Foundation Engineering (Course outline)

- Chapter -1: Introduction (1 hr. , 2 marks)
- Chapter -2: Soil exploration (6 hrs., 12 marks)
- C hapter-3: Lateral earth pressure theories and retaining walls (10 hrs., 16 marks)
- Chapter-4: Arching in soil and braced cuts (3hrs., 4 marks)
- Chapter-5: Flexible retaining structures and coffer dams (3hrs., 4 marks)
- Chapter -6: Bearing capacity and settlement of shallow foundations (6 hrs, 12 marks)
- Chapter -7: Mat foundations (3 hrs., 6 marks)
- Chapter- 8: Pile foundations (6 hrs., 12 marks)
- Chapter -9: Well foundations (4 hrs., 8 marks)
- Chapter -10: Foundation soil improvements (3 hrs., 4 marks)

References:

- Foundation analysis and design, J.E. Bowels, Mc Graw Hill publications, 5th edition
- Principles of foundation engineering, B. M. Das, Thomson's 5 th edition
- Basic and applied soil mechanics, Gopal Ranjan,A.S.R. Rao, New age international publications, second edition
- Soil mechanics and foundation engineering, K. R. Arora, Standard publishers, edition of 1997
- A text book of soil mechanics and foundation engineering, V.N.S. Murthy, CBS Publishers, 4 th edition
- A text book of foundation engineering: R.K. Poudel,Ramesh Neupane , New age international publications, 2 nd edition
- Pile foundation analysis and design, H.G. Polous, E.H. Davis, John Wiley and sons edition of 1980

CHAPTER – 1.0

Introduction

Foundation Engineering:

- In a broad sense, foundation engineering is an art of selecting, designing and constructing the elements that transfer the weight of structure to the underlying soil or rock.
- The role of engineer is to select the type of foundation, its design and supervision of construction.
- Before the engineer can design a foundation intelligently, he must have a reasonably accurate conception of the physical properties and the arrangement of the underlying materials. This requires detailed soil explorations.

The foundation engineer should possess the following information:-

- Knowledge of soil mechanics and background of theoretical analysis
- Composition of actual soil strata in the field.
- Necessary experience-precedents-what designs have worked well under what designs have worked well under what conditions-economic aspects

Engineering judgment or intuition - to find solutions to the problems.

Definition of foundation

The lowest part of a structure which acts as a medium to transfer loads coming from the superstructure to the underlying soil is termed as foundation. In other words , foundation is the lowest load-bearing part of a super structure, typically below ground level.

Requirements (Functional)

- A foundation must be properly located with respect to further influence which may affect the performance of foundation adversely.
- A properly designed foundation is one that transfers the structural load throughout the soil without overstressing of soil which can result in either excessive settlement or shear failure, both of which can damage the structure.

Types of Foundation:

- Shallow foundation : (D_f/B less or equal to 1)
- Deep Foundations : (D_f/B, greater than 4)

Where,

D_f = Depth of foundation
B = Width of foundation

- Shallow foundation located just below the lowest part of the superstructure they support; deep foundations extend considerably deeper in to earth.

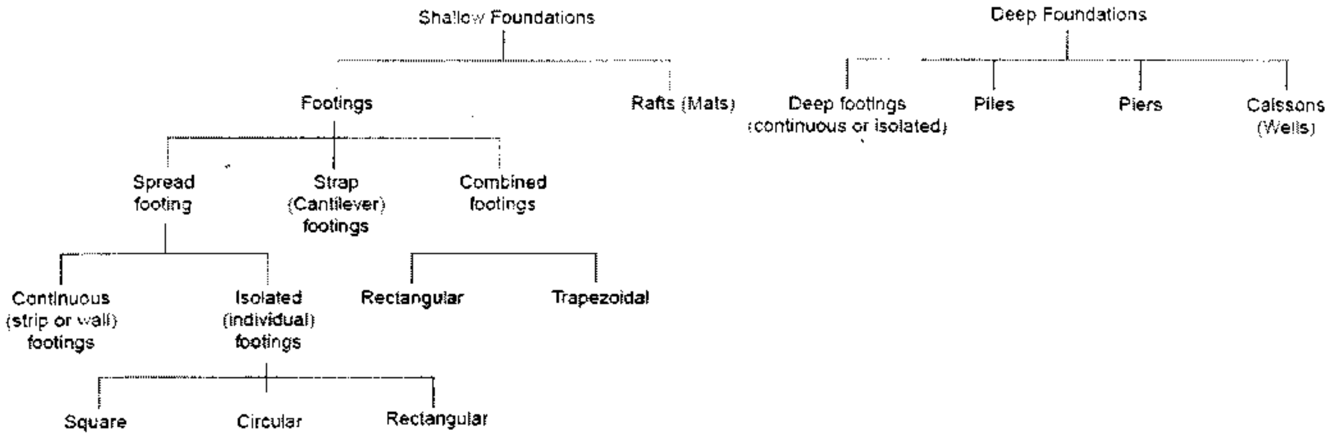
Types of shallow foundation:

1. Wall footing (strip or continuous footing)
2. Spread footing, (wall, isolated column footing)

3.Combined foundations

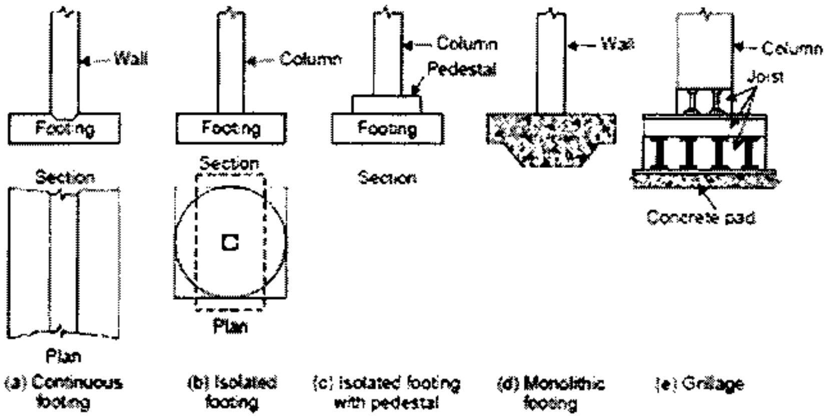
4.Strap or cantilever footing

5.Mat foundation or raft foundation



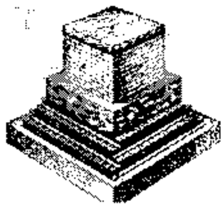
Spread foundation:

Spread footing foundation is basically a pad used to "spread out" loads from walls or columns over a sufficiently large area of foundation soil. These are constructed as close to the ground surface as possible consistent with the design requirements, and with factors such as frost penetration depth and possibility of soil erosion.



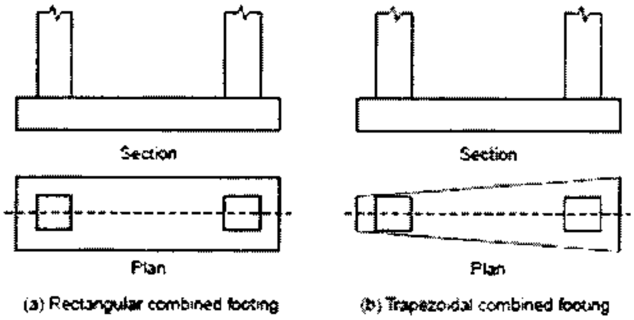
Common types of spread footings

Spread foundations are used to support an individual point load such as that due to a structural column. They may be circular, square or rectangular. They usually consist of a block or slab of uniform thickness, but they may be stepped or hunched if they are required to spread the load from a heavy column. Spread foundations are usually shallow, but deep pad foundations can also be used.

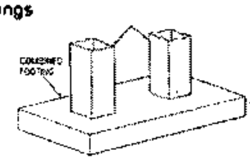


Combined footing:

A combined footing supports two or more columns in a row when the areas required for individual footings are such that they come very near each other. They are also preferred in situations of limited space on one side owing to the existence of the boundary line of private property.



Combined footings



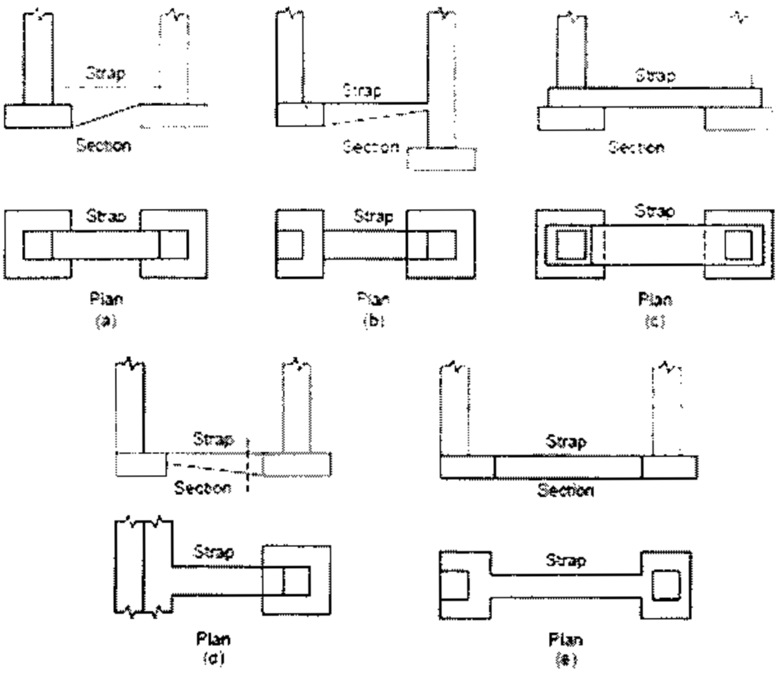
The plan shape of the footing may be rectangular or trapezoidal; the footing will then be called 'rectangular combined footing' or 'trapezoidal combined footing', as the case may be.

Strap or cantilever footing:

Strap footings

A 'strap footing' comprises two or more footings connected by a beam called 'strap'.

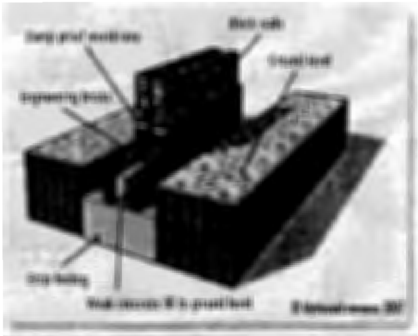
This is also called a 'cantilever footing' or 'pump-handle foundation'. This may be required when the footing of an exterior column cannot extend into an adjoining private property. Common types of strap beam arrangements are shown in aside.

