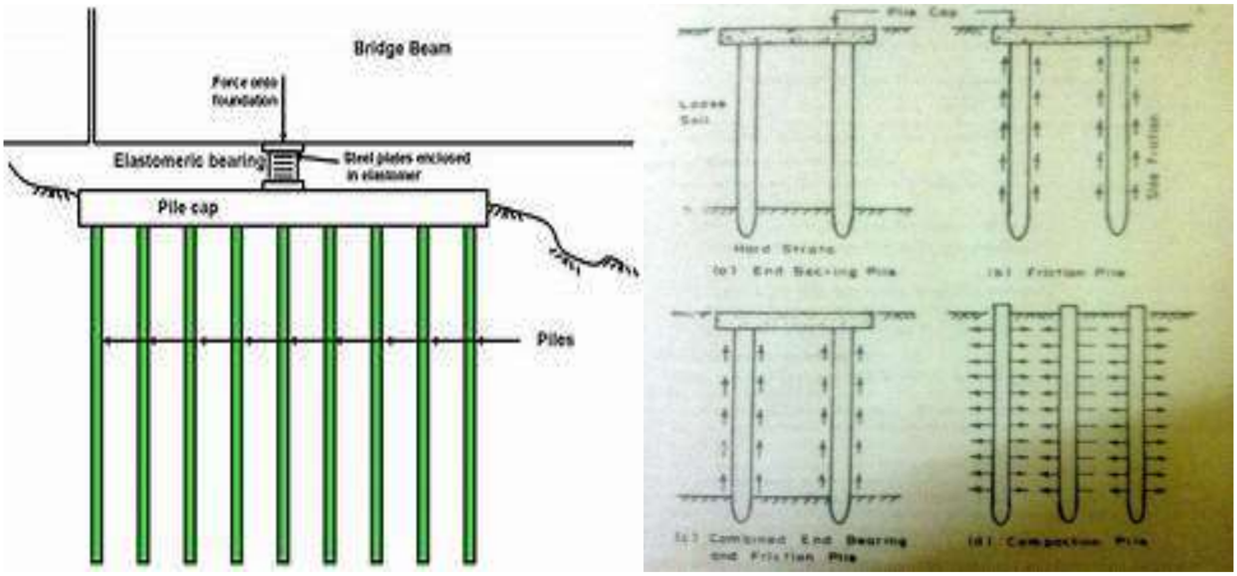


Figure: Typical View of Pile Foundation

Pile foundations are divided into two based on the type of construction;

- **Pre cast Piles:** Precast piles are casted at factory and transported to the site. This kind of piles is readymade and used where there are fewer places to cast pile. Precast piles are not economical and require more money to transport piles to the site.

- **Cast-in-situ piles:** The piles which are casted on site. And don’t require any transportation is called cast-in-situ piles. The cast-In-situ concrete piles are casted in position inside the ground and need not to be reinforced in ordinary cases.

ii) Pier: Pier is a vertical column of relatively large cross-section than a pile. Pier foundation is required:

- When the top strata consists of decomposed rock, overlying a strata of sound rock. In such conditions pile driving becomes very difficult, hence pier foundations are used.
- Also in the case of stiff clays, which offer a lot of resistance to driving of a bearing pile, pier foundations can be conveniently used.

iii) Caisson (Drilled shafts): A caisson is a type of foundation of the shape of hollow prismatic box, which is built above the ground and then sunk to the required depth as a single unit.

A pier and caisson differ basically only in the method of construction. The caissons, has action similar to pile foundations, but are high capacity cast-in-situ foundations. It resists loads from structure through shaft resistance, toe resistance and / or combination of both of these. The s done usi

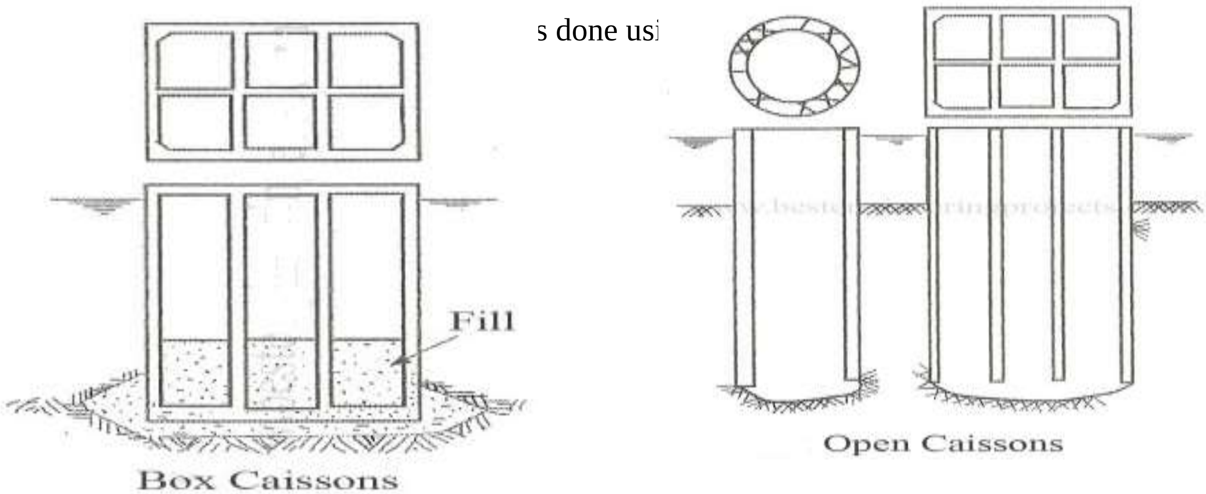


Figure: Typical view of Caisson Foundation

Advantages and Disadvantages of Caissons

- **Advantages of Caissons:**

- Economics
- Minimizes pile cap needs
- Slightly less noise and reduced vibrations
- Easily adaptable to varying site conditions
- High axial and lateral loading capacity

- **Disadvantages of Caissons:**

- Extremely sensitive to construction procedures
- Not good for contaminated sites
- Lack of construction expertise
- Lack of Qualified Inspectors

2.3. General Principles of Foundation Design

- The usual approach to a normal foundation-engineering problem is:

- 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.
- To study the tentative allowable bearing pressures allocated for the various strata below the ground level, as given by the soil investigation report.
- To determine the required foundation depth. This may be the minimum depth based on soil strength or structural requirement considerations.
- To compute the dimensions of the foundation based on the given loading and allowable bearing pressure.
- To estimate the total and differential settlements of the structure.

2.4. Loads on Foundation

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.

d) Earth-quake load: - lateral load coming from earth- quake motion.

e) Dynamic load: - load coming from a vibrating object (machinery**)**

2.4.1. Foundation Loading Types

There are two loading mechanisms of foundations:

a) **Concentrically loaded footing**

In this case the super structural load and the center of gravity of the footing coincide. Hence, pressure distribution below the footing would be a uniform along the width and length of the footing.

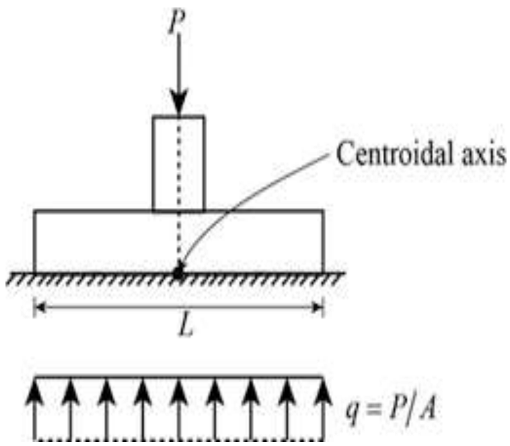
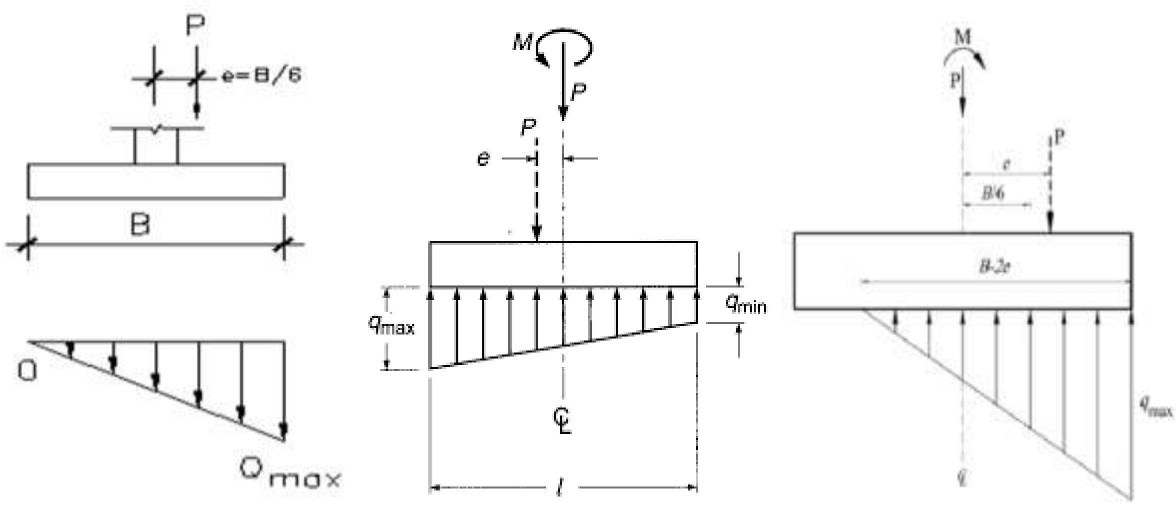


Figure: Concentric loading illustration

b) Eccentrically loaded footing

Eccentricity can be caused by lateral forces due to earthquake or wind and by the lateral soil pressure. Eccentricity has adversary impact on the bearing capacity of the shallow foundation hence can result in the formation of differential settlement. The bearing capacity decreases with increasing in eccentricity. The pressure distribution below the footings varies based on the magnitude of eccentricity as depicted on the following figure. Where a) for e=0, b) e < B/6, c) e > B/6



a) b) c)

Figure: Pressure distribution for different values of eccentricity

2.5. Selection of Foundation Type

In selecting the foundation type the following points must be considered;

- Function of the structure
- Loads it must carry

- Subsurface conditions (Bearing Capacity)
- Cost of foundation in comparison with the cost of the superstructure.
- The depth of foundations of adjacent buildings

CHAPTER THREE
DESIGN OF SHALLOW FOUNDATIONS

3.1. Elements of Reinforced Concrete Design

3.1.1 Design Methods:

Based on design load determination and the corresponding design strength of materials, different methods of design have been introduced.

- **Permissible stress method:** The ultimate strength of the material is divided by a factor of safety to give safe design stresses, which are usually within the elastic range. Stresses caused by the working loads must not exceed the permissible stresses.
- **Load factor method:** The working loads are multiplied by a factor of safety to obtain design loads. Stresses caused by the design load must not exceed the ultimate strength of the material.
- **Limit state method:** The working loads are multiplied by partial factors of safety to obtain design loads and ultimate strengths of materials are divided by further partial factors of safety to obtain design strengths. Stresses caused by the design loads must not exceed the design strength of the material.

The permissible stress method has proved to be a simple and useful method. However, there are certain shortcomings: Because it is based on an elastic stress distribution, it is not entirely applicable to concrete which is a semi-plastic material. Neither is it suitable when deformations are not proportional to the load, as in the case of slender columns.

In the load factor method, the ultimate strengths of the materials are used in the calculations. Because this method does not apply factors of safety to the materials, the variability of the materials cannot directly be taken into account. Furthermore, it cannot be used to calculate deflections and cracking under working loads.

The limit state method overcomes most of the shortcomings of the previous two methods. This is achieved by applying partial factors of safety to both the material strengths and the working loads, and also by varying the magnitude of the factors, depending on whether plastic conditions at the ultimate limit state are being considered, or whether elastic conditions under working loads are being considered.

3.1.2. Limit State Principles:

When dealing with the most economical structure associated with safety and serviceability requirements, the variability exists between construction materials and the construction process itself. We should be able to state a design philosophy to cope with the various criteria required to define the serviceability or usefulness of any structure in a rational manner.

The various criteria required to define the serviceability or usefulness of any structure can be described under the following headlines. The effects listed may lead to the structure being considered 'unfit for use'.

Collapse: failure of one or more critical sections; overturning or buckling.

(i). Deflection: the deflection of the structure or any part of the structure adversely affects the appearance or efficiency of the structure.

(ii). Cracking: cracking of the concrete which may adversely affect the appearance or efficiency of the structure.

(iii). Vibration: vibration from forces due to wind or machinery may cause discomfort or alarm, damage the structure or interfere with its proper function.

(iv). Durability: porosity of concrete.

(v). Fatigue: where loading is predominantly cyclic in character the effects have to be considered.

(vi). Fire resistance: insufficient resistance to fire leading to 1, 2 and 3 above.

When any structure is rendered unfit for use for its designed function by one or more of the above causes, it is said to have entered a limit state. The Code defines the limit states as:

(i). Ultimate limit state: the ultimate limit state is preferred to collapse.

(ii). Serviceability limit states: deflection, cracking, vibration, durability, fatigue, fire resistance and lightning.

The purpose of design then is to ensure that the structure being designed will not become unfit for the use for which it is required, i.e. that it will not reach a limit state. The essential basis for the design method, therefore, is to consider each limit state and to provide a suitable margin of safety. To obtain values for this margin of safety it was proposed that probability considerations should be used and the design process should aim at providing acceptable probabilities so that the structure would not become unfit for use throughout its specified life.

Accepting the fact that the strengths of construction materials vary, as do also the loads on the structure, two partial safety factors will now be used. One will be for materials and is designated γ_m; the other, for loading, is termed γ_f. These factors will vary for the various limit states and different materials. As new knowledge on either materials or loading becomes available the factors can be amended quit easily without the complicated procedures to amend one overall factor used in previous Codes.

The normal procedure is to design for a critical limit state and then to check for the other limit states are satisfied. The critical state for reinforced concrete structures is usually the ultimate limit state. However, water-retaining structures and prestressed concrete is usually designed at the serviceability limit state with checks on the ultimate limit state.

The limit states failure criteria can be summarized as follows:

(Design load effects Q_d) ≤ (Deisgn resistance R_d)

$$\gamma_f Q_n \le \frac{f_k}{\gamma_m}$$

Where Q_d = design load effects = $\gamma_f Q_n$

Qn = nominal load

γ_f = partial safety factor for loads

R_d = design resistance = f_k/γ_m

f_k = characteristic material strength

γ_m = partial safety factor for materials

N.B. Limit state is adopted throughout the design of reinforced concrete foundations.

- **Grades of Concrete**

Grades of concrete	C15	C20	C25	C30	C40	C50	C60
f_{ck}	12	16	20	24	32	40	48

$$f_{ck} = \frac{Grade}{1.25}$$

Where: f_{ck} = Characteristic cylinder compressive strength of concrete.

- ✓ **Safety Factors**

Partial Safety Factors for Materials at ULS.

Design Situations	Concrete, γ_c		Reinforcing Steel, γ_s	
	Class I	Class II	Class I	Class II
Persistent and Transient	1.50	1.65	1.15	1.20
Accidental	1.30	1.45	1.00	1.10

Partial Safety Factors for Actions in Building Structures at ULS.

Design Situation	Action	Factor, γ	Favorable	Unfavorable
Persistent and Transient	Permanent	γ_G	1.00	1.30
	Variable	γ_Q	0.00	1.60
Accidental	Permanent	γ_G	1.00	1.00

Design loads and partial load factor.

Load Acting	P_d
DL + LL.	$1.3G_k + 1.6Q_k$
DL + LL + EQL	$0.8(1.3G_k + 1.6Q_k) + EQL$

Where, G_k is Characteristic dead load.

Q_k is characteristic live load.

EQ is earth quake load.

- ✓ **Design Strength for Concrete**

(a) In compression: $f_{cd} = \frac{0.85 f_{ck}}{\gamma_c}$ (b) In tension: $f_{ctd} = \frac{f_{ctk}}{\gamma_c} = 0.21 f_{ck}^{2/3} / \gamma_c$

- ✓ **Design Strength for Steel**

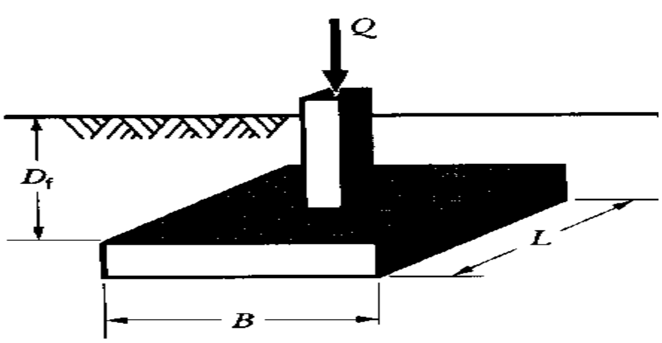
In tension and compression: $f_{yd} = \frac{f_{yk}}{\gamma_s}$

3.2. Analysis and Design of Isolated footings

3.2.1. Concentrically Loaded Isolated Footing

The General Procedure for the Design of Concentrically Loaded Isolated Footing

Given: Column dimensions and reinforcement; column loads (LL, DL); f_{ck} for footing and column; f_{yk} for footing and column; allowable bearing capacity, q_a.



Solution:

(i) Find **Pu = 1.3DL+1.6LL** (Self wt. and backfill usually absent).

(ii) Determine B and L of footing; $A = \frac{(DL+LL)}{q_a}$. For a unique solution, B or L is fixed.

(iii) Find $q_u = \frac{P_u}{BL}$ (Ultimate bearing pressure beneath footing).

(iv) Assume trial effective depth, d, of footing for determination of flexural reinforcement.

(v)Check **d** for punching shear and wide beam shear.

(vi) If step (v) is not fulfilled increase d and repeat starting from step (iv).

