CHAPTER ONE

SITE EXPLORATION

2.1 Definition and Purpose

Definition: Site exploration is a term covering *field* and *lab* investigations of a site for gathering information on the layers of deposits that underlain a proposed structure for economical and safe design of foundation. It shall always be a *prerequisite* for foundation design.

Purpose: Site exploration shall be made:

- To select among alternative sites
- To decide on the type and depth of foundation
- To estimate the load bearing capacity & probable settlement
- To select appropriate method of construction
- To select and locate construction materials
- To evaluate the safety of existing structures
- To determine ground water location

Extent: Site exploration extent depends on:

- Importance of structure
- Complexity of soil conditions
- Foundation arrangement
- Availability of equipment and skill
- Relative cost of exploration
- Information available from performance of existing structures

The least details are required in a highway project, as the soil needs to be explored only up to a depth of 3m or so. More details and deeper explorations are however, required for heavier, multistoried buildings, bridges, dams, etc.

Cost: ranges between 0.1 to 0.5% of the total cost of the project

Information obtained from soil exploration include: general topography and accessibility of the site, location of buried services: power, communication, supply, etc, general geology of the site, previous history and use of the site, any special features (erosion, earth quakes, flooding, seasonal swelling and shrinkage of the soil etc), availability of construction materials, a detailed record of the soil and rock strata including ground water condition, lab and field results of the various strata, results of chemical analysis if any is made etc.

2.1 Planning the Site Exploration (**1**HLCH-1.doc)

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

1 **Assembly of Available Information**

On the specific use of the site, about the site, about the structure- on dimensions, column spacing, type and use, basement or any special requirement etc. Referring building codes is also essential.

2 **Reconnaissance Survey:**

- survey of existing literatures, maps, etc
- a close visual inspection by walking over the site
    - wrinkling of the surface on a hill side slope and leaning trees ⇒ soil creep and potential stability problem
    - Flat low lying areas in the valleys ⇒ lacustrine or river deposits

- Marshy ground with weeds ⇒ shallow GWT
- Cracks, sags, sticking doors and windows in existing light buildings ⇒ expansive soils
- Out crops of rocks and profile of existing gullies and cuts, stream patterns, etc ⇒ indication of the geology of the site

3 **Preliminary Ground Investigation:**

It is done by making few borings, opening pits and field sounding tests to establish the general stratification. Sampling is mostly limited to acquisition of representative samples. The number of quality samples should be limited to a minimum. Important parameters (like shear strength and compression) are mainly estimated from correlations with the index properties obtained on the representative (disturbed) sample. The information obtained at this stage can be used for preliminary design of the foundation.

4 **Detailed Ground Investigation:**

Where the preliminary investigation has indicated the feasibility of the project at the site, more detailed site exploration should be under taken. Depending on the results obtained from the preliminary investigation, additional and deeper boreholes may be sunk; more samples extracted (both disturbed & undisturbed); more sounding field tests undertaken to obtain information which shall be sufficient for final design.

2.1 Methods of Site Exploration

The major methods of soil explorations include boring (and sampling), sounding tests, load tests, shear tests, field density tests, geophysical explorations, etc

1. **BORING:** enables one to extract continuous or discrete samples for visual inspection and testing to determine properties of soils. Methods of boring can be:

a) Test Pits:

- Simplest , cheapest method of shallow investigation
- Provide clear picture of stratification
- Weak lenses and pockets can be identified
- Block samples can be easily extracted from which undisturbed samples are obtained – called chunk sampling
- If GWT is encountered near the ground surface, bore holes are preferred. Pits cannot be dug in silts or sands below the water table or in soft clays because the sides will collapse, endangering the excavation machine & its operator.
- Commonly it is uneconomical to go deeper than 5m
- It is easier to take good undisturbed soil samples from a trial pit than a bore hole; to carry out in-situ tests (such as SPT & vane shear test)

b) Bore Holes:

- Most common for deep investigations
- Mostly done by power-driven machines

Bore hole drilling methods include:

i) Auger boring: boring a hole using augers operated either by hand or machine.

Hand operated auger (Figure 1.1)

- The hand operated augers may be helical types or post-hole auger.
- It can be used for depths up to 3 to 5m. Diameter of holes varies from 5 to 20cm.
- Generally suitable for all types of soils above water table but suitable only below water table in clay soils. Soils with boulders & cobbles are difficult to investigate using augers. Also limited use in sandy soils b/c they do not stick to the auger.

- Are generally used for making subsoil explorations for high ways, runways, railways etc where the explorations are generally confined to depths of about 5m or so.
- *Machine operated auger*: are suitable in all types of soils and can go to deeper depths. The hollow stem can be used for sampling or conducting SPT test and plugged when not in use. They are capable of penetrating up to 50m.

ii) Wash boring: (Figure 1.2)

- Is machine operated boring
- Involves pushing or driving of casings ahead of boring operation and drilling is facilitated through by means of a chopping bit attached at the bottom of flight of hollow drilling rod. Water is pumped which helps in disintegration and facilitates loosening of the soil. Slurry rises up; screened in to soil solids and water.
- The method is rapid except in hard strata and soils with boulders. The machine is light so that it can be easily transported to relatively in accessible areas. It causes not so much disturbance to underlying material.
- Undisturbed samples can be extracted easily by pushing thin walled sampler (split spoon sampler). However the effect of water must be taken in to consideration.
- Disadvantages: - there may be undetected thin layer and high alteration of moisture content.

iii) Rotary Drilling: (Figure 1.3 (a))

- It is generally trailer mounted or lorry mounted.
- Bore hole is advanced by power rotated drilling (cutting) bit (**2**HLCH-1.doc) with simultaneous application of pressure. The drilling bit is carbide or diamond and is attached to the drilling rods.
- Most rapid method in almost all soils. Fluid usually water is used to cool the edges and reduce friction.
- Undisturbed sample can be obtained by attaching special sampler usually split spoon sampler.
- Disadvantage: not suitable for highly fissured rocks (gravelly soils), as gravels do not break easily, but rotate beneath the bit, expensive

iv) Percussion Drilling: (Figure 1.3 (b))

- Involves alternately rising and falling of a heavy chisel-like bit. The drilling activity disintegrates the material below in to the sand silt size. Water is added to loosen soil and chiseler chisels and the loose material (slurry) is scooped out by a bailer. The bailer is generally attached to the boring rod after removing the tool bit at intervals, and then lowered to the hole. It has a non returning valve.
- It can be adopted in almost all types of soils, and is particularly useful in very hard soils or soft rocks.
- Disadvantage: impossible to detect thin compressible layers, high disturbance of soil, expensive

In all types of drilling used in soft soils that may cave in, casing is used. Drilling mud usually bentonite clay may also be used to stabilize the soil instead of casing.

3.2 Layout, Number and Depth of Bore Holes

Layout /Spacing:

- While layout of the structure is not yet ready, evenly spaced grid of bore holes is commonly used
- Whenever possible bore holes should be located close to proposed foundations
- For light weight structure like residential houses, it is wise to locate test pits away from the foundation locations
- Approximate spacing of bore holes may be as follows:
  - Multi storey buildings ------10 m to 50 m
  - One storey industrial buildings-----20 m to 60 m
  - Highways ------250 m to 500 m

- Dams ------- 40 m to 80 m

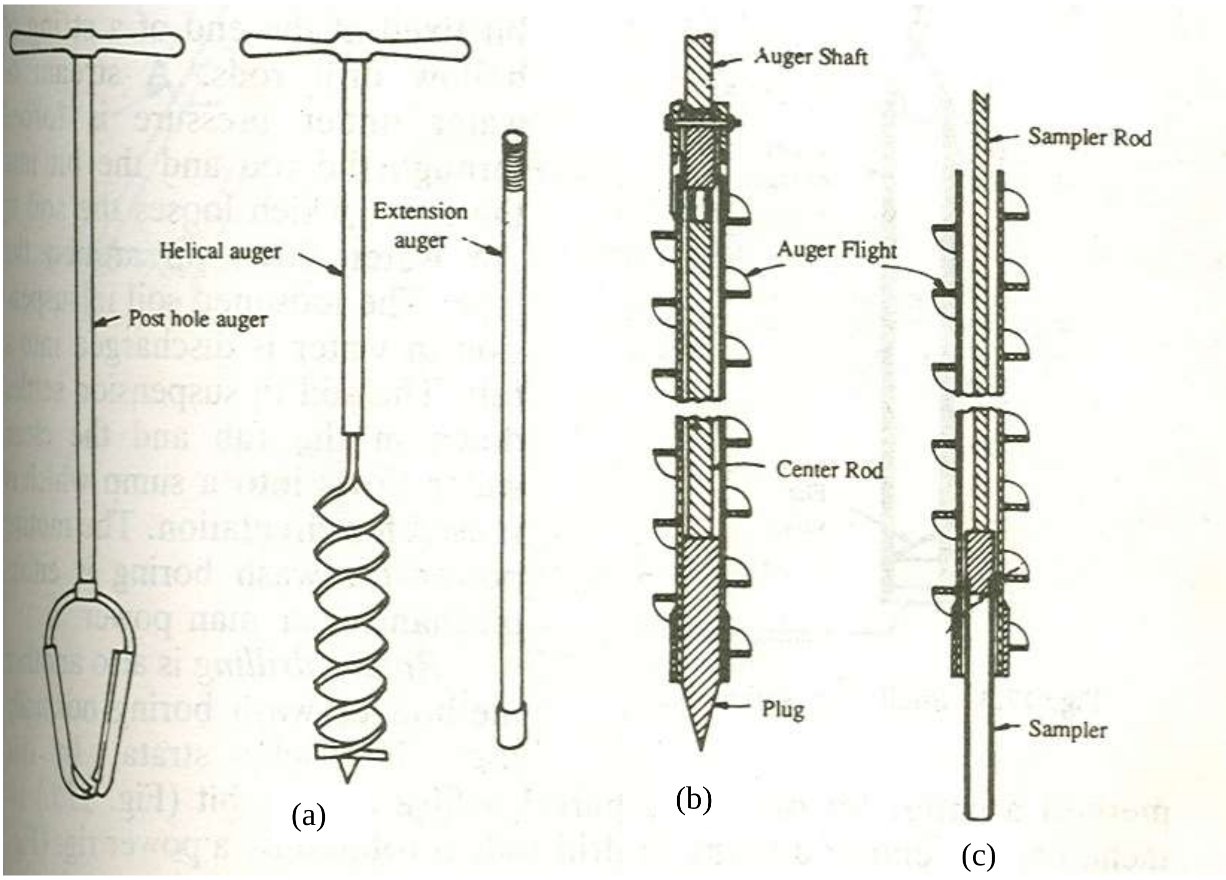
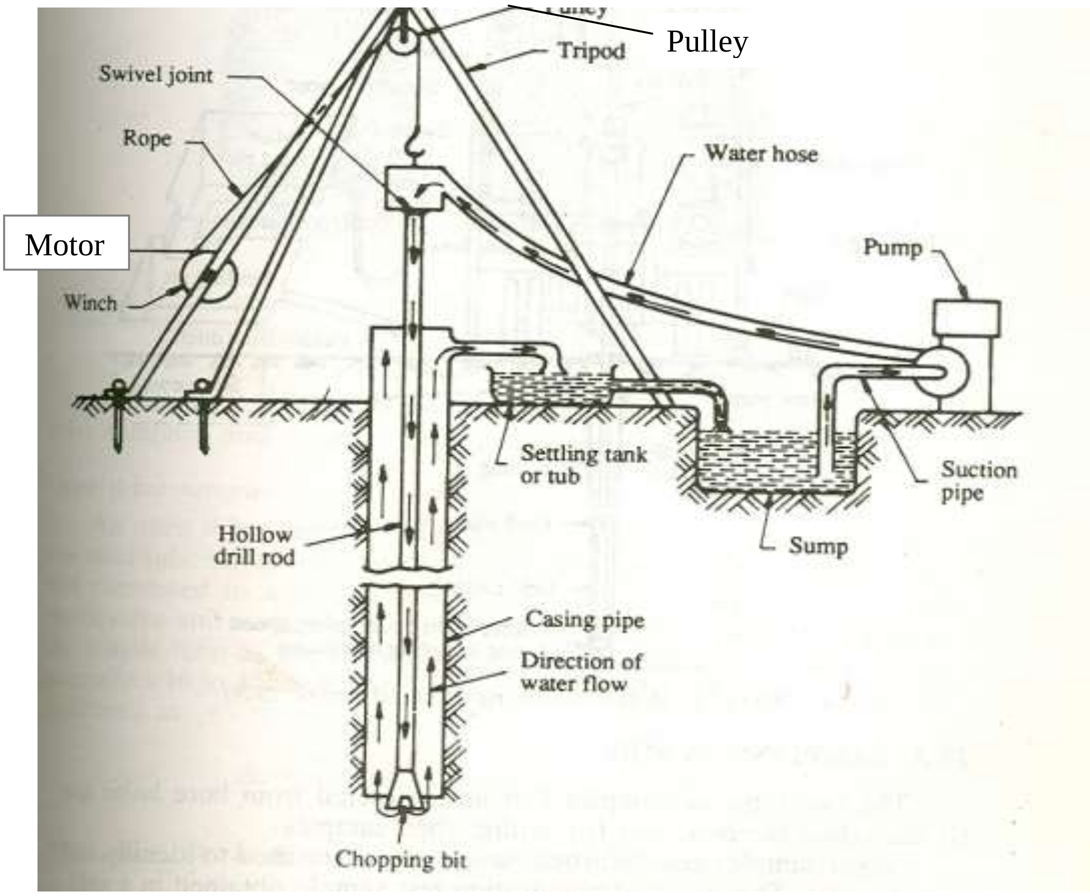
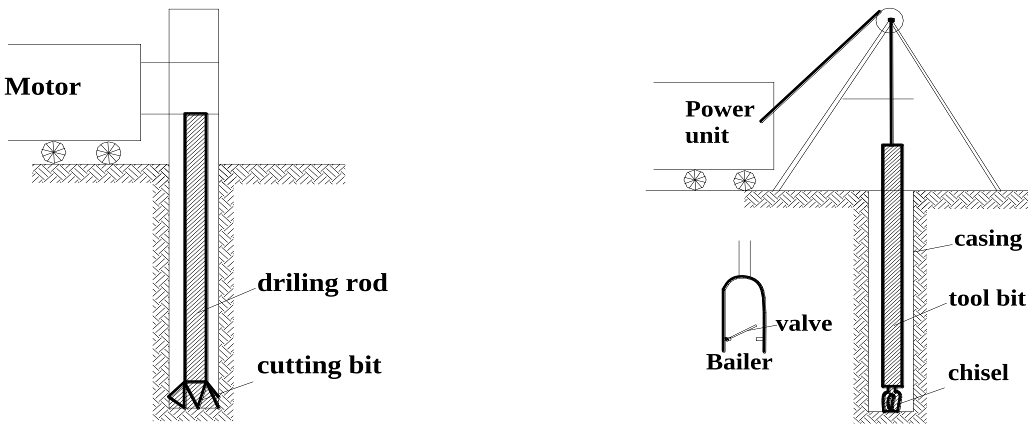


Figure 1.1: (a) Hand Augers (b) Hollow-stem auger plugged while advancing the auger (c) Hollow-stem auger plug removed and sampler inserted to sample soil below auger





(a) Rotary Driling (b) Purcussion Driling

Figure 1.2: Wash boring

Figure 1.3: Drilling types

Number:

- It is recommended that a minimum of three bore holes/pits be employed, where the surface is more or less level and the stratification are not so erratic. But if the stratification and topography are far from uniform, it is advisable to use 5 bore holes. Table 1.1 can be used as a guide line.

Table 1.1: Guidelines for preliminary exploration (EBCS 7, 1995)

Project	Distance b/n borings (m)			Minimum number of borings
	Horizontal stratification of soil			
	Uniform	Average	Erratic	
Multi storey building	50	25	10	2 if supplemented with sounding tests otherwise 4
One or two storey building	60	30	15	2
Bridge piers, abutments, towers, etc	-	30	7.5	1 to 2 for each foundation
High ways	300	150	30	

* Euro Code 7- recommends that the exploration points including sounding from a grid at a spacing of 20 to 40m.

Depth:

- Depends on soil condition and magnitude and type of the construction. For highways and air fields a depth of about 2m would suffice. However, if organic soil, muck or compressible soil is encountered, the boring should extend well below the bad soil.
- Is governed by the depth of influence of the foundation soil contact pressure. Bore holes should go down to at least the depth below the foundation level at which only 5 to 40% q reaches (q=contact pressure). This translates about 2 to 3 times the foundation width below the foundation level.
- It is recommended to make the depth of 1 to 2 bore holes deeper than that of the rest.
- A minimum of 3m drilling in to a rock formation is recommended especially in area, where occurrence of boulders is common so as to conform that it is really a rock, and not large boulder.
- EBCS 7, 1995 recommends: (3HLCH-1.doc)
  - For structures on footings $D = 3B \geq 1.5m$
  - For structures on mat $D = 1.5B$
  - For structures on piles $D \geq D' + 3m$, where D' = pile length from surface
- For preliminary investigation, the depth of exploration may be estimated as:
  - $D = 3 * S^{0.7}$ for light steel and narrow concrete buildings
  - $D = 6 * S^{0.7}$ for heavy steel and wide concrete buildings

where S = number of stories

3.2 Soil Sampling

Samples of soils are taken from boreholes and trial pits so that the soil can be described and tested. The various types of soil samples to be collected can be divided in to:

1. ***Non representative Sample***: consists of mixture of soil from different soil strata. The size of the soil grains, as well as the mineral constituents, might, thus have changed in such samples. Soil samples obtained from auger cuttings and settlings in sump well of wash boring, can be classified in this category. *Such samples may help in determining the depth at which major changes may be occurring in subsurface soil strata*. The rock fragments obtained from percussion drilling, soil samples from auger borings and wash boring *can hardly be used for the determination of index properties* (Like Atterberg limits, grain size distribution, specific gravity, natural moisture content, etc)
2. ***Representative or Disturbed Sample***: is that which contains the same particle size distribution as in the in-situ stratum from which it is collected, though the soil structure may be seriously disturbed. The water content may also have changed. Such disturbed samples can be used for identification of soil types of different strata, for determining Atterberg limits, grain size distribution, specific gravity, natural moisture content, organic and carbonate content, compaction, etc.
3. ***Undisturbed Sample***: is the one that preserves the particle size distribution as well as the soil structure of the in-situ stratum. The moisture content is also tried to be preserved to its original in-situ value. Such soil samples are required for determination of most important properties of the soil to be used for design. Theses properties include shear strength, consolidation or compressibility and permeability.

Extraction of disturbed samples: this is done by pushing or driving an open ended split spoon sampler in to the soil. The sampler is connected at the bottom to a driving shoe and at the top to a flight of drilling rod by means of coupling. If the natural moisture content is needed, then liner must be employed, which will be waxed at the two ends once the sample is retained. This is sent to the lab for testing. (**4**HLCH-1.doc)

Extraction of undisturbed samples: such a sample can be lifted by stopping the boring operation at a certain level and then inserting the appropriate sampler at the bottom of the borehole. When the sampler tube is brought to the surface, some soil is removed from both ends, and molten wax applied in thin layer, to form about 25mm thick seal. Both the ends of the tube are then closed with lids, and transported to the laboratory. Thin-wall samplers with an outer diameter of 5cm (minimum) are used. The common sizes are D_o=5cm and D_o=7.5cm.

The degree of disturbance of the sample mostly depends on: (**4**HLCH-1.doc)

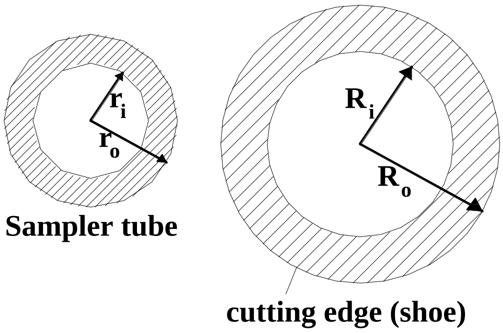
- Natural cause of removal of the overburden, while collecting samples
- The impact applied
- Rate of penetration of the devices
- Dimension of the sampler (cutting edge) and inside wall friction (oil)

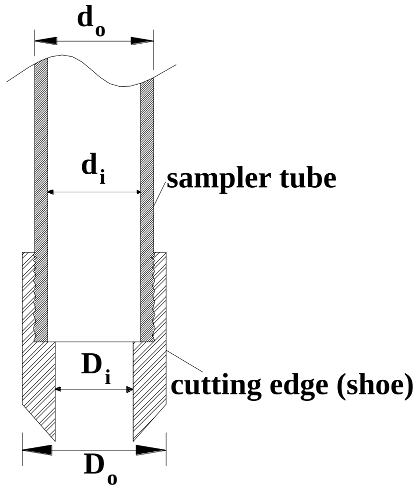
If other conditions are kept constant, the degree of disturbance of a sample is roughly indicated by the:

a) Area ratio:

$$A_r(\%) = \frac{D_o^2 - D_i^2}{D_i^2} * 100\%$$ (**5**HLCH-1.doc)

If $A_r \leq 10\%$, the sample disturbance can be considered as negligible. However, value up to 25% is even considered to be good. Thin walled samplers are preferred to thick wall samplers.





b) Inside clearance:

$$\text{Inside clearance}(\%) = \frac{d_i - D_i}{D_i} * 100\%$$

It should be as low as 1 to 3%. This reduces the frictional resistance between the tube and the sampler. It also allows the slight elastic expansion of the soil sample on entering the tube, and thus assists in sample retention.

c) Out side clearance:

$$\text{Out side clearance}(\%) = \frac{D_o - d_o}{d_o} * 100\%$$

It should not be much greater than the inside clearance. It helps in reducing the force required to withdraw the tube.

Spacing of Soil Sampling

- It is common practice to take undisturbed samples in a depth range of 0.2m to 0.7m for the top investigation and for the following few meters of investigation, continuous sampling is advisable.
- For a fairly good number of boreholes it is usual to extract samples every 1.5m starting from around 0.5m below ground surface or in every layer, which ever is less. (**4**HLCH-1.doc)

2. Sounding Tests

2.1 Standard Penetration Test (SPT): (**6**HLCH-1.doc)

It is really impossible to obtain undisturbed sample from cohesion less soils. Density, strength and compressibility estimates are usually obtained from penetration tests. The objective of SPT is to determine the resistance of the soil to penetration of the standard size of sampler, in order to obtain rough estimate of the properties of the soils in situ. SPT is the most commonly used in situ test in a bore hole. The test is made by making use of a split spoon sampler shown in Figure 1.4 (a). Here a split-spoon sampler is lowered to the bottom of the bore hole by attaching it to the drill rod and then driven by forcing it in to the soil by blows from a hammer (64Kg) falling from a height of 76cm. The sampler is initially driven 15cm below the bottom of the bore hole to *exclude* the disturbed soil while boring. It is then further driven 30cm in two stages (each 15cm). The number of blows required to penetrate the last 30cm is termed as the *SPT* value, or *N*-value. The test is halted if there is refusal (if 50 blows are required for any 15cm penetration, i.e. N=100, or if 10 successive blows produce no advance). After applying some corrections, this blow count is correlated with important properties of the soil, which can be used for design of foundations. The test is run intermittently with almost all types of boring methods and for any type of soils even if it was developed for cohesion less soils. It has clearly the advantages of enabling one to extract representative samples. It is also economical in terms of cost per unit operation.