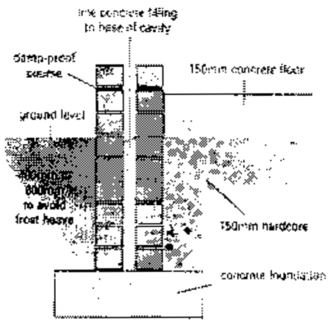


Common arrangement of strap beams in strap footings

Strip Foundation:

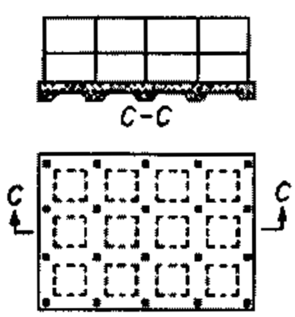
Strip foundations are used to support a line of loads, either due to a load-bearing wall, or if a line of columns need supporting where column positions are so close that individual pad foundations would be inappropriate.



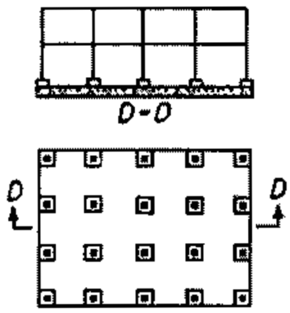


Mat Foundation:

A raft or mat foundation is a large footing, usually supporting walls as well as several columns in two or more rows. This is adopted when individual column footings would tend to be too close or tend to overlap; further, this is considered suitable when differential settlements arising out of footings on weak soils are to be minimised. A mat is required when the loads are heavy and the soil is very weak or highly compressible.



Two-way beam and slab



Flat plate with pedestals

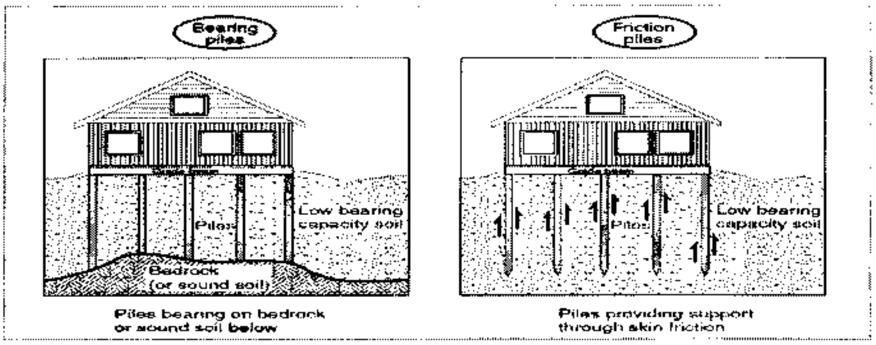
A mat foundation is a thick reinforced concrete slab which supports the entire load (from bearing walls and column loads) of a structure or a large portion of the structure. A mat foundation comes to be more economical than the individual footings when the total base area required for individual footings exceeds about one half of the area covered by the structure.

Types of Deep foundation:

1. Pile foundation
2. Pier foundation
3. Well or caissons

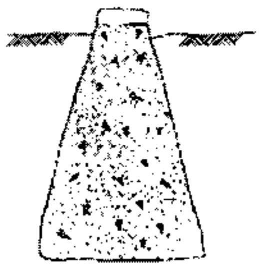
Pile foundation:

A pile is a long slender structural member made up of steel, concrete or wood which is used to transfer load to a deeper strata below ground surface. Pile foundation is a deep foundation (depth greater than 4 times width). It's basically a substructure that is supported by a group of piles which may be either driven in the ground or bored and then cast in situ. Pile foundations are used to transfer loads from soil of low bearing capacity to stiff layers of soil or rock with higher bearing capacity. Piles may be subjected to vertical or lateral loads or a combination of vertical and lateral loads.



Pier foundation:

Pier foundations are somewhat similar to pile foundations but are typically larger in area than piles. An opening is drilled to the desired depth and concrete is poured to make a pier foundation. Much distinction is now being lost between the pile foundation and pier foundation, adjectives such as 'driven', 'bored', or 'drilled', and 'precast' and 'cast-in-situ', being used to indicate the method of installation and construction.



Pier foundation

Usually, pier foundations are used for bridges.

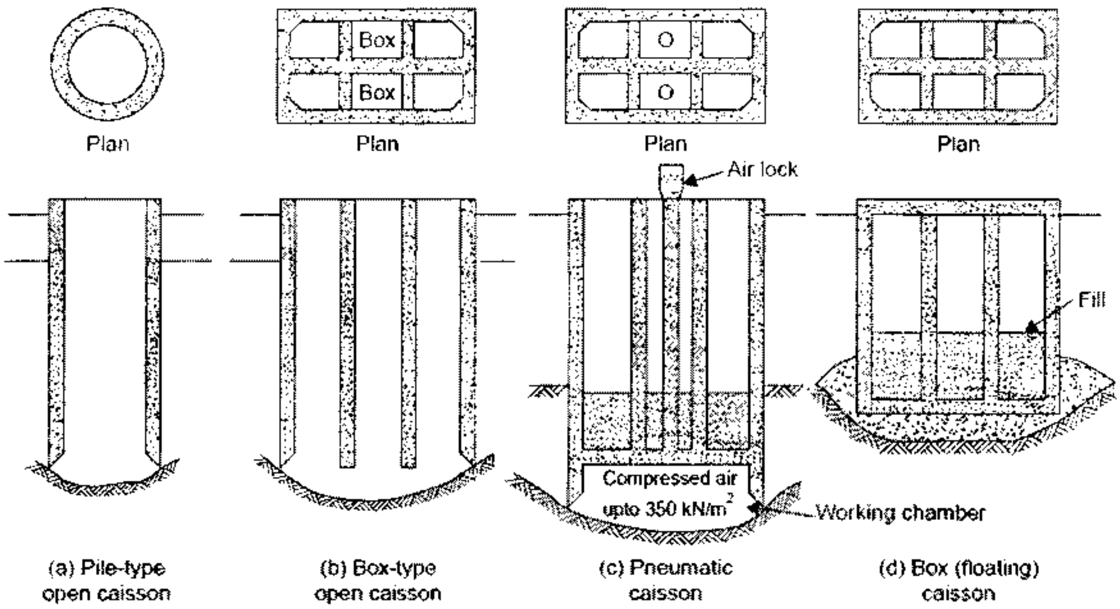
Well foundation :

A caisson is a structural box or chamber that is sunk into place or built in place by systematic excavation below the bottom. Caissons are classified as 'open' caissons, 'pneumatic' caissons, and 'box' or 'floating' caissons. Open caissons may be box-type of pile-type.

The top and bottom are open during installation for open caissons. The bottom may be finally sealed with concrete or may be anchored into rock. Pneumatic caisson is one in which compressed air is used to keep water from entering the working chamber, the top of the caisson is closed.

Excavation and concreting is facilitated to be carried out in the dry. The caisson is sunk deeper as the excavation proceeds and on reaching the final position, the working chamber is filled with concrete.

Box or floating caisson is one in which the bottom is closed. It is cast on land and towed to the site and launched in water, after the concrete has got cured. It is sunk into position by filling the inside with sand, gravel, concrete or water. False bottoms or temporary bases of timber are sometimes used for floating the caisson to the site. The various types of caissons are



Types of caissons (After Teng, 1976)

Loads on foundation:

- Dead Load
- Live load

- Wind loads
- Earthquake Forces

Depth and location of foundation depends on:

1.Zone of significant volume changes in soil
2. Adjacent structures and property lines
3. Ground water
4. Underground defects

Zone of significant volume changes in soil :

- Clays having high plasticity shrink and swell considerably up on drying and wetting respectively.
- Volume change is greatest near ground. Decreases with increasing depth. Volume changes usually insignificant below a depth from 1.5-3.0 m and does not occur below volume changes.

Factors affecting the foundation:

The type of foundation most appropriate for a given structure depends upon several factors:

1) the function of the structure and the loads it must carry,
2) the subsurface conditions,
3) the cost of the foundation in comparison with the cost of the superstructure.

These are the principal factors, although several other considerations may also enter into the picture.

Other factors:

- Constructability
- Service life
- Need of client
- Type of superstructure
- Environmental considerations
- Margin of safety.
- Risk level

Choice of Foundation type and Preliminary Selection

The following are the essential steps involved in the final choice of the type of foundation:

1. Information regarding the nature of the superstructure and the probable loading is required, at least in a general way.
2. The approximate subsurface conditions or soil profile is to be ascertained.
3. Each of the customary types of foundation is considered briefly to judge whether it is suitable under the existing conditions from the point of view of the criteria for stability—bearing capacity and settlement. The obviously unsuitable types may be eliminated, thus narrowing down the choice.
4. More detailed studies, including tentative designs, of the more promising types are made in the next phase.
5. Final selection of the type of foundation is made based on the cost—the most acceptable compromise between cost and performance.

CHAPTER : 2

Site Exploration / Investigation

Introduction:

The process of determining the layers of natural soil deposits that will underlie a proposed structure and their physical properties is generally referred to as site investigation.

Purpose of exploration program:

- Determining the nature of soil at the site and its stratification
- Obtaining disturbed and undisturbed soil samples for visual identification and appropriate laboratory tests
- Determining the depth and nature of bedrock, if and when encountered
- Performing some in situ field tests, such as permeability tests, vaneshear tests, and standard penetration tests
- Observing drainage conditions from and into the site
- Assessing any special construction problems with respect to the existing structure(s) nearby
- Determining the position of the water table
- The principal properties of interest will be the strength, deformation, and hydraulic characteristics.
- The program should be planned so that the maximum amount of information can be obtained at minimum cost.

Objectives of site investigation

- To determine sequence, thickness and lateral extent of soil strata or bedrock.
- To identify the ground water conditions
- To obtain representative soil samples of soil or rocks for conducting lab tests and to determine relevant soil properties.
- To conduct in-situ tests for getting appropriate soil characteristics.
- To explore source of construction materials.
- Information to determine the type of foundation required (shallow or deep) for selection of the type and the depth of foundation suitable for a given structure.
- Information to allow the geotechnical consultant to make a recommendation on the allowable load capacity of the foundation.
- Sufficient data/laboratory tests to make settlement predictions.
- Location of the groundwater table (or determination of whether it is in the construction zone). For certain projects, groundwater table fluctuations may be required. These can require installation of piezometers and monitoring of the water level in them over a period of time.
- Information so that the identification and solution of construction problems (sheeting and dewatering or rock excavation) can be made.
- Identification of potential problems (settlements, existing damage, etc.) concerning adjacent property.
- Identification of environmental problems and their solution.