(vii)Calculate the anchorage length and reinforcement distribution.

(viii) Select the appropriate dowels based on the anchorage length and lap length.

(ix) Complete a design drawing showing all details (footing dimensions, reinforcement Size, spacing cover, etc.)

Note: to avoid repetition of the steps for safe and economical depth you may calculate the d value.

3.2.2. Eccentrically Loaded Spread Footings

The ensuing "load" on the column, and subsequently on the footing, due to supported beams from several spans, can be a combination of a vertical load and moments as shown in figure 4.4.1.

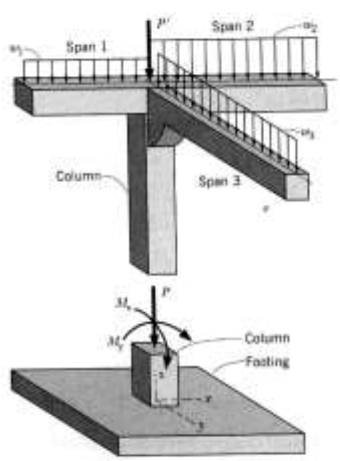


Figure 3.3.1 Example of a loading condition that

The source for the effects of eccentricity on the footing may be either a concentric vertical load and moment combination (fig. 3.3.2a) or a column located eccentrically to the centroid of the footing (fig. 3.3.2b). In order not to overstress the soil under some points of the footing, and to eliminate tilting of column and footing, a footing is proportioned in such a way that a uniform soil pressure distribution is attained.

The difficulty in establishing a fixed location of the load centerline relative to the footing centroid lies on the change of magnitude and direction of the variable loads (such as the live and wind load). Hence, if the column is not centrally located for the sake of having uniform bearing pressure (fig. 3.3.2c), our design is perhaps somewhat hypothetical in a strict sense.

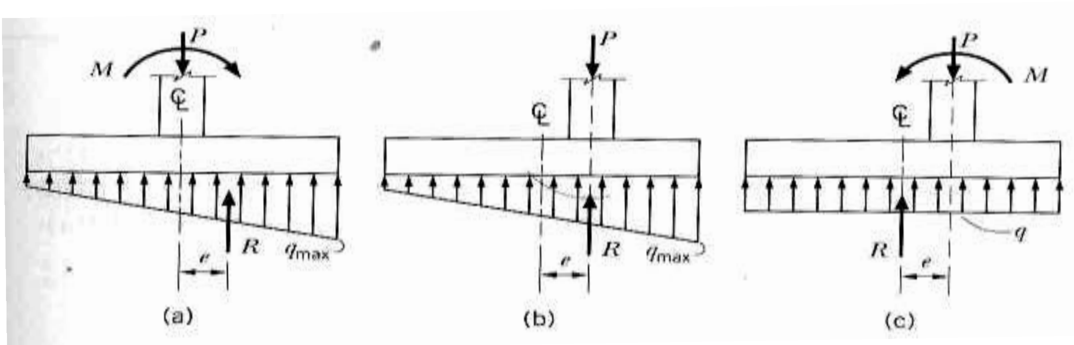


Figure 3.3.2 Soil pressures resulting from eccentric loading

Superimposing the pressures resulting from the direct vertical load to those from moment:

$$q = \frac{P}{A} \pm \frac{M_x c_1}{I_x} \pm \frac{M_y c_2}{I_y}$$

$$M_x = Pe_y \quad and \quad M_y = Pe_x$$

$$I_x = \frac{LB^3}{12} \quad and \quad I_y = \frac{BL^3}{12}$$

$$\therefore \quad q = \frac{P}{BL}\left[1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{B}\right]$$

When q_{max}>q_u the process of load redistribution (similar to concrete beam analysis in ultimate limit state) continues until equilibrium (or failure) is obtained. As a result the assumed rectangular bearing pressure block is being produced (fig. 4.4.3).

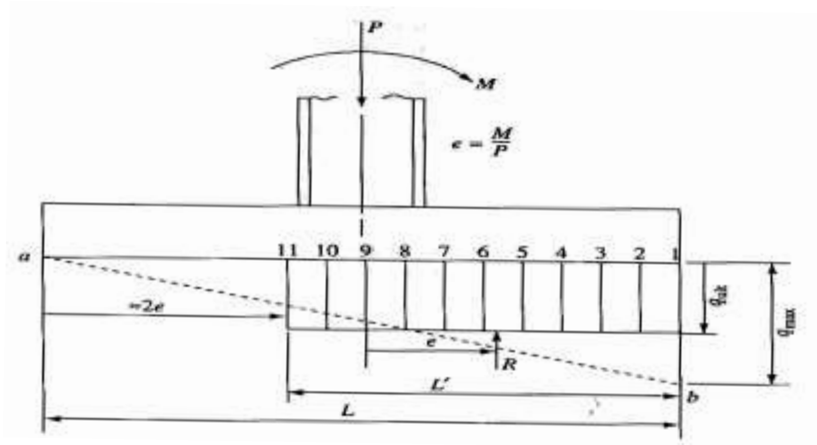


Figure 3.3.3 Soil yielding under *P/A + Mc/I* toe stresses to produce an approximate rectangular pressure zone to resist *P* and to satisfy statics. For overturning stability always take a $\sum M$ check about point 1 at toe.

After careful consideration it appears that the base should be deigned consistent with the procedure for obtaining the bearing capacity. That is use dimensions B', L' for the design also.

This procedure ensures four items of considerable concern:

(i) The resultant soil pressure R (fig. 3.3.3) is never out of the middle one-third of the base so that the overturning stability is always satisfied. This R always gives:

$$SF = \frac{M_{resist}}{M_{overturn}} = \frac{PL}{2M}$$

(ii) The toe pressure will always be such that $q_{toe} \leq q_a$.

(iii) The design is more easily done when a uniform soil

Pressure is used to compute Design moments.

(iv) Approximately the same amount of steel is required as in the design using the triangular stress distribution.

$$L' = L - 2e_x \quad ; \quad B' = B - 2e_y$$

$$e_x = \frac{M_y}{P} \quad ; \quad e_y = \frac{M_x}{P}$$

The amount of steel computed for a unit width is used across the full base dimensions of B and L.

For the punching shear and wide beam shear compute an "average" $q_{u,av} = \frac{P_u}{BL}$ and use this $q_{u,av}$ value.

3.2.3. Inclined Loaded Spread Footings

The General Procedure for the Design of Inclined Loaded Isolated Footing

Given: Column dimensions and reinforcement; column loads (LL, DL); f_{ck} for footing and column; f_{yk} for footing and column; allowable bearing capacity, q_a.

Solution:

(i) The procedure done for eccentrically loaded will be followed and in addition to it the footing should be checked against sliding failure.

3.3. Combined Footings

3.3.1. Rectangular Combined Footing

An isolated footing is likely to result in an uneven soil-pressure distribution for a column very close to a property line. In order to achieve uniform soil pressure, one alternative may be a rectangular-shaped, combined footing. The footing near the property line is connected with an adjacent one.

Generally, it is assumed that the rectangular footing is a rigid member, thus, the pressure is linear. The approach yields a rather conservative design; the moments are somewhat larger than those obtained by treating the footing as a beam on an elastic foundation.

The following is a summary of the procedure:

Given: Typically included in the given part of the problem are column data (loads, sizes, reinforcement, location, and spacing), soil bearing, concrete strength (f_{ck}), and grade of reinforcement (f_{yk}).

Objective: The goal is to determine footing dimensions (width, length, thickness), steel reinforcement (bar sizes, spacing, placement, details, dowels), and relevant details for construction.

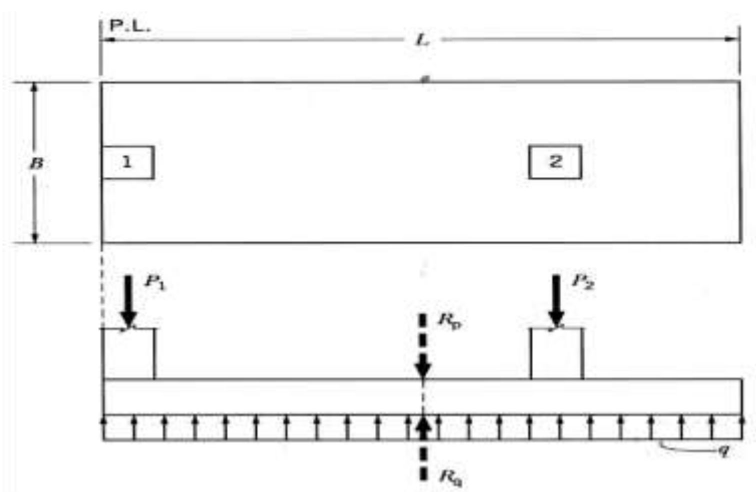


Figure 4.5.1 Rectangular shaped combined footing. For uniform q, the resultant of the applied load is

Procedure: The design is predicated on the assumption that the footing is rigid and that the soil pressure is uniform. The following explanation may illustrate the procedure:

Step 1: Convert the column loads to ULS loads via P_u= 1.3(D.L.)+ 1.6(L.L.). Then convert the allowable soil pressure to ULS pressure via $q_u= (P_{1u} + P_{2u})\, q_a/ (P_1+ P_2)$.

Step 2: Determine the footing length (L) and width (B).

- ✓ First determine the location of the load resultant distance ($\bar{x}$). This point coincides with the midpoint of L,thus yielding the value for L. B is then determined from $B = \sum P_u \big/ L q_u$.

Step 3: Draw shear and moment diagrams. The footing is treated as a beam, loaded with a uniform soil pressure (upward) and column loads (downward), which are treated as concentrated loads.

Step 4: Calculate footing depth based on shear (**Punching shear and wide beam shear**). Critical sections are at 1.5d for punching shear and at the d for a diagonal tension or wide beam, the same as for spread footings. The critical section for wide-beam shear is investigated only at one point (max. shear).

For punching shear, however, an investigation of a three- or four-sided zone for each column may have to be done.

Step 5: Determine the flexural reinforcing steel based on footing depth you calculated in step 4. The longitudinal (flexural) steel is designed using the critical moments (negative and positive) from the moment diagram. Thus, typically, combined footings will have longitudinal steel at both top and bottom of the footing.

Step 6: Determine the steel in the short direction. The steel in the transverse direction is determined based on an equivalent soil pressure q' and subsequent moment, for each column. Even for stiff footings, it is widely accepted that the soil pressure in the proximity of the columns is larger than that in the zone between columns. Thus, for design, we account for this phenomenon by assuming an empirical effective column zone width of s. The soil pressure in this zone, q', is calculated as $q' = P_u/ Bs$, where P_u is the ULS column load, B the footing width and s an equivalent width of footer strip for the column in question. Commonly, the value of s is taken as the width of the column (in the longitudinal direction) plus about 0.75d on each side of that column.

Step 7: Evaluate dowel steel. The requirements are the same as for spread footings.

Step 8: Provide a drawing showing final design. This drawing is to show sufficient detail from which one may construct.

3.3.2. Trapezoid-shaped Footings

A combined footing will be trapezoid-shaped if the column that has too limited a space for a spread footing carries the larger load. In such a case, the resultant of the column loads (including moments) will be closer to the larger column load, and doubling the centroid distance as done for the rectangular footing will not provide sufficient length to reach the interior column. Correspondingly, the soil pressure would not be uniform (recall that our typical objective is uniform soil pressure). For very large column spacing (e.g., say greater than 7m), a strap (cantilever) footing may be a somewhat more economical (i.e. Less material) solution to such a problem. For smaller column spacing, a trapezoid-shaped footing, as shown in fig. 4.6.1 for a two-column arrangement, is usually deemed suitable.

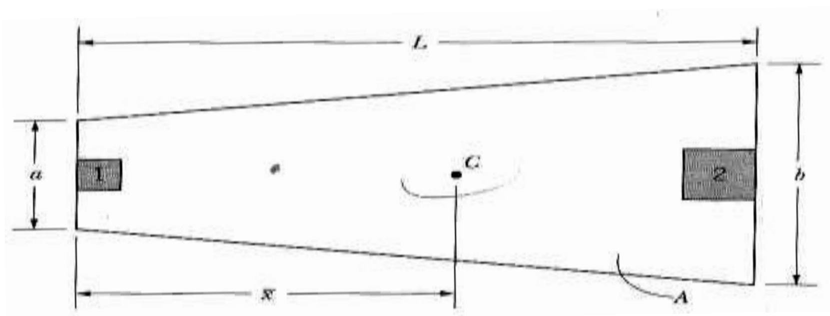


Figure 4.6.1 Trapezoid-shaped footing

Referring to fig. 4.6.1, the area, A, is

$$A = (a+b)L/2$$

From $\bar{x} = \sum Ax / \sum A$, we get

$$\bar{x} = \frac{(aL)(L/2) + [(b-a)(L/2)]2L/3}{aL + (b-a)L/2}$$

$$\bar{x} = (L/3)(a+2b)/(a+b)$$

For the pressure to be uniform, the resultant of the column loads coincides (is collinear) with the resultant of pressure at the centroid (C) as shown.

The following is a summary of the procedure for the design of trapezoid-shaped footings:

Given: Included in the given data are column information (loads, sizes, location, and spacing), length of footing (L), soil bearing values (q_a), concrete strength (f_{ck}), and grade of reinforcement (f_{yk}).

Objective: The goal is to determine footing dimensions (width, thickness), steel reinforcement (bar sizes, spacing, placement, details, dowels), and relevant details for construction.

Procedure: The design is predicated on the assumption that the footing is rigid and that the soil pressure is uniform. The basic steps are:

Step 1: Convert the column loads to ultimate loads via P_u= 1.3(DL) + 1.6(LL); then convert the allowable soil pressure to ultimate; that is, $q_u = (P_{ul} + P_{u2})\, q_a/(P_l + P_2)$.

Step 2: Determine dimensions a andb via simultaneous solutions of two independent equations.

$$A = (a+b)L/2$$
$$\bar{x} = (L/3)(a+2b)/(a+b)$$

Thus, we solve for a and b.

Step 3: Draw the shear and moment diagrams. The footing is treated as a beam, loaded with a uniform soil pressure (upward) and column loads (downward), which are treated as concentrated loads. Note that while the pressure is uniform, the pressure force for-unit length varies with the width [e.g., at the narrow end, the load is a(q_u); and b(q_u) at the wide end, etc.].

Step 4: Determine footing depth based on shear. Critical sections are usually checked for wide-beam shear at the narrow end and punching shear at the wide end.

Step 5: Determine the flexural reinforcing steel. Because the width varies, it is advisable to determine -A_s at several points; the same is now required for + A_s since it is typically governed by ρ_{min}.

Step 6: Determine the steel in the short direction. Assume an average length for the cantilever length; determine the equivalent lengths as for rectangular footings.

Step 7: Determine dowel steel, as for rectangular combined or spread footings.

Step 8: Provide a drawing with details for construction. Here some judgment is necessary to accommodate the steel arrangement in view of the variable width along the footing.

3.4. Strap Footings

A strap footing (cantilever footing) is a composite of two spread (isolated) footings connected by a rigid beam or strap, as shown in fig. 4.6.1. The strap connects an eccentrically loaded footing (e.g., footing 1) with an interior footing, subsequently resulting in a uniform soil pressure and minimum differential settlement.

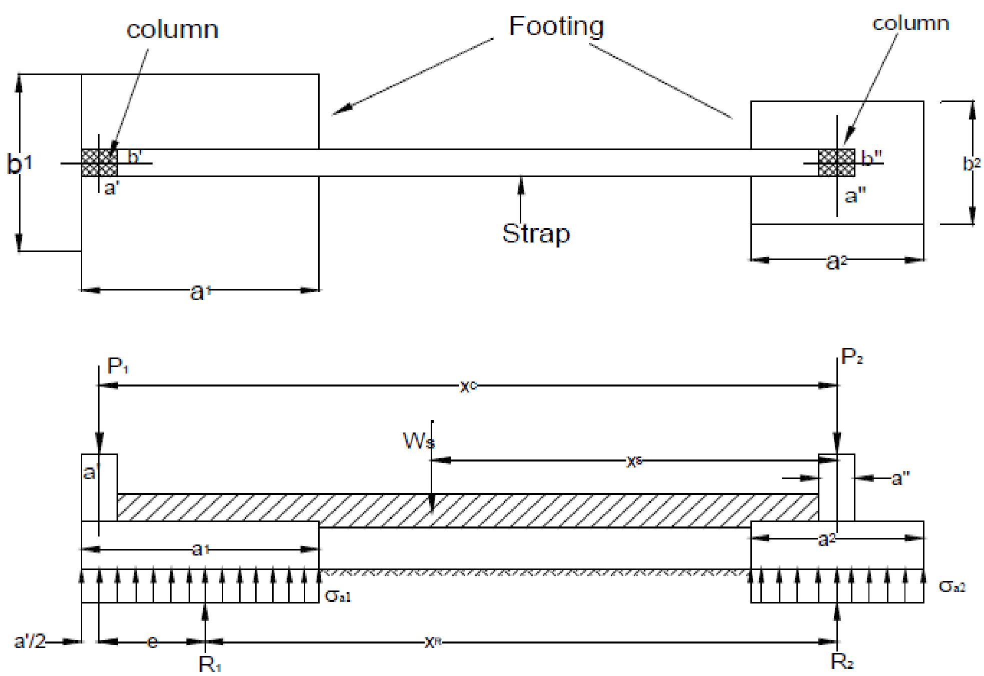


Figure 4.6.1 Typical configuration of a strap footing

The strap is designed as a rigid beam connected to the footings such that it overcomes rotational effects on eccentrically loaded footings; it is assumed to experience no soil pressure. This is accomplished by either forming the strap above the ground or by pouring the strap over a compressible formation, such as loose or spaded soil or semi-rigid styro foam. Hence, the shear is a constant between the footings; the moment varies linearly.