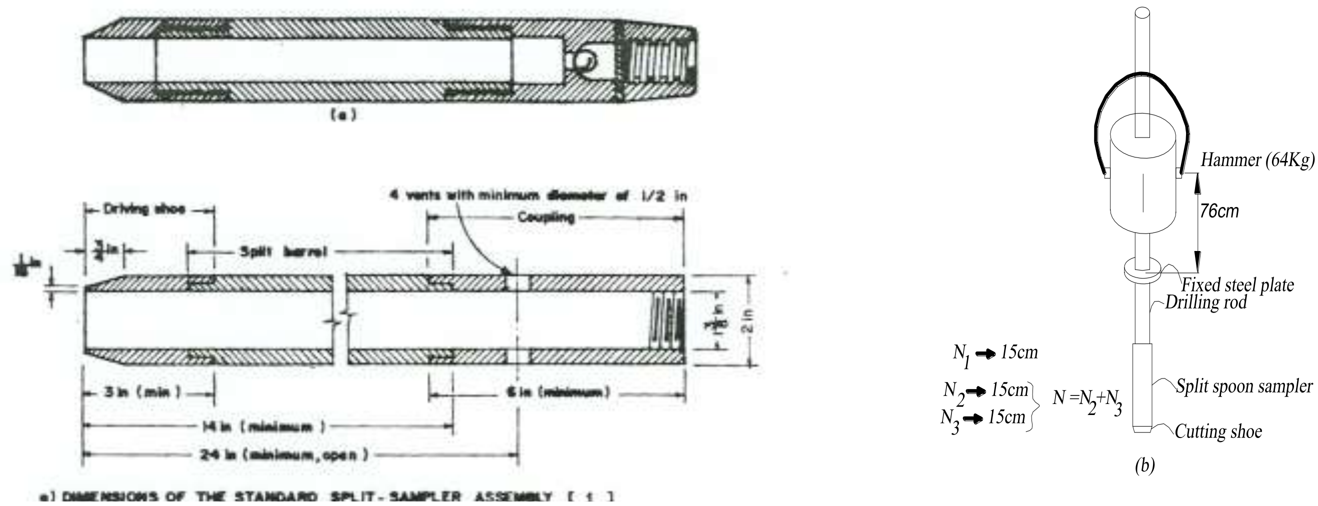


Figure 1.4: Standard Penetration Test (SPT)

Corrections to Observed SPT

It was regularly observed that the N-value in adjacent boreholes or when using different equipment are not the same. The principal factor is the input energy and its dissipation around the sampler in to the surrounding soil. Energy measurements show that the actual in put energy to the sampler is 70 to 100 % of the theoretical input energy. It is believed that the discrepancies arise from the following factors:

- Difference in some features of SPT equipment, drilling rig, hammer and skill of operation
- Driving hammer configuration and the way hammer load is applied
- Whether liner is employed or not
- Amount of overburden pressure- the bigger the over burden pressure the more is N value
- Length of the drill rod- the shorter the rod the more is N value
- Bore hole diameter - the smaller the size of the hole the more is N value

Therefore, in order to get approximately the same value for a given soil type at a given depth, it has been suggested to correct the N value as:

$$N'_{70} = C_N \eta_1 \eta_2 \eta_3 \eta_4 N$$

Where: N'_{70} = corrected or modified blow count

C_N = adjustment for effective overburden pressure $C_N = \sqrt{\dfrac{95.76}{P'_o}}$

P'_o= effective overburden pressure at the depth of interest (in KPa)

η_1 = correction for equipment and hammer type

$$\eta_1 = \frac{E_{r\,(i)}}{E_{r\,(70)}} = \frac{E_{r\,(i)}}{70}$$; $E_{r\,(i)}$ = equipment used for the test (7HLCH-1.doc)

Note: E_r* N = constant for all equipment [i.e. N_{70}*70=N_{60}*60]

η 2 = correction for rod length

$$\eta_2 = \begin{cases} 1.0; & for\ \ L > 10\,m \\ 0.95; & for\ \ 6 < L \le 10\,m \\ 0.85; & for\ \ 4 < L \le 6\,m \\ 0.95; & for\ \ L \le 4\,m \end{cases}$$

η 3 = correction for sample liner

$$\eta_3 = \begin{cases} 1.0; & without\ \ liner \\ 0.8; & with\ \ liner\ in\ dense\ sand\ and\ clay \\ 0.9; & with\ \ liner\ in\ loose\ sand \end{cases}$$

η 4 = correction for bore hole diameter

$$\eta_4 = \begin{cases} 1.0; & for\ 60 \le \phi \le 120mm \\ 1.05; & for\ \phi = 150mm \\ 1.15; & for\ \phi = 200mm \end{cases}$$

Correlations of SPT Results

Although the SPT is not considered as refined and completely reliable method of investigation, the N values give useful information with regards to consistency of cohesive soils and relative density of granular soils.

Cohesion less soils

- The Japanese Railway Standard proposed

$$\phi = \sqrt{18N'_{70}} + 15 \qquad for\ roads\ and\ bridges$$

$$\phi = 0.36N'_{70} + 27 \qquad for\ buildings$$

- Mayerhof (1959) suggested

$\phi = 28 + 0.15D_r$, where D_r = relative density in %

- Yoshida et al (1988) suggested

$D_r(\%) = 25(P'_o)^{-0.12}(N_{60})^{0.46}$, where P'_o =effective pressure in KPa

- Skempton (1986):

$\frac{N'_{70}}{D_r^{\,2}} = 32 + 0.288P'_o$; where P'_o in KPa

- Terzaghi and Peck also gave the following correlation between SPT value, ϕ and D_r.

Table 1.2 : Correlation between N, ϕ, and D_r for Sands

Condition	N'_{70}	ϕ (degree)	D_r(%)
Very loose	0-4	<20	0-15
Loose	4-10	28-30	15-35
Medium	10-30	30-36	35-65
Dense	30-50	36-42	65-85
Very dense	>50	>42	>85

(8HLCH-1.doc)

Cohesive Soils

- The common correlations of N-values with unconfined compressive strength of cohesive soils is: $q_u = K * N$

  Where K- is about 12 and q_u- in MPa
- The following correlations are suggested by Bowels (1995)

Table 1.3 : Correlation between N and q_u for Clays

Consistency	N	q_u(KPa)	γ_{sat}(KN/m^3)
Very soft	0-2	<25	16-19
Soft	2-4	25-50	
Medium	4-8	50-100	17-20
Stiff	8-15	100-200	19-22
Very stiff	15-30	200-400	
Hard	>30	>400	

Note: Other dynamic sounding tests can be conducted by using cone instead of split spoon sampler and driving the cone by hammer blows. Depending on the weight of hammer, the drop height and the tip area we have the different types as summarized in Table 1.2.

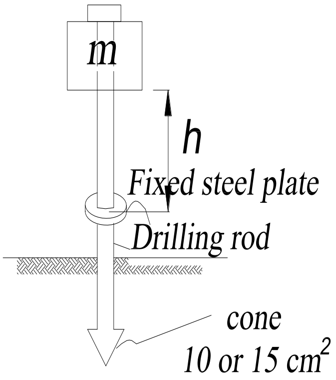


Table 1.4: Proprties of sounding equipment

Type	Mass of hammer, m (Kg)	Drop height , h (cm)	Tip area (cm^2)
Light penetrometer	10	50	10
Medium penetrometer	30	50	10
Heavy penetrometer	50	50	15
SPT	63.5	76.2	tip open

2.2 Cone Penetration Test (CPT) / Dutch Cone Penetration Test (CPT)

It is developed in Dutch and is widely now all over the world. It is a simple test widely used for *soft clays and in fine to medium course sands* instead of SPT. The test does not have any application in gravels and stiff / hard clays. It is performed by pushing the standard cone (metallic wedge of base area 10cm^2 and apex angle of 60^o) in to the ground at a rate of 10 to 20 mm/sec for a depth of 13cm and the force is measured and the end resistance of the cone called the *cone penetration resistance (point resistance)* - q_c is computed as the force required to advance the cone divided by the end area. Then the sleeve is pushed until it touches the top of the wedge followed by pushing both the wedge and the sleeve for 7cm to obtain the combined cone and sleeve resistance, q'_c. Then the side resistance (skin friction) $q_s = q'_c - q_c$. This value is important for pile design. (**9**HLCH-1.doc)

- Data from CPT can be used to estimate soil profile in conjunction with bore hole driving. Supposing one is required to know the soil profile along axis 1-2-3-4 (Figure 1:6), key boring and sounding tests will be done at points 1 & 4. From the results of the boring and sounding tests one may easily deduce the profile of the soil strata by carrying out sounding tests at points 2 & 3. A number of sounding tests may be made including points 1 & 4 depending on the nature of the stratification
- CPT data may also be used to compute bearing capacity of shallow as well as deep foundations.

Correlations of CPT Results

Some correlations are suggested by different researchers

- Lancellotta (1983) and Jumilkawiski (1985) suggested the following correlations for the relative density of granular soil.

$$D_r = -98 + 66 * \log_{10}\left(\frac{q_c}{\sqrt{\sigma'_v}}\right)$$

where q_c=point resistance (metric tone/m^2) and σ'_v= the effective pressure (metric tone/m^2)

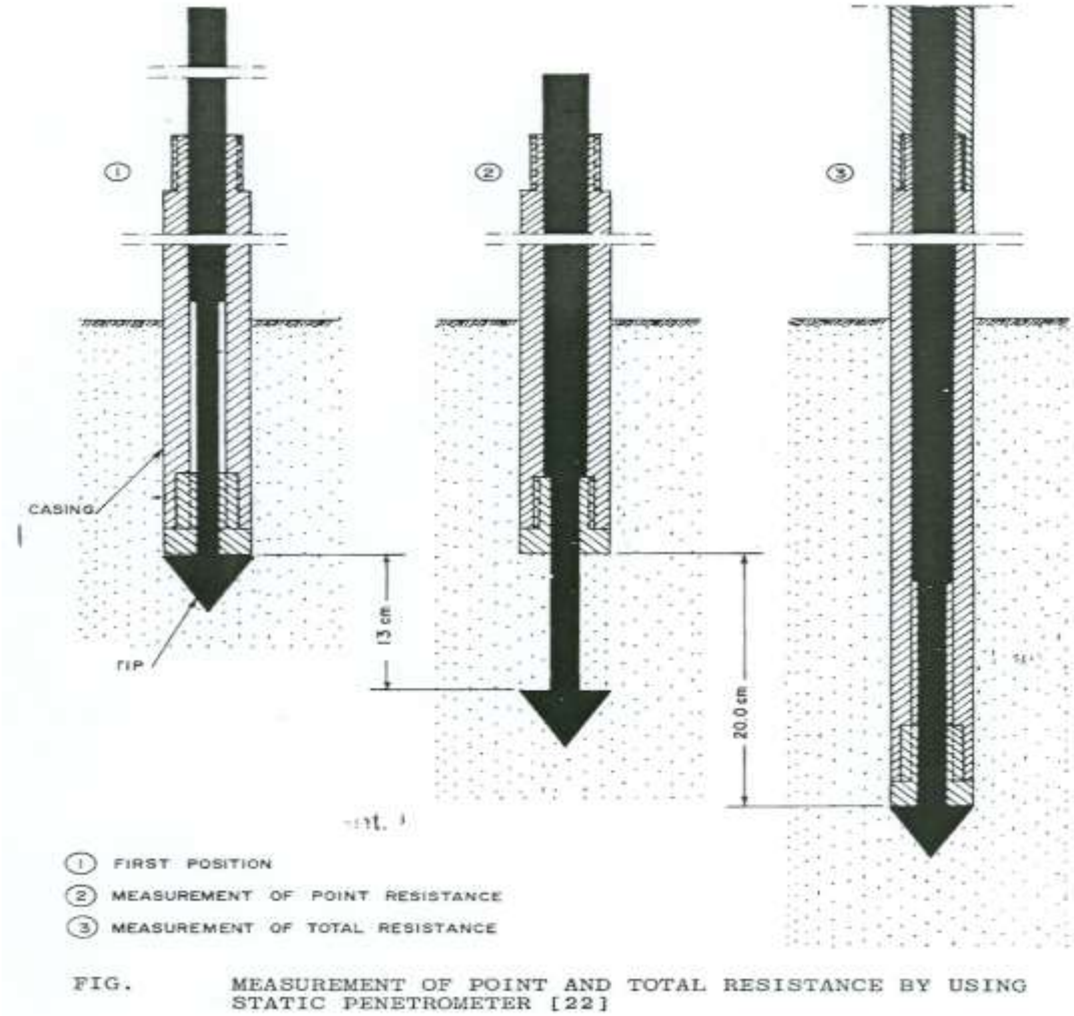


Figure 1.5: Static Penetration Test (CPT)

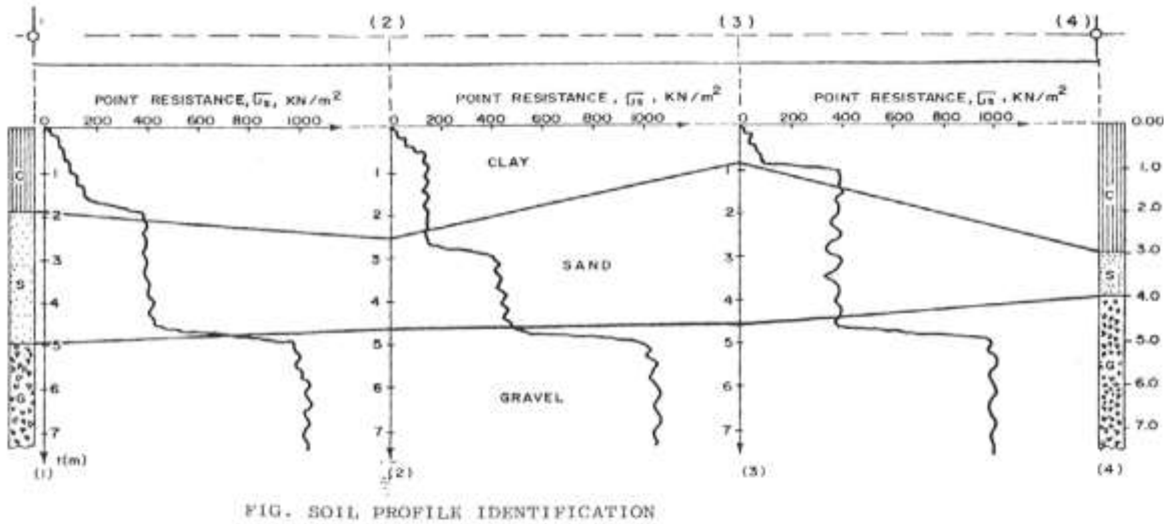


Figure 1.6: Soil Profile identification

- The following table can be used to estimate ϕ and the stress strain modulus of compressibility- E_s of non cohesive soils

Average point resistance q_c (MPa)	compactness	ϕ^o	E_s (MPa)
<5	Very loose (weak)	30	15 - 30
5-10	Loose	32	30 – 50
10-15	Medium dense	35	50 – 80
15-20	Dense	37.5	80 - 100
>20	Very dense	40	100 - 120

- Mayne and kempler (1988) suggested for the undrained shear strength (c_u)

$$c_u = \frac{q_c - \sigma_v}{N_K}$$

where q_c = point resistance (KPa) and σ_v= the total vertical pressure (KPa),

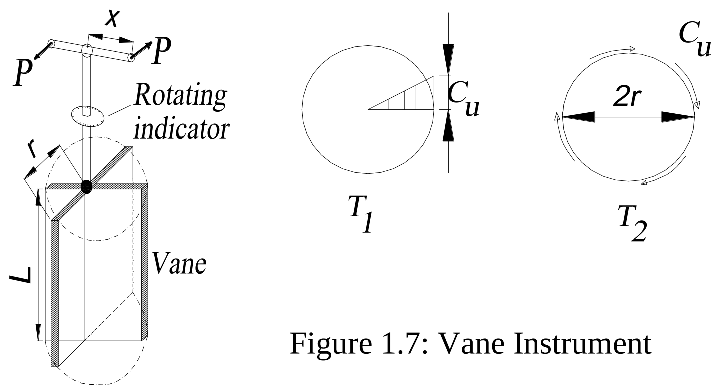
$$N_K = \begin{cases} 15 & \textit{for electric cone penetrometer} \\ 20 & \textit{for mechanical cone penetrometer} \end{cases}$$

- For the over consolidation ratio

$$\text{OCR} = 0.37\left(\frac{q_c - \sigma_v}{\sigma'_v}\right)^{1.01}$$

3. Vane shear Test

It is used for the determination of the *undrained shear strength (*c_u) of soft clays (clays which may be disturbed during the extraction and testing process with cohesions up to 100Kpa). The test is performed at any given depth by first augering to the prescribed depth, cleaning the bottom of the borings, and then carefully pushing the vane instrument (Figure 1.7) in to the stratum to be tested. A torque necessary to shear the cylinder of soil defined by the blades of the vane is applied gradually [by rotating the arm of the apparatus with constant speed of 0.5degree per second] and the peak value noted. The shear strength of the soil can then be estimated by using the formulae derived below.



Vane dimension (mm)			Rod (mm) D_{rod}
L = 4r	r	S	D_{rod}
150	37.5	3	16
100	25	1.6	18

S = blade thickness
D_{rod} = rod diameter (mm)

Figure 1.7: Vane Instrument

The torque is resisted by T_1 and T_2 (moments about the center)

If both ends of the vane are 'submerged' in the soil stratum, and if the maximum shear stress is C_u for all shear surfaces, then

Resisting moment = cylindrical surface resistance + two circular end face resistance

$$T = 2\pi r\, L\, (C_u\, r) + 2[\pi r^2\, C_u\, (2/3r)] = 2\pi r^2 C_u\, (L+2/3r)$$

$$\Rightarrow C_u = \frac{T}{2\pi r^2 (L + \frac{2}{3} r)} \qquad \Rightarrow C_u = \frac{3T}{28\pi r^3} \quad \text{if } L = 4r \text{ (commonly used ratio)}$$

If one end of the vane is 'submerged' in the soil stratum,

Resisting moment = cylindrical surface resistance + one circular end face resistance

$$T = 2\pi r\, L\, (C_u\, r) + \pi r^2\, C_u\, (2/3r) = 2\pi r^2 C_u\, (L+1/3r)$$

$$\Rightarrow C_u = \frac{T}{2\pi r^2 (L + \frac{1}{3} r)} \qquad \Rightarrow C_u = \frac{3T}{26\pi r^3} \quad \text{if } L = 4r$$ (**10**HLCH-1.doc)

The following table gives correlation between consistency and C_u

Consistency	Undrained shear strength C_u(Kpa)	
	BS5930:1981	Terzaghi and Peck: 1967
Very soft	<20	<12
Soft	20-40	12-25
Firm	40-75	25-50
Medium	40-75	25-50
Stiff	75-150	50-100
Very stiff	>150	100-200
Hard		>200

This test is made in every 1 to 2m. Vane shear test is also made in laboratories using small vane instrument.

- Field vane shear test overestimates the undrained shear strength. Therefore reduction factor should be used to estimate the design undrained shear strength.

  $c_{u,\,d} = \lambda\, c_{u},$ The commonly used value of λ is 0.6. Or one can use curves (Figure 1.8) to obtain λ based on the PI value.

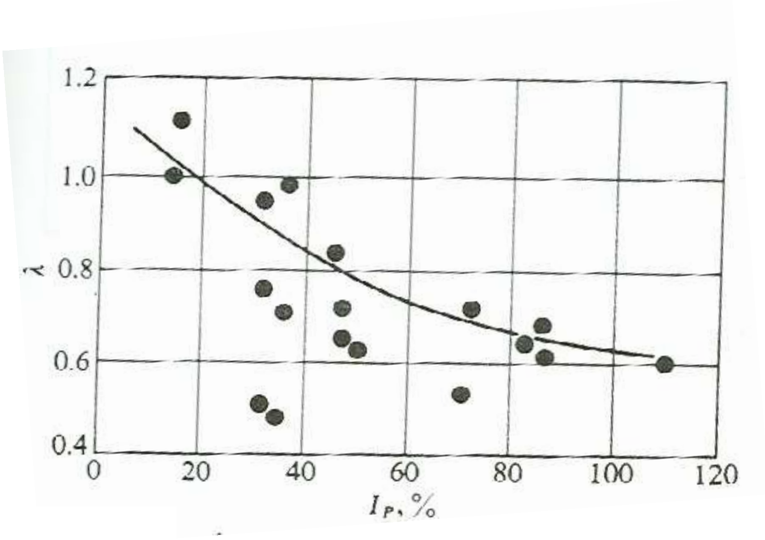


Figure 1.8: Bjerrum's correction factor for vane shear test.

Example 1:

At a depth of 5.8m from the ground level at a site, a shear vane test gave a torque value of 80Nm when fully inserted. The vane is of r=37.5mm.

a) Determine the undrained shear strength of the clay and its consistency

b) If the clay has LL=60%, PL=30%, what would be the undrained shear strength for *design*

Solution: a) Using the formula $C_u = \frac{3T}{28\pi r^3}$ we have,

$$c_u = \frac{3*80*10^{-3}}{28\pi*(0.0375m)^3} = 51.65KPa$$ Thus the clay has a firm consistency.

c) The design c_u will be obtained as $c_{u,d} = \lambda\, c_u$ but $\lambda = f$ (PI)

PI =60-30=30, For PI=30, $\lambda = 0.87$ (Figure 1.8) $\Rightarrow c_{u,d} = 0.87*51.65 = 44.9$KPa

4. Plate Load Test

Obviously the most reliable method of obtaining the ultimate bearing capacity and the settlement characteristics at a site is to perform a load test. The test is also used in the design of highways and runways. The probable settlement of the soil for a given loading and at a given depth can also be determined.

Round plate with standard diameter (30cm and 70cm) or square plate of side (0.3m x 0.3m and 60cmx60cm) is loaded in a pit excavated in the ground, at a depth equal to the roughly estimated depth of the foundation for which the bearing capacity is to be estimated. The procedure is:

- ✓ Excavate a pit to a depth on which the test is to be performed. The test pit should be at least 4B (or 4R) wide as the plate to the depth the foundation is to be placed.
- ✓ A load is applied on the plate by increments ($\Delta P = q_{ult,\ estimated}/5$), and settlements are recorded from dial gauges (at least 3 in no.) for each load increment. Then plots of settlement vs. time and settlement vs. applied stress are made.
- ✓ The test is continued until a total settlement reaches 25mm or until the capacity of the testing apparatus is reached or until the soils fails by shear (plate starts to sink rapidly). Figure 1.9 presents the essential features of the test and typical plots obtained from the plate load test.
- ✓ When the load vs. settlement curve approaches vertical, one interpolates q_{ult}. Sometimes, however, q_{ult} is obtained as that value corresponding to a specified displacement (say, 25mm)

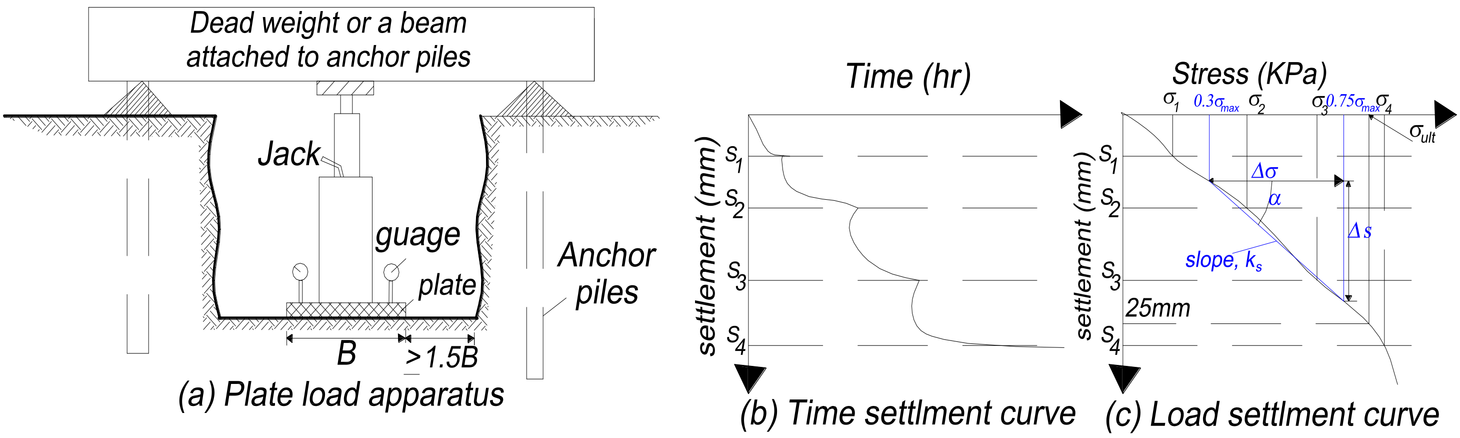


Figure 1.9: Plate Load test

Determination of Bearing Capacity from Plate Load Test

Terzaghi and Peck have suggested the following relation between the settlement of the plate S_P and the settlement of the footing S_F:

For sands $$S_P = S_F\left[\frac{b_P(B+0.3)}{B(b_p+0.3)}\right]^2$$ and $$S_P = \frac{b_P}{B}S_F$$ for clays

where: B= width of footing (least dimension) and b_p= width (diameter) of plate

The permissible settlement value, such as 25mm, should be substituted for S_F in the above equations and the S_P value will be calculated. Then from the load-settlement curve, the pressure corresponding to the computed settlement S_P, is the required value of the ultimate bearing capacity, $q_{ult,P}$, for the plate. The ultimate bearing capacity of the foundation $q_{ult,F}$ is then determined from $q_{ult,P}$ as follows:

For sandy soils $q_{ult,F} = \frac{B_F}{B_P} q_{ult,P}$ and for clays $q_{ult,F} = q_{ult,P}$ (**11**HLCH-1.doc)

The coefficient of sub-grade reaction, k_s, can also be estimated from:

$$k_s = \frac{\Delta\sigma}{\Delta S} = \frac{0.4\sigma_{max}}{\Delta S}\left(KN/m^3\right)$$

This parameter is also employed in immediate settlement computation.

Limitations of the test

1. Size effects: Since the size of the test plate and the size of the prototype foundation are very different, the results of a plate load test do not directly reflect the bearing capacity of the foundation. The bearing capacity of footings in sands varies with the size of footing; thus, the scale effect gives rather misleading results in this case. However, this effect is not pronounced in cohesive soils as the bearing capacity is essentially independent of the size of footing in such soils.
2. Consolidation settlements in cohesive soils, which may take years, cannot be predicted, as the plate load test is essentially a short-term test. Thus, load tests don't have much significance in the determination of q_{all} based on settlement criterion w.r.t cohesive soils.
3. The load test results reflect the characteristics of the soil located only within a depth of about 2B of plate. This zone of influence in the case of a prototype footing will be much larger and ***unless*** the soil is essentially homogenous for such a depth and more, the results could be terribly misleading. Thus it may be misleading if there is weak soil and ground water with in this influence zone.