The several exploration methods for sample recovery*

Method	Depths	Applicability
Disturbed samples taken		
Auger boring†	Depends on equipment and time available, practical depths being up to about 35 m	All soils. Some difficulty may be encountered in gravelly soils. Rock requires special bits, and wash boring is not applicable. *Penetration testing* is used in conjunction with these methods, and disturbed samples are recovered in the split spoon. Penetration counts are usually taken at 1- to 1.5 m increments of depth
Rotary drilling Wash boring Percussion drilling	Depends on equipment, most equipment can drill to depths of 70 m or more	
Test pits and open cuts	As required, usually less than 6 m; use power equipment	All soils
Undisturbed samples taken		
Auger drilling, rotary drilling, percussion drilling, wash boring	Depends on equipment, as for disturbed sample recovery	Thin-walled tube samplers and various piston samplers are used to recover samples from holes advanced by these methods. Commonly, samples of 50- to 100-mm diameter can be recovered
Test pits	Same as for disturbed samples	Hand-trimmed samples. Careful trimming of sample should yield the least sample disturbance of any method

* Marine sampling methods not shown.
†.Most common method currently used.

Planning the Exploration Programme

1. **Assembly of all available information** on dimensions, column spacing, type and use of the structure, basement requirements, any special architectural considerations of the proposed building, and tentative location on the proposed site. Foundation regulations in the local building code should be consulted for any special requirements.

   For bridges the soil engineer should have access to type and span lengths as well as pier loadings and their tentative location.

2. **Reconnaissance of the area.**
   This may be in the form of a field trip to the site, which can reveal information on the type and behavior of adjacent structures such as cracks, noticeable sags, and possibly sticking doors and windows.

   Erosion in existing cuts (or ditches) may also be observed, but this information may be of limited use in the foundation analysis of buildings. For highways, however, runoff patterns, as well as soil stratification to the depth of the erosion or cut, may be observed. Rock outcrops may give an indication of the presence or the depth of bedrock.

The reconnaissance may also be in the form of a study of the various sources of information available, some of which include the following:

Geological maps-Either U.S. government or state geological survey maps. Agronomy maps. Published by the Department of Agriculture (U.S., state, or other governmental agency).

Aerial photographs-Investigator may require special training to interpret soil data, but the non specialist can easily recognize terrain features.

Water and/or oil well logs-Hydrological data. Data collected by the U.S. Corps of Engineers on stream flow data, tide elevations, and flood levels.

Soil manuals by state departments of transportation-State (or local) university publications. These are usually engineering experiment station publications. Information can be obtained from the state university if it is not known whether a state study has been undertaken and published.

3. **A preliminary site investigation.**

In this phase a few borings (one to about four) are made or a test pit is opened to establish in a general manner the stratification, types of soil to be expected, and possibly the location of the groundwater table. If the initial borings indicate that the upper soil is loose or highly compressible, one or more borings should be taken to rock or competent strata. This amount of exploration is usually the extent of the site investigation for small structures.

A feasibility exploration program should include enough site data and sample recovery to establish an approximate foundation design and identify the construction procedures. Certain construction procedures (sheeting, bracing, tiebacks, slurry walls, rock excavation, dewatering, etc.) can represent a very significant part of the foundation cost and should be identified as early as practical.

4. **A detailed site investigation.**

Where the preliminary site investigation has established the feasibility and overall project economics, a more detailed exploration program is undertaken. The preliminary borings and data are used as a basis for locating additional borings, which should be confirmatory in nature, and determining the additional samples required.

Note that if the soil is relatively uniformly stratified, a rather orderly spacing of borings at locations close to critical superstructure elements should be made (requires client furnish the necessary location data).

In the detailed program phase it is generally considered good practice to extend at least one boring to competent rock if the overlying soil is soft to medium stiff. This is particularly true if the structure is multiple-storied or requires settlement control.

Planning of subsurface exploration program:

Before site investigation engineer should have the preliminary data regarding:

- Size of the structure
- Height
- Depth of basement
- Spacing of columns and load bearing walls

- Load to be transmitted by the foundation
- Assembly of all available information on dimensions, column spacing, type and use of the structure, basement requirements, and any special architectural considerations of the proposed building. Foundation regulations in the local building code should be consulted for any special requirements. For bridges the soil engineer should have access to type and span lengths as well as pier loadings. This information will indicate any settlement limitations, and can be used to estimate foundation loads.
- Exploration program depends on type of structure to be built and variability of strata at the site.
- Extent of investigation depends on location of project (in built up area or new)
- For planning a detailed knowledge of soil engineering, experience and engineering judgment is required.
- Sometimes exploration program may change, e.g.- as variability in soil strata is found the extent of exploration is to be increased and vice versa.
- The cost of investigation varies from 0.05 to 0.2% of entire cost of structure and in unusual cases it may be even up to 1%.

Stages in subsurface exploration program:

The investigation is generally carried out in 3 stages.

1) Preliminary investigation
2) Detailed investigation
3) Investigation during construction

1. Preliminary investigation (for providing rough idea of site)

A). Fact finding survey (Topography, Geology, Aerial photograph, Seismicity, Hydrological and meteorological data, Building codes from available literatures, technical journals, published reports, geological map or topo map etc.)

B) Reconnaissance Survey (actual visit to site to verify data obtained from fact finding survey)

- Rapid depressions in the land indicate swallow holes.
- Leaning trees indicates creep of soil.
- Surface indication of ground water implies the presence of springs or marshy lands.

C) Trial boring

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

2. Detailed investigation

When primary investigation does not provide sufficient information then detailed investigation is required.

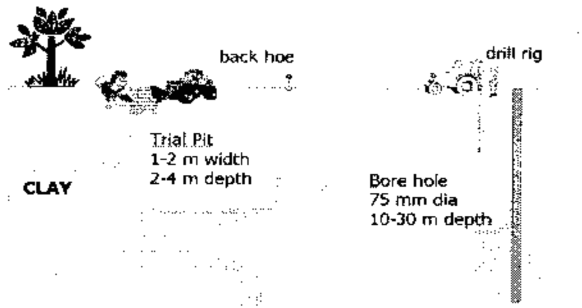
a) Accessible investigation (Digging of trial pits and test trenches for obtaining true and representative samples.)

b) Inaccessible investigation (Auger boring, Wash boring, Percussion drilling, Rotary drilling)

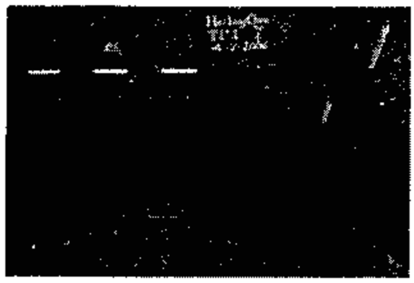
The various operations involved in detailed investigation are:

i) Boring
ii) Sampling: Disturbed samples / Undisturbed samples
iii) Testing
- Laboratory test and In-situ testing

Test pits:



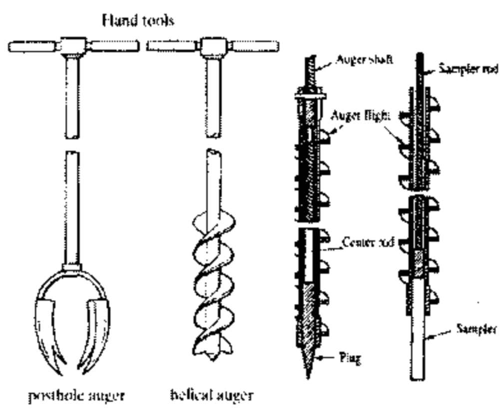
- Permits visual inspection of subsurface conditions in natural state.
- Max. depth limited to 18 -20 feet.
- Especially useful for gravelly soil where boreholes may be difficult. Sampling/testing done on



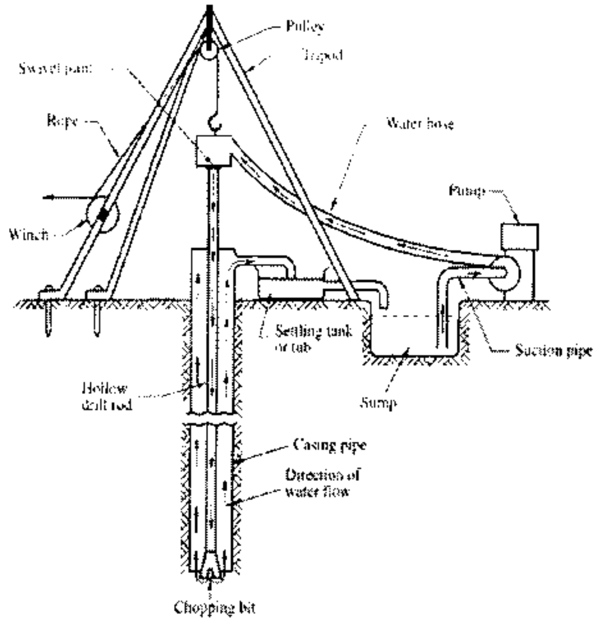
Boring

Method of boring Auger boring:

- Simplest method of exploration and sampling.
- Power driven or hand operated.
- Max. depth 10 m
- Suitable in all soils above GWT but only in cohesive soil below GWT
- Hollow stem augers used for sampling or conducting Standard Penetration Tests.

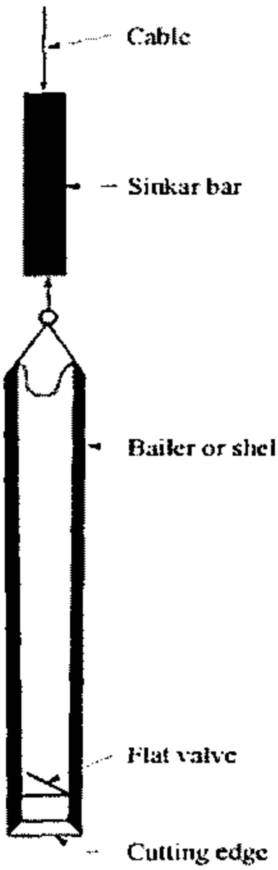


Wash Boring:



- ❖ A casing is driven with a drop hammer. A hollow drill rod with chopping bit is inserted inside the casing.
- ❖ Soil is loosened and removed from the borehole using water or a drilling mud jetted under pressure.
- ❖ The water is jetted in the hole through the bottom of a wash pipe and leaves the hole along with the loose soil, from the annual space between the hole and wash pipe.