The footings are treated as isolated footings. The interior footing (e.g., footing 2) is generally square-shaped and is designed as a spread footing, with appropriate negative (top) longitudinal steel provided to resist the negative moment transmitted via the strap. While this spread-footing approach also applies to footing 1, one carefully scrutinizes the zone near column 1 for some additional transverse steel requirements, as typically included for rectangular or trapezoid-shaped footings discussed in the preceding sections.

The following procedural summary illustrates the recommended approach for a strap footing design.

Given Typically, included in the given part of the problem are column data (loads, sizes, reinforcement, location, and spacing), allowable soil bearing, q., concrete strength (fck), and grade of reinforcement (fyk).

Objective: The goal is to (a) determine the footing dimensions (length, width, and thickness) proportioned such that the soil pressure is reasonably uniform and differential settlement is minimal, (b) design the strap, (c) design the footings, and (d) show a drawing with pertinent details for construction purposes.

Procedure: The design assumes no soil pressure under the strap (other than that necessary to support the weight of the strap; hence, the weight of the strap is neglected). The footings are designed as isolated footings subjected to column loads and strap reactions.

Step 1 (a) Convert to Puand qu, as previously described.

(b) Try a value for e. This establishes the position of R1

$$e = x_c - x_R$$

$$a_1 = 2(\frac{a'}{2} + e)$$

(c) From equilibrium (i.e., ∑M= 0 and ∑Fy= 0), determine the values for R1 and R2.

By taking moment about R2

$$R_1 = P_1 \frac{x_c}{x_R} + W_s \frac{x_s}{x_R}$$

Where, P1 is the column load.

Ws- is the weight of the strap.(May be neglected)

From ∑Fy= 0,

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

Step 2 Determine footing dimensions, L and B. Note that q will be uniform when R coincides with the centroid of that footing. Also, for minimum differential settlement, q should be the same for both footings.

$$b_1 = \frac{R_1}{a_1 \sigma_{a1}} \qquad b_2 = \frac{R_2}{a_2 \sigma_{a2}}$$

Step 3 Draw the shear (V) and moment (M) diagrams.

Step 4 Design the strap as a beam. Use maximum SF and M in the section between footings. Affix the strap to the footings to effectively prevent footing rotation.

Step 5 Design the footings as spread (isolated) footings with reinforcement in both directions including -As steel to accommodate the negative moment. Some special assessment for the transverse steel near column 1 is recommended.

Step 6 Provide the final drawing showing details for construction.

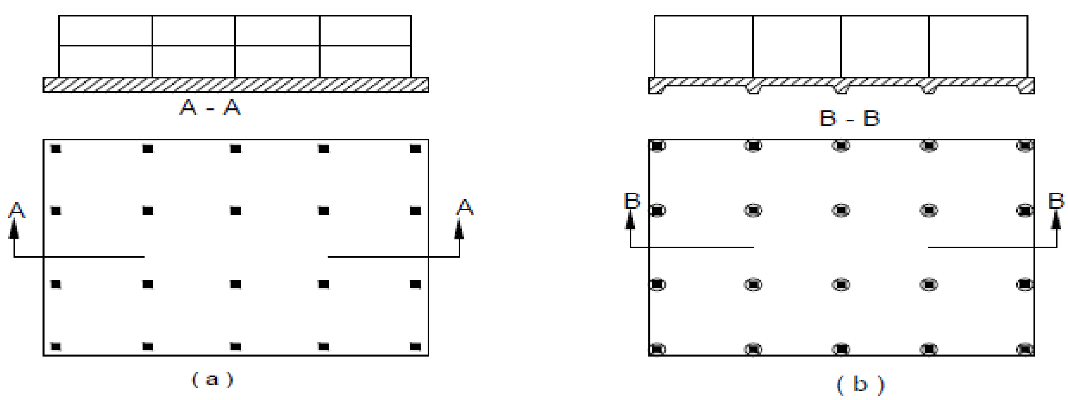
3.5. Analysis and Design of Mat Foundations.

A mat (raft) foundation is a structural reinforced concrete slab that supports a number of columns distributed in both horizontal directions or supports uniform pressure, as from a tank. Rafts are used to bridge over soft spots if the spots are very localized, and to reduce the average pressure applied to the soil.

4.8.1. General

Mat or raft foundation is a large concrete slab supporting a number of columns. It is used where the supporting soil has low bearing capacity. The bearing capacity is increased by combining all individual footings into one mat – since bearing capacity is proportional to width and depth of foundation (Fig. 5.8). In addition to increasing the bearing capacity, mat foundations tend to bridge over irregularities of the soil and the average settlement does not approach the extreme values of isolated footings. Thus mat foundations are often used for supporting structures that are sensitive to differential settlement.

Mat foundations may have different forms as indicated in (Fig. 5.1.). The design of mat foundation presents problems of highly statically indeterminate nature. The methods of design depend on the assumption one uses. Basically there are two methods of design, namely the *Rigid Method* and the *Elastic Method.*



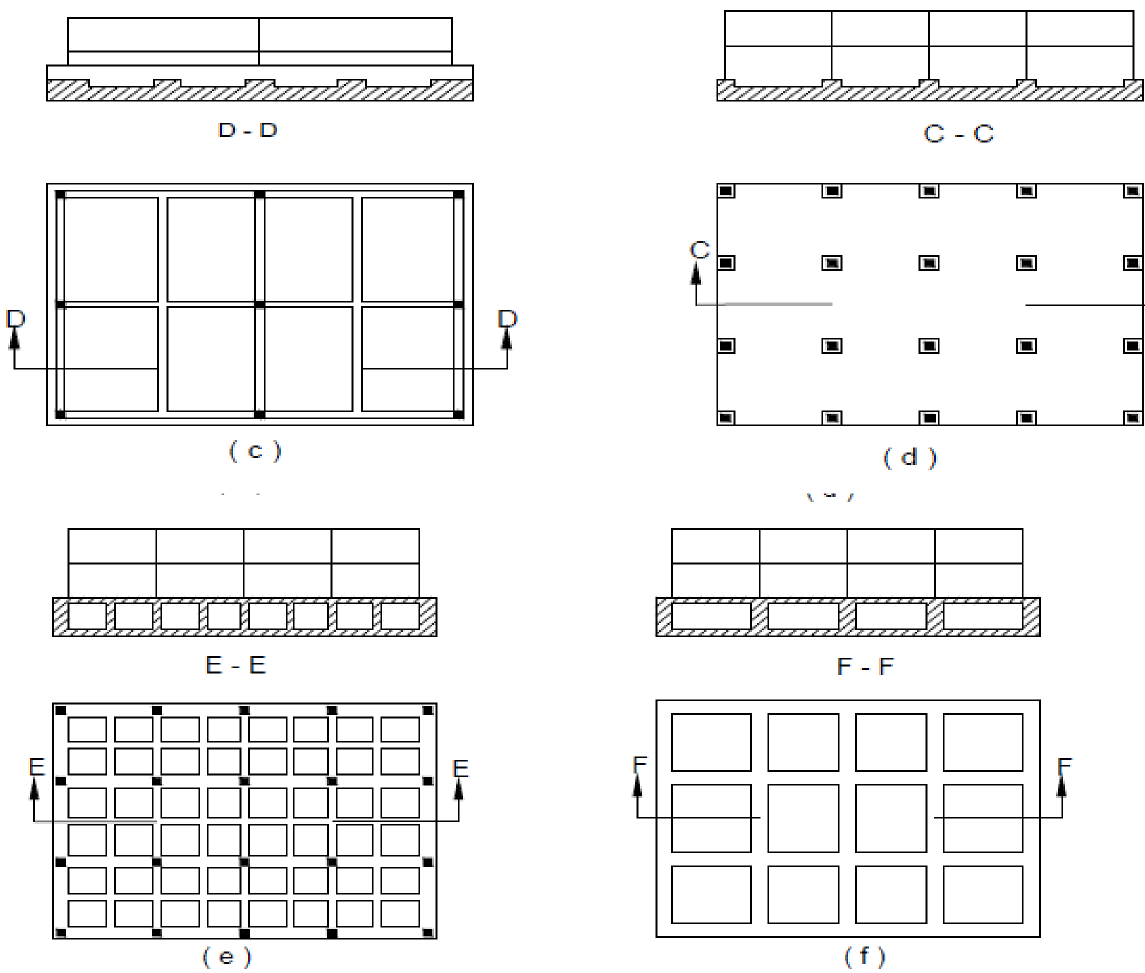


Fig. 5.9 : Different forms of mat foundations

a) Flat plate b)Flat plate thickened under columns
c) Two - way beam and ribbed slab
d) Flat plate with pedestals e) cellular construction
f) Basement walls as rigid frame [19]

4.8.2. Rigid Method of design

A mat foundation is considered rigid if it supports a rigid superstructure or when the column spacing is less than $\frac{1.75}{\lambda}$.

$$\lambda = \left[\left(\frac{K_s b}{4 E_c I} \right) \right]^{1/4} \qquad (5.36)$$

where

K_s = coefficient of subgrade reaction

B = width of a strip of mat between centers of adjacent bays

E_c = modulus of elasticity of concrete

I = moment of inertia of the strip of width b

λ = characteristic coefficient.

It should, however, be noted that Eq. (5.36) is valid for relatively uniform column loads (loads not varying more than 20% between adjacent columns) and relatively uniform columns spacing.

4.8.2.1. Uniform Mat

For uniform mat, the following procedure for design is suggested:

(i) Compute the maximum column and wall loads.
(ii) Determine the line of action of the resultant of all the loads
(iii) Determine the contact pressure distribution using the flexure equation:

$$\sigma = \frac{\sum P}{A} \pm \frac{\sum P e_y y}{I_x} \pm \frac{\sum P e_x x}{I_y}$$

where

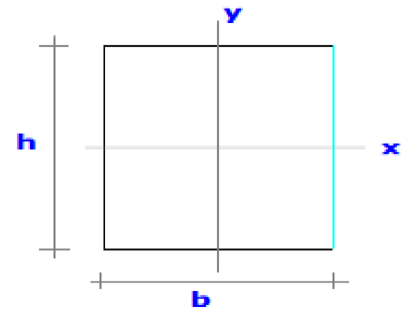
$\sum P$ = total loads on the mat

A = total area of the mat

x, y = co-ordinate of any given point on the mat with respect to the x and y axes passing through the centroid of the area of the mat

e_x, e_y = eccentricities of the resultant force in x and y direction

I_x, I_y = moment of inertia of the area of the mat with respect to the x and y axis respectively.



$$I_x = \frac{bh^3}{12} \text{ and } I_y = \frac{hb^3}{12}$$

(iv) Analyze the mat in one of the following approximate methods:

Method A

Convert the contact pressure calculated using Eq. (5.37) to a uniform contact pressure distribution using engineering judgment. Take a system of column strip with width W_s as shown in Fig. (5.12a).

Draw 45° diagonal lines from the edges of pedestals (columns) to form the system of lines indicated in the figure.

The central slabs, like for instance R S T U (shaded), are designed as two-way rectangular slabs with fixed edges supported by the strips, in which the supports are located at an imaginary location inside the appropriate strips at a distance of 20% of the width of the column strip but not exceeding the effective depth d[8]. The same reinforcements are used for bottom and top of the slab.

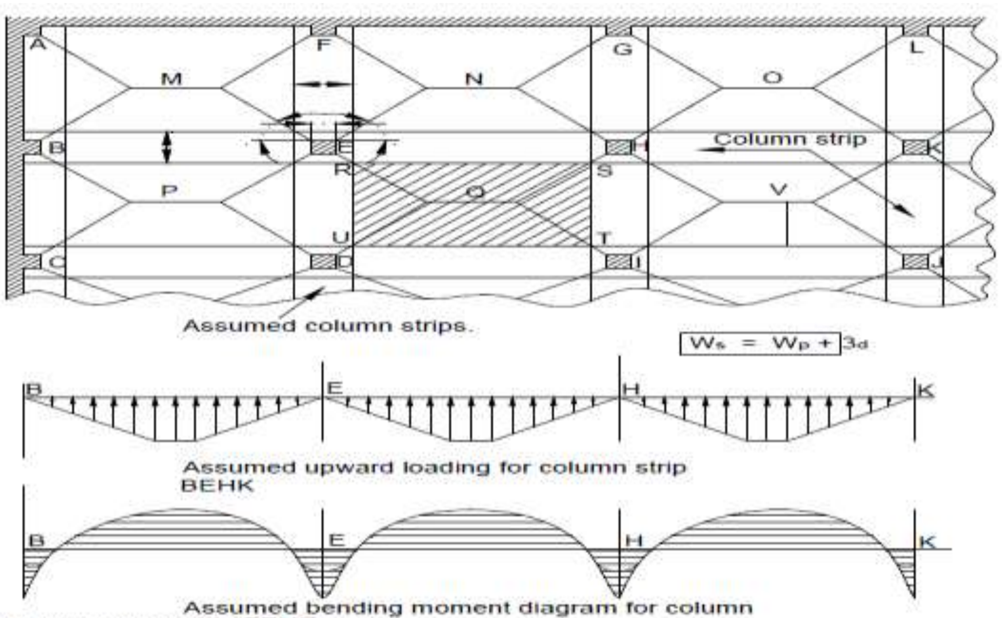
The column strips, like BEHK, should support the loads from BPEM, EQHN, etc., and are designed as a series of fixed-end beams with triangular loading [8] (Fig. 5.12a).

Method B

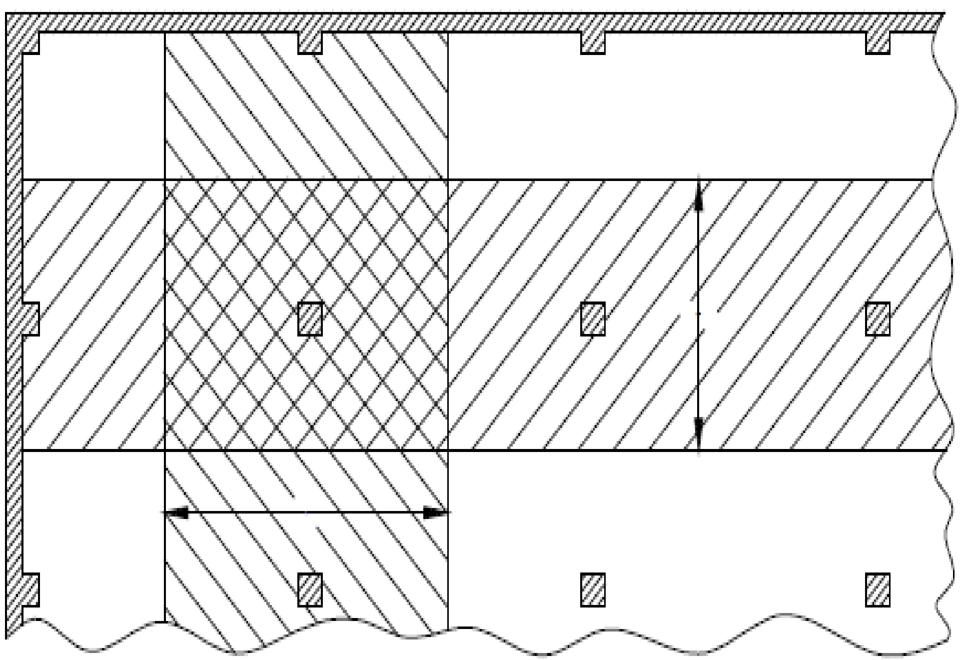
In the case where the column loads and spacing do not vary more than 20% from each other, divide the slab into perpendicular bands (Fig. 5.12b). Each band is assumed to act as an independent beam subjected to known contact pressure and known column loads. Determine the magnitudes of the positive and negative moment using $M = \frac{wl^2}{10}$ for interior spans and $M = \frac{wl^2}{8}$ for exterior spans [4].

(v) Check wide-beam and punching shear.

(vi) Provide the necessary reinforcement. Use a cover of not less than 5cm.



a) Analysis according to method A



a) Analysis according to method B

Fig . 5.12 : Approximate methods of analysis of large mat

CHAPTER 4

Analysis and Proportioning of Retaining walls

Retaining Walls: are structures used to provide lateral stability of earth or other material where conditions disallow the mass to assume its natural slope.

4.1.Common Types of Retaining Walls

1. Gravity Retaining wall

Made of plain concrete or stone masonry. The stability of gravity retaining walls depends upon its weight. It is trapezoidal in section with the base projecting beyond the face and back of the wall. No tensile stress in any portion of the wall and it is economically used for walls less than 6m high.

2. Cantilever wall

Made of reinforced concrete material that consists of a thin stem and a base slab. It is inverted T-shaped in section with each projecting acts as a cantilever. Economically used for walls 6 to 7.5m high.