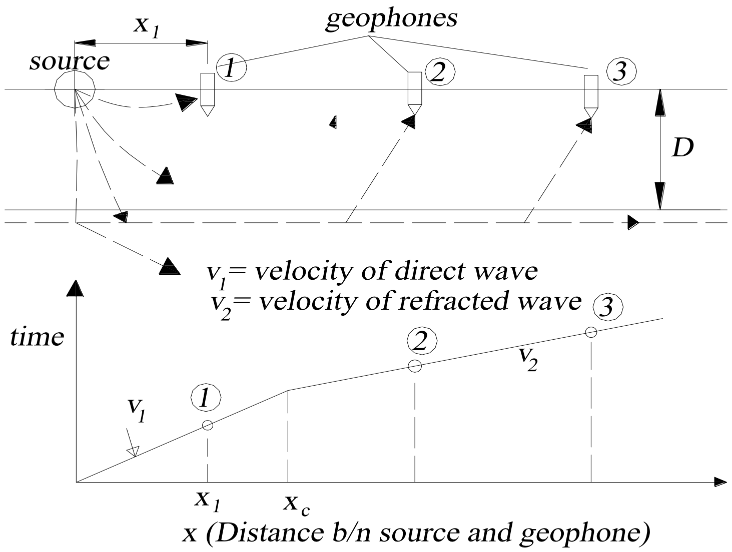
5. Indirect Geophysical Methods of Soil Exploration

Geophysical methods correlate speed and condition of wave propagation in a soil media with soil properties. They help us in *checking* and *supplementing* the soil test results. They are generally useful in preliminary investigation stage when they can give us ideas about position of the water table, strata boundaries of vastly differing soils, depth of existing bedrocks, etc. The results inferred from such tests must, however, be checked and confirmed from the boreholes, by lifting soil samples, and examining and testing them. Some of these methods are discussed below.

1) **Seismic exploration**

Such method is based on the simple fact that the seismic waves move through different types of soils at different velocities (4000 to 7000m/s in sound rocks, 500-700m/s in clays, and as low as 30m/s in loose weathered materials) and are also refracted when they cross the boundary between two different types of soils. Here shock waves are induced by producing an explosion at the surface (drop hammer or 3Kg sledge hammer adequate for 20m penetration; deeper with explosive shock source). The waves are then picked up through geophones placed at various points.

This method can help us in plotting the soil profiles, economically, but would fail to detect a layer having velocity lesser than that of the upper layer. Hence a layer of clay laying below a layer of compacted gravel, would go undetected in this method. It is reliable for relatively thick and distinct layers.



(12HLCH-1.doc)

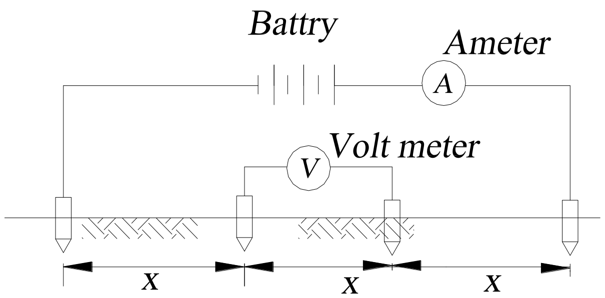
Figure 1.10: Seismic Exploration

- Interpretation of the test results of seismic exploration should be done with care. Reliable information is only obtained when the soil profile consists of relatively thick and distinct layers. The test results may lead to inaccurate conclusion if the soil profile consists of relatively thin layers. The velocity of longitudinal waves is correlated with the soil type as given in the table below. Shear waves may also be correlated with the soil type.

Soil type	Velocity of Longitudinal Waves V_l (m/s)
Non cohesive	200 – 1500
Soils with little cohesion	1000 – 1600
Cohesive soils	1600 – 2000
Rocks	2000 – 6000

2) **Electrical Resistivity Method:**

This method uses the principle that different soils exhibit different resistivity. As a result four electrodes are inserted in to the ground and current is made to flow. The resistance is then measured. This method requires good contrast in resistivity between the soil layers. If difference between the layers is not substantial, or if the soil is wet and contains a considerable amount of dissolved salt, the reading may be wrong. Clean dense sand above the water table, will therefore have high resistivity, because it will have very small saturation and dissolved salts. Saturated clay of high void ratio will similarly have low resistivity, because there would be a lot of pore water and free ions in it, so as to act as good conductors of electricity, offering very low resistance.



$$\rho = 2\pi\, x \frac{E}{I}$$

where: ρ= apparent resistivity in Ohms/m

x = electrode spacing

E = potential drop

I = circuit current

Figure 1.11: Electrical Resistivity method

By increasing electrode spacing, there will be an increase in influence depth. As long as the stratum does not change, ρ remains the same and if ρ changes a new stratum is encountered at a certain depth (approximately at a depth equal to x).

Interpretation of the test results of electrical resistivity method can be made with the help of the following table.

Soil type	Resistivity Ohms/m
Clay and saturated silt	0 – 1000
Sandy clay	1000 – 2700
Clayey sand and saturated sand	2700 – 5400
sand	5400 – 16400
gravel	16,400 – 50,000

6. ROCK CORE SAMPLING

In rocks, except for very soft or partially decomposed sandstone or lime stone, blow counts are at refusal level (N >100). When rock layer is encountered during driving, rock coring is necessary to check the soundness of the rock. Unconfined compressive strength could also be determined using rock cores. Rock coring is the process in which a sampler consisting of a tube (core barrel) with a cutting bit at its lower end cuts an annular hole in a rock mass, thereby creating a cylinder or core of rock which is recovered in the core barrel. Rock cores are normally obtained by rotary drilling.

Standard rock cores range from about 1 ¼ inches to nearly 6 inches in diameter. The recovery ratio R_r defined as the percentage ratio between the length of the core recovered and the length of the core drilled on a given run, is related to the quality of rock encountered in boring, but it is also influenced by the drilling technique and the type and size of core barrel used. A better estimate of in situ rock quality is obtained by a modified core recovery ratio as the rock quality designation (RQD) which is expressed as

$$RQD = \frac{\sum \text{Length of intact pices of core} > 100\text{mm length}}{\text{Total length of the core advance}}$$

Breaks obviously caused by drilling are ignored. The diameter of the core should preferably not less than 2 1/8 inches. The table below gives the rock quality description, modulus of Elasticity and unconfined compressive strength as related to RQD

RQD (%)	Rock Quality	E_{field}/E_{lab}	$q_{u,field}/q_{u,lab}$
90-100	Excellent	0.7 – 1.0	0.7 – 1.0
75-90	Good	0.3 – 0.7	0.3 – 0.7
50-75	Fair	0.25	0.25
25-50	Poor	0.2	0.2
0-25	Very Poor	0.15	0.15

- If rock is close to the ground surface, it is recommended to drill 2m in sound rock and 3 to 6m in weathered rock.
- If rock is encountered at deeper depth, it is recommended to drill 3 to 4m in to the rock, especially below the location of the foundation elements.

7. Ground Water

The presence of water table near the foundation affects the load bearing capacity of a foundation. The water table may change seasonally. In many cases establishing the highest and the lowest possible levels of water during the life of the project is necessary. If water is encountered in bore hole during field exploration, the fact should be recorded. In soils with high coefficient of permeability, the level of water in bore hole will stabilize in a bout 24 hrs after completion of the bore hole drilling. The depth of the water table can then be measured using steel tape. In soils with low K-values, this process may take a week. If the seasonal ground water table variation is to be measured, piezometer may be installed in to bore hole and the variation is recorded for longer time. (**14**HLCH-1.doc)

8. Soil Exploration Report

At the end of all soil exploration programs, after the required information has been collected, a soil exploration report is prepared for the use in the design office. It is a good practice to divide the report in to two:

1. Factual report: include all gathered data
2. Interpretative report: include interpreted data which serve as a basis for design.

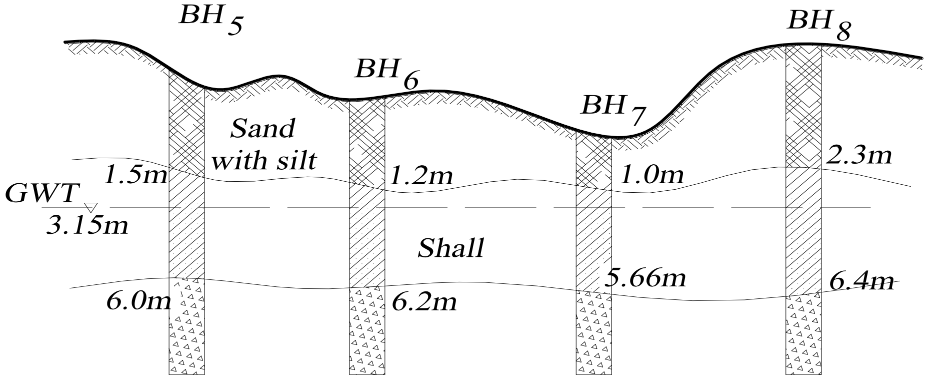
The report may be presented in the following sections

a) **Introduction**: which contains information like
- For whom?
- Why?
- Method and approach
- Terms of reference (TOR) if there is any

b) **General description of the site**: which should describe
- General configuration and surface features like trees, shrubs, buildings, quarries, marshy ground, fill areas, etc
- Any useful information derived from past records
- Other peculiar observations- wind, earth quakes, slopes, subsidence, etc

c) **General geology of the area**: which include notes on the geology of the area based on comparison with existing published information and special geologic features like, faults, springs, mine shafts, etc

d) **Preparation of the soil profile**: This shall describe the various strata in the deposit. It can be best presented by passing an imaginary section through a series of bore holes. The water table location shall be indicated if possible.



e) **Laboratory test results** : a brief mention of the various tests done is made
   - Due emphasis on unusual tests
   - For detailed results reference should be made to approximate curves or tables
   - For non standard tests, it is necessary to describe the detailed procedure followed.

f) **Discussion of Results** : this is made in relation to implication on design and construction

   For example:
   - In case of shallow foundations, one can recommend depth of foundation, safe bearing capacity, expected settlements a result of superstructure loads provided, advantages and disadvantages of going deeper
   - In case of pile foundations, one can recommend the bearing stratum, depth of penetration in the bearing stratum, method of installation of the pile, the type of pile to be used (friction/end bearing)

If any detrimental effects on existing structure are possible, it must be well discussed.

g) **Conclusions**: a summary of the main findings of investigation and the interpretation is given.

CHAPTER TWO

TYPES OF FOUNDATIONS AND THEIR SELECTION

2.1 Types of Foundations

Commonly encountered foundations in practice may be broadly classified into two main categories:

1. shallow foundations (1HLCH-2.doc)
    a. Wall or continuous footings (Figure 2.1 (a))
    b. Spread or isolated footings and combined footings (Figure 2.1 (b))
    c. Mat or raft foundations (Figure 2.1 (c))
2. Deep foundations
    a. Pile foundation (Figure 2.1 (d))
    b. Piers and caissons (Figure 2.1 (e))
    c. Under shallow foundations

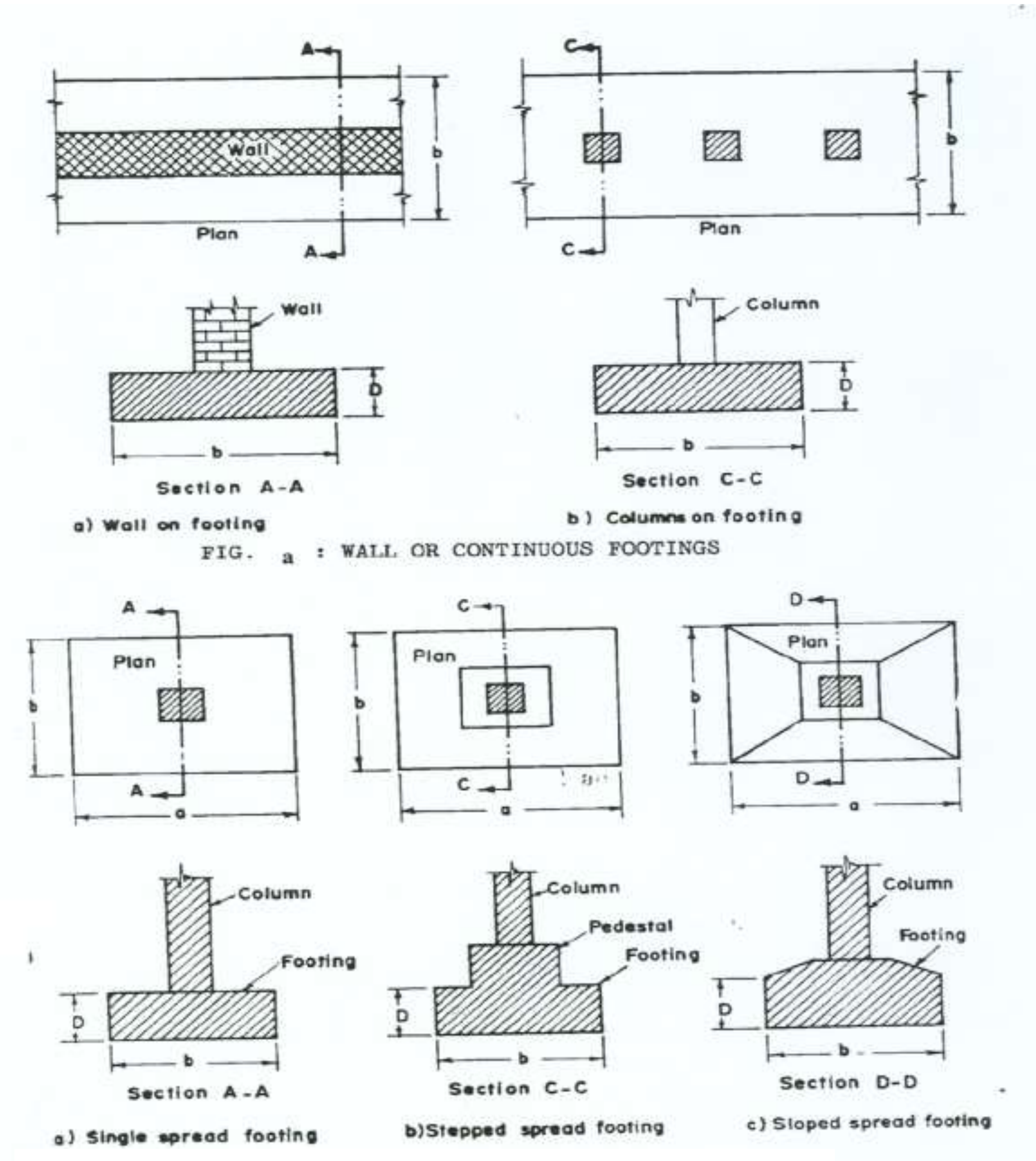
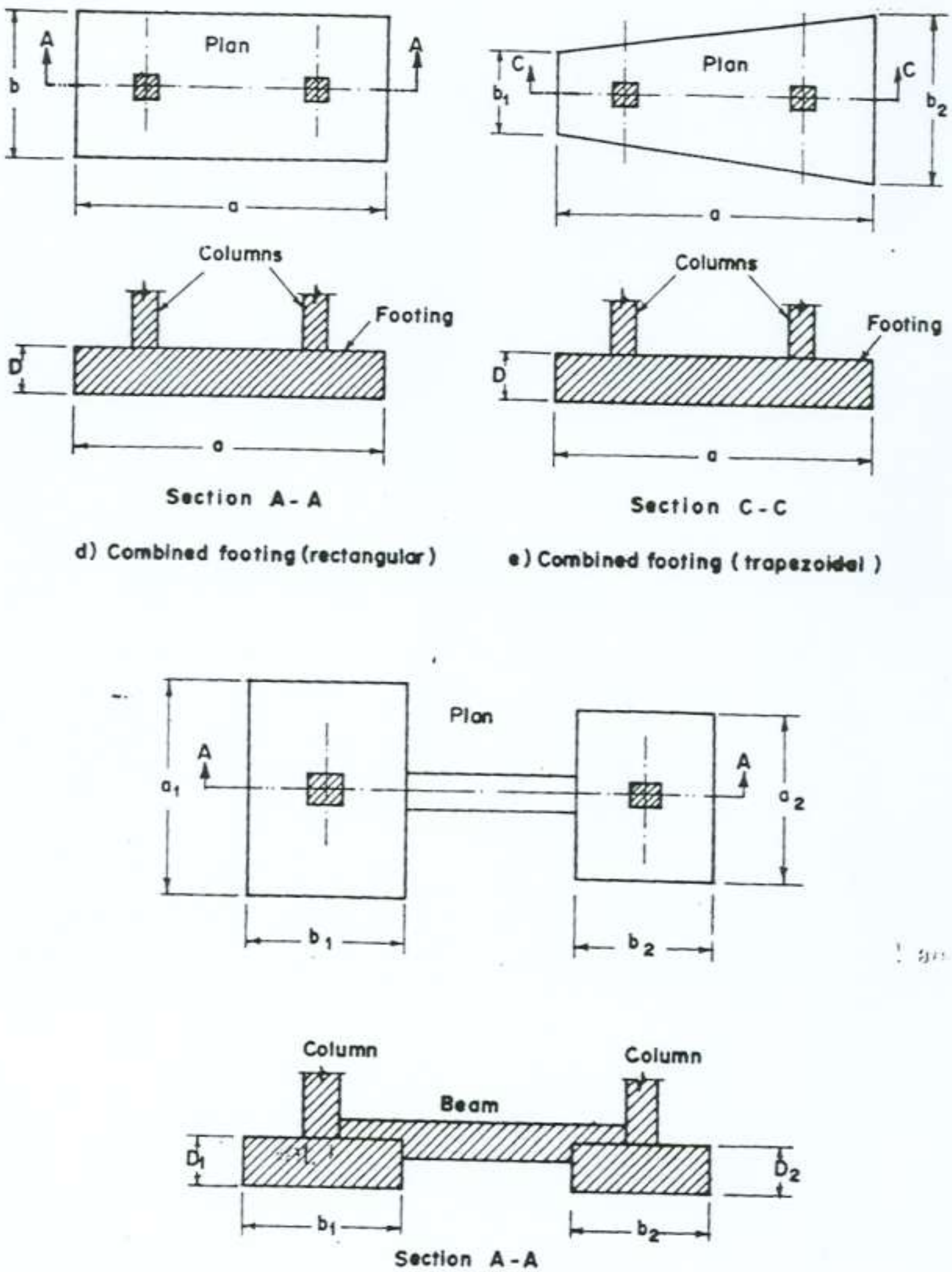