  The water reaches the ground level where the soil in suspension is allowed to settle and mud is re-circulated.



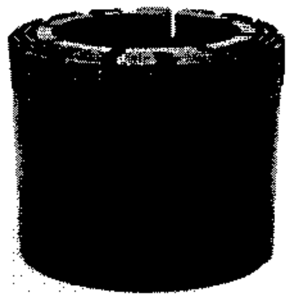
Percussion drilling:

- ❖ Grinding the soil by repeated lifting and dropping of heavy chisels or drilling bits.
- ❖ Water is added to form slurry of cuttings.
- ❖ Slurry removed by bailers or pumps.
- ❖ In general, a machine used to drill holes is called a drill rig (generally power driven, but may be hand driven).

A winch is provided to raise and lower the drilling tools into the hole.

Rock Core Drilling:

- Done with either tungsten carbide or diamond core bits
- Use a double or triple tube core barrel when sampling weathered or fractured rock
- Used to determine Rock Quality Designation.



Diamond coring bit

Sampling:

Extraction of samples from sampling instruments.

- Types of samples:

a) Disturbed samples b) Undisturbed samples

- Disturbed samples are those in which natural structure of soil gets disturbed during sampling but samples represent the composition and mineral content of the soil. Disturbed samples can be used for the determination of index properties (grain size plasticity characters, specific gravity)
- Undisturbed samples may be defined as those in which the material has been subjected to minimum disturbance so that the samples are suitable for strength tests and consolidation tests. In undisturbed samples natural structure of soil and water content is retained. Tube samples and chunk samples are considered to fall in this category.
- It is impossible to get truly undisturbed sample. Even the removal of soil from ground produces a change in stress and causes some disturbance.
- Least disturbance is obtained on a sample carefully exposed and trimmed from a block sample obtained from pit.
- Undisturbed samples are used for determining engineering properties of soil such as compressibility, shear strength and permeability.
- The smaller is the disturbance more reliable the sample is

Disturbed samples may be further subdivided as:

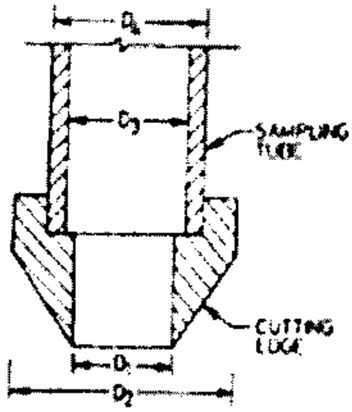
a) Representative samples b) Non-representative samples

- Representative samples contain all the mineral constituents of the soil, but the structure of the soil may be significantly disturbed. The water content may also have changed. They are suitable for identification and for the determination of certain physical properties such as Atterberg limits, classification test and grain specific gravity.

- Non-representative samples consist of mixture of materials from various soil or rock strata or are samples from which some mineral constituents have been lost or got mixed up. Soil samples obtained from auger borings and wash borings are non-representative samples. These are suitable only for providing qualitative information such as major changes in subsurface strata.

Requirements of a sampler

- Cutting edge properly machined and taper should not exceed 20 degree.
- The tube should be seamless and should be round without any protrusions.



Area ratio:

A_r = Maximum cross sectional area of cutting edge *100 %
area of soil sample

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

D_1 = Inner diameter of cutting edge
D_2 = Outer diameter of cutting edge

Thicker the wall, greater the disturbance. Obtaining good quality of undisturbed sample, the area ratio should be 10% or less.

Ar may be as high as 30% for a thick wall sampler like split spoon and may be as low as 6 to 9% for thin wall samplers like shelby tubes.

Inside clearance ratio

$$C_i = \frac{D_3 - D_1}{D_1} \times 100$$

Inside clearance allows elastic expansion of the sample when it enters the tube which helps to reduce frictional drag on the soil sample from the wall of the tube.

For undisturbed soil sample the inside clearance ratio should be between 0.5 to 3%.

Outside clearance ratio:

$$C_O = \frac{D_2 - D_4}{D_4} \times 100$$

Outside clearance ratio fascinates in withdrawal of sampling tube.

For reducing driving force, the outside clearance ratio should be between 0 to 2% for cohesive soils and preferably zero for cohesionless soil.

Recovery ratio:

$$R_r = \frac{Length\ of\ core\ recoverd}{Length\ of\ core\ drilled\ in\ a\ given\ run}$$

It is related to the quality of soil or rock encountered in boring, but it is also influenced by the drilling technique and the type and size of core barrel used.

A recovery ratio greater than 1 indicates the loosening of the sample from rearrangement of stones, roots or removal of pre load.

Sampling procedure:

Before taking sample the bottom of the borehole should be cleaned.For sampling in borehole above water table, borehole should be kept in dry condition. Sampler should be pushed into soil as far as practicable, driving is done only when it is not possible to push further. The pushing and driving should be stopped 50 to 80mm before reaching full length.Sample should be separated from the soil by turning the sampler one or two rounds.

Types of samplers

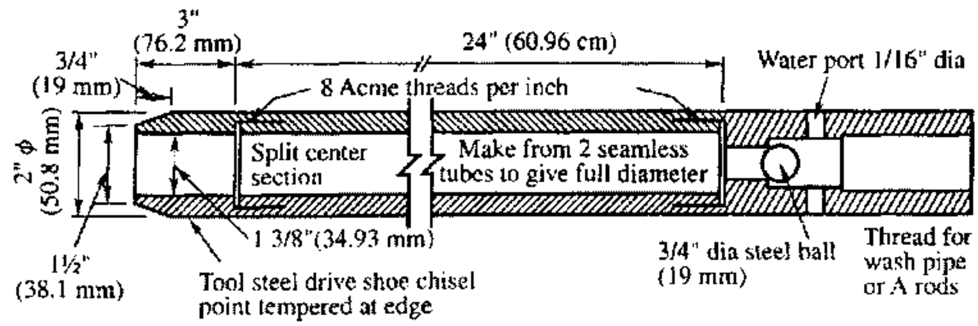
- Split spoon sampler (Split barrel sampler)
- Scraper bucket
- Thin walled sampler (Shelby tubes)
- Piston sampler
- Denison sampler
- Modified California sampler
- Continuous soil sampler
- Other types of soil samplers

Split spoon sampler

The split spoon sampler is most commonly used sampler used to obtain disturbed samples in all types of soils.

It mainly consists of 3 parts i) Driving shoe, made of tool- steel, about 75mm long ii) steel tube about 450mm long, splits into two halves iii) coupling at the top about 150mm long. The inside and outside diameter of split tube are 38mm and 50mm.

It is used in conjunction with standard penetration test (SPT), attached to drill rod.



Split barrel sampler for standard penetration test

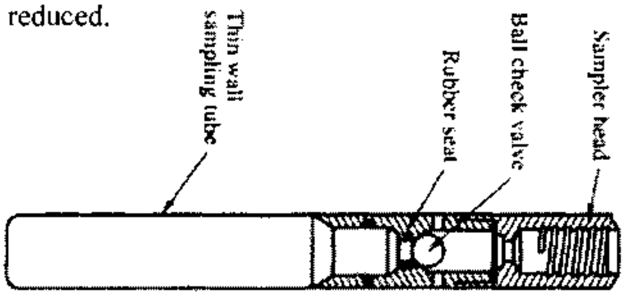
Scraper bucket sampler

- In sandy deposits containing pebbles, it is not possible to obtain samples by split spoon sampler.
- A scraper bucket sampler consists of a driving point which is attached to its bottom end and a vertical slit in the upper portion of the sampler.
- As the sampler is rotated, the scrapings of the soil enter the sampler through slit. When the sampler is filled with the scrapings, it is lifted and sample is withdrawn.
- The samples obtained by this sampler are quiet disturbed but representative.

Thin walled sampler (Shelby)

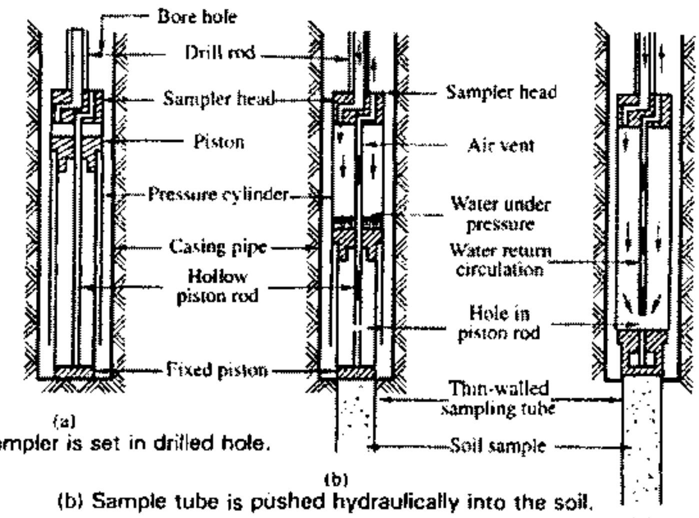
- The thin walled tube (Shelby) sampler is commonly used to obtain relatively undisturbed samples of cohesive soils for strength and consolidation testing.
- They are made up of seamless steel with bottom sharpened and beveled and then pushed into the soil.
- The sampler is commonly used has 76mm outside and 73mm inside dia. And area ratio is less than 15% and inside clearance between 0.5% to 3%.

Larger dia. Sampler tubes are often used where higher quality samples are required and the sampling disturbance must be reduced.



Piston sampler

- A piston sampler is basically a thin wall tube sampler with piston, piston rod, and a modified soil sampler head.
- Piston keeps the lower end of the sampling tube closed when the sampler is lowered to the bottom of the hole.
- The presence of the piston prevents rapid squeezing of the soft soils into the tube and reduces the disturbance of the sample.



(a) Sampler is set in drilled hole.
(b) Sample tube is pushed hydraulically into the soil.
(c) Pressure is released through hole in piston rod.
Osterberg Piston Sampler

- A vacuum is created on the top of the sample, which helps in retaining the sample.
- Piston samplers are used for getting undisturbed soil samples from soft and sensitive clays.
- From this sampler 100% recovery of sample is enhanced.