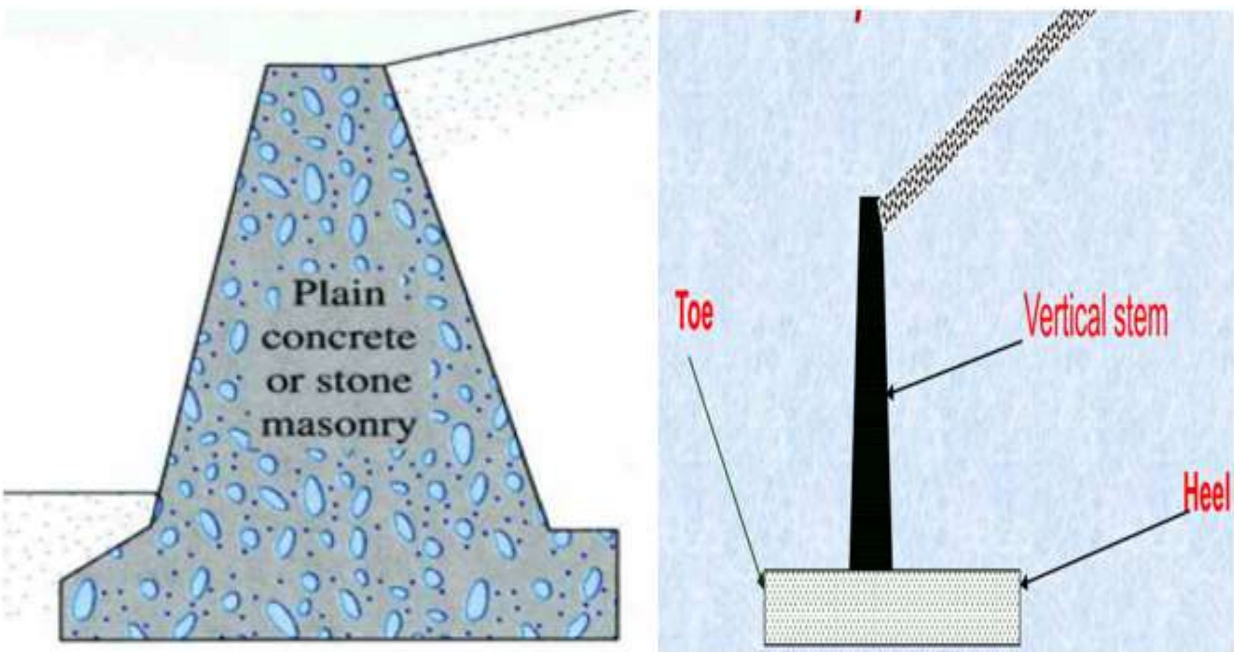


Figure 4.1 (a) Gravity Wall (b) Cantilever Wall

3. Counter Fort Retaining Wall

Are similar to cantilever retaining walls. They have thin vertical concrete slabs known as counterforts that tie the wall and the base slab together. The purpose of the counterforts is to reduce the shear and the bending moments. It is ordinarily used for walls height greater than 6.0m

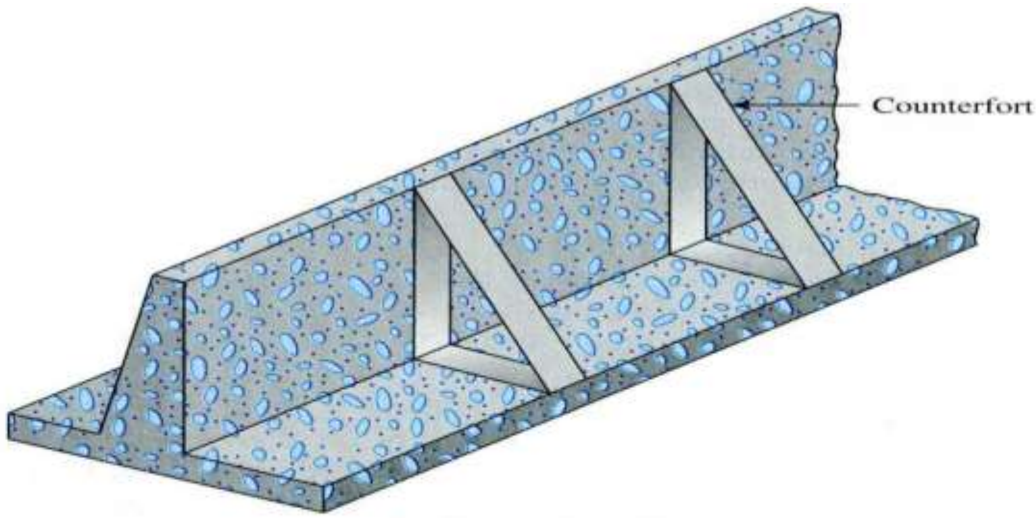


Figure 4.2 Counterfort Wall

4. Buttress Wall

Same as counter fort except that the vertical brackets are on the opposite side of the backfill

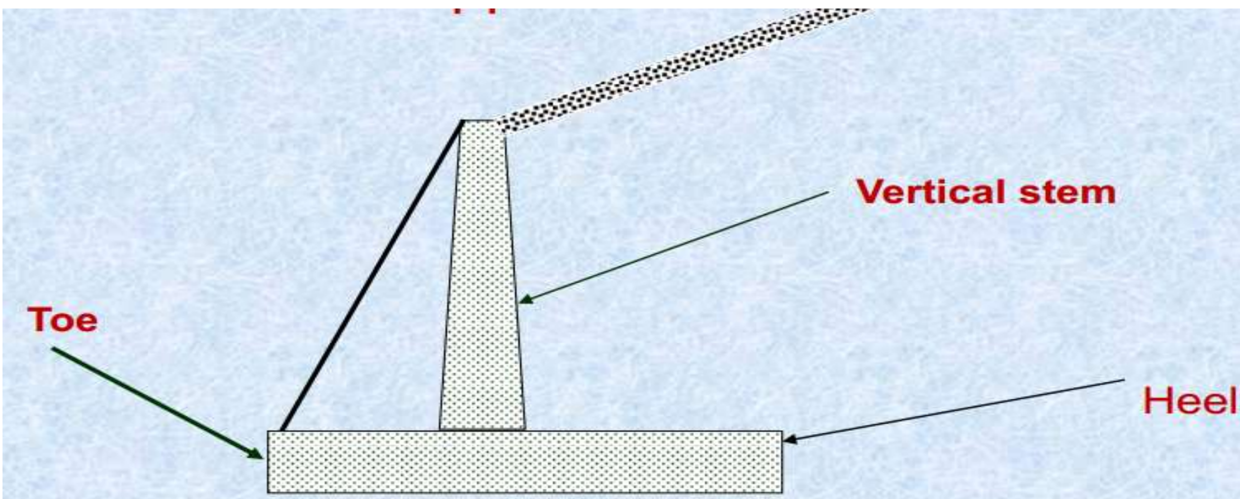


Figure 4.3 Buttress Wall

4.2. Common Proportions of Retaining Walls

The usual practice in the design of retaining walls is to assign tentative dimensions and then check for the overall stability of the structure. Note that the top of the stem of any retaining structure should not be less than 0.3 m for proper placement of concrete. The depth, D, to the bottom base of the slab should be a minimum of 0.6 m. However, the bottom of the base slab should be positioned below the seasonal frost line.

For counterfort retaining walls, the general proportion of the stem and the base slab is the same as for cantilever walls. However, the counterfort slabs may be about 0.3 m thick and spaced at center-to-center distances of 0.3H to 0.6H.

checks: (a) gravity wall; (b) cantilever wall

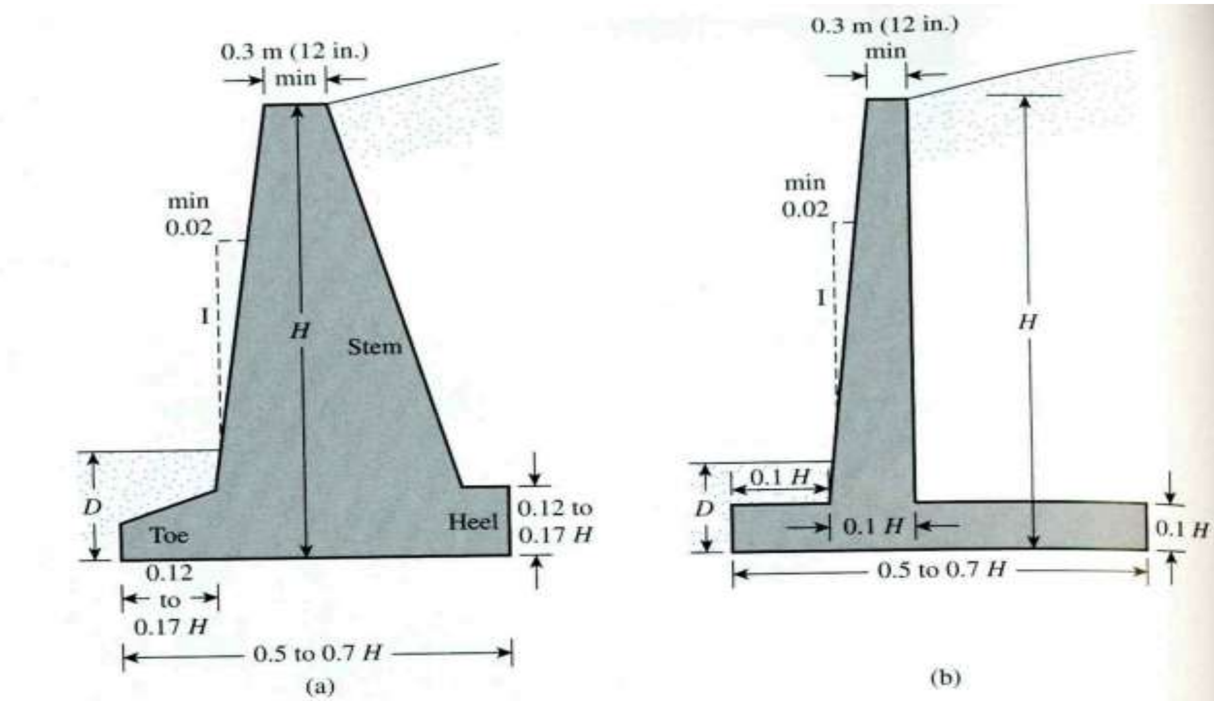


Figure 4.4 Approximate dimensions for various components of retaining wall for initial stability checks: a) Gravity wall; b) Cantilever wall

4.3.Forces on Retaining Walls

The forces that should be considered in the design of retaining walls include

Active and passive earth pressures, dead weight including the weight of the wall and portion of soil mass that is considered to act on the retaining structure, Surcharge including live loads, if any Water pressure, if any contact pressure under the base of the structure.

4.4.Stability of Retaining Walls

Retaining walls should be designed to provide adequate stability against sliding along its base, overturning about its toe, foundation bearing failure, deep-seated shear failure (overall or deep foundation failure) and excessive settlement.

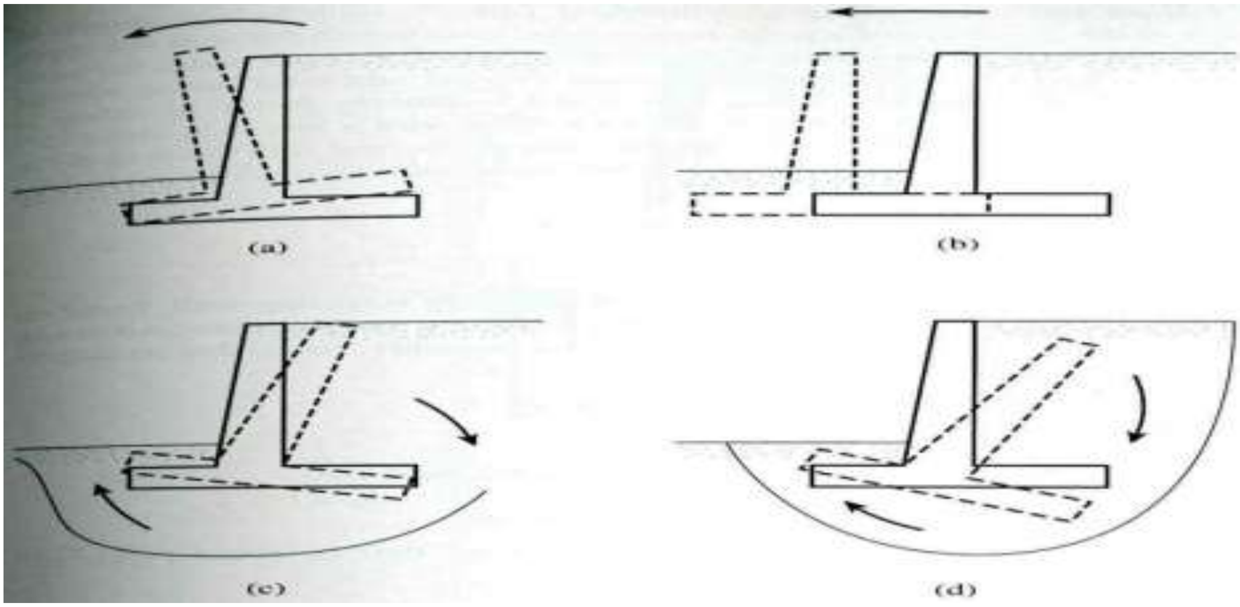


Figure 4.5 Failure of retaining wall: (a) by overturning; (b) by sliding; (c) by bearing capacity failure; (d) by deep-seated shear failure

4.4.1. Sliding stability

The factor of safety against sliding may be expressed by the equation

$$FS_{(\text{sliding})} = \frac{\Sigma F_{R'}}{\Sigma F_d}$$

where

$\Sigma F_{R'}$ = sum of the horizontal resisting forces
ΣF_d = sum of the horizontal driving forces

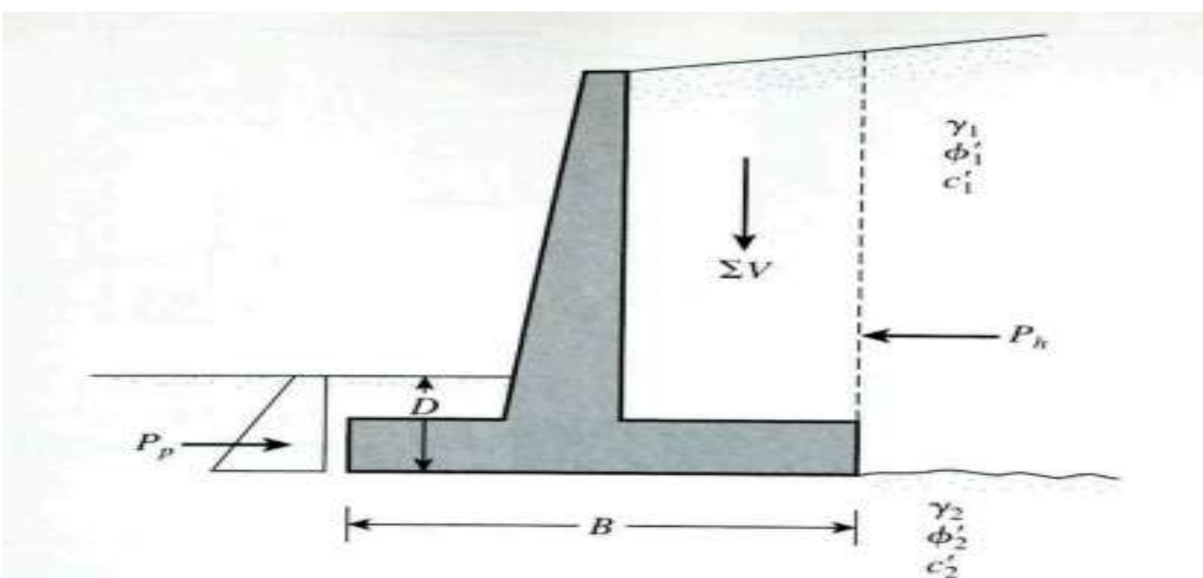


Figure 4.6 Sliding stability along the base

The passive force shown on figure is also a horizontal resisting force.

$\Sigma F_{R'} = (\Sigma V)\tan\delta' + Bc'_a + P_p$ Where, δ' = angle of friction between the soil and the base slab
c'_a = adhesion between the soil and the base slab

The only horizontal force that will tend to cause the wall to slide (a *driving force*) is the horizontal component of the active force P_a, so

$$\Sigma F_d = P_a \cos\alpha$$

Therefore,

$$FS_{(sliding)} = \frac{(\Sigma V)\tan\delta' + Bc'_a + P_p}{P_a\cos\alpha}$$

Factor of safety ≥1.5 for granular soils.

Factor of safety ≥2.0 for cohesive soils.

4.4.2. Overturning Stability

The factor of safety against overturning about the toe expressed as:

$$FS_{(overturning)} = \frac{\Sigma M_R}{\Sigma M_o}$$

where

ΣM_o = sum of the moments of forces tending to overturn about point C
ΣM_R = sum of the moments of forces tending to resist overturning about point C

The overturning moment is

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

where $P_h = P_a\cos\alpha$.

To calculate the resisting moment, $\sum M_R$ (neglecting Pp), a Table such as 4.1 can be prepared. The weight of the soil above the heel and the weight of the concrete or masonry are both forces that contribute to the resisting moment. Note that the force Pv also contributes to resisting moment. Pv is the vertical component of the active force Pa, or

$$P_v = P_a \sin\alpha$$

The moment of the force P_v about C is

$$M_v = P_v B = P_a \sin\alpha B$$

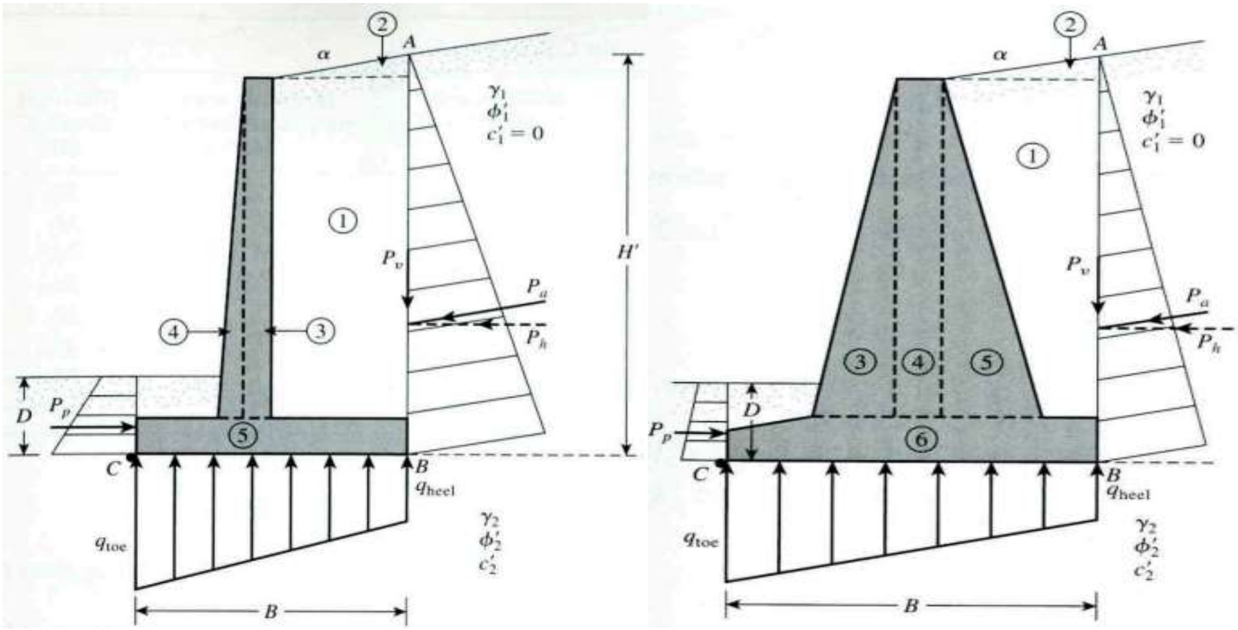


Figure 4.7 Overturning stability assuming that Rankine is valid

where B = width of the base slab.