Figure 2.1: Different Types of Foundations



f) Strap footing

FIG. b COMMON TYPES OF SPREAD FOOTINGS

Figure 2.1………………………continued

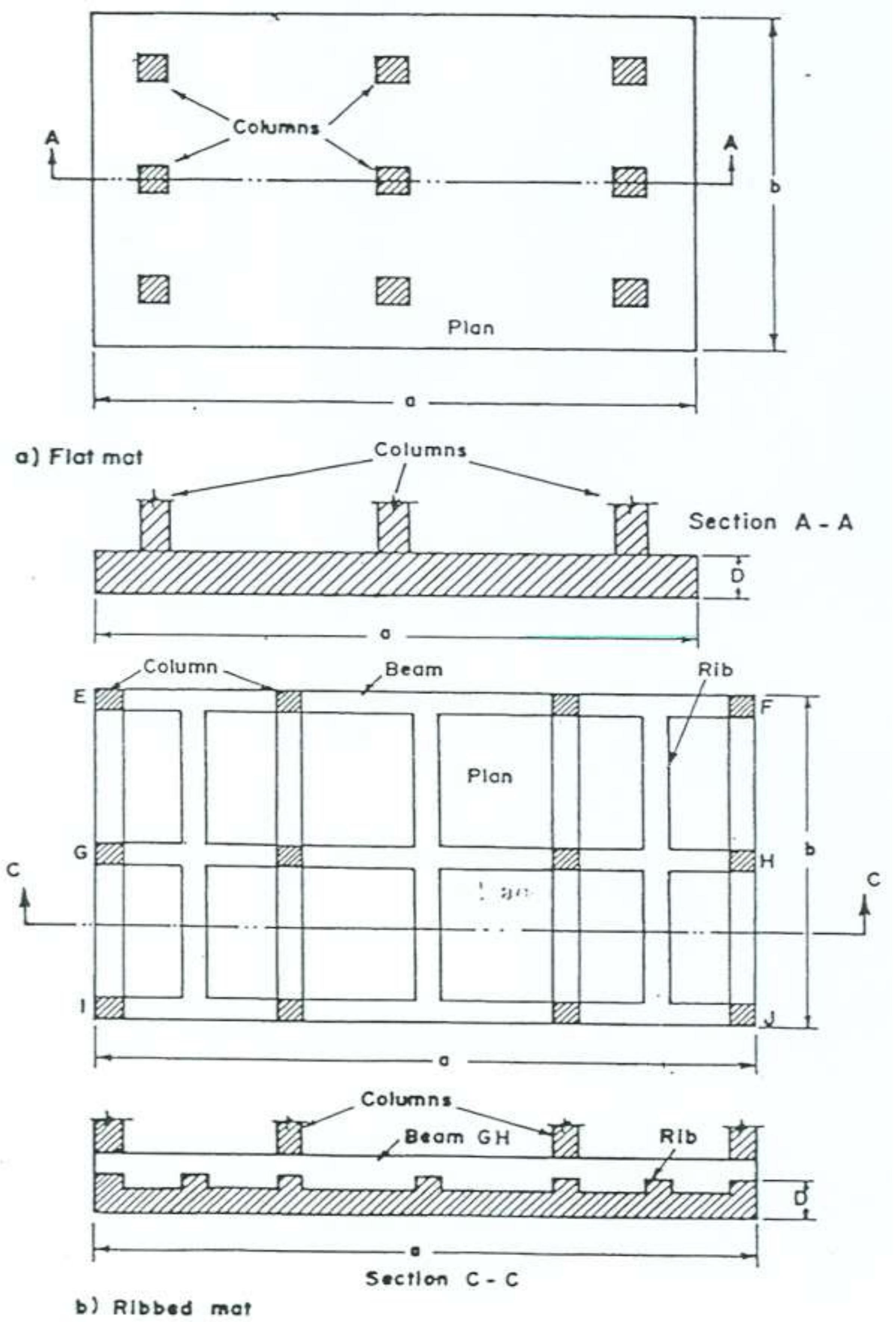


FIG. C TYPICAL MAT OR RAFT FOUNDATIONS

Figure 2.1……………………….continued

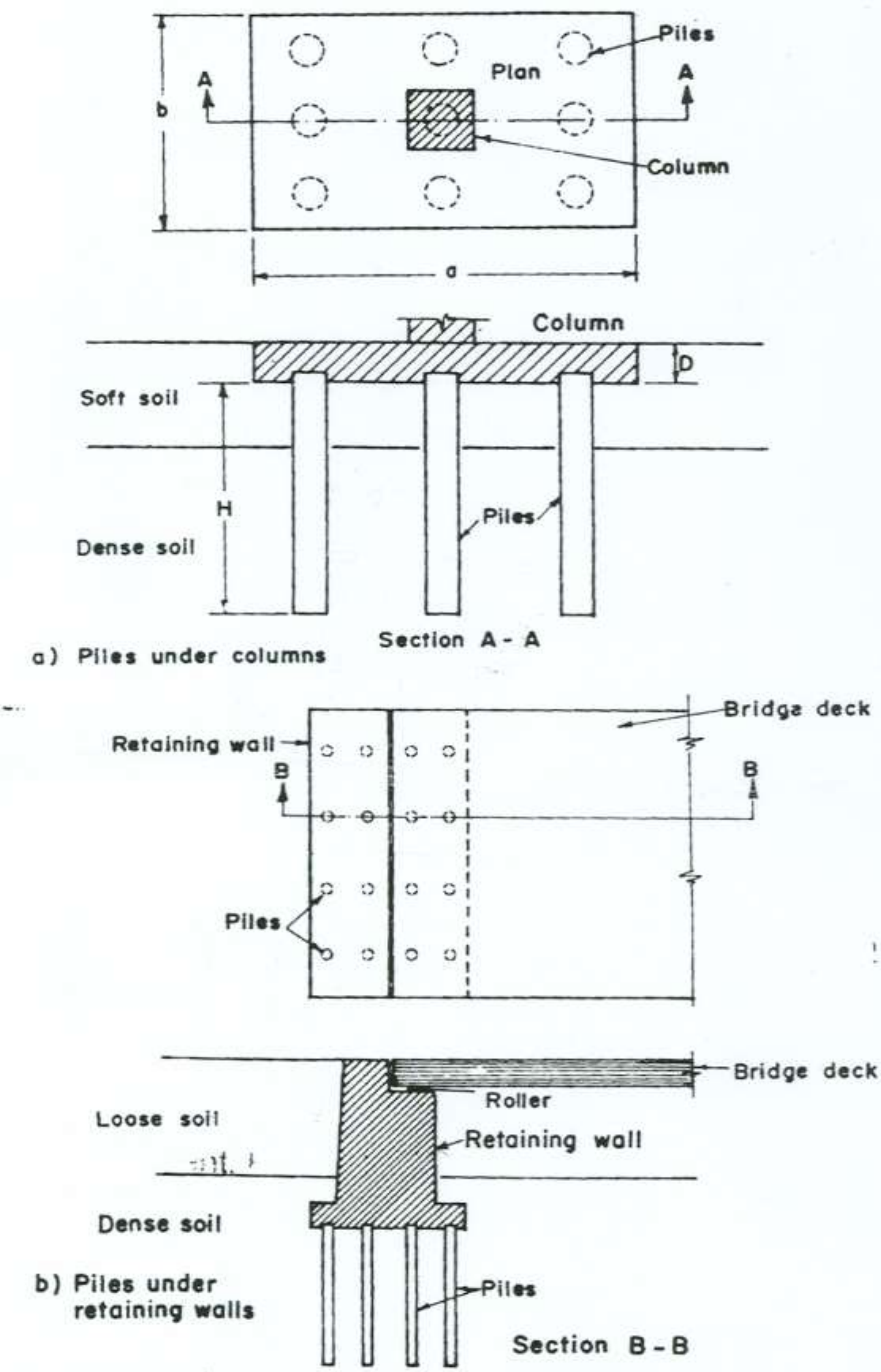


FIG. d TYPICAL EXAMPLES OF PILE FOUNDATIONS

Figure 2.1………………………continued

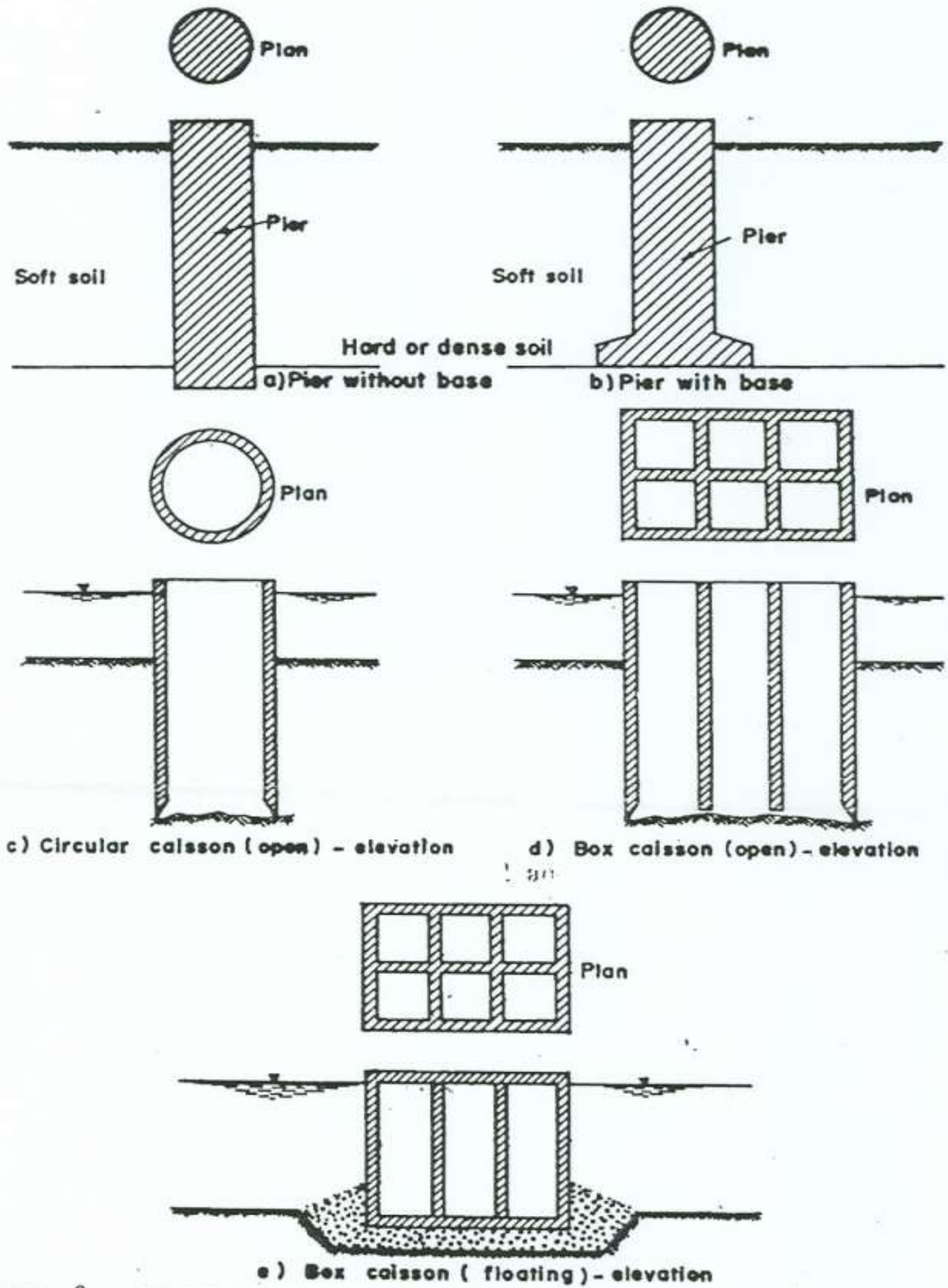


FIG. e : TYPICAL EXAMPLES OF PIER AND CAISSON FOUNDATIONS

Figure 2.1………………………continued

2.2. *Selection of Foundation Types*

In selecting the foundation types the following must be considered:

a) Function of the structure
b) Loads it must carry
c) Subsurface conditions
d) Cost of foundation in comparison with the cost of the superstructure (2HLCH-2.doc)

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

1) Obtain at least approximate information concerning the superstructure and the loads to be transmitted to the foundation
2) Determine the subsurface conditions in a general way
3) Consider each of the usual types of foundations in order to judge whether or not
    a. They could be constructed under existing conditions
    b. They are capable of carrying the required load.
    c. They experience serious differential settlements
4) Undertake a detailed study of the most promising types. Such a study may require additional information on loads and subsurface conditions.
5) Determine the approximate size of footings, piers or caissons or the approximate length and number of piles required.
6) Prepare an estimate for the cost of each promising type of foundation
7) Select the type that represents the most acceptable compromise between performance and cost.

CHAPTER THREE

SOME CONSIDERATIONS FOR DESIGN OF SHALOW FOUNDATIONS

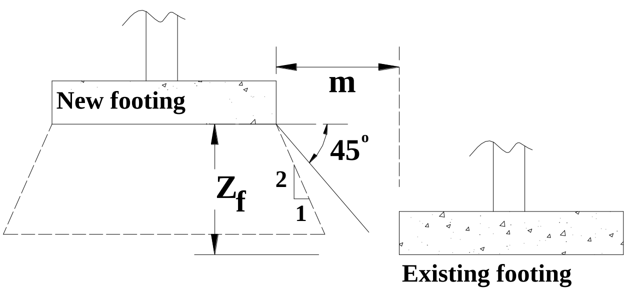
3.1 General Requirements (1HLCH-3.doc)

In the design of shallow foundations, the following factors should be considered properly

- Footing depth and location:
- Net and gross bearing capacity
- Erosion problems for structures adjacent to flowing water
- Corrosion protection and sulfate attack
- Water table fluctuation
- Foundations in sand, silt and clays
- Foundations on expansive soils

Footings should be carried below

- Top soil, organic material, peat or muck
- Unconsolidated material such as abandoned (or closed) garbage dumps and similar filled in areas
- Zones of high volume change due to moisture fluctuations



- Use an approximate spacing of footings as **m** $> Z_f$ to avoid interface between 'old' and 'new' footings
- If the 'new' footing is in the relative position to the 'existing' footing of this figure, interchange the words 'existing' and 'new'.

- It is difficult to compute how close one may excavate to existing footings with out having a detrimental effect on the existing footing. If excavation of a new footing is at a depth greater than that of the existing footing there might be a possible settlement of the existing footing because of (a) loss of lateral support of the soil wedge beneath the existing footing (b) loss of overburden pressure-q N_q term of the bearing capacity equation. Thus, it is recommended to construct a wall (sheet pile wall or other material) to retain the soil in essentially the K_o state out side the excavation.

3.2 SETTLEMENT AND BEARING CAPACITY

3.2.1 SETTLEMENT

1. Definition of settlement

Foundations placed on the soil introduce change in stresses which will compress and deform the underlying soil. The statistical accumulation of the movements in the direction of interest (usually in the vertical direction) is referred to as settlement, S.

A structure may undergo 'uniform settlement' or 'differential settlement'. Uniform settlement or equal settlement under different points of the structure does not cause much harm to the structural stability of the structure. However, differential settlement or different magnitudes of settlement at different points underneath a structure-especially a rigid structure is likely to cause supplementary stress and thereby cause harmful effects such as cracking, permanent and irreparable damage, and ultimate yield and failure of the structure. As such, differential settlement must be guarded against. (2HLCH-3.doc)

2. Data for Settlement analysis:

To estimate the settlements we need:

- To obtain the soil profile-which gives an idea of the depths of various characteristic zones of soil at the site of the structure, as also the relevant properties of soil such as initial void ratio, grain specific gravity, water content, and the consolidation and compressibility characteristics
- To estimate the stresses transmitted to the subsurface strata, using a theory such as Boussinesq's for stress distribution in soil.

3. Total Settlement

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

a) Initial settlement or elastic compression.
b) Consolidation settlement or primary compression.
c) Secondary settlement or secondary compression.

Initial Settlement or Elastic Compression

This is also referred to as the 'immediate or distortion or contact settlement' and it is usually taken to occur immediately on application of the foundation load (within about 7 days).

Immediate settlement computation (3HLCH-3.doc)

The settlement of the **corner of a rectangular base (flexible) of dimensions B' X L'** on the surface of an elastic half-space can be computed from an equation from the Theory of Elasticity [e.g., Timoshenko and Goodier (1951)] as follows:

$$S_i = q_o\, B'\left(\frac{1-v^2}{E_s}\right) I_S\, I_F$$

q_o = intensity of contact pressure in units of E_s

B' = least lateral dimension of contributing base area in units of S.

E_s, v = elastic soil parameters

I_i = influence factors, which depend on L'/B', thickness of stratum H, Poisson's ratio v, and base embedment depth D. The influence factor I_s (see Figure 3.1 for identification of terms) can be computed using equations given by Steinbrenner (1934) as follows:

$$I_S = I_1 + \frac{1-2v}{1-v} I_2 \qquad \text{with } I_1 \text{ and } I_2 \text{ as follows:}$$

$$I_1 = \frac{1}{\pi}\left[M \ln\left(\frac{(1+\sqrt{M^2+1})(\sqrt{M^2+N^2})}{M(1+\sqrt{M^2+N^2+1})}\right) + \ln\left(\frac{(M+\sqrt{M^2+1})(\sqrt{1+N^2})}{M+\sqrt{M^2+N^2+1}}\right)\right]$$

$$I_2 = \frac{N}{2\pi}\tan^{-1}\left[\frac{M}{N\sqrt{M^2+N^2+1}}\right] \qquad \tan^{-1} \text{ in radians}$$

$$\text{where; } \mathrm{M} = \frac{\mathrm{L'}}{\mathrm{B'}} \quad \textit{and} \quad \mathrm{N} = \frac{\mathrm{H}}{\mathrm{B'}}$$

$B' = \frac{B}{2}$ for center and $B' = B$ for corner I_i ; $L' = \frac{L}{2}$ for center and $L' = L$ for corner I_i

I_F = influence factor from the Fox (1948b) equations, which suggest that the settlement is reduced when it is placed at some depth in the ground, depending on Poisson's ratio and L/B. Figure 3.1 can be used to approximate I_F.

Note: if your base is "rigid" you should reduce the I_s factor by about 7 percent (that is, $I_{s,\ rigid}$ = O.931 $I_{s,\ flexible}$)

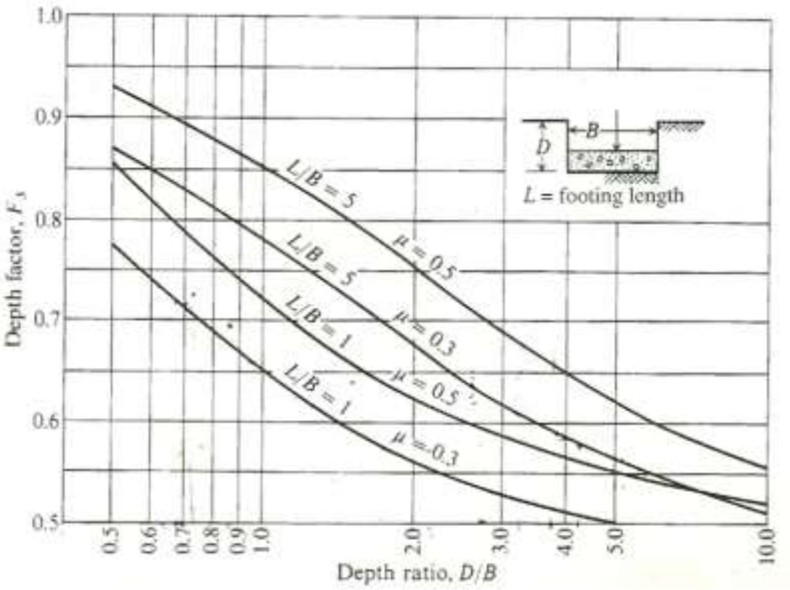


Figure 3.1: Influence factor I_F for footing at a depth D. Use actual footing width and depth dimension for this D/B ratio.

Determination of E_s: Determination of E_s-the modulus of elasticity of soil, is not simple because of the wide variety of factors influencing it. It is usually obtained from a consolidated undrained triaxial test on a representative soil sample, which is consolidated under a cell pressure approximating to the effective overburden pressure at the level from which the soil sample was extracted. The plot of deviator stress versus axial strain is never a straight line. Hence, the value must be determined at the expected value of the deviator stress when the load is applied on the foundation. If the thickness of the layer is large, it may be divided into a number of thinner layers, and the value of E_s, determined for each.

Consolidation Settlement or Primary Compression

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

- The total consolidation settlement, S_c. may be obtained from one of the following equations:

$$S_c = \frac{H\, C_c}{(1+e_o)} \log_{10}\left(\frac{\sigma'_o - \Delta\sigma}{\sigma'_o}\right)$$

$$S_c = m_v\, \Delta\sigma\, H$$

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

Where, C_c = compression index from the e versus log P plot

e_o = in situ void ratio in the stratum where C_c was obtained

H = stratum thickness. If the stratum is very thick (say >6m) it should be subdivided into several sub layers of H_i = 2 to 3m, with each having its own e_o and C_c. Compute the several values of S_{ci} and then sum them to obtain the total consolidation settlement.

σ'_o = effective overburden pressure at mid-height of H

$\Delta\sigma$ = average increase in pressure from the foundation loads in layer H and the same units of σ'_o. The vertical pressure increment $\Delta\sigma$ at the middle of the layer has to be obtained by using the theory of stress distribution in soil.

m_v = constrained modulus of elasticity determined from consolidation test =1/Es

- *Time-rate of settlement:* Time-rate of settlement is dependent, in addition to other factors, upon the drainage conditions of the clay layer. If the clay layer is sandwiched between sand layers, pore water could be drained from the top as well as from the bottom and it is said to be a case of double drainage. If drainage is possible only from either the top or the bottom, it is said to be a case of single drainage. In the former case, the settlement proceeds much more rapidly than in the latter. The calculations are based upon the equation:

$$T_v = \frac{C_v}{H^2}$$

Secondary Settlement or Secondary Compression

Settlement due to secondary compression is believed to occur during and mostly after the completion of primary consolidation or complete dissipation of excess pore pressure. It is the continuing readjustment of the soil grains in to a closer (or more dense) state under compressive load. In the case of organic soils and micaceous soils, the secondary compression is comparable to the primary compression; in the case of all other soils, secondary settlement is considered insignificant.

4. Differential Settlement

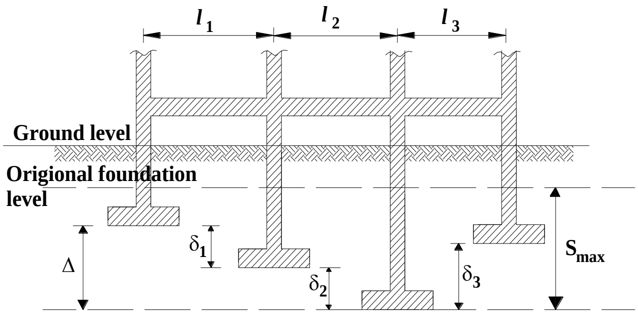
Non-uniform or differential settlement is settlement in which part of a foundation or two adjoining footings settle differently. If the effect of differential settlement is not taken in to the design of the structure, the structure may crack very badly and the safety of the structure becomes questionable. Basically there are two methods of estimating the allowable differential settlement of a given structure:

1 Analytical methods: expressions derived by introducing simplifying assumptions where stiffness used as a criterion. They may be sometimes misleading and are not used in practice.

2 Empirical methods: previous knowledge or results of field or lab tests are used to determine the settlements.

The magnitudes of the settlements obtained by using the above methods are compared with the permissible amount of settlement.

From statistical analysis Skempton and MacDonald concluded that as long as the angular distortion, δ / l , of a building is less than 1/300, there should be no settlement damage.(Figure 3.2).



δ_1, δ_2, δ_3 = differential settlements
Δ = greatest differential settlement
S_{max} = maximum total settlement
l_1, l_2, l_3, = bay width
δ / l = angular distortion

Figure 3.2: Definition of differential settlement

Having established the permissible limits of differential settlement, various authors have recommended the magnitude of maximum permissible total settlement S_{max} for practical purposes. If the maximum total settlement is kept within the permissible limit, the differential settlement, being a function of the total settlement, will also be taken care of.

5. Allowable magnitude of recommended settlement (7HLCH-3.doc)

If the computed settlements are with in the values in the parentheses in the table below, statistically the structure should adequately resist that deformation.

Table: Tolerable settlements of buildings in **mm** (After Skempton and MacDonald)

Recommended maximum values in parentheses

Criterion	*Isolated foundation*	*Rafts*
Angular distortion (cracking)	1/300	
Greatest differential settlement		
Clays	45 (35)	
Sands	32 (25)	
Maximum settlement		
Clays	75	75-125 (65-100)
Sands	50	50-75 (35-65)

According to EBCS 7 (1995), the permissible total settlement is 50mm and 75mm on sand and clayey soils respectively for isolated footings and correspondingly 75mm and 125mm for rafts.

3.2.2 Bearing Capacity

1. *Introduction*

To ensure stability, foundations must provide an adequate factor of safety against ***shear or bearing failure of the underlying soil*** and the *structure* must be capable of withstanding the ***settlements*** that will result, in particular the differential settlements. Thus the ***criteria*** for the determination of the bearing capacity of a foundation are based on the requirements for the stability of the foundation. The design value of the safe bearing capacity would be the smaller of the two values, obtained from the two criteria:

1. Shear failure criterion (**10.1**HLCH-3.doc)
2. Settlement criterion

The soil's limiting shear resistance is referred to as the *ultimate bearing capacity*, q_u, of the soil. For design, one uses an allowable bearing capacity, q_{all}, obtained by dividing the ultimate bearing capacity by a suitable safety factor (i.e. $q_{all}=q_u/FS$). (**10.2**HLCH-3.doc)

Some **analytical** methods of estimating bearing capacity are given below.

2. *Terzaghi's Bearing Capacity Theory*

Terzaghi obtained expressions for the ultimate bearing capacity for general shear conditions as:

$$\text{Long footings:} \qquad q_u = cN_c + \gamma D_f N_q + \frac{1}{2}B\gamma N_\gamma$$

$$\text{Square footings:} \qquad q_u = 1.3cN_c + \gamma D_f N_q + 0.4\gamma B N_\gamma$$

$$\text{Circular footings:} \qquad q_u = 1.3cN_c + \gamma D_f N_q + 0.3\gamma B N_\gamma$$

$$\text{where:} \quad N_q = \frac{a^2}{2\cos^2(45+\phi/2)} \quad ; \qquad N_c = [N_q - 1]\cot\phi \quad ;$$

$$N_\gamma = \frac{1}{2}\tan\phi\left[\frac{K_{p\gamma}}{\cos^2\phi} - 1\right] \qquad \text{with} \qquad a = e^{\left(\frac{3\pi}{4}-\frac{\varphi}{2}\right)\tan\phi}$$

$$N_{p\gamma} = 3\tan^2\phi\left[45 + \left(\frac{\phi+33}{2}\right)\right] \qquad \text{(After S. Husain)}$$

Table 3.1 below gives the values for the various bearing capacity factors recommended for the above equations.

Table 3.1: Terzaghi's N-factors

ϕ^0	0	2	4	6	8	10	12	14	16	18	20	22	24
N_c	5.7	6.3	6.97	7.73	8.6	9.61	10.76	12.11	13.68	15.52	17.69	20.27	23.36
N_q	1	1.22	1.49	1.81	2.21	2.69	3.29	4.02	4.92	6.04	7.44	9.19	11.4
N_γ	0	0.18	0.38	0.62	0.91	1.25	1.7	2.23	2.94	3.87	4.97	6.61	8.58
ϕ^0	26	28	30	32	34	36	38	40	42	44	46	48	50
N_c	27.09	31.61	37.16	44.04	52.64	63.53	77.5	95.67	119.67	151.95	196.2	258.29	347.52
N_q	14.21	17.81	22.46	28.52	36.51	47.16	61.55	81.27	108.75	147.74	204.2	287.86	415.16
N_γ	11.35	15.15	19.73	27.49	36.96	51.7	73.47	100.39	165.69	248.29	427	742.61	1153.2

The results obtained here are quite within acceptable limits for shallow footings (e.g. $D_f/B \le 1$) subjected to ***only vertical loads***. But they are limited to concentrically loaded horizontal footings; they are not suitable for footings that support eccentrically-loaded columns or to tilted footings. Furthermore, they are regarded as somewhat overly conservative.

Terzaghi developed his bearing-capacity equations assuming a general shear failure in a dense soil and a local shear failure for a loose soil. For the local shear failure he proposed reducing the cohesion and ϕ as:

$$c'' = \frac{2}{3}c$$

$$\phi'' = \tan^{-1}\left(\frac{2}{3}\tan\phi\right)$$

3. *Meyerhof's Bearing Capacity Equation*

Meyerhof proposed a bearing capacity equation similar to that of Terzaghi but added shape factors, s, depth factors, d, and inclination factors, *i*.

$$\text{Inclined Load}: \quad q_u = cN_c s_c d_c i_c + \gamma D_f N_q s_q d_q i_q + \frac{1}{2} B\gamma N_\gamma s_\gamma d_\gamma i_\gamma$$

$$\text{where}: \quad N_q = e^{\pi \tan\phi} \tan^2(45 + \phi/2) \qquad N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi)$$

The N values are given in Table 3.2 (a) and (b).

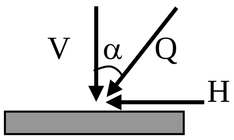
Table 3.2 (a): Meyerhof's N- factors

ϕ^0	0	2	4	6	8	10	12	14	16	18	20	22	24
N_c	5.1	5.63	6.19	6.81	7.53	8.34	9.28	10.37	11.63	13.1	14.83	16.88	19.32
N_q	1	1.2	1.43	1.72	2.06	2.47	2.97	3.59	4.34	5.26	6.4	7.82	9.6
N_γ	0	0.01	0.04	0.11	0.21	0.37	0.6	0.92	1.37	2	2.87	4.07	5.72
ϕ^0	26	28	30	32	34	36	38	40	42	44	46	48	50
N_c	22.25	25.8	30.14	35.49	42.16	50.59	61.35	75.32	93.71	118.37	152.1	199.27	266.89
N_q	11.85	14.72	18.4	23.18	29.44	37.75	48.93	64.2	85.38	115.31	158.51	222.31	319.07
N_γ	8	11.19	15.67	22.02	31.15	44.43	64.08	93.69	139.32	211.41	328.74	526.47	873.89

Table 3.2 (b): Meyerhof's factors (s, d, *i*)

ϕ	Shape	Depth	Inclination
Any ϕ	$s_c = 1 + 0.2K_p \dfrac{B}{L}$	$d_c = 1 + 0.2\sqrt{K_p}\dfrac{D}{B}$	$i_c = i_q = \left(1 - \dfrac{\alpha}{90^0}\right)^2$
For $\phi = 0^0$	$s_q = s_\gamma = 1.0$	$d_q = d_\gamma = 1.0$	$i_\gamma = 1.0$
For $\phi \geq 10^0$	$s_q = s_\gamma = 1 + 0.1K_p \dfrac{B}{L}$	$d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\dfrac{D}{B}$	$i_\gamma = \left(1 - \dfrac{\alpha}{\phi}\right)^2$

$K_p = \tan^2\left(45 + \dfrac{\phi}{2}\right)$ where α = angle of resultant measured from vertical axis



When triaxial ϕ_{tr} is used for plain strain, adjust ϕ_{tr} to obtain $\phi_{ps} = \left(1.1 - 0.1\dfrac{B}{L}\right)\phi_{tr}$

Meyerhof suggested that footing dimensions $B' = B - 2e_y$ and $L' = L - 2e_x$ be used in determining the total allowable load eccentrically applied in the x and y directions, respectively (i.e., $Q_u = q_u B' L'$), and in the corresponding terms in the ultimate bearing capacity equations and in the various correction factors for shape and inclination.