Block Sampling

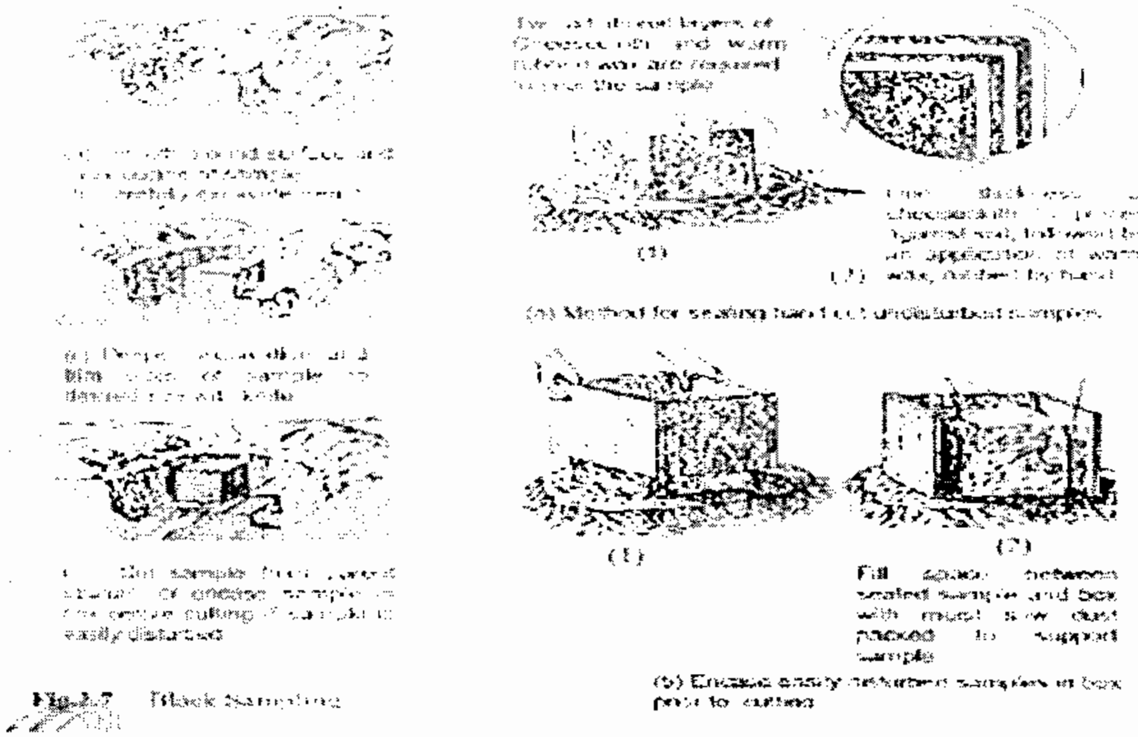


Figure : Block sampling

Sampler	*Disturbed / Undisturbed*	*Appropriate Soil Types*	*Method of Penetration*	*% Use in Practice*
Split-Barrel (Split Spoon)	Disturbed	Sands, silts, clays	Hammer driven	85
Thin-Walled Shelby Tube	Undisturbed	Clays, silts, fine-grained soils, clayey sands	Mechanically Pushed	6
Continuous Push	Partially Undisturbed	Sands, silts, & clays	Hydraulic push with plastic lining	4
Piston	Undisturbed	Silts and clays	Hydraulic Push	1
Pitcher	Undisturbed	Stiff to hard clay, silt, sand, partially weather rock, and frozen or resin impregnated granular soil	Rotation and hydraulic pressure	<1
Denison	Undisturbed	Stiff to hard clay, silt, sand and partially weather rock	Rotation and hydraulic pressure	<1
Modified California	Disturbed	Sands, silts, clays, and gravels	Hammer driven (large split spoon)	<1
Continuous Auger	Disturbed	Cohesive soils	Drilling w. Hollow Stem Augers	<1
Bulk	Disturbed	Gravels, Sands, Silts, Clays	Hand tools, bucket augering	<1
Block	Undisturbed	Cohesive soils and frozen or resin impregnated granular soil	Hand tools	1

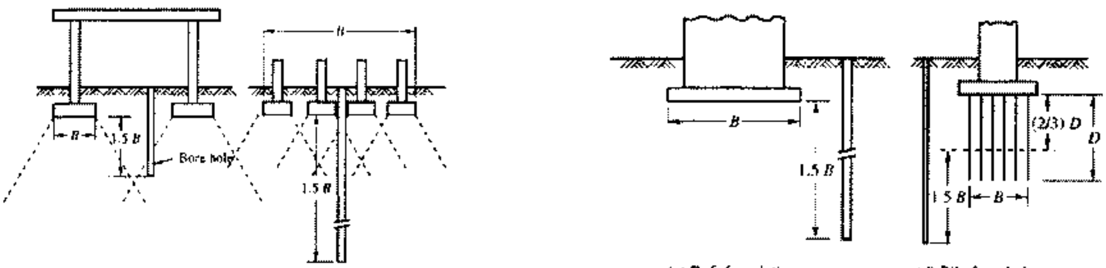
Vertical and lateral extent of boring:

Number of boreholes:

- ❖ As no. of boreholes increases we get much reliable site information but it increases cost of site investigation.
- ❖ For all small structures at least two or preferably 3 boreholes should be sunk so that true dip of strata could be established.
- ❖ For small and less important building 1 borehole or test pit at centre is sufficient. For buildings on area greater than 0.4 hectare(4000 sq m) 4 boreholes at corner and 1 at centre is required. For multistoried building 4 boreholes at corner and other at important locations is required.
- ❖ For bridge pier/ Transmission tower 1-2 boreholes at location are required.

Depth of boring

- ❖ The depth of boring should be equal to the significant depth (Depth upto which superimposed loads can produce significant settlement and shear stresses).
- ❖ Significant depth is generally taken as the depth at which vertical stress is 20% of the load intensity and usually depth of exploration should be about 1.5 times width of footing below the base of footing.
- ❖ Hence depth of boring should be 1.5 times width of square footing.



(a) Footings placed far apart (b) Footings placed at closed intervals (c) Raft foundation (d) Pile foundation

- ❖ For pile foundations the depth of exploration is 1.5 times width of pile group from tip of piles.
- ❖ For friction pile group the depth of boring should be 1.5 times width of pile group below lower one third length of pile.

When foundation is founded on bed rock then it should not be mistaken with large boulders. So a minimum of 3m core boring should be done to conform it as rock.

For multistoried building the depth of boring can be estimated by following formula given by Sower and Sower (9170).

$$D = CS^{0.7}$$

- ❖ D = Depth of boring (m)
- ❖ C = 3 (for light steel building and narrow concrete building.)
- ❖ =6 (for heavy steel buildings and wide concrete buildings)
- ❖ S= No. of storey
- ❖ For exploration of deep excavation the depth of exploration should be 1.5 times depth of exploration. For road cuts it should be equal to width of cut.
- ❖ For road fills minimum depth of exploration should be 2m below ground or height of fill whichever is greater.
- ❖ For gravity dams minimum depth of exploration is twice the height of dam.

Lateral extent of explorations:

- Lateral extent or spacing of boring depends upon the variability of soil strata in the horizontal direction.
- For small and less important building 1 borehole or the test pit at the centre is sufficient but for compact building on 0.4 hectare, there should be 5 boreholes 1 at the centre and 4 at the corners.
- For multistoried buildings boring should be done at the 4 corners and other important locations.
- Lateral extent or spacing of boring depends upon the variability of soil strata in the horizontal direction.
- For small and less important building 1 borehole or the test pit at the centre is sufficient but for compact building on 0.4 hectare, there should be 5 boreholes 1 at the centre and 4 at the corners.
- For multistoried buildings boring should be done at the 4 corners and other important locations.
- The spacing is usually kept between 10m to 30m depending on subsurface conditions.
- For highways the spacing varies between 150 to 300m along the proposed centre line. For erratic subsurface it may be even upto 30m.
- For concrete dams, the spacing of bore holes generally varies between 40 to 80m.

Table 18.1 Spacing of Borings

Project	Boring spacings	
	m	ft
One-story buildings	25–30	75–100
Multistory buildings	15–25	50–75
Highways	250–300	750–1000
Earth dams	25–50	75–150
Residential subdivision planning	60–100	200–300

The American Society of Civil Engineers (1972) recommended the following rules of thumb for estimating the boring depths for buildings.

1. Estimate the variation of the net effective stress increase, $\Delta\sigma'$, that will result from the construction of the proposed structure with depth. This variation can be estimated by using the principles outlined in Chapter 10. Determine the depth D_1 at which the value of $\Delta\sigma'$ is equal to 10% of the average load per unit area of the structure.
2. Plot the variation of the effective vertical stress, σ'_o, in the soil layer with depth. Compare this with the net stress increase variation, $\Delta\sigma'$, with depth as determined in Step 1. Determine the depth D_2 at which $\Delta\sigma' = 0.05\sigma'_o$.
3. The smaller of the two depths, D_1 and D_2, is the approximate minimum depth of the boring.