Once ΣM_R is known, the factor of safety can be calculated as

$$FS_{(\text{overturning})} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_v}{P_a \cos\alpha (H'/3)}$$

The usual minimum desirable value of the factor of safety with respect to overturning is 2 to 3.

Table 4.1 Procedures for calculating $\sum M_R$

Section (1)	Area (2)	Weight/unit length of wall (3)	Moment arm measured from *C* (4)	Moment about *C* (5)
1	A_1	$W_1 = \gamma_1 \times A_1$	X_1	M_1
2	A_2	$W_2 = \gamma_1 \times A_2$	X_2	M_2
3	A_3	$W_3 = \gamma_c \times A_3$	X_3	M_3
4	A_4	$W_4 = \gamma_c \times A_4$	X_4	M_4
5	A_5	$W_5 = \gamma_c \times A_5$	X_5	M_5
6	A_6	$W_6 = \gamma_c \times A_6$	X_6	M_6
		P_v	B	M_v
		ΣV		ΣM_R

(*Note:* γ_l = unit weight of backfill
γ_c = unit weight of concrete)

Some designers prefer to determine the factor of safety against overturning with the formula

$$FS_{(\text{overturning})} = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6}{P_a \cos\alpha (H'/3) - M_v}$$

4.4.3. Foundation stability

$$q_t = \frac{Rv}{B}\left(1 \pm \frac{6e}{B}\right)$$

Where e= eccentricity of R_v

$q_t \leq q_{all}$, qall = qult/F.S

F.S = Factor of safety = 2 and 3 for granular and cohesive soils, respectively.

CHAPTER 5

Design and Analysis of Soil Retaining Structure

The design and construction of retaining structures forms an integral part of many civil engineering projects. They comprise a number of elements and may restrain the soil by virtue of their mass, or because they are embedded, propped and/or anchored. To design a retaining structure it is necessary to understand the soil and its behavior, the ground water conditions, how wall is constructed and how the soil and the structure interact.

Figure 5.1: Retaining Wall

5.1. Classification of Retaining Walls

In general retaining walls classified into two major categories further subdivisions:

1. Conventional retaining walls
   1. Gravity retaining walls
   2. Semi-gravity retaining walls
   3. Cantilever retaining walls
   4. Counter fort retaining walls
2. Mechanically stabilized earth walls (MSE)

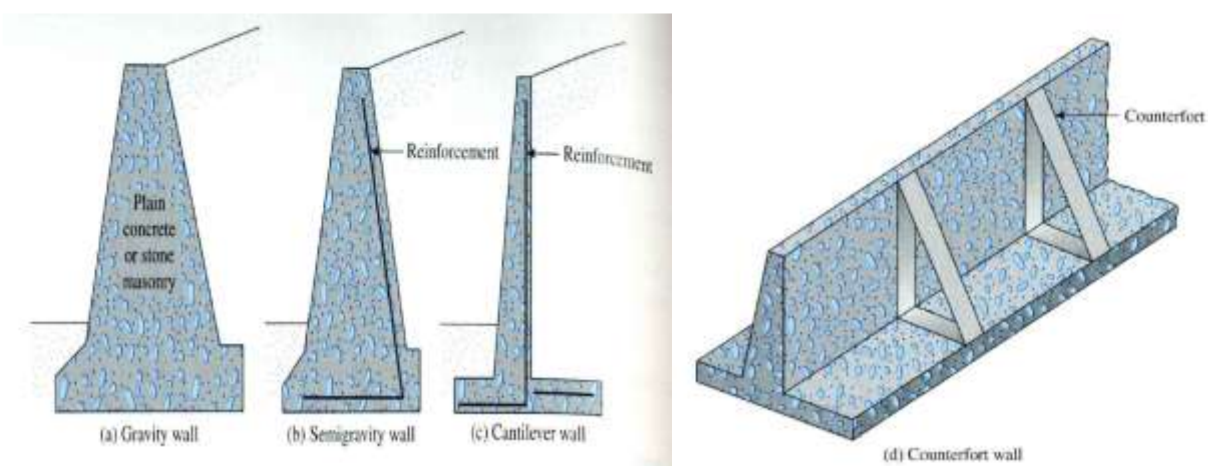


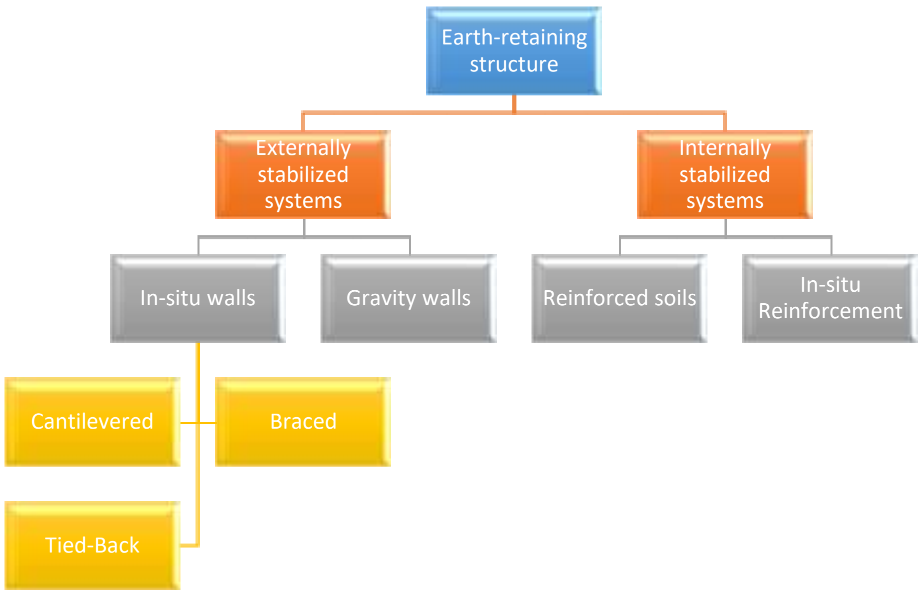
Figure 5.2 Classification of Retaining Wall

O'Rourke & Jones classification

O'Rourke & Jones (1990) classified earth-retaining structures into two broad categories:

1. Externally stabilized systems: are those that resist the applied earth loads by virtue of their weight & stiffness.
2. Internally stabilized systems: reinforce the soil to provide the necessary stability.

- Reinforced soils e.g. MSE
- In-situ reinforcement e.g. soil nailing



5.2. Design of conventional Retaining wall

The stability of a gravity wall is maintained mostly by its weight and partly by the passive resistance mobilized by the soil at the front of the wall. There are two phases of design in retaining wall:

1st the LEP known, the structure as a whole is checked for stability, overturning, sliding, and bearing capacity failures.

2nd each component of the structure is checked for strength, and the steel reinforcement of each component is determined.

Design Steps; Determine:

1. earth pressures
2. resultant thrust behind wall
3. soil reactions at base of wall (footing)
4. location of resultant soil reaction on base

    = take moments of all forces about toe of wall

Note:- N must be located within the middle third of the base to avoid tensile stress under the heel, the contact pressure at the toe must be equal to or less than the allowable bearing pressure and the settlement of the toe must be within the tolerable limits.

5.2.1. Gravity and Cantilever Walls

In designing retaining walls, an engineer must assume some of the dimensions proportioning. If the stability analysis yields undesirable results, the section can be changed and rechecked. The figures in the next slide shows the general proportions of various retaining-wall components that can be used for initial check.

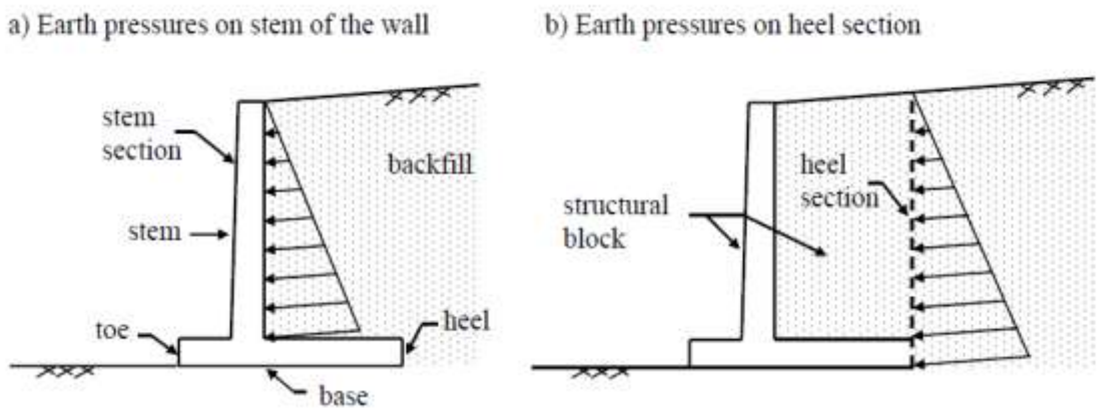


Figure 1. Sketch of design lateral earth pressures on cantilever retaining wall for a) internal stability and b) global stability.

Global stability (e.g. sliding and overturning) and

Internal stability (e.g. structural adequacy of the stem and base of the wall).

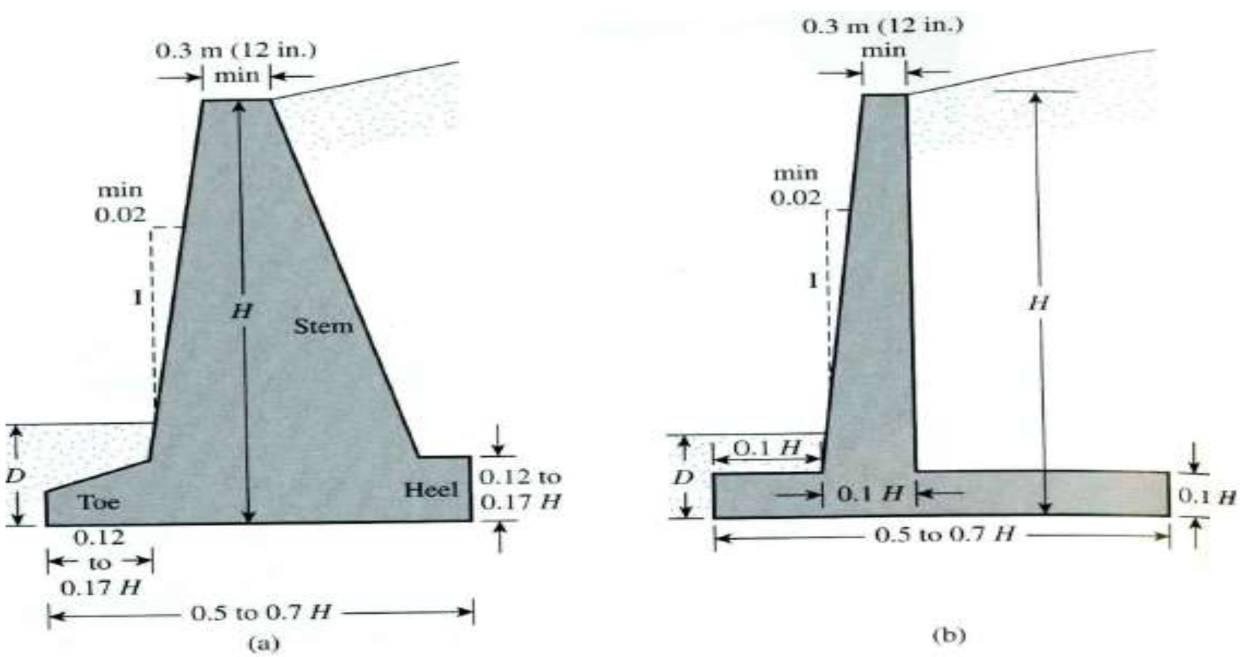


Figure 5.4 Gravity and Cantilever Retaining Wall

5.2.2. Counterfort Retaining Wall

Long cantilever retaining walls are often supported along their length by counterforts to increase their rigidity and strength. The general proportioning of the stem and the base slab is the same as for cantilever walls. However, the counterfort slabs may be about 0.3 m thick and spaced at center-to-center distances of 0.3H to 0.7H.

5.3. Application of Lateral Earth Pressures Theories to Design

5.3.1. Cantilever Retaining Wall

In the case of cantilever walls, the use of Rankine's earth pressure theory for stability checks involves drawing a vertical line AB through the point A as shown in the next slide. In the analysis of the wall's stability, the force $P_{a(Rankine)}$, the weight of the soil above the heel, and the weight W_c of the concrete all should be taken into consideration. The assumption for the development of Rankine active pressure along the soil face AB is theoretically correct if the shear zone bounded by the line AC is not obstructed by the stem of the wall.

The angle, η, that the line AC makes with the vertical is:

$$\eta = 45 + \frac{\alpha}{2} - \frac{\phi'}{2} - \frac{1}{2}\sin^{-1}\left(\frac{\sin\alpha}{\sin\phi'}\right)$$

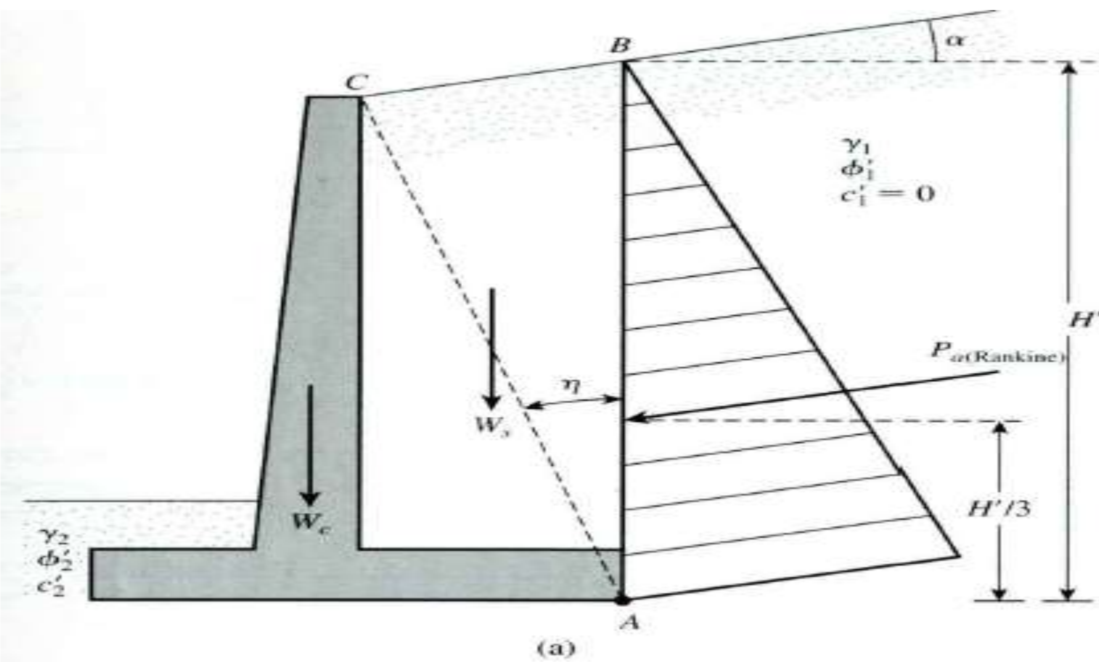


Figure 5.5 Cantilever Retaining Wall

5.3.2. Gravity walls

Similar analysis can be carried out for gravity walls in Rankine's theory. Coulomb's active earth pressure theory can also be used for gravity walls with the consideration of only $P_{a(Coulomb)}$ and W_c

.

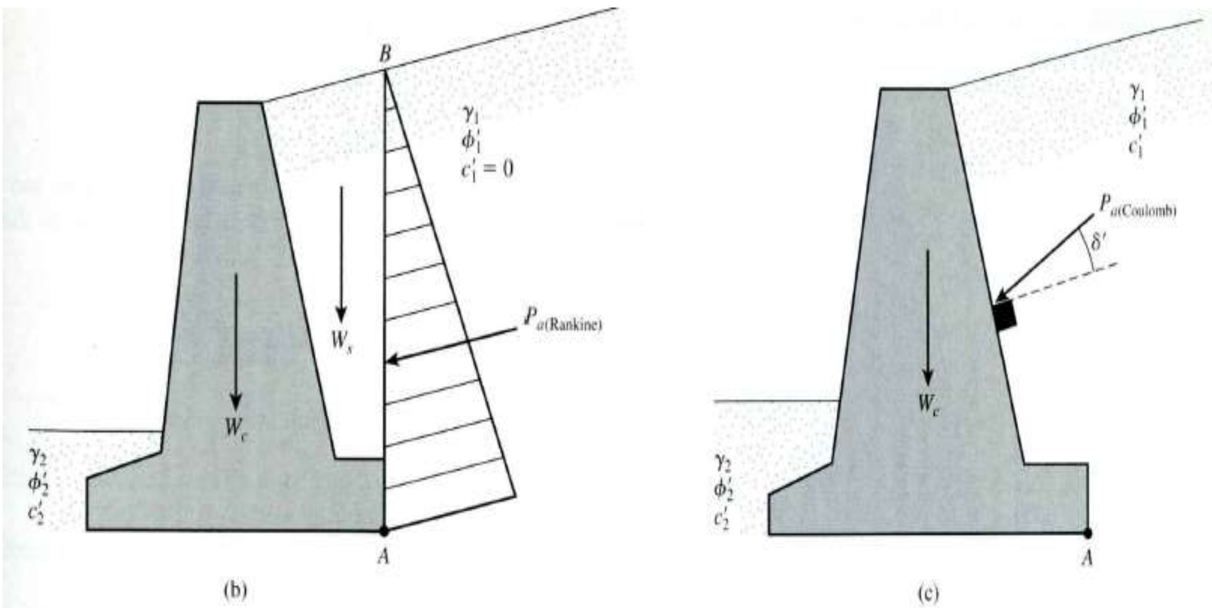


Figure 5.6 Gravity Wall

5.4. Failure Mechanism of Retaining Walls

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

a. It may overturn about its toe.

b. It may slide along its base.