4. *Hansen's Bearing Capacity Equation*

Hansen proposed the *general bearing capacity equation* which includes ground factors and base factors to include conditions for a footing on a slope.

$$q_u = cN_c s_c d_c i_c b_c g_c + \gamma D_f N_q s_q d_q i_q b_q g_q + \frac{1}{2} B\gamma N_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

where: $N_q = e^{\pi \tan\phi} \tan^2(45 + \phi/2)$; $N_c = (N_q - 1)\cot\phi$;

$N_\gamma = 1.5(N_q - 1)\tan\phi$ [Table 3.3 (a)]

Expressions for inclination, shape, depth, base, and ground inclination expressions proposed by Hanson are given in Table 3.3 (b).

Table 3.3 (a): Hansen's N- factors

ϕ^0	0	2	4	6	8	10	12	14	16	18	20	22	24
N_c	5.1	5.63	6.19	6.81	7.53	8.34	9.28	10.37	11.63	13.1	14.83	16.88	19.32
N_q	1	1.2	1.43	1.72	2.06	2.47	2.97	3.59	4.34	5.26	6.4	7.82	9.6
N_γ	0	0.01	0.05	0.11	0.22	0.39	0.63	0.97	1.43	2.08	2.95	4.13	5.75
ϕ^0	26	28	30	32	34	36	38	40	42	44	46	48	50
N_c	22.25	25.8	30.14	35.49	42.16	50.59	61.35	75.32	93.71	118.37	152.1	199.27	266.89
N_q	11.85	14.72	18.4	23.18	29.44	37.75	48.93	64.2	85.38	115.31	158.51	222.31	319.07
N_γ	7.94	10.94	15.07	20.79	28.77	40.05	56.18	79.54	113.96	165.58	244.65	368.68	568.59

Table 3.3 (b): Hansen's factors (s, d, *i*, b, g)

Shape factors

$$s_c = 0.2\frac{B'}{L'} \quad \text{(for } \phi=0\text{)}$$

$$s_c = 1 + \frac{N_q}{N_c}\frac{B'}{L'} \quad \text{(for } \phi>0\text{)}$$

$$s_q = 1.0 + \frac{B'}{L'}\sin\phi$$

$$s_\gamma = 1.0 - 0.4\frac{B'}{L'} \geq 0.6$$

Depth factors

$$d_c = 0.4k \quad \text{(for } \phi=0\text{)}$$

$$d_c = 1.0 + 0.4k \quad \text{(for } \phi>0\text{)}$$

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 k$$

$$d_\gamma = 1.0$$

$$k = \frac{D}{B} \quad \text{if } \frac{D}{B} \leq 1$$

$$k = \tan^{-1}\left(\frac{D}{B}\right) \quad \text{if } \frac{D}{B} > 1$$

k in radians

Inclination factors

$$i_c = \frac{1}{2} - \frac{1}{2}\sqrt{1 - \frac{H_i}{A_f C_a}} \quad \text{(for } \phi=0\text{)}$$

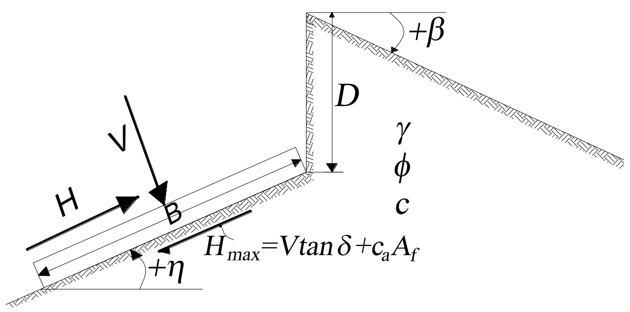
$$i_c = i_q - \frac{1-i_q}{N_q - 1} \quad \text{(for } \phi>0\text{)}$$

$$i_q = \left[1 - \frac{0.5H_i}{V + A_f C_a \cot\phi}\right]^{\alpha_1}$$

$$2 \leq \alpha_1 \leq 5$$

$$i_\gamma = \left[1 - \frac{\left(0.7 - \frac{\eta}{450^0}\right)H_i}{V + A_f C_a \cot\phi}\right]^{\alpha_2}$$

$$2 \leq \alpha_2 \leq 5$$



$\beta+\eta \leq 90^0$; $\beta \leq \phi$; D measured vertically.

For $L/B \leq 2$ use ϕ_{tr}

For $L/B>2$ use $\phi_{ps}=1.5\,\phi_{tr} - 17^0$ but for $\phi_{tr} \leq 34^0$ use $\phi_{tr} = \phi_{ps}$

δ = friction angle between base and soil ($0.5\phi \leq \delta \leq \phi$)

$A_f = B'\,L'$ (effective area)

c_a = base adhesion (0.6c to 1.0c)

Ground factors (base on slope)

$$g_c = \frac{\beta^0}{147^0} \quad \text{(for } \phi=0\text{)}$$

$$g_c = 1.0 - \frac{\beta^0}{147^0} \quad \text{(for } \phi>0\text{)}$$

$$g_q = g_\gamma = (1 - 0.5\tan\beta)^5$$

Base factors (tilted base)

$$b_c = \frac{\eta^0}{147^0} \quad \text{(for } \phi=0\text{)}$$

$$b_c = 1 - \frac{\eta^0}{147^0} \quad \text{(for } \phi>0\text{)}$$

$$b_q = e^{-2\eta\tan\phi} \quad \eta \text{ in radians}$$

$$b_\gamma = e^{-2.7\eta\tan\phi} \quad \eta \text{ in radians}$$

Notes

- ✓ Failure can take place either along the long side or along the short side and thus shape , depth and inclination factors shall be calculated in both sides
- ✓ Use H_i as either H_B or H_L for inclination factors

5. *Vesic's Bearing Capacity Equation*

The Vesic procedure is essentially the same as the method of Hansen with select changes. The N_c and N_q terms are those of Hansen but N_γ is slightly different as is given by: $N_\gamma = 2(N_q + 1)\tan\phi$ (also see Table 3.4 (a))

There are also differences in the i_i, b_i and g_i, terms (Table 3.4 (a)).

Table 3.4 (a): Vesic's N_γ - factors

ϕ^0	0	5	10	15	20	25	26	28
N_γ	0	0.4	1.2	2.6	5.4	10.9	12.5	16.7
ϕ^0	30	32	34	36	38	40	45	50
N_γ	22.4	30.2	41	56.2	77.9	109.3	271.3	761.3

(**10.3**HLCH-3.doc)

Table 3.4 (b): Vesic's factors (s, d, *i*, b, g)

Shape factors

$$s_c = 1.0 + \frac{N_q}{N_c}\frac{B}{L}$$

$$s_q = 1.0 + \frac{B}{L}\tan\phi$$

$$s_\gamma = 1.0 - 0.4\frac{B}{L} \geq 0.6$$

Depth factors

$d_c = 0.4k$ (for ϕ=0)

$d_c = 1.0 + 0.4k$ (for $\phi > 0$)

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 k$$

$$d_\gamma = 1.0$$

$$k = \frac{D}{B} \quad \text{if } \frac{D}{B} \leq 1$$

$$k = \tan^{-1}\left(\frac{D}{B}\right) \quad \text{if } \frac{D}{B} > 1$$

k in radians

Inclination factors

$i_c = 1 - \dfrac{mH_i}{A_f C_a N_c}$ (for ϕ =0)

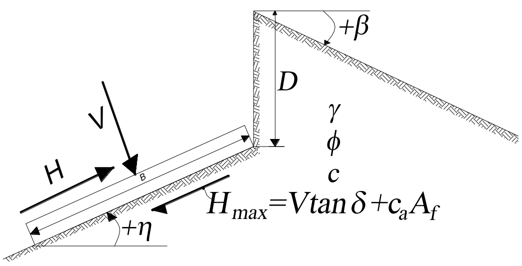
Where: $m = m_B = \dfrac{2 + B/L}{1 + B/L}$

$m = m_L = \dfrac{2 + L/B}{1 + L/B}$

$i_c = i_q - \dfrac{1 - i_q}{N_q - 1}$ (for $\phi > 0$)

$$i_q = \left[1 - \frac{H_i}{V + A_f C_a \cot\phi}\right]^m$$

$$i_\gamma = \left[1 - \frac{H_i}{V + A_f C_a \cot\phi}\right]^{m+1}$$



$\beta + \eta \leq 90^0$; $\beta \leq \phi$; D measured vertically.

For $L/B \leq 2$ use ϕ_{tr}

For $L/B > 2$ use $\phi_{ps} = 1.5\,\phi_{tr} - 17^0$ but for $\phi_{tr} \leq 34^0$ use $\phi_{tr} = \phi_{ps}$

δ = friction angle between base and soil ($0.5\phi \leq \delta \leq \phi$)

A_f = B' L' (effective area)

c_a = base adhesion (0.6c to 1.0c)

Ground factors (base on slope)

$g_c = \dfrac{\beta}{5.14}$ $-- \beta$ in rad (for ϕ=0)

$g_c = i_q - \dfrac{1 - i_q}{5.14\tan\phi}$ (for $\phi > 0$)

$$g_q = g_\gamma = (1.0 - \tan\beta)^2$$

Base factors (tilted base)

$b_c = g_c$ (for ϕ=0)

$b_c = 1 - \dfrac{2\beta}{5.14\tan\phi}$ (for $\phi > 0$)

$b_q = b_\gamma = (1 - \eta\tan\phi)^2$ η in radians

Notes

- ✓ Compute $m = m_B$ when $H_i = H_B$ (H parallel to B) and $m = m_L$ when $H_i = H_L$ (H // L). If you have both H_B and H_L use $m = \sqrt{m_B^2 + m_L^2}$. Note use of B and L, not B',L'.
- ✓ When ϕ =0 and $\beta \neq 0$, use $N_\gamma = -2\sin(\pm\beta)$ in N_γ term
- ✓ Always i_q, $i_\gamma \geq 0$. For Vesic use B' in the N_γ term even when $H_i = H_L$

6. *Effect of Water Table on Bearing Capacity*

The water table location can be in one of the following cases (Fig 3.2):

1. Water Table Above the base of the Footing

Fig 3.2(a) depicts a case of the water table located between the ground surface and base of the footing. When this condition is encountered, both the 2nd and 3rd terms of the bearing capacity equations are affected by a lower value of γ [= γ_b (γ') or γ_{sub}].

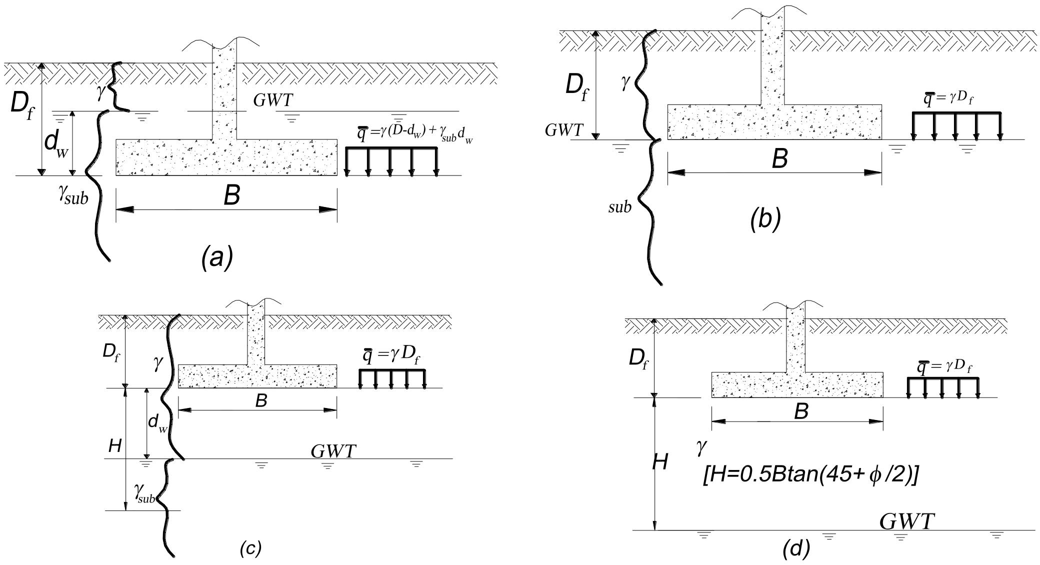


Fig 3.2: Effect of Water Table

2. Water at the Base of Footing

For this case, the γ in the second term (N_q) requires no adjusting. The third term will be γ_b (Fig 3.2 (b)).

3. The Water Table Below the Base of Footing but with in the wedge zone

When the water table lies with in the wedge zone [depth approximately H=0.5Btan(45+ϕ/2) from base of footing], some small difficulty may be obtained in computing the effective unit weight to use in the Nγ term [Fig 3.2 (c)]. In many cases this term for such situation can be ignored to obtain a conservative solution. However, one can compute effective weight (γ_e) for the soil within the wedge zone as

$$\gamma_e = (2H - d_w)\frac{d_w}{H^2}\gamma + \frac{\gamma_{sub}}{H^2}(H - d_w)^2$$

Where: H=0.5B tan (45+ϕ/2); d_w =depth of water table below base of footing

γ and γ_{sub} (= γ - γ_w) are wet and submerged unit weight of the soil respectively

4. The Water Table Below the wedge zone

When the water table is below the wedge zone [depth approximately H=0.5Btan(45+ϕ/2) from base of footing], the water table effects can be ignored for computing the bearing capacity.

7. *Bearing Capacity Based on Tolerable Settlement*

For the second criterion, the tolerable values of the total and differential settlements which a particular structure, on a particular type of foundation in a given soil, can undergo without sustaining any harmful effects are to be decided up on. These values have already been specified, basing on experience and judgment. Once the limiting values of settlement are fixed, the procedure involves determining that pressure which causes settlements just equal to the limiting value. This is allowable bearing capacity on the basis of the settlement criterion. It is to be noted that there is no need to apply a further factor of safety to this pressure, since it would have been applied even at the stage of fixing up tolerable settlement values.

The smaller pressure of the values obtained from the two criteria is termed the 'allowable bearing pressure', which is used for design of the foundation.

The bearing capacity based on settlement criterion may be determined from the field load tests or plate load tests, standard penetration tests or from the charts like those prepared by Terzaghi and Peck, based on extensive investigation.

i. *Bearing Capacity From SPT*

The SPT is widely used to obtain the bearing capacity of soils directly. According to Bowels, the allowable bearing capacity is obtained as follows:

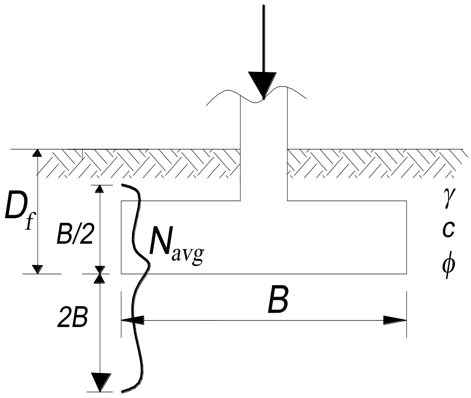
- For an allowable settlement of $S_{max} = 25mm$

$$q_{all}\,(KPa) = 25\,N'_{70}\,K_d \qquad ; \qquad B \le 1.2m$$

$$q_{all}\,(KPa) = 16\,N'_{70}\left[\frac{B+0.3}{B}\right]^2 K_d; \qquad B \ge 1.2m$$

$$\text{where} \quad K_d = 1 + \frac{D_f}{3B} \le 1.33$$

$$\text{For mat foundation } (B \ge 1.2m), \quad \left[\frac{B+0.3}{B}\right]^2 \cong 1$$



- For $S_{max} \geq 25mm$

$$q_{all} = \frac{S(mm)}{25mm}(q_{all})_{25mm}$$

ii. *Bearing Capacity From CPT*

✓ Meyerhof (1956,1965) suggested for S_{max} =25mm and ***sands***

$$q_{all}(KPa) = \frac{q_c}{30}; \qquad \text{for} \quad B \le 1.2m \qquad \text{(a)}$$

where q_c= point resistance in KPa

$$q_{all}(KPa) = \frac{q_c}{50}\left(\frac{B+0.3}{B}\right)^2; \quad \text{for} \quad B > 1.2m \qquad \text{(b)}$$

Meyerhof proposed doubling the result obtained from (b) for mat foundations.

✓ Schmertmann (1975) gave for footings on ***sands***

$N_\gamma = \frac{q_c}{80}$ with this value of N_γ, ϕ is determined followed by other factors. Then Meyerhof's bearing capacity equation is employed to determine q_{ult}. This approximation should be applicable for $D/B \leq 1.5$. q_c is averaged over the depth interval from about B/2 above to 1.1B below the footing base.

✓ For ***clays*** one may use[Schmertmann]:

$$\text{Strip}: \; q_{all}(KPa) = 200 + 28q_c$$

$$\text{Square}: q_{all}(KPa) = 500 + 34q_c \qquad q_c \text{ in KPa}$$

iii. *Bearing Capacity From Field Load Tests (Refer Chapter 1)*

8. *Bearing Capacity Based on Building Codes (Presumptive Pressure)*

Table 3.5 indicates representative values of building code pressures. These values are primarily for illustrative purposes, since it is generally agreed that in all but minor construction projects some soil exploration should be undertaken. Major drawbacks to the use of presumptive soil pressures are that they don't reflect the depth of the footing, size of footing, location of water table, or potential settlements.

Table 3.5 Presumed Design Bearing Resistances* under Vertical Static Loading (EBCS 7, 1995)

Supporting Ground Type	Description	Compactness** or Consistency***	Presumed Design Bearing Resistance (KPa)	Remarks
Rocks	Massively crystalline igneous and metamorphic rock (granite, basalt, gneiss)	Hard and sound	5600	These values are based on the assumptions that the foundations are carried down to un weathered rock
	Foliated metamorphic rock (slate, schist)	Medium hard and sound	2800	
	Sedimentary rock (hard shale, siltstone, sandstone, limestone)	Medium hard and sound	2800	
	Weathered or broken-rock (soft limestone)	Soft	1400	
	Soft shale	Soft	850	
	Decomposed rock to be assessed as soil			
Non-cohesive Soils	Gravel, sand and gravel	Dense	560	Width of foundation *(B)* not less than 1 m
		Medium dense	420	
		Loose	280	
	Sand	Dense	420	Ground water level assumed to be depth not less than *B* below the base of the foundation
		Medium dense	280	
		Loose	140	
Cohesive soils	Silt	Hard	280	
		Stiff	200	
		Medium stiff	140	
		Soft	70	
	Clay	Hard	420	
		Stiff	280	
		Medium stiff	140	
		Soft	70	
		Very soft	Not Applicable	

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

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

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

9. *Bearing Capacity for Footings on Layered Soils*

If the thickness of the stratum from the base of the footing d_1 is less than the H distance [$H = 0.5\ B \tan(45 + \phi/2)$], the rupture zone will extend in to lower layer(s) depending on their thickness and require some modification of q_{ult}. There are three general cases.

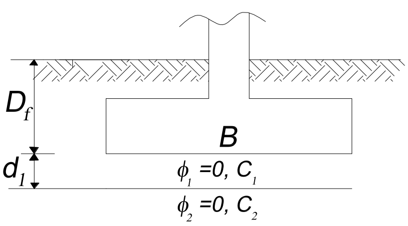
Case 1: Layered cohesive soil layers with $\phi_1 = \phi_2 = 0$, $C_1 \neq C_2$ and strength ratio $C_R = C_2 / C_1$

a) For $C_R \leq 1$ obtain N_c [Brown and Meyerhof] as follows,

- For strip and rectangular footings:

$$N_c = \frac{1.5 d_1}{B} + 5.14 C_R \leq 5.14$$

- For circular footings with B=diameter:



$$N_c = \frac{3d_1}{B} + 6.05C_R \leq 6.05$$

1. If $C_R > 0.7$, reduce the above bearing capacity factors by 10%.

b) For CR >1 obtain N_c [Brown and Meyerhof] as follows,

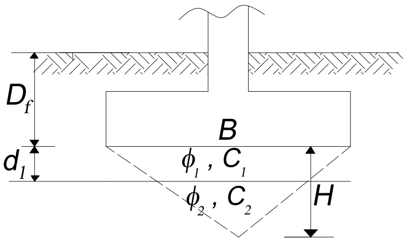
$$N_c = 2\left[\frac{N_{c1}N_{c2}}{N_{c1}+N_{c2}}\right] \quad \textit{with}$$

$$\left.\begin{aligned} N_{c1} &= 4.14 + \frac{0.5B}{d_1} \\ N_{c2} &= 4.14 + \frac{1.1B}{d_1} \end{aligned}\right\} \text{strip and rectangular footings}$$

$$\left.\begin{aligned} N_{c1} &= 5.05 + \frac{0.33B}{d_1} \\ N_{c2} &= 5.05 + \frac{0.66B}{d_1} \end{aligned}\right\} \text{circular footings}$$

2. When the top layer is very soft with a small d_1/B ratio, one should consider placing the footing deeper on to the stiff clay or using some kind of soil replacement because the top soil may squeeze out (i.e. if $q_{ult} > 4C_1+\gamma D_f$) beneath the footing.

Case 2: Stratified c-ϕ soil



- Using ϕ_1, compute H =0.5Btan (45+ϕ/2)
- If H < d_1, compute q_{ult} using C_1 and ϕ_1
- If H > d_1, use modified C_{avg} and ϕ_{avg} to compute q_{ult} with,

$$C_{avg} = \frac{C_1 d_1 + C_2(H - d_1)}{H}$$

$$\phi_{avg} = \frac{\phi_1 d_1 + \phi_2(H - d_1)}{H}$$

Case 3: Footings on sand overlaying clay or on clay overlaying sand

- Using ϕ_1, compute H =0.5Btan (45+ϕ/2)
- If H < d_1, compute q_{ult} using C_1 and ϕ_1
- If H > d_1, estimate q_{ult} as follows,

$$q'_{ult} = q''_{ult} + \frac{P\sigma_{vh}K_s\tan\phi}{A_f} + \frac{Pd_1C_1}{A_f} \leq q_{ult}$$

where: q_{ult} = bearing capacity of top layer

q''_{ult} = bearing capacity of lower layer computed using B = footing dimension, C and ϕ of lower layer and q = γd_1

P = total perimeter for punching [P =2 (B+L) or P = π*diameter]

A_f = area of footing (converts perimeter shear forces to a stress)

σ_{vh} = total vertical pressure from footing base to lower soil

K_s = lateral earth pressure coefficient $K_a < K_s < K_p$. Use $K_s = K_o$

Pd_1C_1 = cohesion on perimeter as a force

tanϕ = coefficient of friction b/n $\sigma_{vh}K_s$ and perimeter shear zone wall

A possible alternative for c-ϕ soil with a number of thin layers is to use average values of c and ϕ in the bearing capacity equations obtained as:

$$C_{avg} = \frac{C_1H_1 + C_2H_2 + \ldots\ldots + C_nH_n}{\sum H} \qquad \phi_{avg} = \frac{\phi_1H_1 + \phi_2H_2 + \ldots\ldots + \phi_nH_n}{\sum H}$$

10. *Bearing Capacity of Foundations Subjected to Uplift or Tension Forces*

Footings in industrial applications such as for legs of elevated water tanks, anchorages for the anchor cables of transmission towers, and bases for legs of power transmission towers-and in a number of industrial equipment installations are subjected to uplift or tension forces. Footings that can develop tensile resistance or drilled piers with or without enlarged base are commonly used as foundations for these types of structures. The bearing capacity of these types of foundations may be computed using the following equations. (**11**HLCH-3.doc)

For shallow foundations (D/B < 2.5):

$$\text{Circular :}\qquad T_{ult} = \pi BCD + s_f \pi B \gamma \left(\frac{D^2}{2}\right) K_u \tan\phi + W$$

$$\text{Rectangular :}\quad T_{ult} = 2(B+L)CD + \gamma D^2 (2s_f B + L - B) K_u \tan\phi + W$$

Where: $s_f = 1 + \frac{mB}{D}$; B= width or diameter of footing; D= depth of footing;

L = length of footing; C = cohesion; γ = unit weight ; ϕ = angle of internal friction

K_u = earth pressure coefficient; W= weight of backfill and footing

For deep foundations (H/B > 2.5):

$$\text{Circular :}\qquad T_{ult} = \pi BCH + s_f \pi B \gamma (2B - H)\left(\frac{H}{2}\right) K_u \tan\phi + W$$

$$\text{Rectangular :}\quad T_{ult} = 2(B+L)CH + \gamma(2B - H)(2s_f B + L - B) H\, K_u \tan\phi + W$$

Where: $s_f = 1 + \frac{mB}{H}$; B= width or diameter of footing; D= depth of footing;

L = length of footing; C = cohesion; γ = unit weight ; ϕ = angle of internal friction

K_u = earth pressure coefficient; W= weight of backfill and footing

Obtain shape factors s_f, ratios m and H/B [all $f(\phi)$] from the following table-interpolate as necessary:

ϕ (^{o})	20	25	30	40	45	48
Max [D/B or H/B]	2.5	3	4	7	9	11
m	0.05	0.1	0.15	0.35	0.5	0.6
s_f	1.12	1.3	1.6	4.45	5.5	7.6

11. *Bearing Capacity of Rocks*

It is common to use *building code values* for the allowable bearing capacity of rock; however, geology, rock type, and quality (as RQD) are significant parameters which should be used together with the recommended code value. (**13**HLCH-3.doc)

One may use Terzaghi's bearing capacity equations to obtain the bearing capacity of rocks using ϕ and c of rock from high pressure triaxial tests. Bearing capacity factors to be used are:

$$N_q = \tan^6(45 + \phi/2);\ N_c = 5\tan^4(45 + \phi/2); \qquad N_\gamma = N_q + 1$$

We could estimate ϕ = 40 ^{o} for most rock except limestone or shale where values between 38^o and 45^o should be used. Similarly we could in most cases estimate S_u=5Mpa as a conservative value. Finally we may reduce the ultimate bearing capacity based on RQD as

$$q'_{ult} = q_{ult}(RQD)^2$$

One can also estimate the bearing capacity using the unconfined compressive strength, q_u, determined in the laboratory using core samples (intact rock). The allowable bearing capacity is estimated as: $q_{all} = q_u$ to $2.5q_u$

CHAPTER FOUR

DESIGN OF SHALOW FOUNDATIONS

4.1 DESIGN OF ISOLATED FOOTINGS (1HLCH-4.doc)

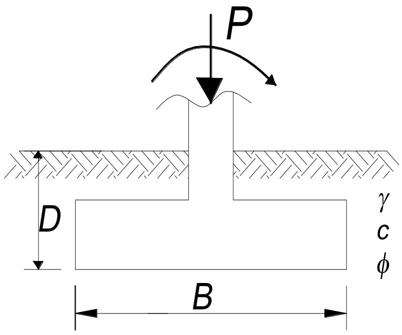
A footing carrying a single column is called a spread footing; since its function is to "spread" the column load laterally to the soil so that the stress intensity is reduced to a value that the soil can safely carry. These members are sometimes called *single or isolated footings*. Single footings may be of constant thickness or either stepped or slopped.