Guidelines for spacing and number of boreholes, depth

30 A Text Book of Foundation Engineering

Table 2.2 Guideline for Minimum no. of Boreholes and Spacing

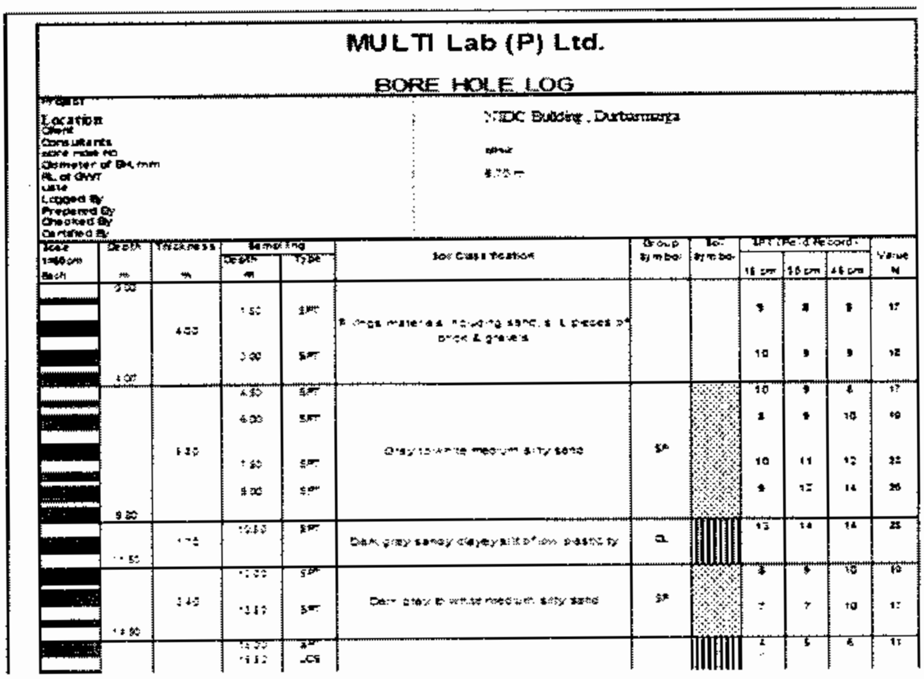
S. No.	Type of job	Type of soil in horizontal extent: Uniform	Average	Erratic	Minimum no. of boring
		Spacing, m			
1	1 or 2 storied structure	60	30	15	3
2	Multi - storied structure	45	30	15	4
3	Bridge piers, abutments		30	15	1 - 2 per foundation
4	Transmission tower		30	15	1 - 2 per foundation
5	Highway and airports	100	150	100	
6	Borrow pits	300 - 150	150 - 100	30 - 15	

Table 2.3 Depth of Boring According as Structure Type

S. No.	Structure type	Design consideration	Depth of boring	Remarks
1	Building	Settlement, total and differential	10 m minimum, or up to depth where the increase in stress due to structure is 10 % of stress imposed by structure	D = depth of exploration [Fig. 2.14 (a)]
2	Retaining walls	Bearing capacity and settlement	D = 0.5 to 2H	D = depth of hole and H = height of wall [Fig. 2.14 (b)]
3	Terraces and fill	Bearing capacity and settlement	D = 1.25 L for terraces and 0.5 L for fill	D = depth of hole and L = projected slope length [Fig. 2.14 (c)]
4	Deep cuts	Stability of slopes	D = 0.75 B to 1.0 B	D = depth of hole and B = bottom width of [Fig. 2.15 (a)]
5	Earth dam and levee	Bearing capacity	D = L	D = depth of hole and 2L = bottom width of dam [Fig. 2.15 (b)]
6	Concrete dams	Bearing capacity	D = 1.5 to 2H	D = depth of hole and H = height of dam
7	Highways, railway and airports	General stability and drainage conditions	1 to 2 m for light loads and 2 to 3.5 m for heavy loads	
8	Tunnels	Stability of materials and pressure exerted against tunnel lining	D = B	D = depth of hole and B = gross width of tunnel

Borehole logs

- ❖ Bore hole log is made for making a detailed record of various boring operations involved and other in situ tests carried out during boring operations.
- ❖ The soil is classified based upon the visual examinations of the collected disturbed samples.



Example of bore hole log

Site investigation report:

- At the end of all subsoil exploration programs, the soil and/or rock specimens collected from the field are subjected to visual observation and appropriate laboratory testing. After the compilation of all of the required information, a soil exploration report is prepared for the use of the design office and for reference during future construction work. Although the details and sequence of information in the report may vary to some degree is depending on the structure under consideration and the person compiling the report.

Subsoil Exploration Report:

1. A description of the scope of the investigation
2. A description of the proposed structure for which the subsoil exploration has been conducted
3. A description of the location of the site, including any structures nearby, drainage conditions, the nature of vegetation on the site and surrounding it, and any other features unique to the site
4. A description of the geological setting of the site
5. Details of the field exploration - that is, number of borings, depths of borings, types of borings involved, and so on
6. A general description of the subsoil conditions, as determined from soil specimens and from related laboratory tests, standard penetration resistance and cone penetration resistance, and soon
7. A description of the water-table conditions
8. Re commendations regarding the foundation, including the type of foundation recommended, the allowable hearing pressure, and any special construction procedure that may he needed; alternative foundation design procedures should also be discussed in this portion of the report.
9. Conclusions and limitations of the investigations.

The following graphical presentations should he attached to the report:

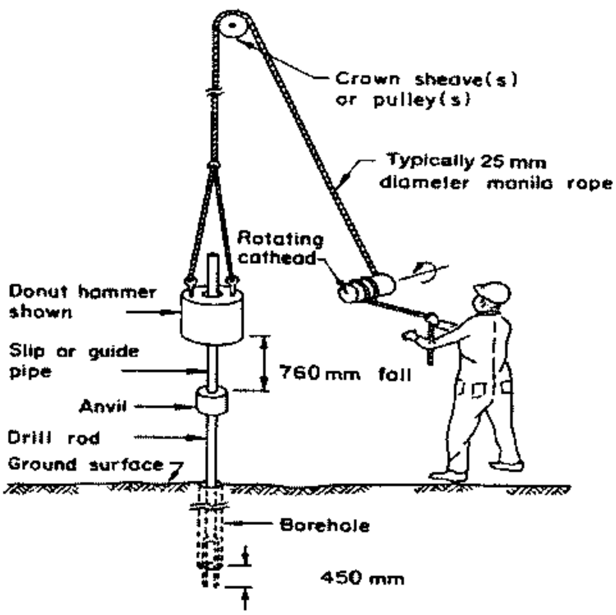
1. A site location map
2. A plan view of the location of the borings with respect to the proposed structures and those nearby
3. Boring logs
4. Laboratory test results
5. Other special graphical presentations

FIELD TESTS:

1. **Penetration tests**
   a. Standard Penetration Test, SPT
   b. Static Cone Penetrometer test (Push Cone Penetrometer Test, PCPT or DSCPT)
   c. Dynamic cone penetration test
2. **Borehole tests**
   a. Pressuremeter test
   b. Dialatometer test
   c. Field vane shear test, Torvane

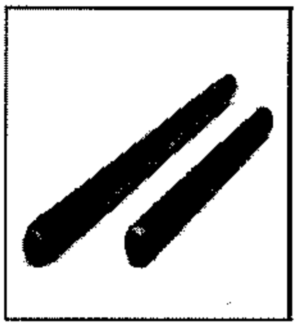
Standard Penetration Test (SPT):

- ❖ Most commonly Used In-Situ Test Specially for cohesion less soils which can't be easily sampled.
- ❖ Test is conducted in a bore hole using a standard split spoon sampler.
- ❖ When bore hole has been drilled to the desired depth, the drilling tools are removed and sampler is lowered to the bottom of the hole.
- ❖ In general SPT samples are taken in both granular and cohesive soils, and thin walled tube samples are taken in cohesive soils
- ❖ The sampling interval varies between individual projects and between regions.
- ❖ A common practice is to obtain split barrel samples at 0.75 m intervals in the upper 3m and 1.5 m interval below 3m.
- ❖ In some instances, a greater sample interval , often 3m, is allowed below depths of 30m
- ❖ The test is very useful to determine angle of internal friction, cohesion.

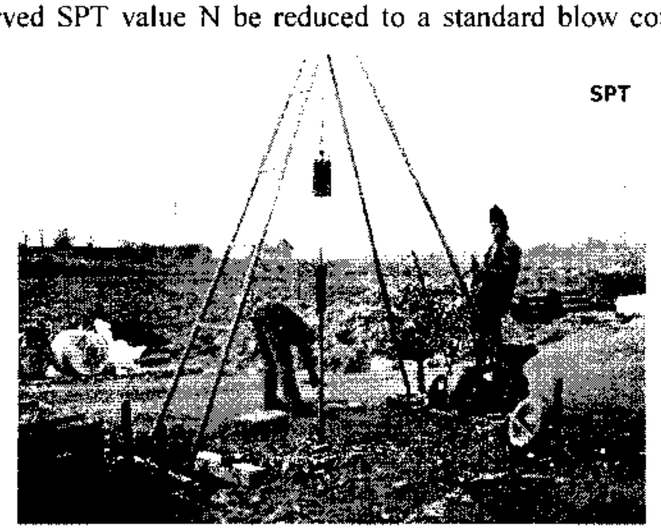


- ❖ The theoretical input energy may be expressed as $E_{in} = Wh$
- ❖ where W = weight or mass of the hammer (63.5 kg)
- ❖ h = height of free fall (760mm)
- ❖ Investigation has revealed that the actual energy transferred to the driving head and then to the sampler ranged from about 30 to 80 percent.

$$R_{e1} = \frac{\text{Actual hammer energy to sampler, } E_a}{\text{Input energy, } E_{in}}$$



- ❖ It has been suggested that the SPT be standardized to some energy ratio , Bowles (1996) suggests that the observed SPT value N be reduced to a standard blow count corresponding to 70 percent of standard energy. Terzaghi, et al., (1996) suggest 60 percent.



Refusal of the SPT test:

1. When the number of blows for 150 mm penetration exceeds 50 the test is discontinued.

Corrections:

❖ Then SPT value has to be corrected for dilatancy correction and Overburden Correction.

Dilatancy means phenomenon exhibited by some fluids, solutions , and gel in which they become more viscous when stress is applied.

Dilatancy Correction

❖ Silty fine sands and fine sands below the water table develop pore pressure which is not easily dissipated. The pore pressure increases the resistance of the soil and hence the penetration Number.

Terzaghi and Peck

❖ They recommended following correction in case of silty fine sands when $N_{obs} > 15$

❖ The corrected penetration Number

$$N' = 15 + 1/2\,(N_o - 15)$$

where, N_o = observed SPT value

N' = corrected value for dilatation effect

Bazara (1967), gives the following equation for correcting N

$$N' = 0.6\,N_o$$

where N_o is greater than 15.

Over Burden Pressure Correction:

❖ With the increase in confining pressure in sands N values are increased considerably as compared to shallow depth.

❖ So, N value has been underestimated at shallow depth and overestimated at great depth.

The equation as developed by Gibbs and Hotlz (1957)

$$N = \frac{4N'}{0.01\,p_o' + 0.8} = C_N N'$$

where, N' = SPT value corrected for dilatancy,

p_o' = the effective overburden pressure in kPa,

C_N = correction factor.