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

d. It may undergo deep-seated shear failure

e. It may go through excessive settlement.

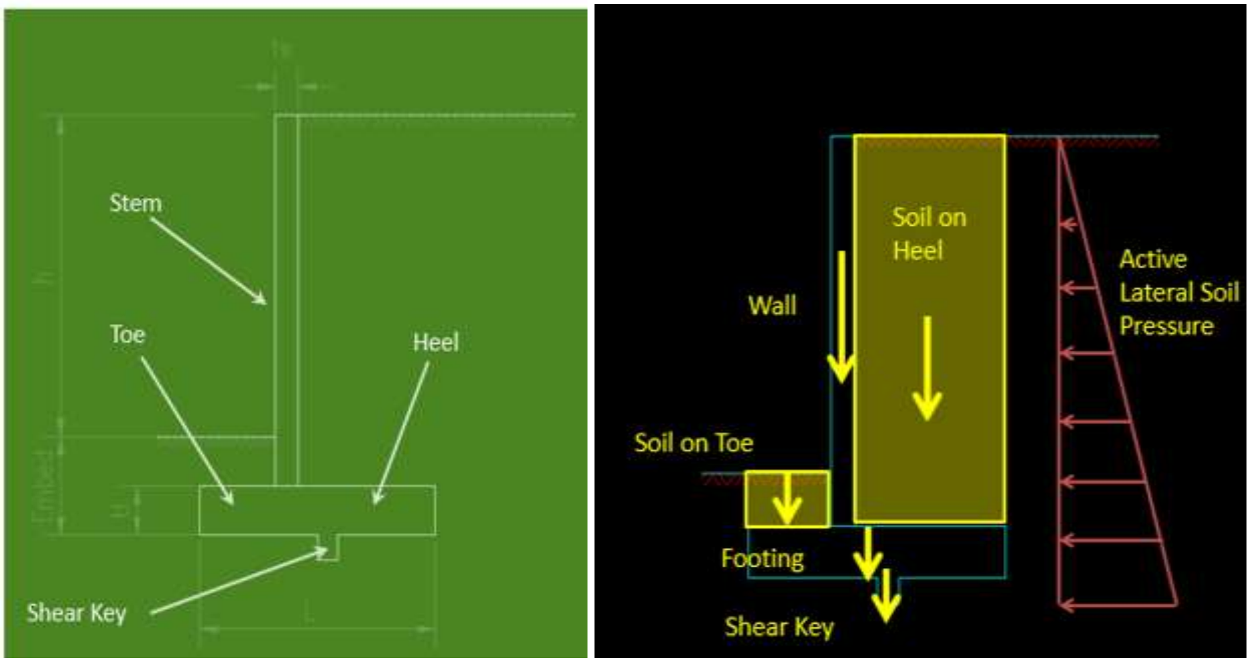


Figure 5.7 Forces Acting on Cantilever Retaining Wall

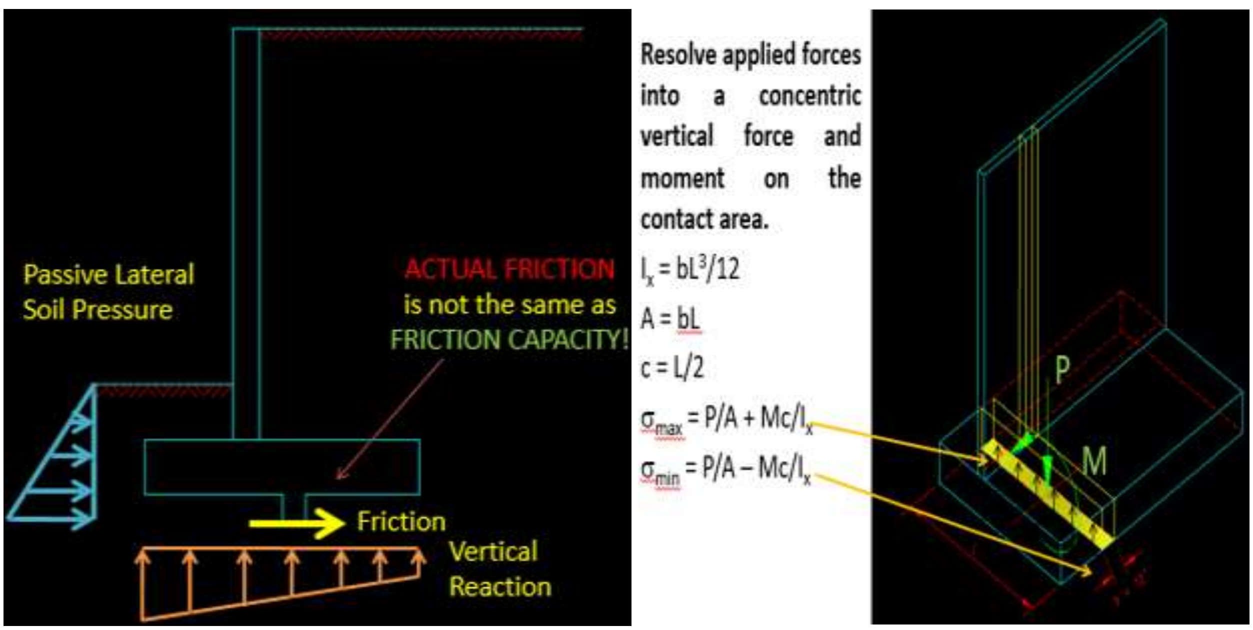


Figure 5.8 Reactions and Soil Bearing Stress

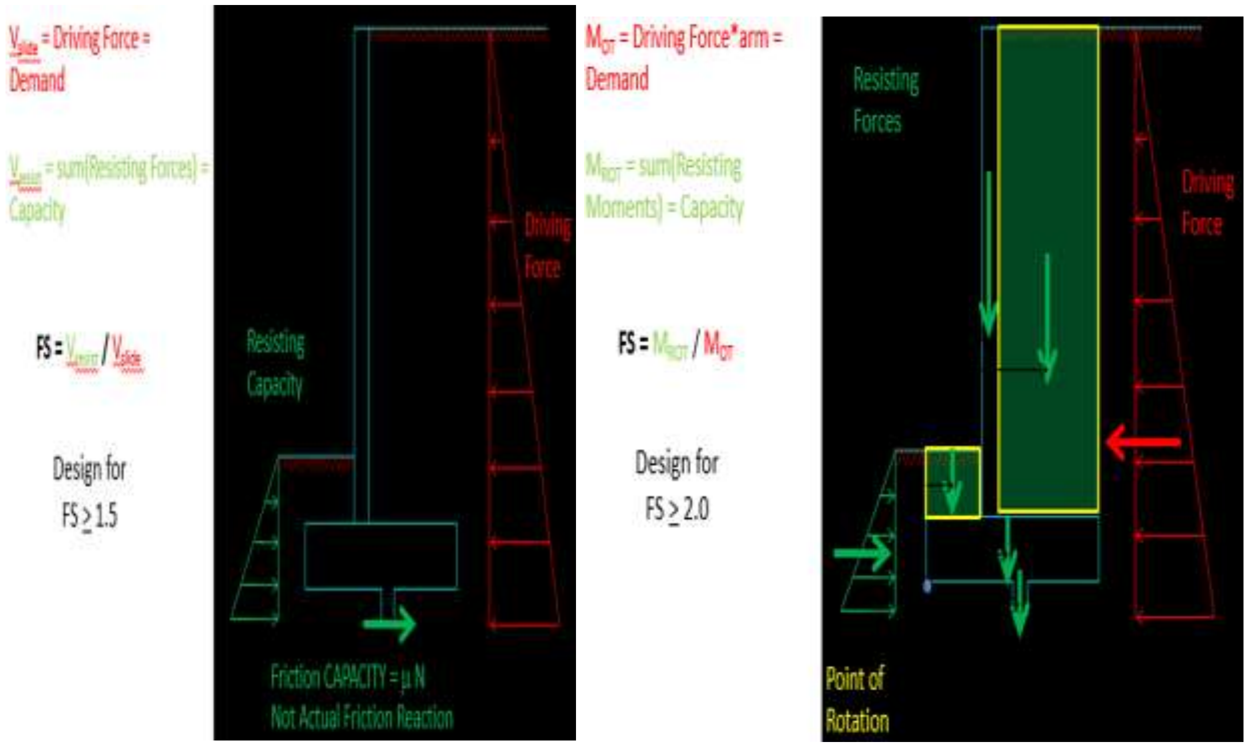


Figure 5.9 (a) Sliding and (b) Overturning

Stability Criteria

The stability of the retaining wall should be checked against :

(i) *FOS* against overturning (recommended FOS = 2.0)

$$FOS = \frac{Resisting\ moment}{Disturbing\ moment}$$

(ii) *FOS* against sliding (recommended FOS = 2.0)

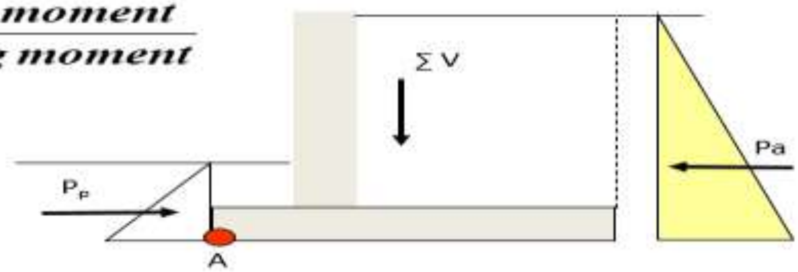
$$FOS = \frac{R_V\ tan\ \delta + (0.5 - 0.7)\ P_p + c_w B}{R_H}$$

Stability Analysis

The stability of the retaining wall should be checked against :

FOS against overturning
(recommended FOS = 2.0)

$$FOS = \frac{Resisting\ moment}{Disturbing\ moment}$$

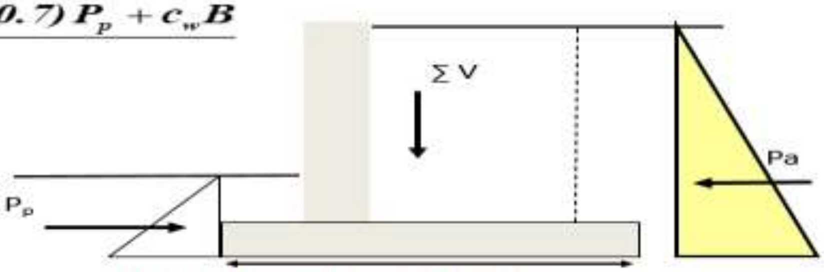


.. overturning about A

Stability Criteria

FOS against sliding (recommended FOS = 2.0)

$$FOS = \frac{R_V\ \tan\delta + (0.5 - 0.7)\,P_p + c_w B}{R_H}$$



Friction & wall base adhesion

Stability Criteria

For base pressure (to be compared against the bearing capacity of the founding soil. Recommended FOS = 3.0)

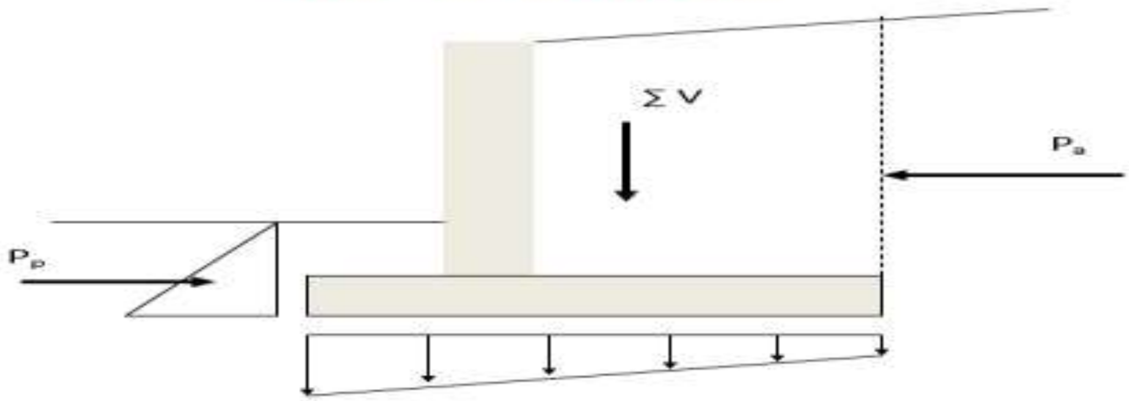
$$q_b = \frac{R_V}{B}\left\{1 \pm \frac{6e}{B}\right\}$$

Now, Lever arm of base resultant

$$\bar{x} = \frac{\sum Moment}{R_V}$$

Thus eccentricity $\quad e = \dfrac{B}{2} - \bar{x}$

Stability Analysis



Base pressure on the founding soil

Worked example :

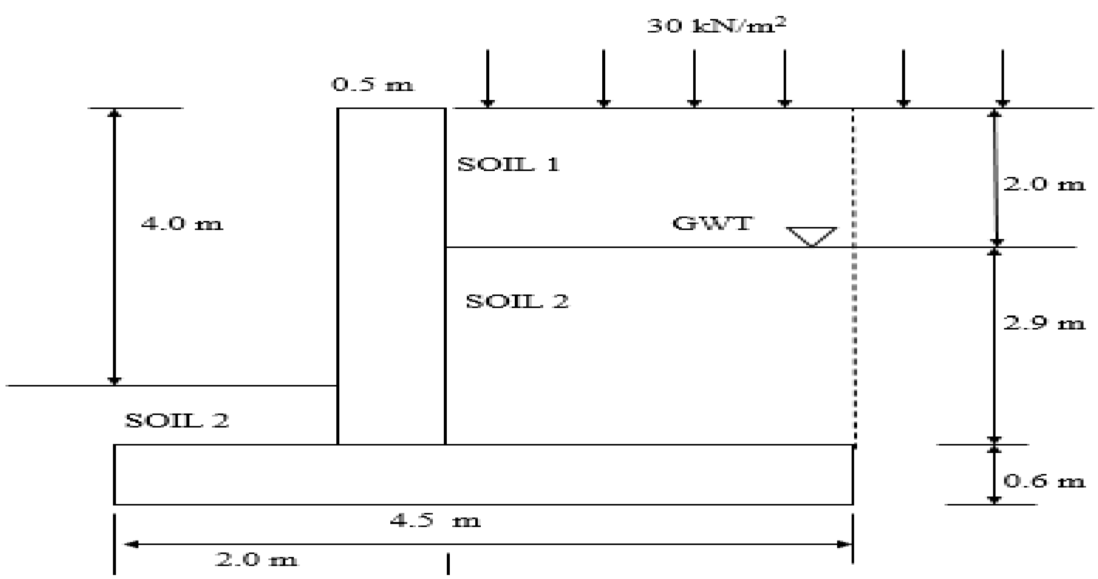
Figure below shows the cross-section of a reinforced concrete retaining structure. The retained soil behind the structure and the soil in front of it are cohesionless and has the following properties:

SOIL 1 : $\phi = 35°$, $\gamma_d = 17\ kN/m^3$,
SOIL 2 : $\phi = 30°$, $\delta = 25°$, $\gamma_d = 18\ kN/m^3$, $\gamma_{sat} = 20\ kN/m^3$

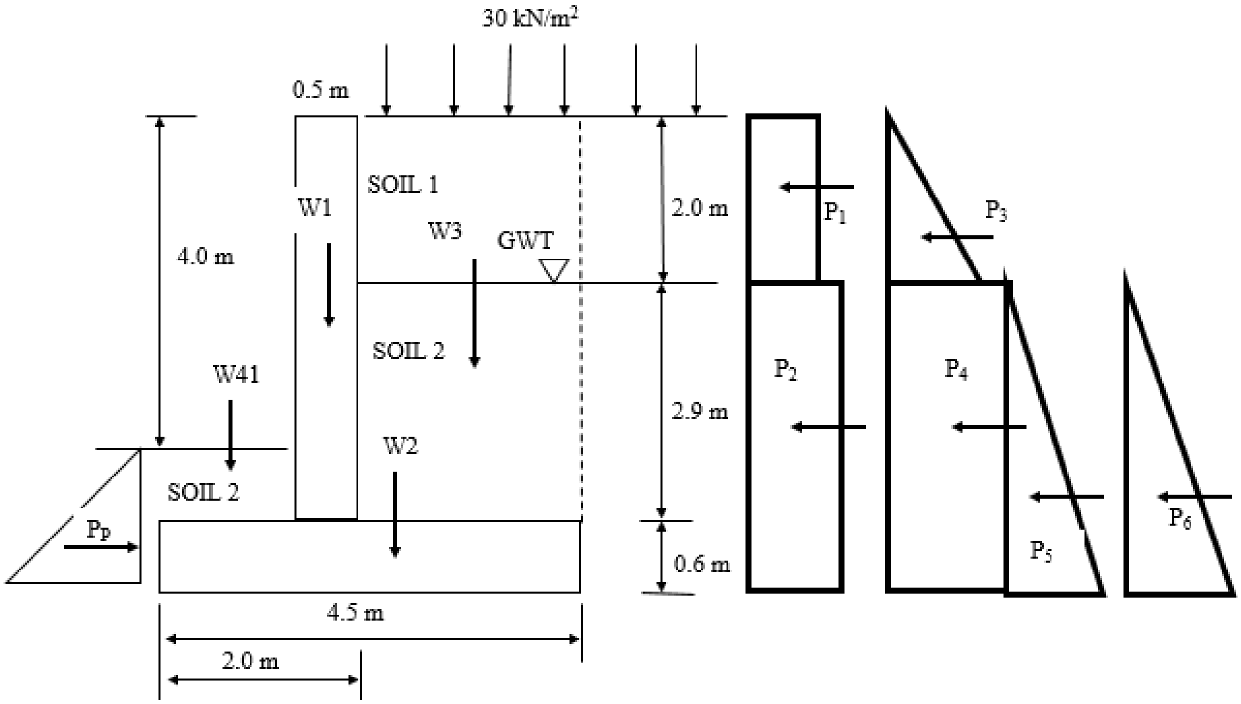
The unit weight of concrete is 24 kN/m³. Taking into account the passive resistance in front of the wall, determine a minimum value for the width of the wall to satisfy the following design criteria:

Factor of safety against overturning > 2.5
Factor of safety against sliding > 1.5
Maximum base pressure should not exceed 150 kPa

PROBLEM



SOLUTION



Determination of the Earth Pressure Coefficients

$$K_{a1} = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1 - \sin 35°}{1+\sin 35°} = 0.271$$

$$K_{a2} = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1 - \sin 30°}{1+\sin 30°} = 0.333$$

$$K_{p2} = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1 + \sin 30°}{1-\sin 30°} = 3.00$$

ELEM.	FORCE (kN/m)	TOTAL	L. ARM (m)	MOMENT (kNm/m)
HORIZONTAL				
Active				
P1	0.271 x 30 x 2	16.26	4.5	73.17
P2	0.333 x 30 x 3.5	34.97	1.75	61.20
P3	0.5 x 0.271 x 17 x 2 x 2	9.21	4.17	38.41
P4	0.333 x 17 x 2 x 3.5	39.63	1.75	69.35
P5	0.5 x .333 x (20-9.81) x 3.5 x 3.5	20.78	1.167	24.25
P6	0.5 x 9.81 x 3.5 x 3.5	60.09	1.167	70.13
	SUM	180.94		336.50
Passive				
Pp	0.5 x 3 x 18 x 1.5 x 1.5	60.75	0.5	30.38
VERTICAL				
W1	0.5 x 4.9 x 24	58.8	1.75	102.90
W2	0.6 x 4.5 x 24	64.8	2.25	145.80
W3	2 x 2.5 x 17 + 2.9 x 2.5 x 20 + 30 x 2.5	305	3.25	991.25
W4	0.9 x 1.5 x 18	24.3	0.75	18.23
	SUM	452.9		1288.55

To check for stability of the retaining wall

(i) FOS against overturning > 2.5

$$FOS = \frac{\text{Resisting moment}}{\text{Disturbing moment}} = \frac{1288.55}{336.50} = 3.83 \qquad > 2.5\text{, thus it is OK}$$

(ii) FOS against sliding > 1.5

$$\boldsymbol{FOS} = \frac{\boldsymbol{R_V \tan \delta + 0.5\, P_p}}{\boldsymbol{R_H}} = \frac{452.9\, \boldsymbol{\tan 25^o + 0.5 \text{ x } 60.75}}{180.94} = 1.34 \quad < \boldsymbol{1.5}$$

Thus it is not OK

(iii) For base pressure

$$q_b = \frac{R_V}{B}\left\{1 \pm \frac{6e}{B}\right\}$$

Now, Lever arm of base resultant

$$\bar{x} = \frac{\sum \text{Moment}}{R_V} = \frac{1288.55 - 336.5}{452.9} = 2.10$$

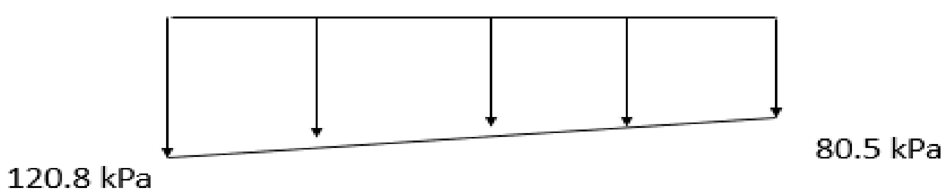
Thus eccentricity $e = \frac{B}{2} - \bar{x} = 2.25 - 2.10 = 0.15$

Therefore $q_b = \frac{452.9}{4.5}\left\{1 \pm \frac{6 \times 0.15}{4.5}\right\} = 120.8 \,\&\, 80.5 kPa$

q_b = 120.8 and 80.5 kPa

Since maximum base pressure is less than the bearing pressure of the soil, the foundation is stable against base pressure failure.

DISTRIBUTION OF BASE PRESSURE



In conclusion the retaining wall is not safe against sliding. To overcome this the width of the base may be increased or a key constructed at the toe.

Structural Design Process

- Select the overall dimensions (height, embedment, footing length and position, and estimated footing & wall thicknesses) based on stability (sliding and overturning) and soil strength (max/min bearing pressures) using service level loads.
- Check slab (wall and footing) thicknesses using shear criteria and factored loads. Adjust thicknesses as necessary, rechecking stability and soil strength of the values change.
- Select the flexural steel for the three cantilever slab elements using factored loads.
- Select the temperature and shrinkage steel for wall and footing.
- Draw the resulting wall cross section (to scale!)

Design Elements to Prevent Failure

Relieve water pressure

(for all 3 types of failure)

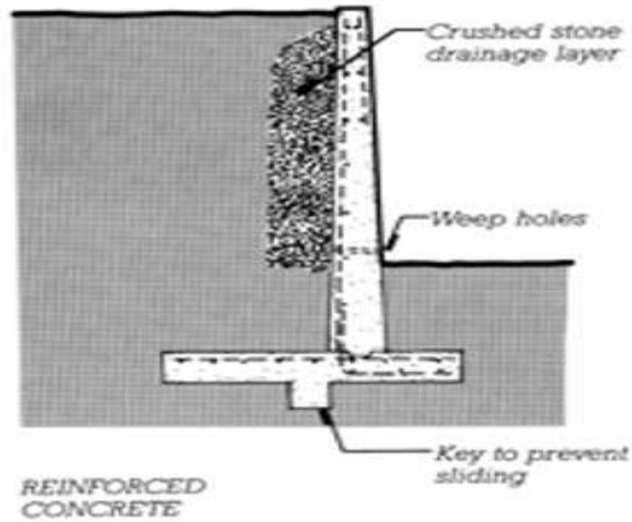
- Crushed stone
- Weeps

Overturning

- Cantilevered Footing
- Reinforcing

Sliding

- Key



5.5. Mechanically Stabilized Retaining Walls

Recently, soil reinforcement has been used in the construction and design of foundations, retaining walls, embankment slopes, and other structures. Depending on the type of construction, the reinforcements may be galvanized metal strips, geotextiles, geogrids, or geocomposites. Reinforcement materials such as metallic strips, geotextiles, and geogrids are now being used to reinforce the backfill of retaining walls, which are generally referred to as mechanically stabilized retaining walls.

Figure 5.10 Mechanically Stabilized Earth (MSE) Retaining Walls

The beneficial effects of soil reinforcement derive from: (1) the soil's increased tensile strength and (2) the shear resistance developed from the friction at the soil reinforcement interfaces. Such reinforcement is comparable to that of concrete structures.

Improved Earth Walls or Internally Stabilized Systems

Improved earth walls consist of stabilized backfill soil and facing elements. Improvement of the soil (apart from compaction) is carried out either by means of chemicals such as cement or lime or other chemicals or by using inclusions. Woven. Soils stabilized by cement or lime normally fall into the category of a Mohr-Coulomb material with improved shear strength parameters.

Soil reinforcement

A mechanically stabilized soil is reinforced by strips or grids that may be metallic, polymeric or organic. A mixture of soil and polymeric elements of fine diameter and small length has also been used. The main objective is to transfer the tensile stresses to reinforcement elements. Anchored earth systems combined by soil reinforcement have been developed and applied successfully in highway construction.

5.6. Considerations in Soil Reinforcement

Metal Strips: In most instances, galvanized steel strips are used as reinforcement in soil.

Non-biodegradable Fabrics: Non-biodegradable fabrics are generally referred to as geotextiles.

5.6.1. Reinforcement Material

There are two types of materials used

1. **Metallic reinforcements:** Typically mild steel (galvanized / epoxy coated)
2. **Nonmetallic reinforcement:** Generally polymeric materials (polypropylene, polyethylene or polyester)

Figure 5.11 Galvanized Steel Strips

Figure 5.12 Galvanized Wire Mesh

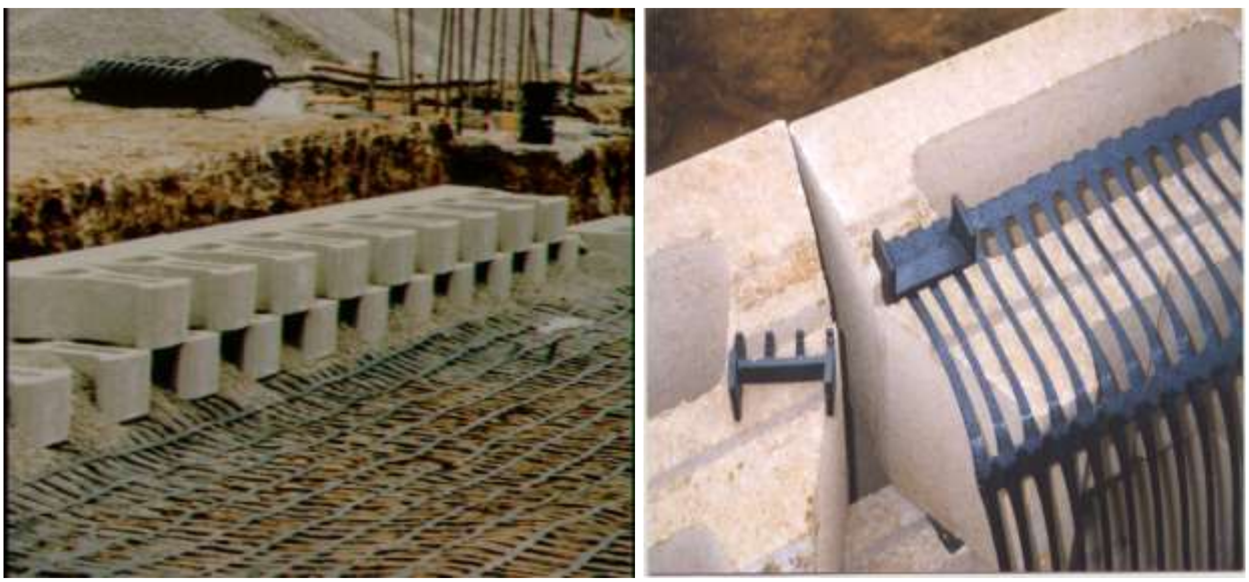


Figure 5.13 Geogrids

5.7. Primary uses Geo-textiles

Geo-textiles have four primary uses in foundation engineering:

1. **Drainage:** The fabrics can rapidly channel water from soil to various outlets, thereby providing a higher soil shear strength and hence stability.
2. **Filtration:** When placed between two soil layers, one coarse grained and the other fine grained, the fabric allows free seepage of water from one layer to the other. However, it protects the fine-grained soil from being washed into the coarse-grained soil.
3. **Separation:** Geo-textiles help keep various soil layers separate after construction and during the projected service period of the structure. For example, in the construction of highways, a clayey sub-grade can be kept separate from a granular base course.

4. **Reinforcement:** The tensile strength of geo-fabrics increases the load-bearing capacity of the soil.

5.8. Geogrids

Geogrids are high-modulus polymer materials, such as polypropylene and polyethylene, and are prepared by tensile drawing. The major function of geogrids is reinforcement. Geogrids are relatively stiff netlike materials with openings called apertures that are large enough to allow interlocking with the surrounding soil or rock to perform the function of reinforcement or segregation (or both).

5.8.1. Geogrid types

Geogrids generally are of two types:

1. Uniaxial and
2. Biaxial

Uniaxial TENSAR grids are manufactured by stretching a punched sheet of extruded high-density polyethylene in one direction under carefully controlled conditions - results in a product with high one-directional tensile strength and a high modulus.

Biaxial TENSAR grids are manufactured by stretching the punched sheet of polypropylene in two orthogonal directions. This process results in a product with high tensile strength and a high modulus in two perpendicular directions.

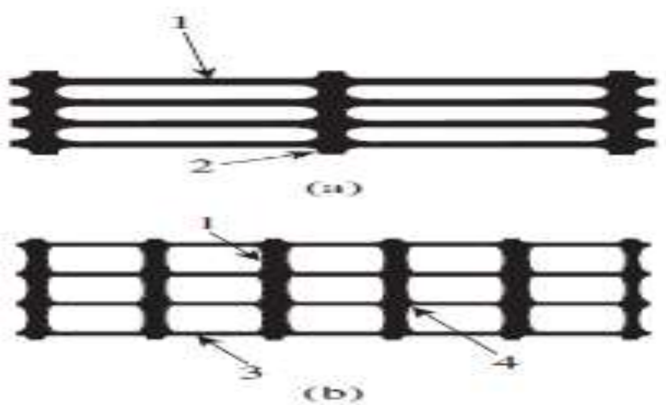


Figure 5.14 Geogrids: (a) uniaxial; (b) biaxial (Note: 1—longitudinal rib; 2—transverse bar; 3—transverse rib; 4—junction)

5.9. General Design Considerations

The general design procedure of any mechanically stabilized retaining wall can be divided into two parts:

1. Satisfying internal stability requirements
2. Checking the external stability of the wall

The internal stability checks involve determining tension and pullout resistance in the reinforcing elements and ascertaining the integrity of facing elements. The external stability checks include checks for overturning, sliding, and bearing capacity failure.

5.10. Retaining Walls with Metallic Strip Reinforcement

Reinforced-earth walls are flexible walls. Their main components are

1. Backfill, which is granular soil
2. Reinforcing strips, which are thin, wide strips placed at regular intervals, and
3. A cover or skin, on the front face of the wall

The simplest and most common method for the design of ties is the Rankine method.

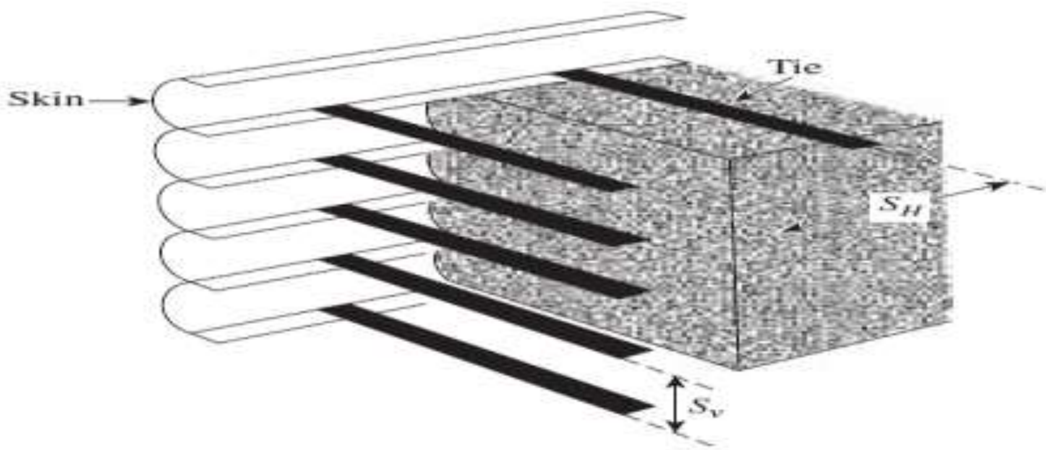


Figure 5.15 Tie and Skin

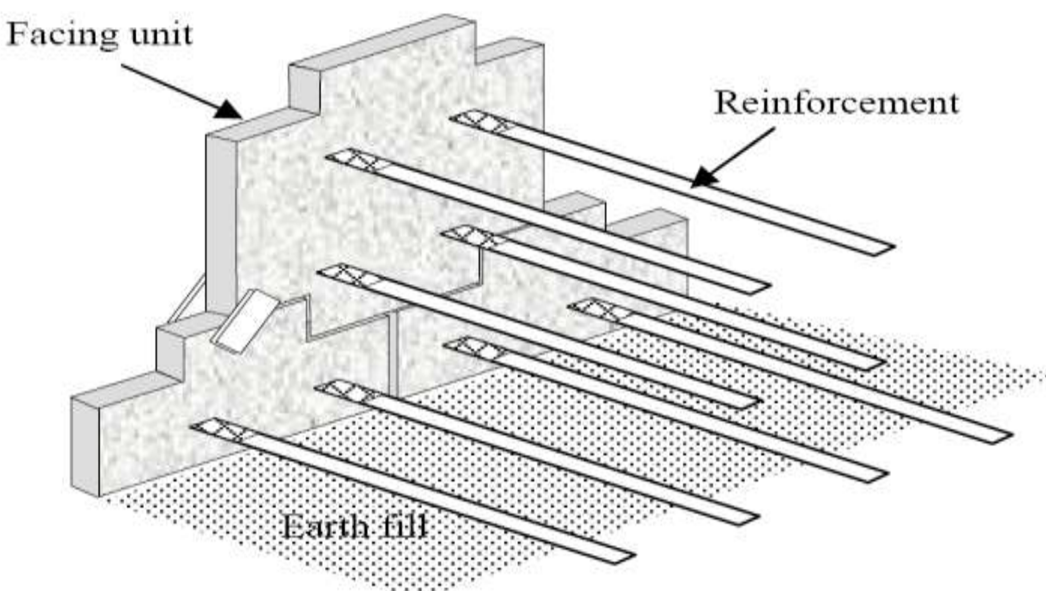


Figure 5.16 Component of Earth Retaining Wall

5.11. Sheet Pile Walls

Steel sheet piles were originally used for temporary cofferdams. Since 1930 they have been used for permanent maritime works, such as quay walls, jetties, locks, dikes, dry docks and floodwalls. However, their use was limited by a lack of understanding their structural behavior. Steel sheet piles are also used for bridge abutments, retaining walls in cuts and underpasses for railroads and highways, support elements in underground parking.