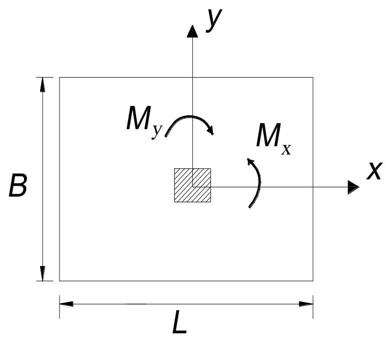
Assumptions used in footing design- [Contact pressure distribution]

Theory of elasticity analysis and observations indicate that the stress distribution beneath symmetrically loaded footings is not uniform. The actual stress distribution depends on the rigidity of the footing and the stiffness of the soil. However, linear pressure distribution is assumed for design purpose. Also the few field measurements reported indicate this assumption is adequate. (2HLCH-4.doc)

The approximate contact pressure under a given symmetrical foundation can be obtained from the flexural formula, provided that the considered load lies with in the kern of the footing [i.e. $e_y < B/6$ and $e_x < L/6$].

$$\sigma(x,y) = \frac{P}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \qquad (4.1)$$





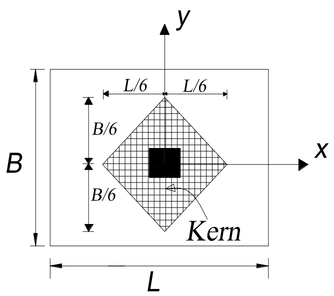
By substituting the following in equation (4.1) we obtain equation (4.2),

$$A = B \times L; \quad e_x = \frac{M_y}{P}; \quad e_y = \frac{M_x}{P}; \quad I_x = \frac{LB^3}{12}; \quad I_y = \frac{BL^3}{12}$$

$$and \quad x = L/2; \qquad y = B/2 \ \ (for\ the\ corners)$$

$$\sigma_{\substack{\max \\ \min}} = \frac{P}{BL}\left(1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{B}\right) \qquad (4.2)$$

If we want to know when we will have negative contact pressure (separation), we proceed as follows



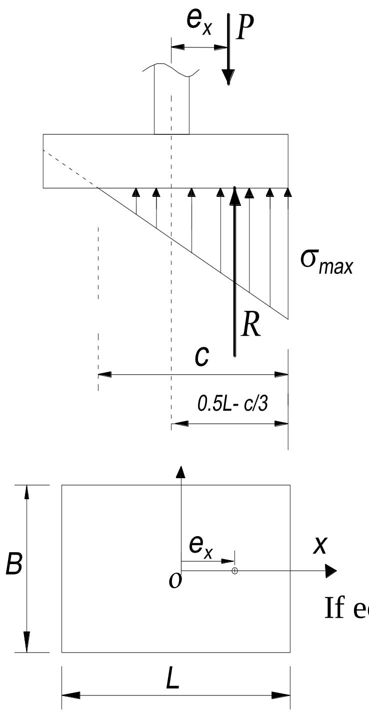
$$\sigma_{\min} = \frac{P}{BL}\left(1 - \frac{6e_x}{L} - \frac{6e_y}{B}\right) \geq 0$$

$$\Rightarrow \qquad 1 \geq \frac{6e_x}{L} + \frac{6e_y}{B}$$

If $e_x = 0 \Rightarrow e_y \leq B/6$ and if $e_y = 0 \Rightarrow e_x \leq L/6$.
Thus as long as the load is within the kern (see Figure), no separation takes place.

A design should not allow as much as possible separation, because that would lead to uneconomical design and potential tilting of the column.

But if there is separation for some reason, then σ_{max} will be determined as follows. Consider eccentricity along L only [$e_x > L/6$].



The new compressive contact pressure alone must equilibrate the superstructure force and moment. Thus,

$$\sum F_z = 0 \Rightarrow R = P$$

$$\Rightarrow \frac{1}{2}\sigma_{max} B c = P = R; \Rightarrow c = \frac{2P}{\sigma_{max} B} \quad (*)$$

$$\sum M_o = 0 \Rightarrow P e_x = R\left(\frac{L}{2} - \frac{c}{3}\right) \qquad (**)$$

From (*) and (**) we have,

$$\sigma_{max} = \frac{4P}{3B(L-2e_x)} \quad and$$

$$c = 3\left(\frac{L}{2} - e_x\right)$$

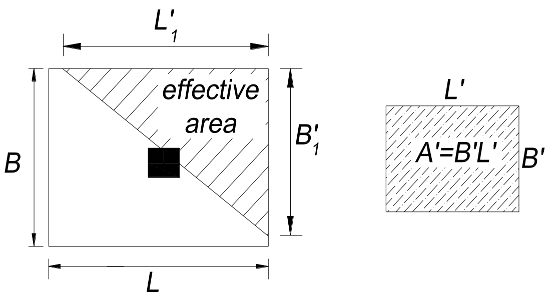
If eccentricity is only along B [$e_y > B/6$] one can similarly get:

$$\sigma_{max} = \frac{4P}{3L(B-2e_y)} \quad and \quad c = 3\left(\frac{B}{2} - e_y\right)$$

If the load is eccentric about both axes, trial and error is needed to determine the maximum soil pressure under any footing. Graphical methods are also available. The curves of Plock shown in Figure 4.1 can be used to locate the *zero-pressure line* and also determine the magnitude of the maximum contact pressure.

- For bearing capacity calculation consider the following,

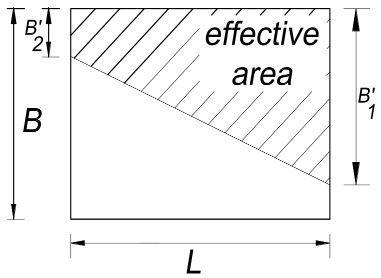
Case 1: $e_x \geq L/6$ and $e_y \geq B/6$



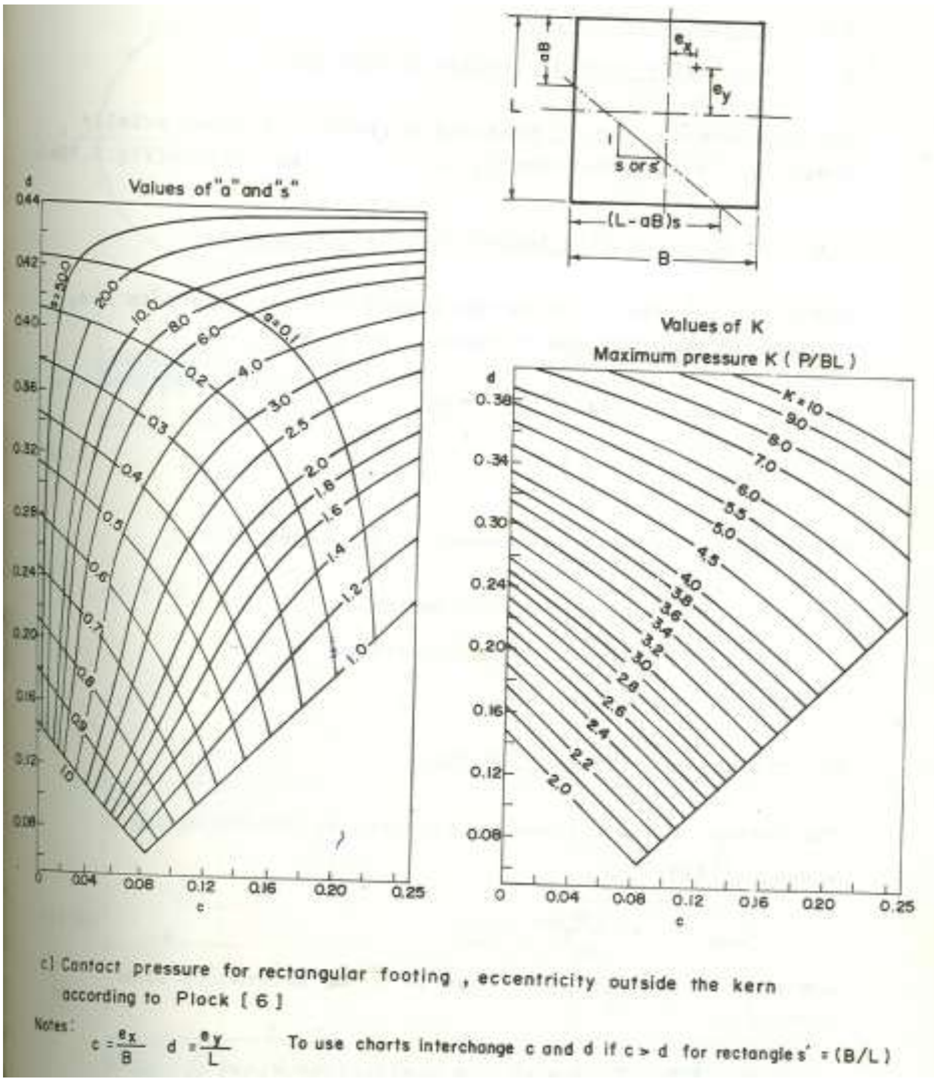
- $L'_1 = 3\left(\frac{B}{2} - e_y\right) \quad and \quad B'_1 = 3\left(\frac{L}{2} - e_x\right)$
- Larger of L'_1 or B'_1 will be L'
- $A' = B'L' \Rightarrow B' = \frac{A'}{L'}$
- Use B' and L' to compute shape factors
- Use B and L to compute other factors
- Use either B' or L' with Nγ in the bearing capacity equation based on the direction of the horizontal load.

Example\9.xls

Case 2: $e_x < L/6$ and $e_y < B/2$



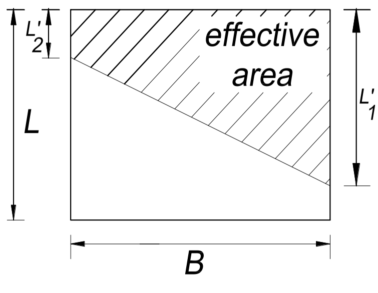
- $A' = \frac{1}{2}(B'_1 + B'_2)L \quad and \quad L' = L \qquad \Rightarrow B' = \frac{A'}{L'}$
- Obtain B'_1 and B'_2 using the above curves
- Use B' and L' for shape factors
- Use B and L to compute other factors
- Use either B' or L' with Nγ in the bearing capacity equation based on the direction of the horizontal load.



Example\Example.xls

Figure 4.1: Approximate Contact Pressure Distribution under Eccentrically Loaded Strip and Rectangular Foundations

Case 3: $e_x < L/2$ and $e_y < B/6$



- $A' = \frac{1}{2}(L'_1 + L'_2)B$
- Obtain L'_1 and L'_2 using the above curves
- L' is taken as greater of L'_1 or L'_2 and $B' = \frac{A'}{L'}$
- Use B' and L' for shape factors
- Use B and L to compute other factors
- Use either B' or L' with Nγ in the bearing capacity equation based on the direction of the horizontal load.

4.1.1 Proportioning of Footings

After having the allowable soil pressure q_{all} for a given soil, one may determine the area and subsequently the proportions of a footing necessary to sustain a given load or combinations of loads. Footings are designed as rigid.

The allowable soil pressure, q_{all} is substituted in place of σ_{max} in the equation, $\sigma_{max} = \frac{P}{BL}\left(1 + \frac{6e_x}{L} + \frac{6e_y}{B}\right)$. Thus, $q_{all} = \frac{P}{BL}\left(1 + \frac{6e_x}{L} + \frac{6e_y}{B}\right)$.

In this equation all other quantities are known except the area A= B L of the footing.

4.1.2 Structural Design of Footings

Before going in to the structural design, one should check if the settlement of the selected footing is with in the prescribed safe limits. If the settlement exceeds the safe limits, one should increase the area of the footings until the danger of settlement is eliminated.

One then should design for the following modes of failures:

1. Shear failure
   - Punching shear
   - Wide beam shear (diagonal tension)

   } to avoid these provide adequate depth
2. Flexural failure --- provide adequate depth and reinforcement
3. Bond failure
   - column bar pullout
   - Flexural reinforcement bars failed in bond

   } to avoid these provide adequate development or anchorage length

(4,5,6HLCH-4.doc)

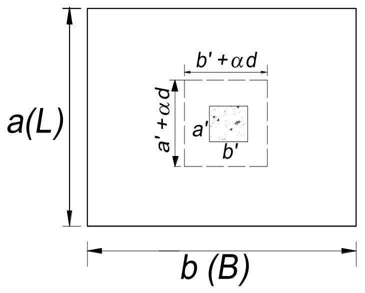
(i) Determination of Thickness

The thickness of a given footing is usually governed by *punching shear* (for square and centrally loaded footings) or *wide beam shear* (for rectangular footings with large L/B ratio or eccentrically loaded footings).

(a) Thickness from Punching Failure (3HLCH-4.doc)

It is common practice to provide adequate depth to sustain the shear stress developed without reinforcement. The critical section for punching is as shown in the figure below.

Rectangular columns:



$\alpha = 1.0$

i.e. $\frac{d}{2}$ distance around the column

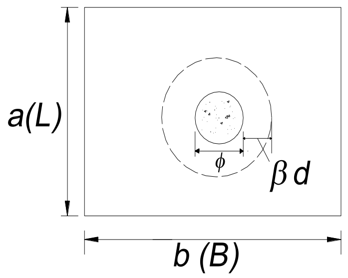
- punching resistance: $V_r = 2\left[(a'+d)+(b'+d)\right] v_{up} d$

- acting punching shear force: $V_a = \left[ab-(a'+d)(b'+d)\right]\sigma$ Or $V_a = P_{col} - (a'+d)(b'+d)\sigma$

- equating the two above expressions, one can now solve for d from

$$(4v_{up}+\sigma)d^2 + (2v_{up}+\sigma)(a'+b')d - (ab-a'b')\sigma = 0 \quad (4.3a)$$

Circular columns:



$\beta = 0.5$

- punching resistance: $V_r = \pi(\phi+d)\, v_{up} d$

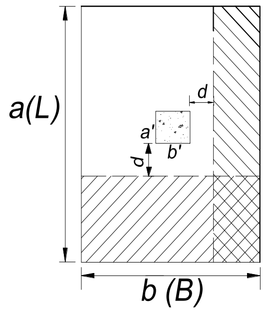
- acting punching shear force: $V_a = \left[ab - \frac{\pi}{4}(\phi+d)^2\right]\sigma$ Or $V_a = P_{col} - \frac{\pi}{4}(\phi+d)^2\sigma$

- equating the two above expressions, one can now solve for d from

$$(v_{up}+\frac{\sigma}{4})d^2 + (v_{up}+\frac{\sigma}{2})(\phi)d - (ab-\frac{\pi}{4}\phi^2)\frac{\sigma}{\pi} = 0 \qquad (4.3c)$$

(b) Thickness from wide beam Shear (Diagonal Tension)

The selected depth using punching shear criterion may not be adequate to withstand the diagonal tension developed. Hence one should also check the safety against diagonal tension. The critical sections for wide beam shear are as shown in the figure below.



Acting shear force (wide beam shear)

Short direction: $V_{SS} = \left(\frac{a}{2} - (\frac{a'}{2} + d)\right) b\sigma$

Long direction: $V_{LL} = \left(\frac{b}{2} - (\frac{b'}{2} + d)\right) a\sigma$

Resisting shear (wide beam shear)

Short direction: $V_{rS} = b\,d\,v_{uw}$

Long direction: $V_{rL} = a\,d\,v_{uw}$

At the limiting state we have,

$$V_{SS} = V_{rS} \qquad \Rightarrow \left(\frac{a}{2} - (\frac{a'}{2} + d)\right) b\sigma = b\,d v_{uw} \;\Rightarrow d = \frac{(a-a')\sigma}{2(\sigma + v_{uw})}$$

$$V_{LL} = V_{rL} \qquad \Rightarrow \left(\frac{b}{2} - (\frac{b'}{2} + d)\right) a\sigma = a\,d v_{uw} \;\Rightarrow d = \frac{(b-b')\sigma}{2(\sigma + v_{uw})}$$

Thus if d is calculated from punching, the above calculated d's for wide beam shear must be less than that d from punching. Or if the thickness is already obtained from punching requirement, then we need only to check that the wide beam shear strength is not exceeded.

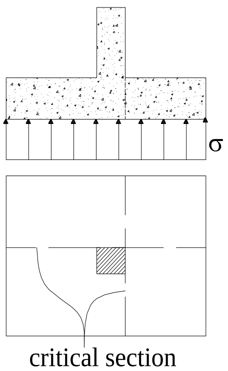
$$Short\ direction: \qquad v_w = \frac{V_{SS}}{b\,d} = \frac{\left(\frac{a}{2} - (\frac{a'}{2} + d)\right)\sigma}{d} \le v_{uw}$$

$$Long\ direction: \qquad V_w = \frac{V_{LL}}{a\,d} = \frac{\left(\frac{b}{2} - (\frac{b'}{2} + d)\right)\sigma}{d} \le V_{uw}$$

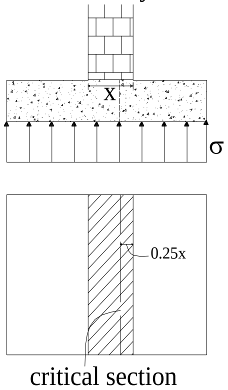
(ii) Determination of Flexural Reinforcement

The critical section for bending moment may vary according to the types of columns as shown in the figure below.

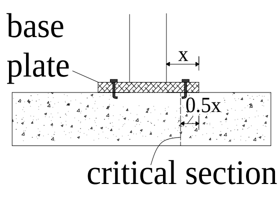
- column/ wall(except masonry)



Masonry



- Steel column resting on concrete



(iii) Bond Strength and Development Length

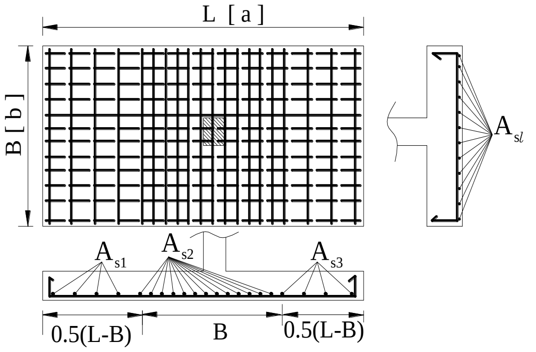
The development length is determined from available formulae and it should be grater or equal to the available length. The available development length can be calculated as length from critical section to extreme side of the footing less concrete cover.

(iv) Placement of Reinforcement Bars

a) For square footings reinforcements are distributed uniformly in both directions

b) For rectangular footing:

- Longitudinal steel in the long direction (usually placed on bottom) shall be uniformly spaced
- Steel in the short direction based on ACI code is as shown below



- $A_{sl} = A_{s\,a-a}$
- $A_{s1} = A_{s3} = \frac{1}{2}\left(\frac{L-B}{L+B}\right)A_{s\,b-b}$
- $A_{s2} = \frac{2B}{L+B}A_{s\,b-b}$

Allowable stresses according to EBCS 2 (1995): [for LSD]

1. Punching shear resistance

$$v_{up} = 0.5\mathrm{f}_{ctd}(1+50\rho_e) \qquad \Rightarrow V_{up} = 0.5\mathrm{f}_{ctd}(1+50\rho_e)\,\mathrm{U\,d}$$

where: f_{ctd} = design tensile strength of concrete; $\mathrm{f}_{ctd} = \frac{0.35}{\gamma_c}\sqrt{\mathrm{f}_{cu}}$; $\gamma_c = 1.5$

ρ_e = effective geometrical ratio of reinforcement

$\rho_e = \sqrt{\rho_{ex}\rho_{ey}} \leq 0.008$, ρ_{ex} and ρ_{ey} are geometrical steel ratios in the x and y directions respectively

U = perimeter of the critical section

d = effective depth

2. Wide beam shear resistance

$$v_{uw} = 0.3\mathrm{f}_{ctd}(1+50\rho) \qquad \Rightarrow V_{uw} = 0.3\mathrm{f}_{ctd}(1+50\rho)\,\mathrm{b}_w\,\mathrm{d}$$

where: $\rho = \frac{A_s}{b_w d} \leq 0.02$; b_w = width of web or rib of a member

3. Development length

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

where: $f_{yd} = f_{yk}/\gamma_s$; γ_s=1.15; $f_{bd} = f_{ctd}$

Allowable stresses according to ACI: [for USD]

1. Punching shear resistance

$$v_{up} = \frac{\phi}{3}\sqrt{\mathrm{f}'_c} = 0.33\phi\sqrt{\mathrm{f}'_c}$$

2. Wide beam shear resistance

$$v_{uw} = \frac{\phi}{6}\sqrt{\mathrm{f}'_c} = 0.17\phi\sqrt{\mathrm{f}'_c}$$

3. Embedment of reinforcing bars of diameter < 35mm

$$l_d = \frac{0.19 A_b\, f_y}{\sqrt{f'_c}}$$

where: ϕ = reduction factor and is 0.85 for shear

f'_c = 28-day cubic concrete strength in MPa, f_y =yield strength of steel(MPa)

A_b = area of single bar in mm^2 and l_d(mm)

4. The permissible bearing pressure

$$f_b = 0.60 f'_c \sqrt{\frac{A_2}{A_{col}}} \quad ; \qquad \sqrt{\frac{A_2}{A_{col}}} \le 2$$

Where A_2 = base area of the bearing frustum = (b' + 4d) (a' + 4d)

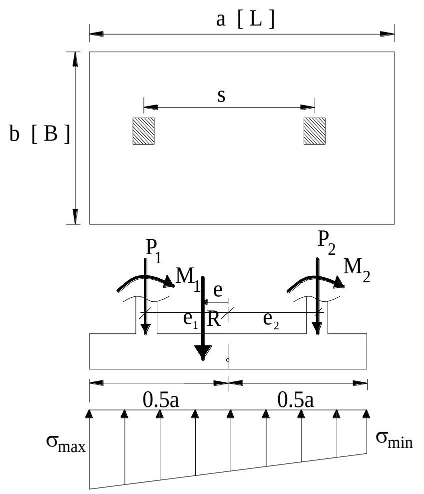
A_{col} = area of the columns = b' a'

4.2 DESIGN OF COMBINED FOOTINGS

When a footing supports a line of two or more columns, it is called combined footing. A combined footing may have either rectangular or trapezoidal shape. It may not be possible to place columns near property line or near mechanical equipment. Columns located off center will result in a non uniform pressure and it may not be also stable against overturning. In order to avoid this problem, an alternative is to enlarge the footing and place one or more columns on one footing.

4.2.1 DESIGN OF RECTANGULAR COMBINED FOOTINGS (8HLCH-4.doc)

The footing is designed such that the centroid of the footing area coincides with the resultant of the column loads. This produces uniform bearing pressure over the entire area and prevents the tendency of tilting. Thus the proportioning is done using the flexural formula.