The equation developed by Bazaraa (1967) based on effective overburden pressure

For $p_o' \leq 75$ kPa,

$$N = \frac{4N'}{0.04\,p_o' + 1} = C_N N'$$

For $p_o' > 75$ kPa,

$$N = \frac{4N'}{0.01\,p_o' + 3.25} = C_N N'$$

Considering the following points

1. **For $p_o' \leq 75$ kPa, $N > N'$.**
2. **For $p_o' > 75$ kPa, $N < N'$.**
3. **When N' indicates a relative density $D_r < 0.5$, no correction is required. C_N may be taken as equal to unity up to $D_r = 0.5$.**
4. **C_N is limited to 2.**

Peck et al (1974) proposed the following relations for P'_o>25kPa

$$N = 0.77\,N' \log_{10} \frac{2000}{p_o'} = C_N N'$$

where, p_o' is in kPa. This equation has the following characteristics:

1. $N \to \infty$ as $p_o' \to 0$.
2. $N = N'$ at $p_o' = 100$ kPa.
3. $N < N'$ for $p_o' > 100$ kPa.

Note: Some researchers empahasis on application overburden correction first and then dialatency correction.

OTHER CORRECTIONS:

1. Hammer Efficiency Correction, E_h

- Different types of hammers are in use for driving the drill rods. Two types are normally used in USA. They are (Bowles, 1996)
- 1. Donut with two turns of manila rope on the cathead with a hammer efficiency $E_h = 0.45$.
- 2. Safety with two turns of manila rope on the cathead with a hammer efficiency as follows:
- Rope-pulley or cathead = 0.7 to 0.8;
- Trip or automatic hammer = 0.8 to 1.0.

2. Drill Rod, Sampler and Borehole Corrections

Correction factors are used for correcting the effects of length of drill rods, use of split spoon sampler with or without liner, and size of bore holes. The various correction factors are (Bowles, 1996).

a) Drill rod length correction factor C_d

Length (m)	Correction factor (C_d)
> 10 m	1.0
4–10 m	0.85–0.95
< 4.0 m	0.75

b) Sampler correction factor, C_s

Without liner C_s = 1.00

With liner,

Dense sand, clay = 0.80

Loose sand = 0.90

c) Bore hole diameter correction factor, C_b

Bore hole diameter	Correction factor, C_b
60-120 mm	1.0
150 mm	1.05
200 mm	1.15

3 Correction Factor for Overburden Pressure in Granular Soils, C_N

The C_N as per Liao and Whitman (1986) is

$$C_N = \left[\frac{95.76}{p'_o}\right]^{1/2} \tag{9.5}$$

where, p'_o =effective overburden pressure in kN/m²

There are a number of empirical relations proposed for C_N. However, the most commonly used relationship is the one given by Eq. (9.5).

N_{cor} may be expressed as

$$N_{cor} = C_N N E_h C_d C_s C_b \tag{9.6}$$

N_{cor} is related to the standard energy ratio used by the designer. N_{cor} may be expressed as N_{70} or N_{60} according to the designer's choice.

In Eq (9.6) $C_N N$ is the corrected value for overburden pressure only. The value of C_N as per Eq. (9.5) is applicable for granular soils only, whereas $C_N = 1$ for cohesive soils for all depths.

Example

The observed standard penetration test value in a deposit of fully submerged sand was 45 at a depth of 6.5 m. The average effective unit weight of the soil is 9.69 KN /m³. The other data given are (hammer efficiency = 0.8, (b) drill rod length correction factor = 0.9, and (c) borehole correctionfactor = 1.05. Determine the corrected SPT value for standard energy (a) Res - 60 percent, and (b) Res - 70 percent.

Correlation of N with Engineering properties:

Terzaghi and Peck also give the following correlation between SPT value, D_r, and ϕ:

Table 18.4 Correlation between N, D_r and ϕ

S. No.	Condition	N	D_r	ϕ
1	Very loose	0 – 4	0 – 15%	Less than 28°
2.	Loose	4 – 10	15 – 35%	28° – 30°
3.	Medium	10 – 30	35 – 65%	30° – 36°
4.	Dense	30 – 50	65 – 85%	36° – 42°
5.	Very dense	Greater than 50	Greater than 85%	Greater than 42°

For clays the following data are given:

Table 18.5 Correlation between N and q_u

S. No.	Consistency	N	q_u (kN/m^2)
1	Very soft	0 – 2	Less than 25
2.	Soft	2 – 4	25 – 50
3	Medium	4 – 8	50 – 100
4.	Stiff	8 – 15	100 – 200
5.	Very stiff	15 – 30	200 – 400
6.	Hard	Greater than 30	Greater than 400

The correlation for clays is rather unreliable. Hence, vane shear test is recommended for more reliable information.

Factors affecting SPT value:

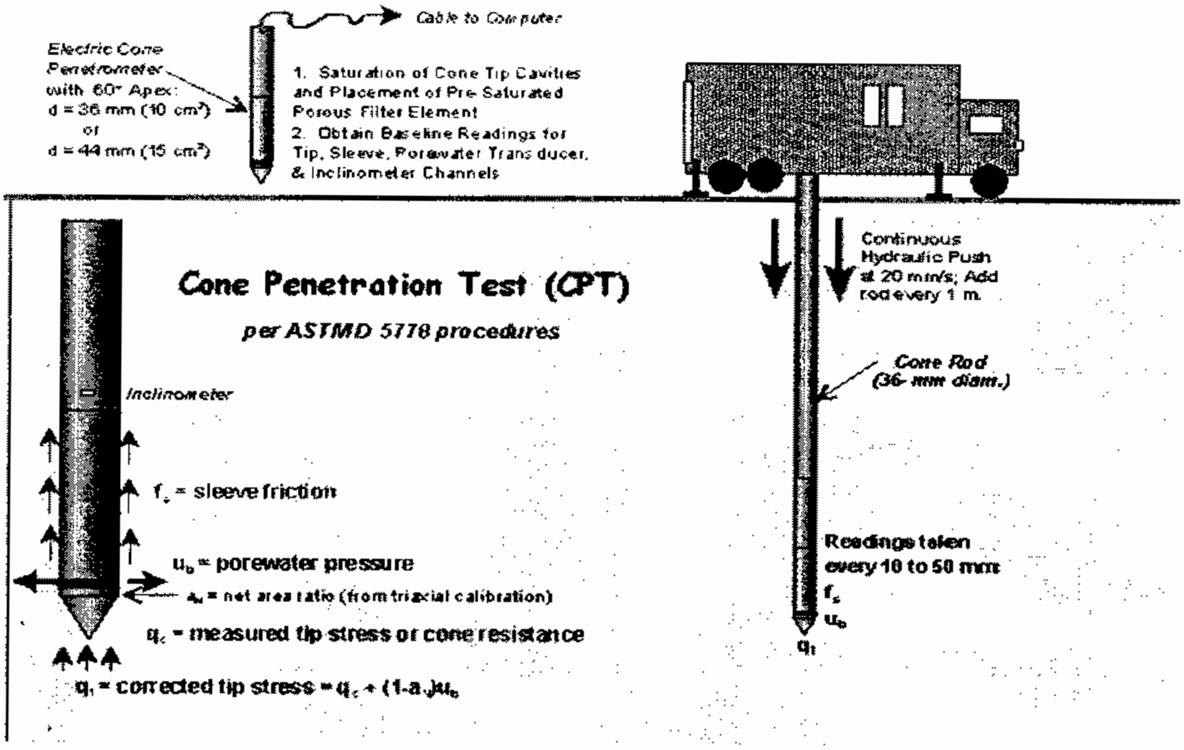
- Height of fall of monkey hammer
- Interference of drop hammer by guide rod
- Diameter and Condition of Winch
- Number of Turns of rope in winch
- Condition of rope
- Improper seating of sampler in the bottom of the hole
- Effect of isolated stone met in the hole
- Effect of water table in silts and fine sands
- Carelessness on the part of the tester
- Effect of overburden pressure

Example

The field N value of fully submerged fine sand was 40 at a depth of 6m. The average saturated unit weight of soil is 19 KN/m^3. Calculate the corrected N value for overburden and dilatancy correction.

Cone Penetration test (CPT):

- ❖ CPT is an in situ test which has been proved valuable for soil profiling.
- ❖ History: Introduced nearly 50 years ago in Holland and Belgium (introduced by soil mechanics laboratory at Delft).
- ❖ Due to this reason static cone penetration test is also referred as Dutch cone penetration test.

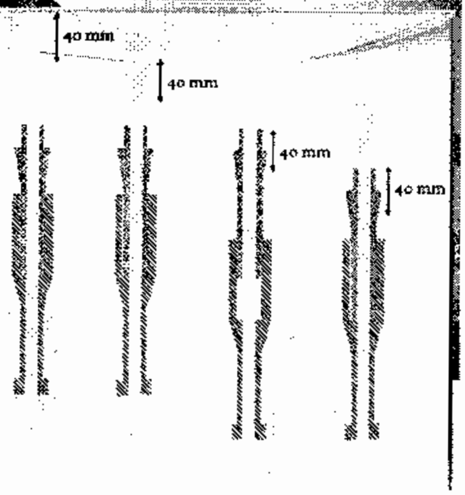


Types of CPT

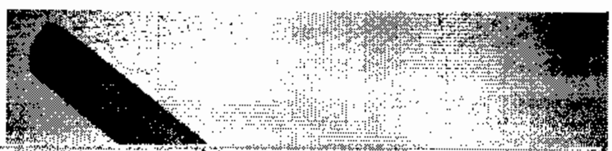
1. Static (SCPT)
2. Dynamic(DCPT)
   a. Dry DCPT
   b. Wet DCPT

METHODOLOGY:

Static cone penetration test:



Static Cone Penetrometer test



Cone Penetrometer with pore water pressure measurement capability are known as piezocones

- ❖ Dutch cone has an apex angle of 60 degree and overall diameter of 35.7mm giving an end area of 10 cm^2.
- ❖ Cone and jacket are pushed at the standard rate of 10mm per seconds and for the depth of 40mm at a time.
- ❖ The cone is pushed downwards by applying the thrust.
- ❖ Resistance offered by cone and sleeve are measured individually by burden

gauges provided in the driving mechanism.

- Also called Dutch cone penetration test.
- The cone is pushed continuously into the ground and automatic measurements are taken of the force on the tip and sleeves
- Depth is also recorded
- Cone resistance q_c and friction resistance f_s are used together with charts to obtain soil parameters

PENETROMETER

- 60° cone
- Base are =10 cm2
- Diameter of cone= 35.7mm
- Frictional sleeve=15 cm2

Suitability

- In coarse grained soils.
- CPT is very functional (useful) in small scale pile load model test.