Figure 5.17 Sheet pile Walls

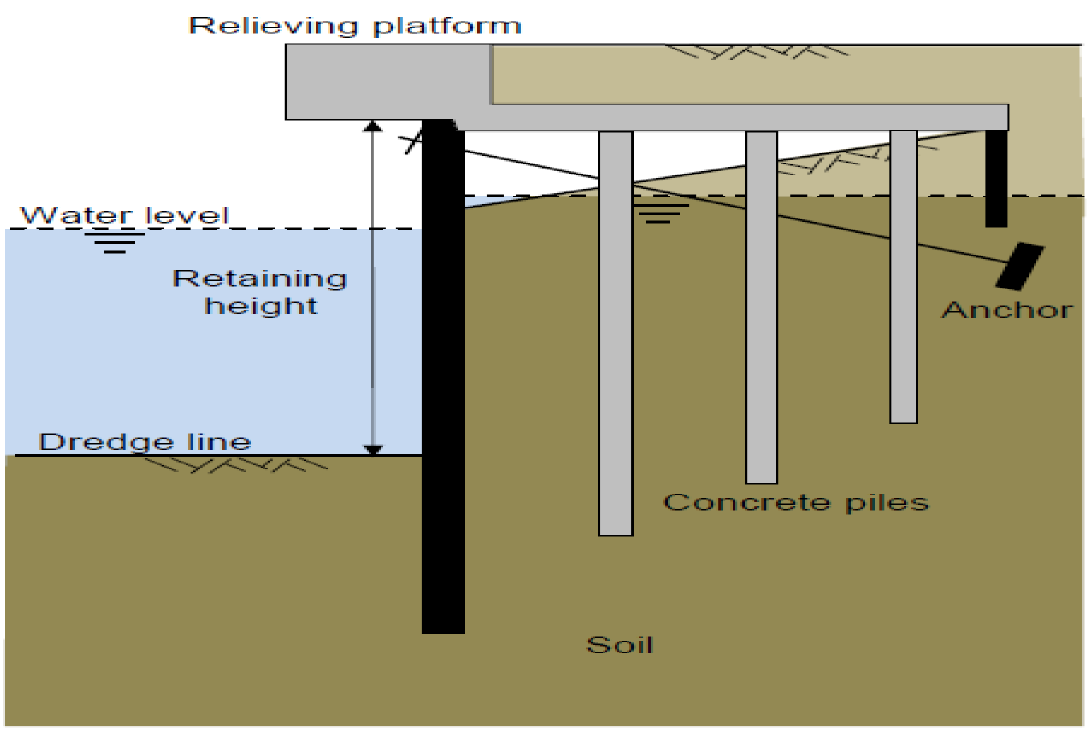


Figure 5.18 Schematic view of the quay wall

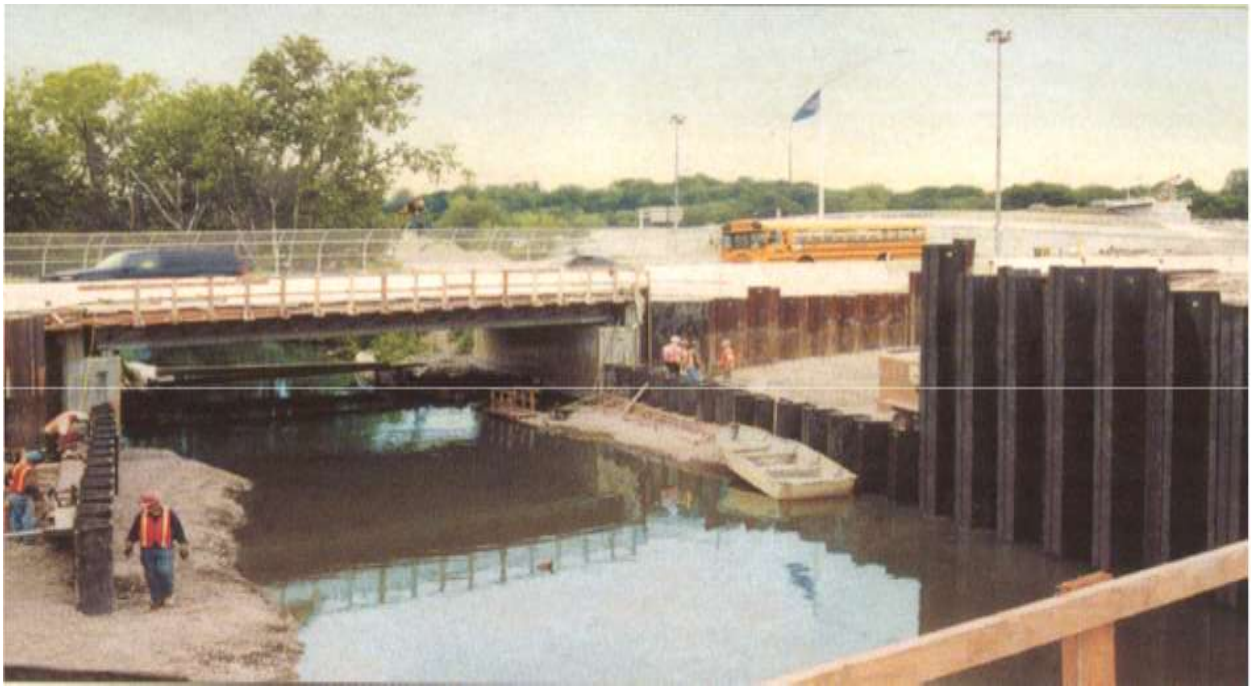


Figure 5.19 Sheet piling used to expand a bridge's width, while maintaining highway and river traffic

5.11.1. Types of Sheet Piles

Wooden sheet piles: are typically 2"x12" tongue-and-groove boards. They are only useful for very low walls with a free height of H 3 m. Currently they are only used to stabilize the walls of temporary trenches in order to install sewers and other utilities. They are placed to a working stress of $f_w = 0.65$ to $0.95\ f_y$.

Pre-cast concrete panels.

Steel sheet piles: are the most commonly used because they are the only type that can be driven into soft rocks or gravelly soils. They are driven or vibrated into position with high working $f_w = 0.60 - 0.90\ f_y$. The advantages of using steel sheet piling for walls are: they are resistant to high driving stresses, relatively of light weight, pile length can be increased by welding a new sheet on top of the driven sheet, and they can be removed from the ground when finished, and reused.

Aluminum sheet piles.

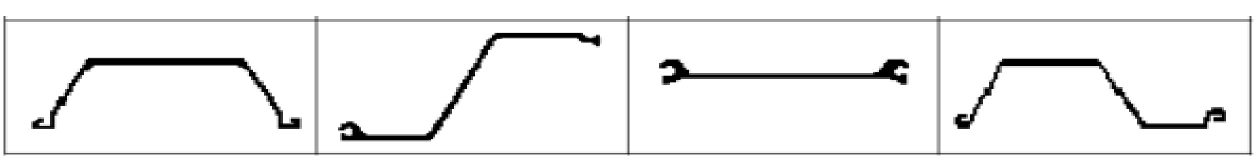
Plastic sheet (PVC) piles.

Figure 5.20 Sheet Piles

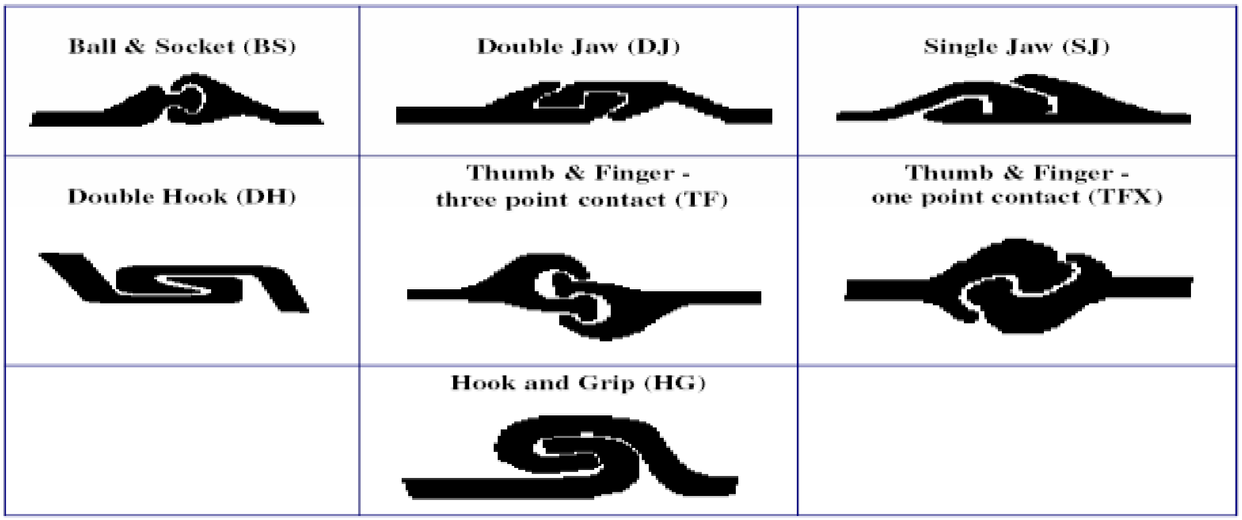
Figure 5.21 Sheet-piling used to divert a river while building a dam and locks on the White River in Arkansas

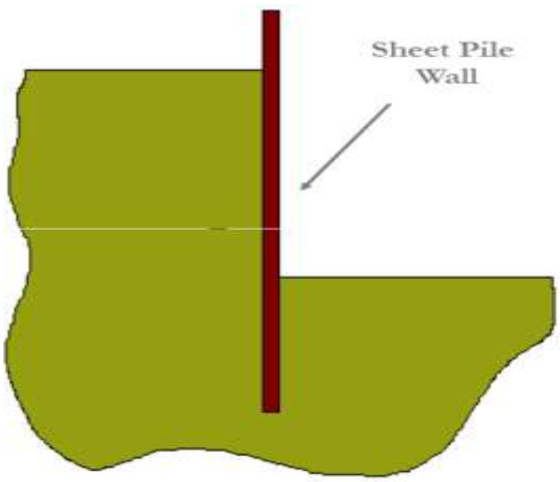
5.11.2. Shape Classifications

Z-type (Z)	*Larssen and other "U" types (U)*	*Flat or straight - web types (SA), (S)*	*Arch shaped and lightweight "gauge" sheets (A)*
used for intermediate to deep wall construction	**used for applications similar to Z-piles.**	**with strong interlocks, and little beam strength, for filled cell construction**	**used for shallower wall construction.**

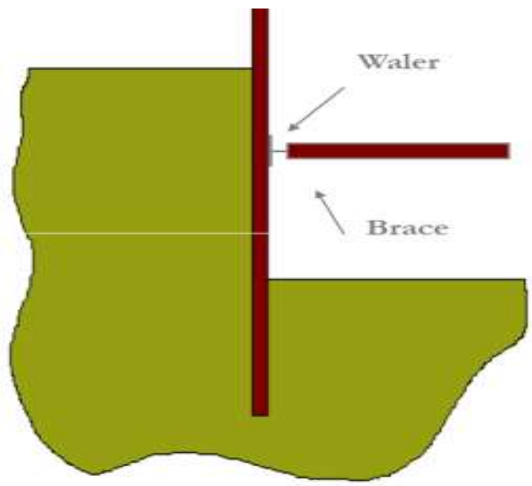


Types of interlocks

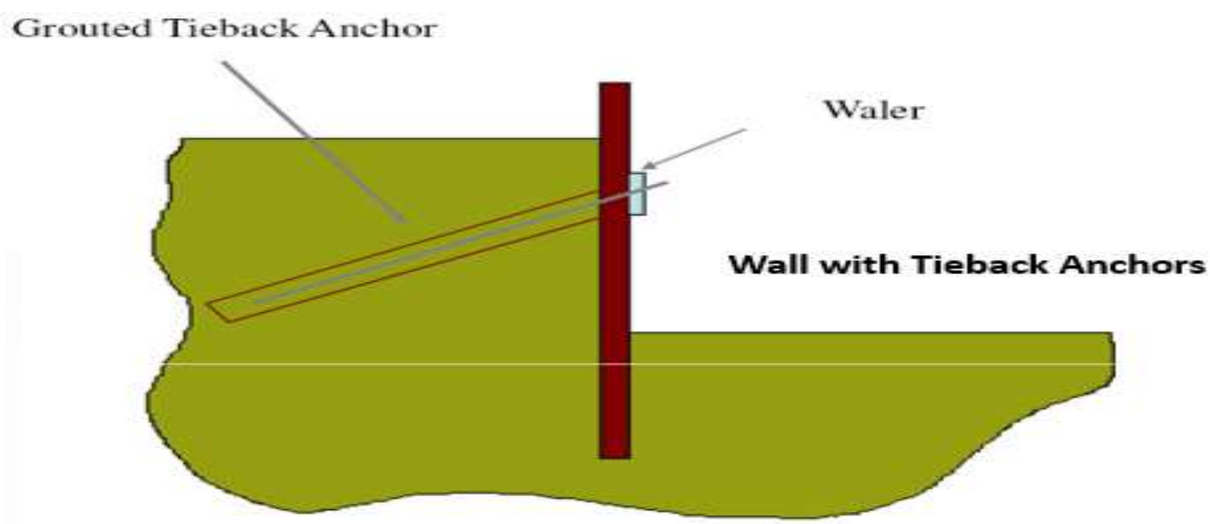




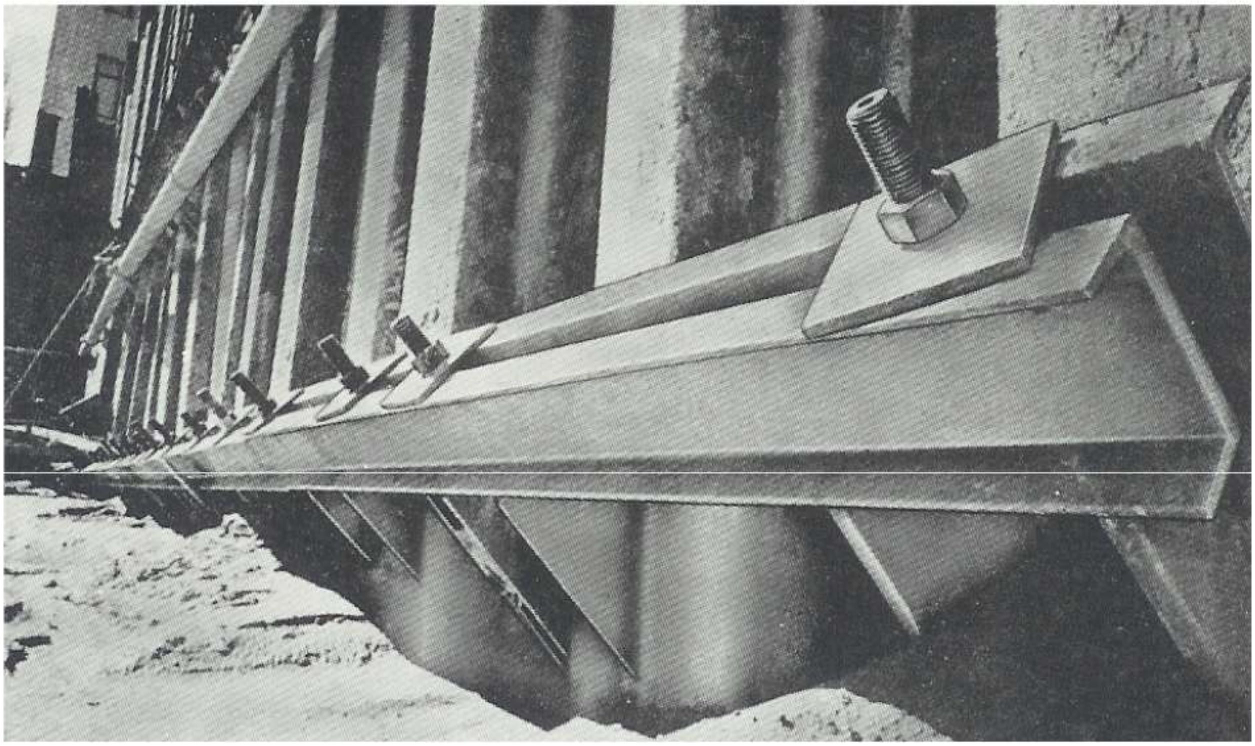
The cantilever wall is like a balcony beam



Wall with internal bracing



Earth anchors (tiebacks) are tensile members that apply a stabilizing forces onto the slope. They generally consist usually of steel rods or post-tensioning cables. Their anchoring must be sufficiently extended well beyond the critical failure surface. They are more expensive than internal bracing, such as struts or rakers, but provide an uncluttered excavation and increase the safety of the site.



This anchored steel sheet pile wall uses double channel beams for wales. The anchors are Dywidag (threaded) bars bearing on steel plates.

5.11.3. Construction methods

Sheet pile walls (SPW) may be divided into two basic categories:

1. Cantilever and
2. Anchored

Construction method generally can be divided into 2 categories:

a. Backfilled structure
b. Dredged structure