First determine location of the resultant:

- $R = P_1 + P_2$
- $R * e_x = M_{1y} + M_{2y} + P_2 e_{2x} - P_1 e_{1x}$

$$\Rightarrow e_x = \frac{M_{1y} + M_{2y} + P_2 e_{2x} - P_1 e_{1x}}{R}$$

- $R * e_y = M_{1x} + M_{2x} + P_2 e_{2y} - P_1 e_{1y}$

$$\Rightarrow e_y = \frac{M_{1x} + M_{2x} + P_2 e_{2y} - P_1 e_{1y}}{R}$$

Then use flexural formula to determine the planar dimensions of the footing

$$\sigma_{\substack{max \\ min}} = \frac{P}{BL}\left(1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{B}\right) \le \left(q_{all} \; or \; q_{ult}\right)$$

General Design Procedure

1 Determine the location of the resultant R, eccentricities e_a [e_L or e_x] and e_b [e_B or e_y]

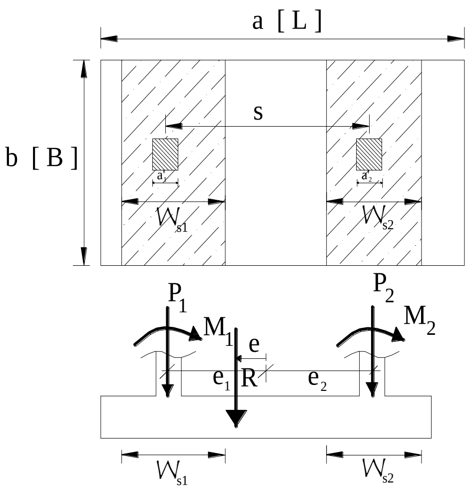
2 Determine the planar dimension in such a way that

$$\sigma_{\substack{max \\ min}} = \frac{P}{BL}\left(1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{B}\right) \le \left(q_{all} \; or \; q_{ult}\right)$$

3 Treating it like a beam in the longitudinal direction draw BMD and SFD

4 Make a structural design using the SF and BM. The critical sections are same as that of isolated spread footing with the thickness determined based on punching and wide beam shear and flexural steel is determined from BM

5 Determine short direction reinforcement as spread footing. Here width of footing around column is assumed to be effective to transfer the column loads to the soil. The effective zones are obtained by adding 0.75d in ether side of the columns from the face of the column.



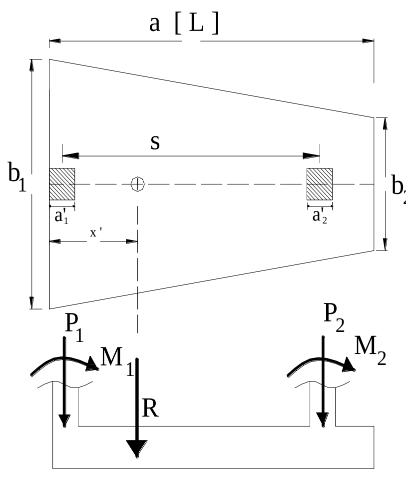
$$W_{s1} = a'_1 + 1.5d$$

$$W_{s2} = a'_2 + 1.5d$$

(9HLCH-4.doc)

4.2.2 DESIGN OF TRAPEZOIDAL COMBINED FOOTINGS

A combined footing will be trapezoid-shaped if the column that has too limited space for a spread footing carries the larger load. In this 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 rectangular footing (to achieve a uniformly distributed contact pressure) will not provide sufficient length to reach the interior column. Thus one has to use a wider section near the column with larger load. The footing geometry is as shown below.



$$R = P_1 + P_2$$

$$R\frac{a}{2} = P_1\left(\frac{a'_1}{2}\right) + M_1 + P_2\left(s + \frac{a'_1}{2}\right) + M_2$$

$$\Rightarrow a = \frac{2}{R}\left[P_1\left(\frac{a'_1}{2}\right) + M_1 + P_2\left(s + \frac{a'_1}{2}\right) + M_2\right]$$

Trapezoidal footing will be used if the out-to-out distance between columns is greater than 2a *i.e* $2a < \left(\frac{a'_1}{2} + s + \frac{a'_2}{2}\right)$ unless the distance s is so great that a cantilever (or strap) footing would be more economical.

Area of trapiezium $A = \frac{L}{2}(b_1 + b_2)$

Centroid of trapiezium $x' = \frac{L}{3}\frac{(2b_2 + b_1)}{b_1 + b_2}$ by taking moment of area.

For uniform contact pressure distribution line of action of the resultant R should pass through the centroid of the area.

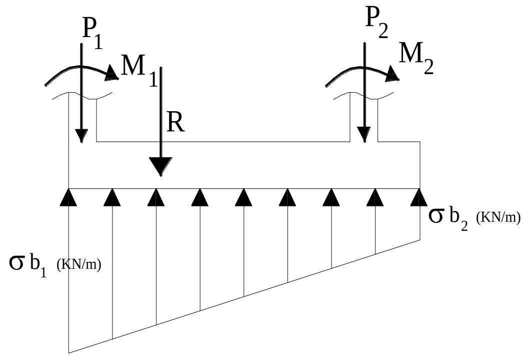
For $b_2 = 0$ (i.e triangle), $x' = \frac{L}{3}$ and for $b_2 = b_1$ (i.e rectangle), $x' = \frac{L}{2}$; it follows that a trapezoidal footing is a solution for L/3 < x' <L/2 with a minimum value of L as out-to-out of the column faces. In most cases a trapezoidal footing would be used with only two columns, but the solution proceeds similarly for more than two columns. The forming and reinforcing steel for trapezoid footing is somewhat awkward to place.

General Design Procedure

1. Determine the location of the resultant R, eccentricities e_a [e_L or e_x] and e_b [e_B or e_y]
2. Calculate a [or L] from, $a = \frac{2}{R}\left[P_1\left(\frac{a'_1}{2}\right) + M_1 + P_2\left(s + \frac{a'_1}{2}\right) + M_2\right]$. Then trapezoidal footing will be used if $2a < \left(\frac{a'_1}{2} + s + \frac{a'_2}{2}\right)$ unless the distance s is so great that a cantilever (or strap) footing would be more economical.
3. Determine the planar area, A in such a way that

   $$\sigma \le \frac{P}{A} \qquad \text{[uniform stress distribution is implied]}$$

4. Determine dimensions b_1 and b_2 from $A = \frac{L}{2}(b_1 + b_2)$ and $x' = \frac{L}{3}\frac{(2b_2 + b_1)}{b_1 + b_2}$
5. After b_1 and b_2 are determined the footing is treated like a beam in the longitudinal direction similar to rectangular footings except that the "beam" pressure diagram will be linearly varying (1st degree) from b_1 and b_2 not being equal.



6. Draw BMD (3rd degree curve) and SFD(2nd degree curve)
7. Make a structural design using the SF and BM. The critical sections are same as that of isolated spread footing with the thickness determined based on punching and wide beam shear and flexural steel is determined from BM
8. Determine short direction reinforcement as spread footing. Here width of footing around column is assumed to be effective to transfer the column loads to the soil. The effective zones are obtained by adding 0.75d in ether side of the columns from the face of the column. (10HLCH-4.doc)

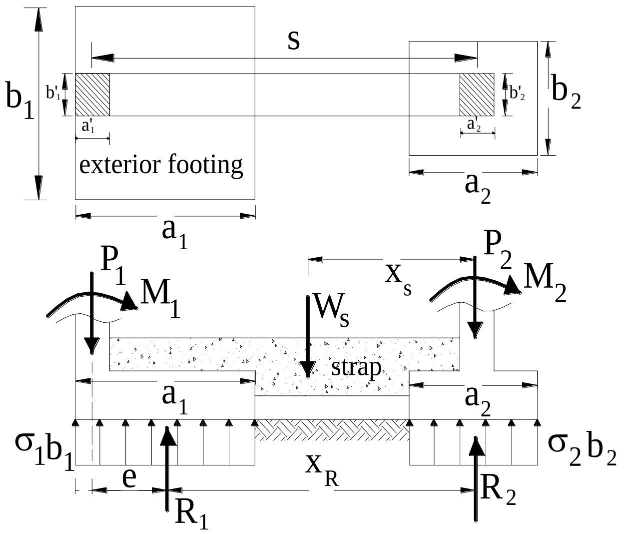
4.2.3 DESIGN OF STRAP (OR CANTILEVER) FOOTINGS

Essentially a strap footing consists of a rigid beam connecting two pads (footings) to transmit unbalanced shear and moment from the statically unbalanced footing to the second footing so that a uniform soil pressure is computed beneath both footings. The strap serves the same purpose as the interior portion of a combined footing but is much narrower to save on materials. Thus strap footings are used as alternative to combined footings when the cost of combined footings is relatively high. It may be used in lieu of a combined rectangular or trapezoid footing if the distance between columns is large (say > 8m) and /or the allowable soil pressure is relatively large so that the additional area is not needed.

Proportioning

In the proportioning of footings, three basic assumptions are used. Theses are:

1. The strap or beam connecting the two footings is perfectly rigid. Perhaps $I_{strap}/I_{footing} > 2$ (Bowels). This rigidity is necessary to avoid rotation of the exterior footing
2. Footings should be proportioned for approximately equal soil pressure and avoidance of large differences in b to minimize differential settlement
3. Strap should be out of contact with soil so that there is no soil reactions



Procedures for proportioning the footins are:

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

$$a_1 = 2(0.5a'_1 + e) \quad \Rightarrow \quad x_R = S - e$$

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

$$R_1 X_R - P_1 S - W_s X_S + M_1 + M_2 = 0 \quad \Rightarrow R_1 = \frac{P_1 S + W_s X_S - M_1 - M_2}{X_R}$$

where Ws= weight of strap (it can be neglected if the strap is relatively short)

c. Determine the magnitude of R_2 from $\Sigma F_y = 0$

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

d. Compute the widths of the footings

$$b_1 = \frac{R_1}{a_1 \sigma} \qquad \text{then make } b_1 = b_2 \text{ and hence } a_2 = \frac{R_2}{b_2 \sigma}$$

e. Structural design : SFD and BMD are drawn and the footings are designed as spread footings (11HLCH-4.doc)

4.2.4 DESIGN OF MAT FOUNDATION

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 individual footings in to one mat as the bearing capacity is proportional to width and depth of foundations. In addition to increasing the bearing capacity mat foundations tend to bridge over irregularities of the soil and 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 shown in the Figure 4.2.

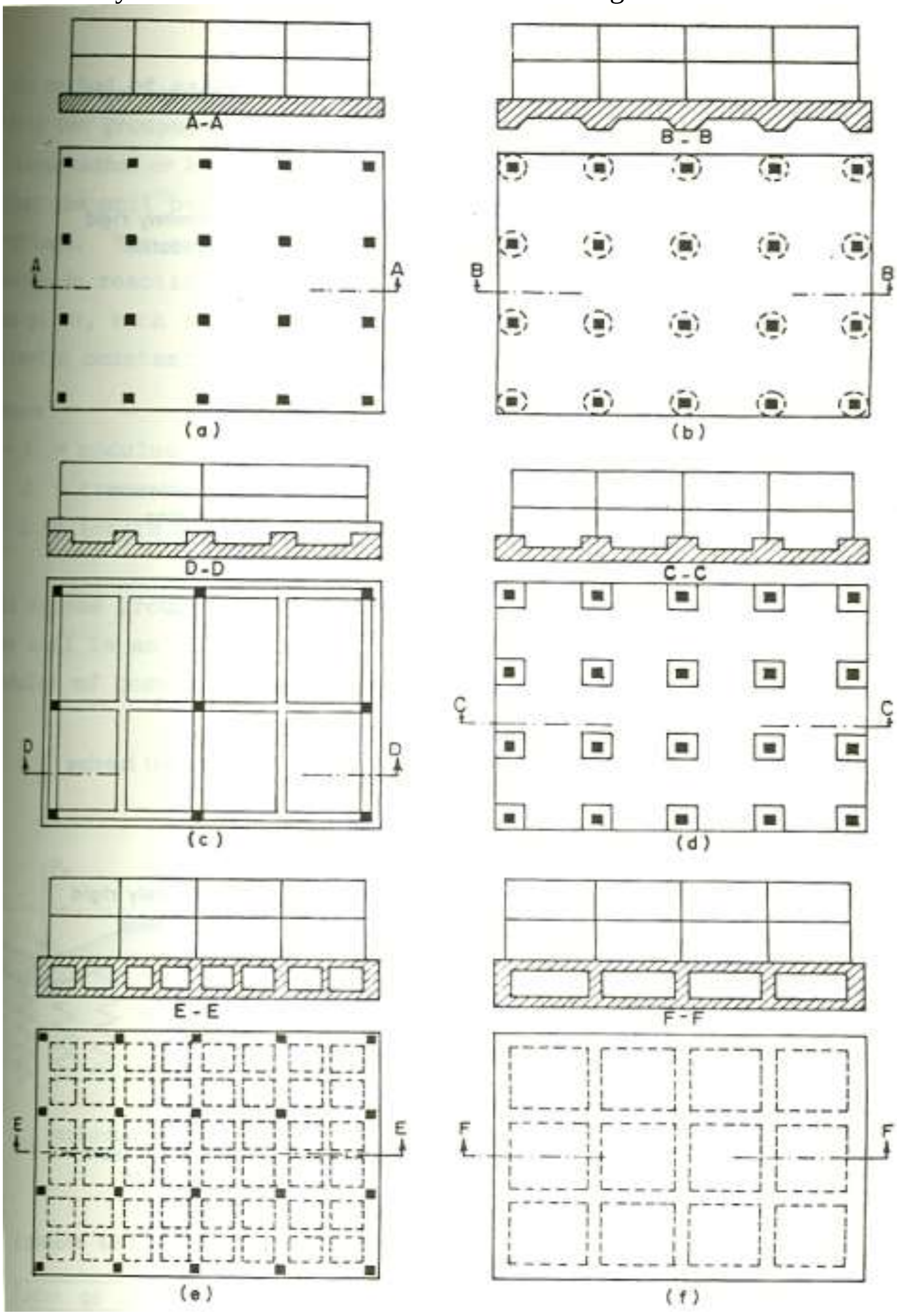


Figure 4.2: Different Forms of Mat foundations

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

Probably the most common mat design consists in a flat concrete slab 0.75m to 2m thick and with continuous two way reinforcing top and bottom. This type of foundation tends to be heavily over designed for three major reasons:

1. Additional cost of, and uncertainty in, analysis
2. The extra cost of over design of this element of the structure will generally be quite small for reasonable amounts of over design relative to total project cost
3. The extra safety factor provided for the additional cost

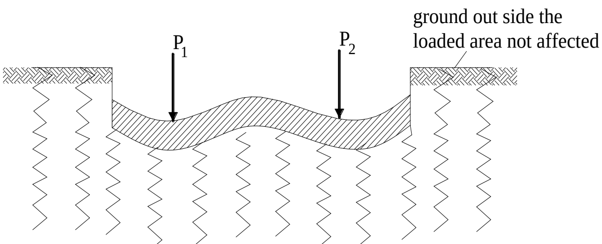
Design Methods

In structural action a mat is very similar to a flat slab or flat plate, upside down, i.e. loaded upward by the bearing pressure and downward by the concentrated column reactions. The method of design depends on the assumption made regarding the distribution of bearing pressures which act as up ward loads on the foundation. Basically there are two methods of design, namely the rigid method and elastic method.

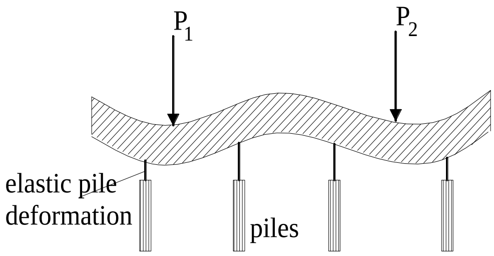
1. Elastic Method

This method may be divided into two groups.

The first group is known as the simplified elastic method or Winkler method, is based on the assumption that the soil behaves like individual separate elastic springs. The spring constant is taken to be the modulus of sub-grade reaction of the soil. In the case of a raft resting on piles, each pile is considered as a spring having an elastic constant equal to $\frac{EA}{I}$ where, E = modulus of elasticity of pile, A = cross-sectional area of pile, L = length of pile.

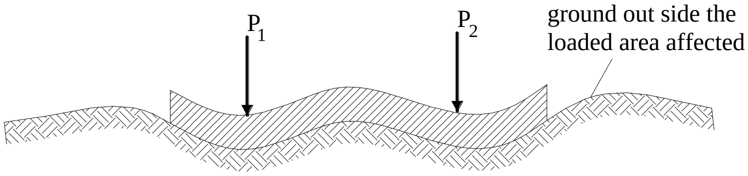


(a) Mat resting on Soil



(b) Mat resting on Piles

The second group known as the true elastic method assumes that the soil is elastic continuum with a constant or variable modulus of compressibility.



True Elastic method

2. Rigid Method

Here it is assumed that the mat is infinitely rigid in comparison with the sub soil. The contact pressure under the mat is assumed to be linearly distributed and the centroid of the bearing pressure coincides with the line of action of the resultant force of all the loads acting on the mat. Then all loads, the downward column loads as well as the upward bearing pressures are known. Hence, moments and shear forces in the foundation can be found by statics alone. Once theses are determined the design of the mat foundation is similar to that of inverted flat slabs or plates. However, approximate methods of analysis of mats can be used.

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

$$\lambda = \left[\frac{K_S\, b}{4E_c\, I}\right]^{1/4} \qquad \text{-------} \quad (a)$$

where λ = characteristic coefficient

K_s =coefficient of sub-grade reaction

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

Ec = modulus of elasticity of concrete

I = moment of inertia of strip of width b

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

a. Rigid Method for Uniform Mat Design

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 flexural equation:

$$\sigma = \frac{R_{tot}}{A} \pm \frac{R_{tot} e_x\, x}{I_x} \pm \frac{R_{tot} e_y\, y}{I_y} \qquad (b)$$

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

Method A

Convert the contact pressure calculated using equation (b) to a uniform contact pressure distribution using the engineering judgment. Take a system of column strip with width Ws as shown in Figure 4.3 (a). Draw 45^0 diagonal lines from the edges of pedestals (columns) to form the system of lines indicated in the figure.

The central slabs, like for instance RSTU (shaded), are designed as two way rectangular slabs with fixed edges supported by 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. 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 [Figure 4.3 (a)].

Method B

In the case where the column loads and spacings do not vary more than 20% from each other, divide the slab into perpendicular bands [Figure 4.3(b)]. 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 $M = \frac{w\,l^2}{10}$ for interior spans and $M = \frac{w\,l^2}{8}$ for exterior spans.

(v) Check wide beam and punching shear

(vi) Provide the necessary reinforcement.

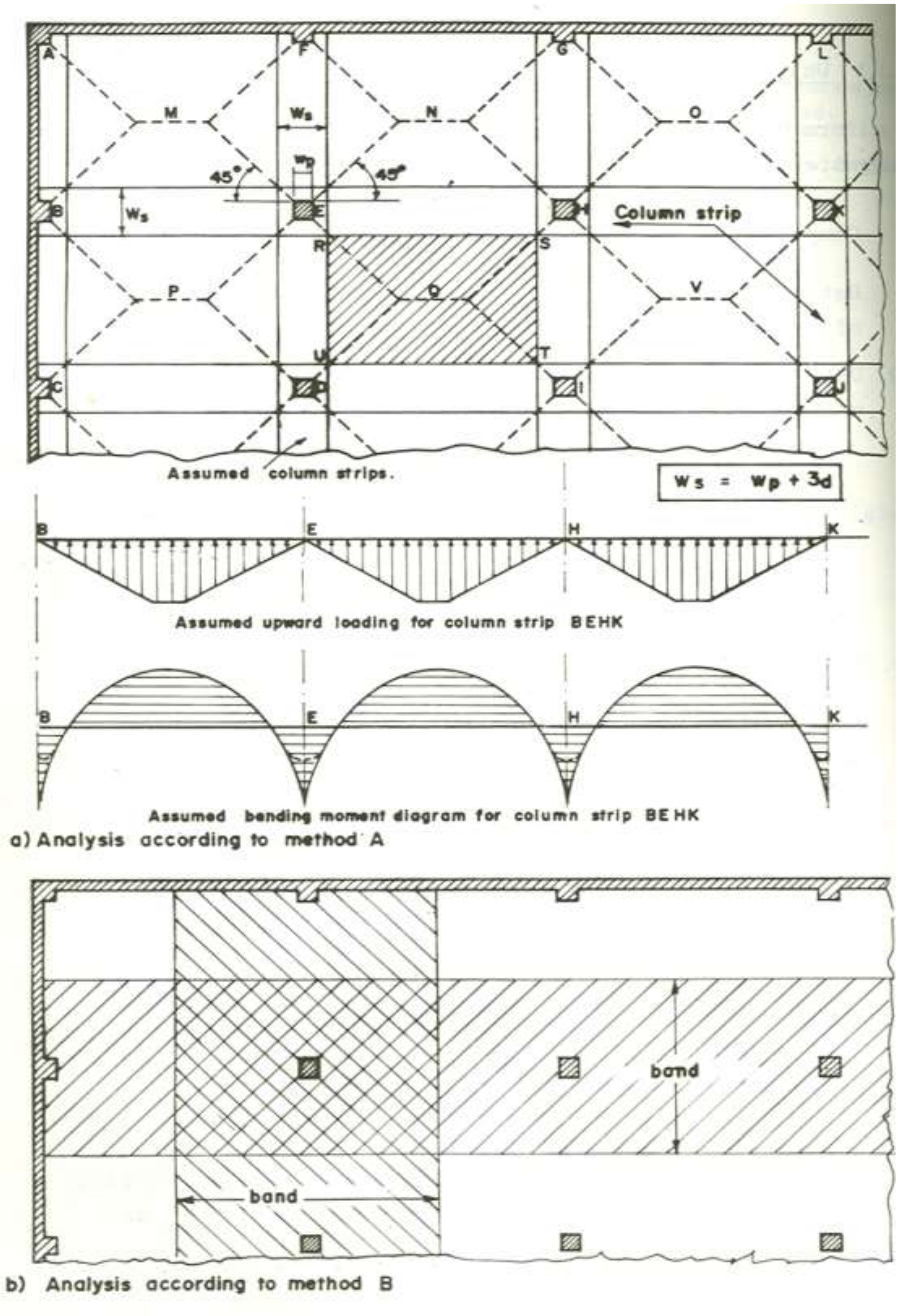


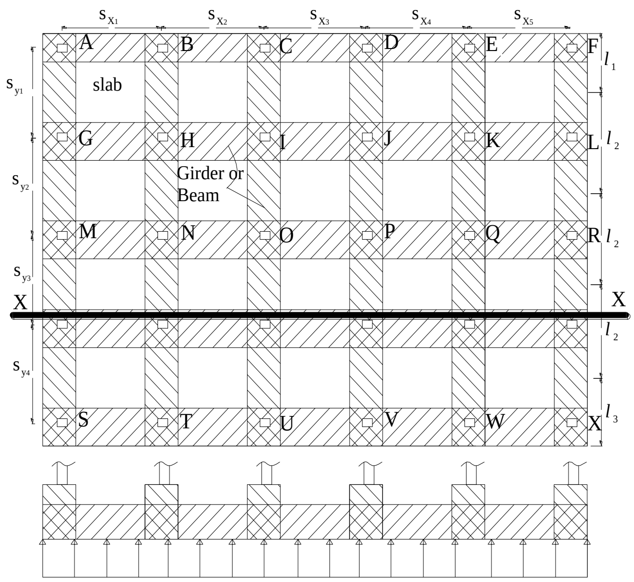
Figure 4.3: Approximate Methods of analysis of Large Mat

b. **Ribbed Mat Design**

Ribbed mats are frequently used in the practice and are found to be economical than uniform mats especially for heavy structures. In the case of ribbed mat, systems of beams are introduced both in the x- and y- directions to stiffen the slab. Ribbed mat could be designed as two way slab or using a simplified method. And the beams (girders) have to be designed for both bending and shear.

Simplified Method

Considering the figure below,



SECTION X-X

RIBBED MAT

Slab design:

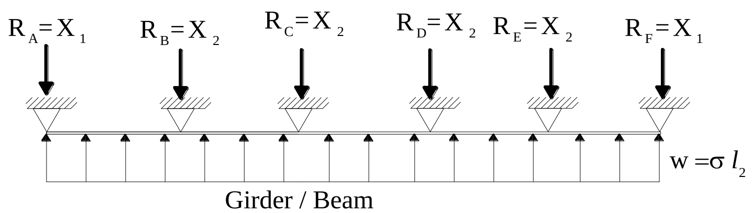
- Along the X- direction
  - ✓ Calculate the moment from, $M = \frac{(\sigma\, S_y)S_x^{\,2}}{10}$
  - ✓ Using M determine the reinforcement and provide the same steel area at the top and bottom
- Along the Y- direction
  - ✓ Calculate the moment from, $M = \frac{(\sigma\, S_x)S_y^{\,2}}{10}$
  - ✓ Using M determine the reinforcement and provide the same steel area at the top and bottom

Beam (Girder) design:

- Along the X -direction

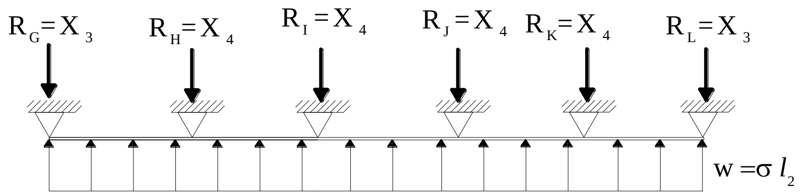
Edge beams (beam A-B-C-D-E-F and S-T-U-V-W-X)

$$n\, X_2 + 2\, X_1 = L_{tx}\, w \text{ --------- (a)}$$



Interior beams (beams like G-H-I-J-K-L etc)

$$n\, X_4 + 2\, X_3 = L_{tx}\, w \quad \text{--------- (b)}$$

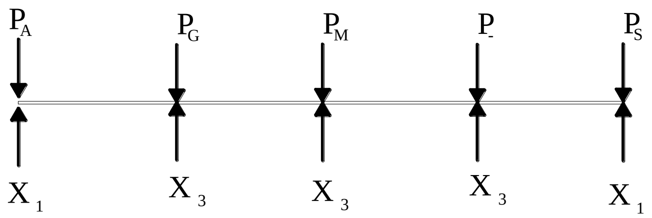


Girder / Beam

- Along the Y -direction

Edge beams (beam A-G- - -M-S and F-L-R- - -X)

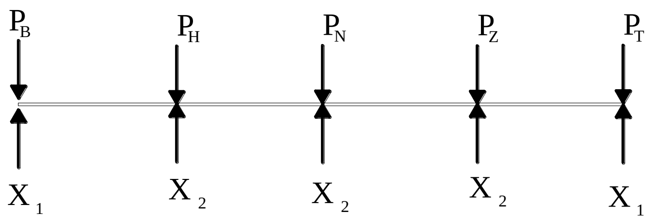
$$m\,X_3 + 2\,X_1 = \sum P_i \qquad \text{---------}\ (c)$$



Girder / Beam

Interior beams (beams like B-H-N-Z-T etc)

$$m\,X_4 + 2\,X_2 = \sum P_i \qquad \text{---------}\ (d)$$



Girder / Beam

Other additional relationships are,

$$\frac{X_1}{X_3} = \frac{\sigma l_1}{\sigma l_2} = \frac{l_1}{l_2} \qquad and \qquad \frac{X_2}{X_4} = \frac{\sigma l_1}{\sigma l_2} = \frac{l_1}{l_2} \qquad ------\ (e)$$

One can solve for the unknown reactions from equations (a), (b), (c), (d) and (e) and hence draw BMD and SFD. The beams are then designed for flexure and shear accordingly. (**12**HLCH-4.doc)

CHAPTER FIVE

RETAINING WALLS

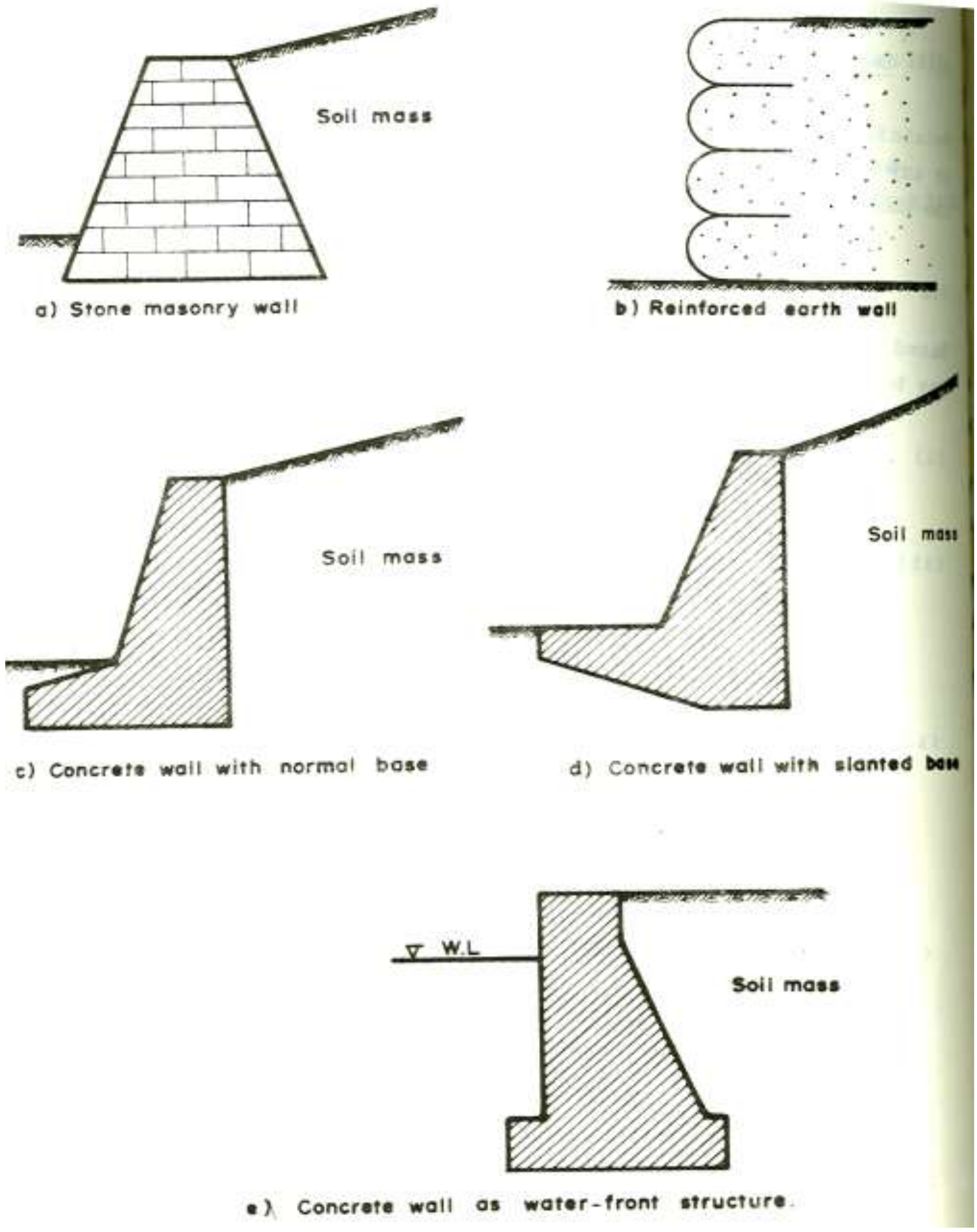
1. Types of Retaining Walls

Retaining walls are structures used to retain a mass of earth or any other material where prevailing conditions do not allow the mass to assume its natural slope. They commonly support vertical or nearly vertical slopes of soil.

Various types of retaining walls are shown in Figure 5.1 & 5.2 and are widely employed in civil engineering works ranging from their use in road and rail construction to support cuts and fills where space is limited to prevent the formation of appropriate side slopes, to the construction of marine structures such as docks, harbours and jetties.

Based on the method of achieving stability, retaining walls may be categorized into the following types. .

a) **Gravity walls**: the stability of the walls depends on their weight (Figure 5.1)



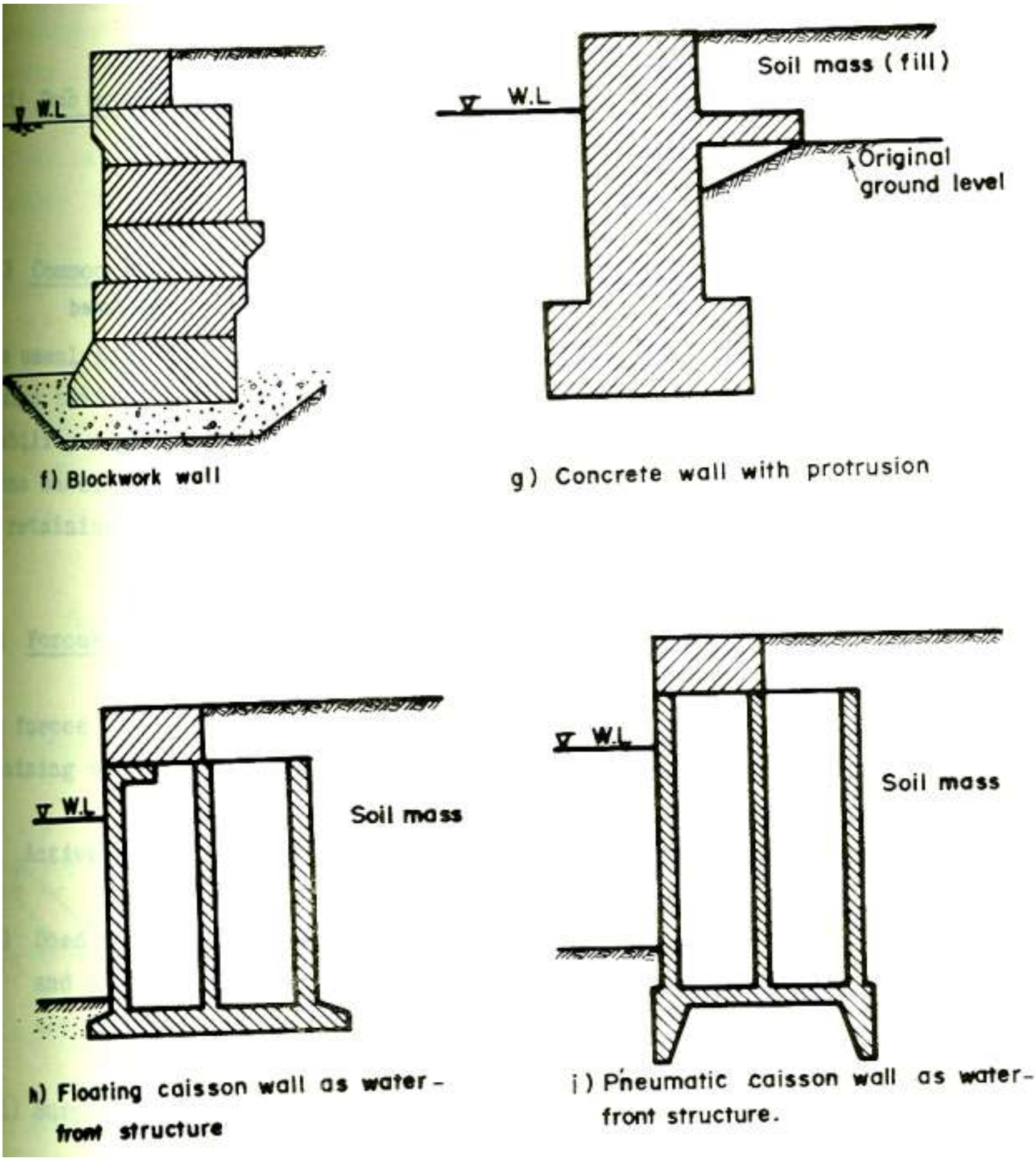


Figure 5.1: Gravity walls

b) **Cantilever walls:** these are reinforced concrete walls that utilize cantilever action to retain the mass of earth or any other material behind them (Figure 5.2 b)

c) **Semi-Gravity walls:** these are walls that are intermediate between gravity and cantilever walls. Here a small amount of reinforcement is added to reduce the mass of concrete.

d) **Counterfort retaining walls:** these are high walls similar to cantilever walls with the difference that vertical bracing is provided to tie the walls and the base together. (Figure 5.2 c)

e) **Buttresses retaining walls:** these walls are similar to the counterfort retaining walls with the difference that the bracing is in front of the wall and is subjected to compressive force instead of a tension force (Figure 5.2 d)

f) **Crib walls:** the walls are built up members of pieces of timber, metal or pre-cast concrete and filled with granular material (Figure 5.2 g)

g) **Sheet pile walls:** Sheet pile walls are sheet like retaining structures that are commonly used in place of conventional retaining walls. They are commonly used in: water front constructions, temporary constructions, places where massive excavation is not possible due to limited space (Figure 5.2 e & f)

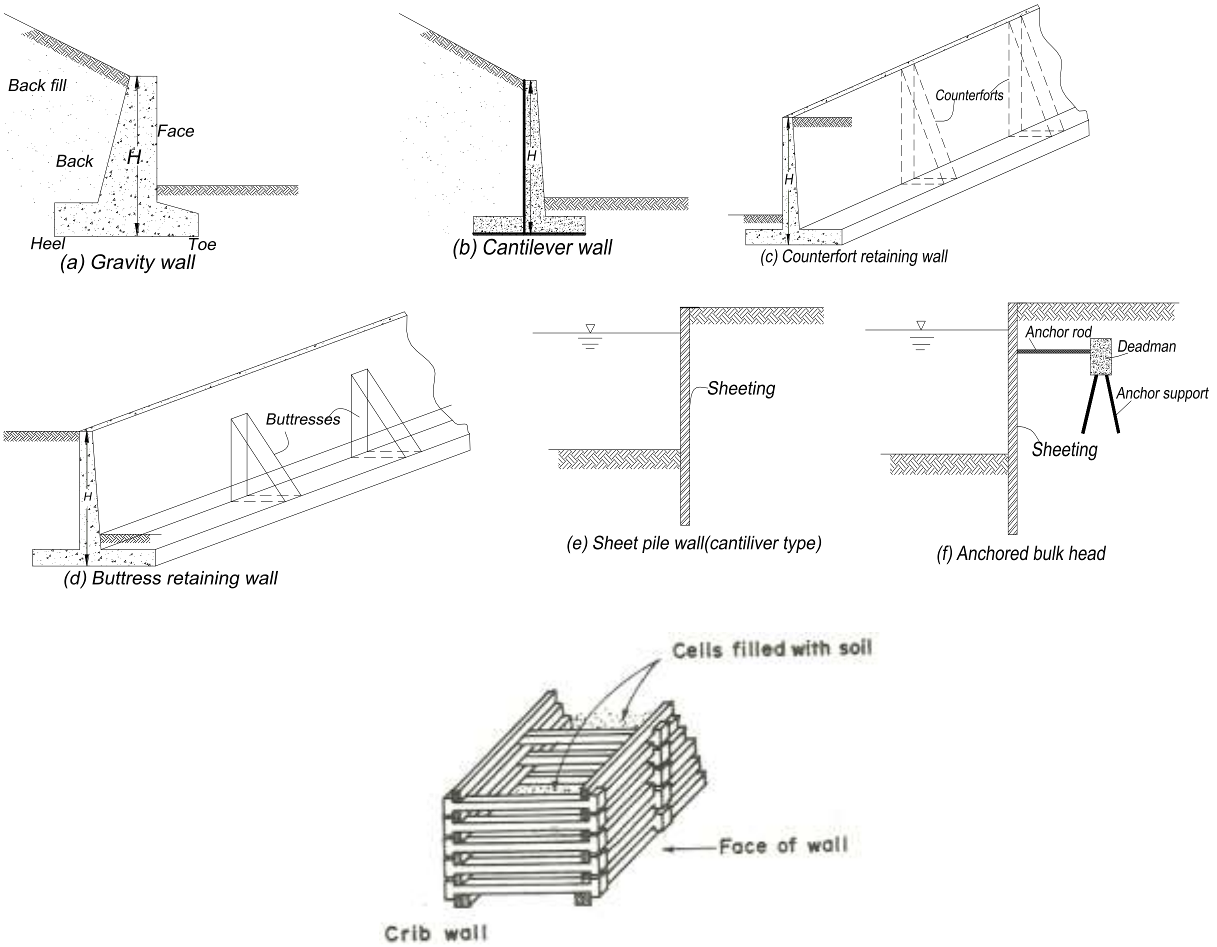
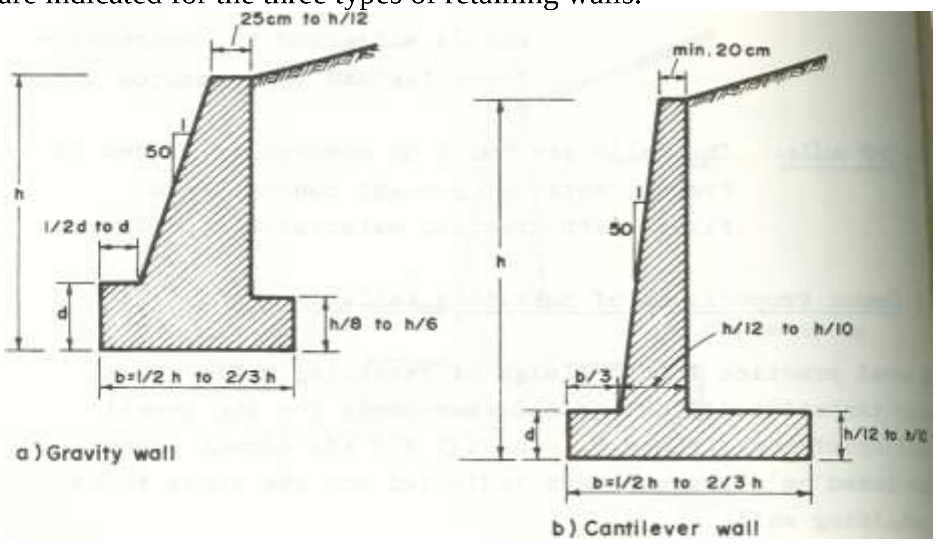


Figure 5.2 Types of Earth retaining structures

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. In Figure 5.3 the common proportions based on experience are indicated for the three types of retaining walls.



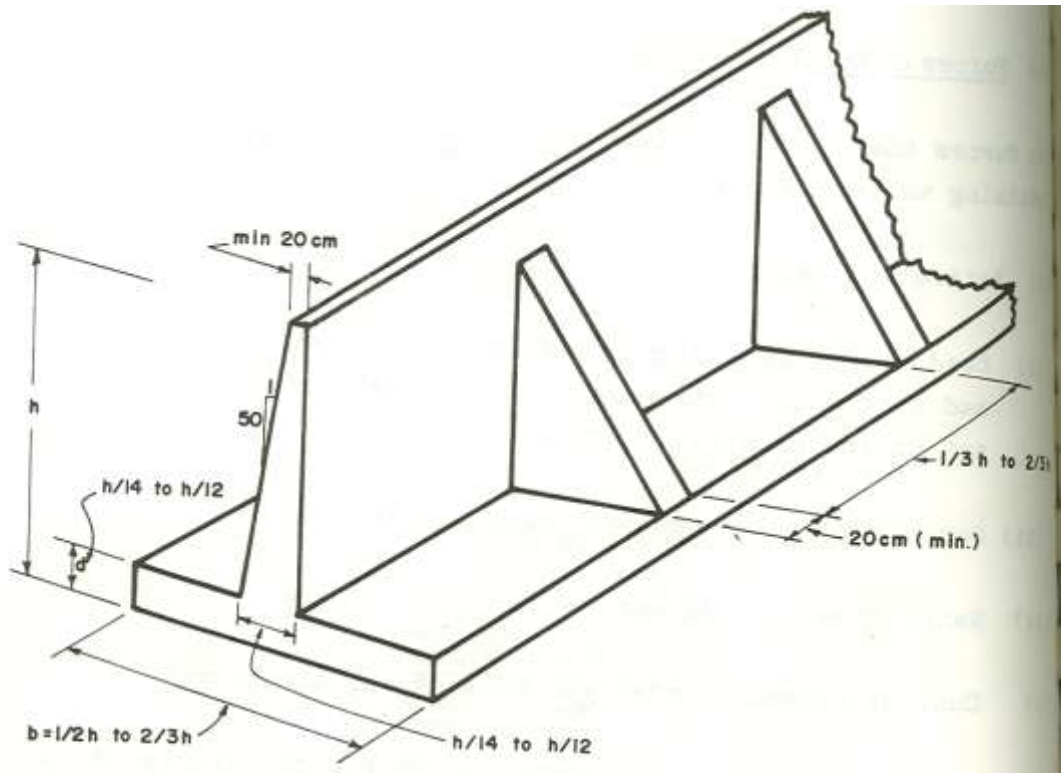


Figure 5.3: Common Design Proportions of Retaining Walls

3. Forces Acting on Retaining Walls

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

- **a)** Active and passive earth pressures
- **b)** Dead weight including the weight of the wall and portion of soil mass that is considered to act on the retaining structure
- **c)** Surcharge including live loads, if any
- **d)** Water pressure, if any
- **e)** Contact pressure under the base of the structure

The active and the passive earth pressures are calculated using the classical theories of Rankin and Coulomb. The distribution of the contact pressure under the base of the retaining wall is assumed to be planar and hence the usual flexural formula is used. The stability of the retaining wall is checked for sliding and overturning and deep foundation failure. The factor of safety against sliding, overturning and deep foundation failure is normally fixed in accordance with prevailing Building Codes. However, in all cases a minimum factor of safety of 1.5 should be maintained.

4. Procedures for the Design of Retaining Walls

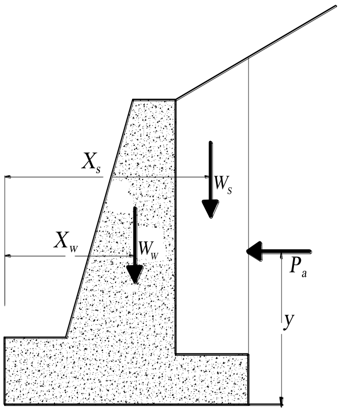
For the complete analysis of retaining walls it is common to follow the following steps:

1) Select height, shape and type of retaining wall according to field requirements and tentative dimensions
2) Compute all the vertical and horizontal loads acting on the wall (like weight of the wall, weight of soil above the wall, active and passive earth pressures, water pressures, etc)
3) Check stability of the wall (like sliding, overturning, bearing capacity, deep foundation failure, settlement etc)
4) Structural design: for gravity walls the above steps are sufficient but for cantilever retaining wall, in addition to stability check, the stem, the heel and the toe should be designed structurally for shear and flexure.

5. Stability Check

1. Overturning Stability

Considering the wall shown,



- Acting moment $M_a = P_a\ y$
- Resisting moment $M_r = W_s\ x_s + W_w\ x_w$
- Factor of safety:

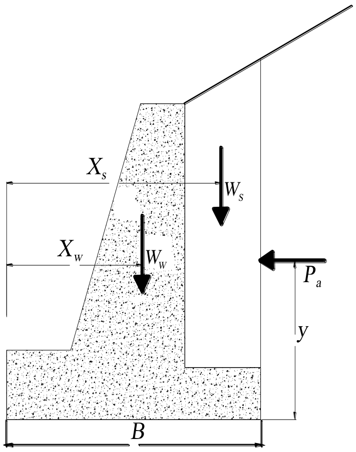
$$FS = \frac{M_r}{M_a} \geq 1.5$$

If FS < 1.5, the design ***shall be revised***

- The effect of passive resistance shall be neglected.

2. Sliding Stability

Considering the wall shown,



- Horizontal acting force is: $H_a = P_a$
- Horizontal resisting force is :

$$H_r = (\sigma \tan\phi_b)B + C_a B$$
$$= V \tan\phi_b + C_a B$$

where: ϕ_b =0.5 ϕ to 2/3 ϕ and C_a =0.5C to 0.7C

ϕ = angle of internal friction of the foundation soil

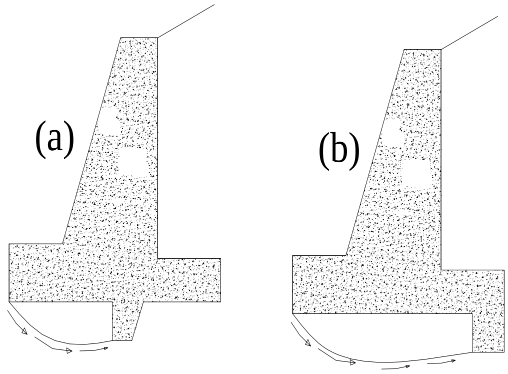
C = cohesion of the foundation soil

- Factor of safety:

$$FS = \frac{H_r}{H_a} \geq 1.5$$

- The effect of passive resistance shall be neglected unless it is very significant.

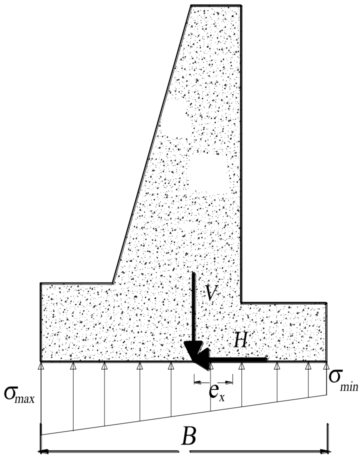
In some cases factor of safety of 1.5 may not be found. To increase the sliding resistance, either the base slab width may be increased or key may be provided which ever is economical. There are different opinions on the location of the base key. However, it is possible to mobilize more sliding resistance when the base key is on the back fill side.



The advantage of opinion (a) is that one can extend the reinforcement of the stem in to the key.

3. Bearing Capacity

The vertical pressure as transmitted to the soil by the base slab should be checked against the bearing capacity of the soil.



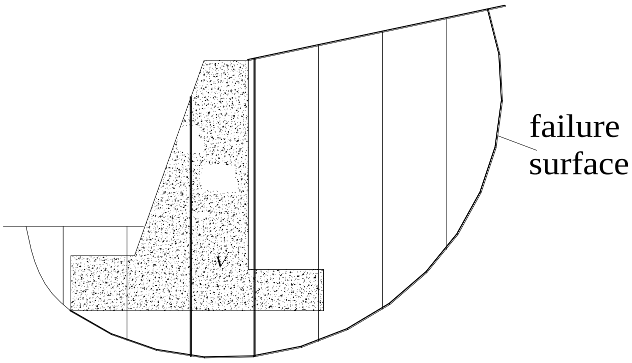
$$\sigma_{max} = \frac{V}{B*1}\left(1+\frac{6e_b}{b}\right) \le \sigma_{all}$$

$$\sigma_{min} = \frac{V}{B*1}\left(1-\frac{6e_b}{b}\right) \ge 0$$

$\sigma_{min} = \ge 0 \Rightarrow$ the load should be within the middle $\frac{1}{3}$rd

4. Deep Foundation Failure

In addition to the three types of possible failures for retaining walls discussed previously, deep shear failure could also occur if there is weak soil deposit within a depth of 1.5h below the base of the foundation. Therefore, it is necessary to check deep foundations failure as slope stability analysis.



- The critical slip surface is obtained by trial like in slope stability analysis
- The FS $\geq$ 1.5

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