Force required for the inner rod to push the tip (Q_c) and the total force required to push both the tip and the sleeve ($Q_c + Q_f = Q_t$) will be measured

$$q_c = \frac{Q_c}{A_c}$$

In the same way, the local side friction f_c is

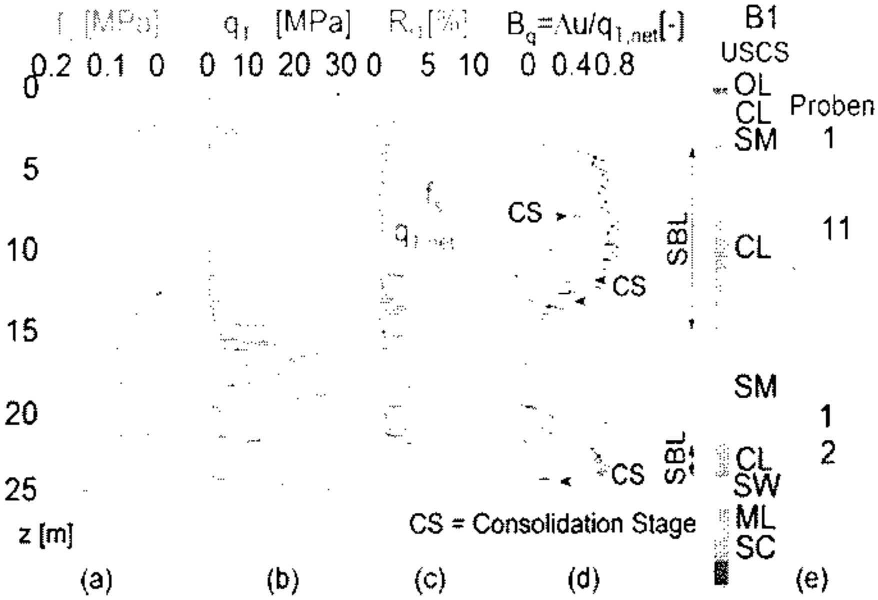
$$f_c = \frac{Q_f}{A_f}$$

where, $Q_f = Q_t - Q_c$ = force required to push the friction jacket,
Q_t = the total force required to push the cone and friction jacket together in the case of a mechanical penetrometer,
A_f = surface area of the friction jacket.

Friction ratio, R_f is expressed as

$$R_f = \frac{f_c}{q_c}$$

Various correlations have been developed to determine soil strength parameters (c, ϕ, ect) from R_f



Use of friction ratio for soil classification:

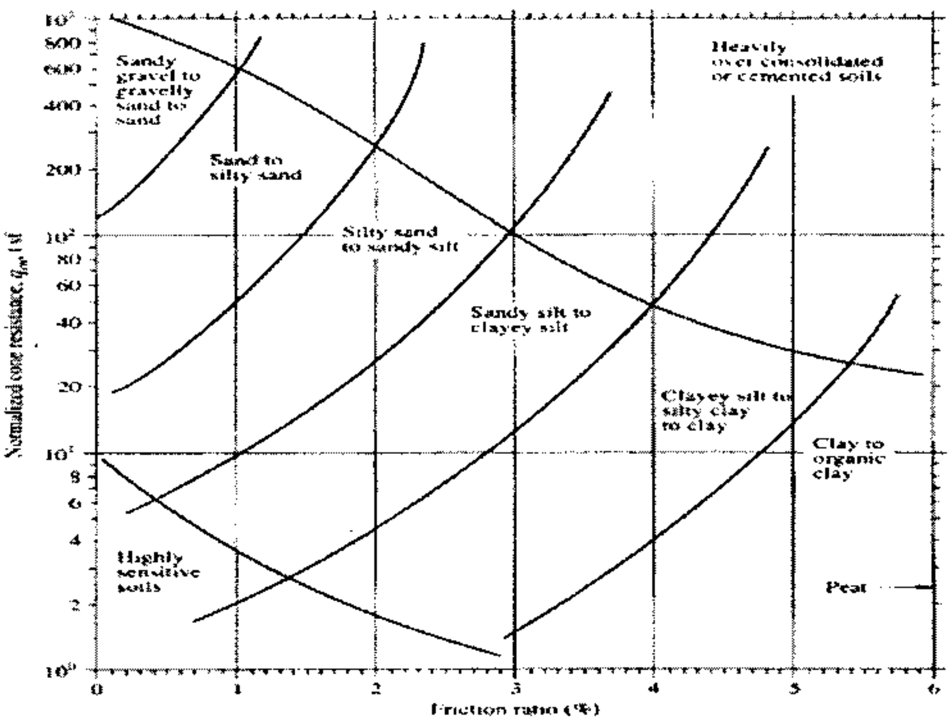


Figure 9.16 A simplified classification chart (Douglas, 1984)

Co-Relation with q_c and N:

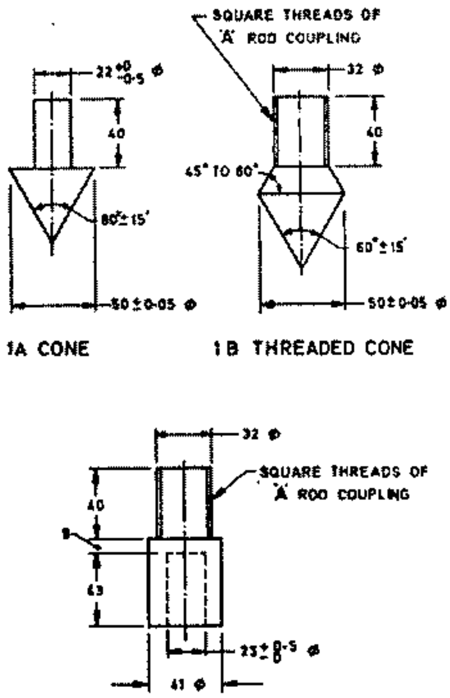
- Sands q_c = 500 N to 600N
- Silty Sands q_c = 300 N to 400N
- Silts and clayey soil q_c = 200 N
- Where qc is in KN/m^2.

Dynamic CPT

- It can be Dry or wet DCPT.
- Test is conducted by driving the cone by blows of hammer.
- Number of blows for driving the cone through a specified distance is the measure of the dynamic cone resistance (N_{cbr}).

Dry DCPT (Without using bentonite slurry)

- 50mm cone without bentonite slurry used in cased borehole to eliminate skin friction.
- Driving energy- 65kg hammer falling through 75cm.
- The number of blows required for 30cm of penetration is taken as cone penetration resistance (Ncbr).



All dimensions in millimetres.

CONE AND CONE ADOPTER

Co-Relation with SPT(N):

For 50mm cone

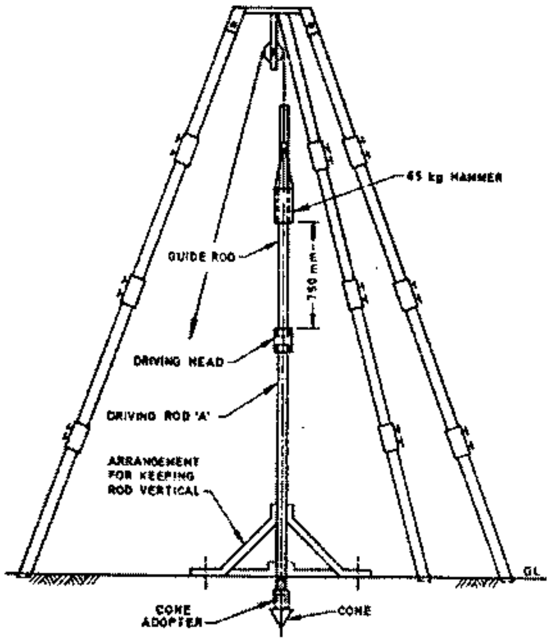
- N cbr = 1.5N (For depths upto 3m)
- N cbr = 1.75N(for depth greater than 3-6m)
- N cbr = 2N (for depth greater than 6m)

For 65mm dia. cone without using bentonite slurry (source Arora)

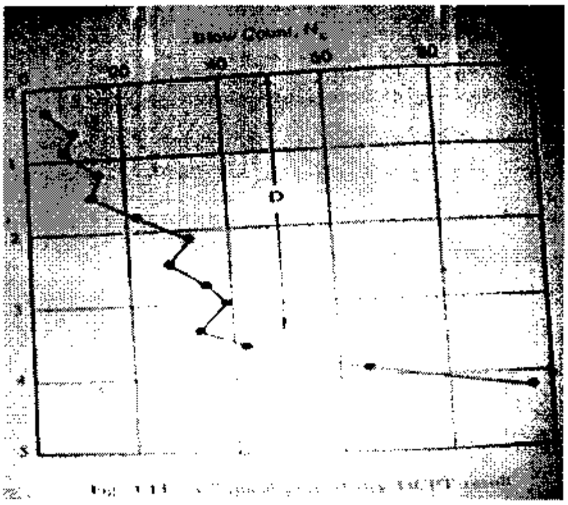
(From Central Building Research, Roorkee)

- Ncbr =1.5N (For depths upto 4m)
- Ncbr =1.75N(for depth greater than 4-9m)
- Ncbr = 2N (for depth greater than 9m)

Fig showing variation of depth in Ncbr in dry DCPT.

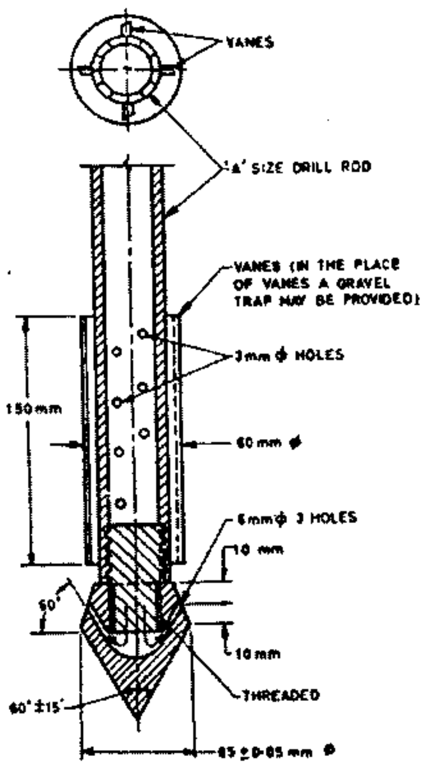


TYPICAL ASSEMBLY OF EQUIPMENT FOR CONE PENETRATION TEST

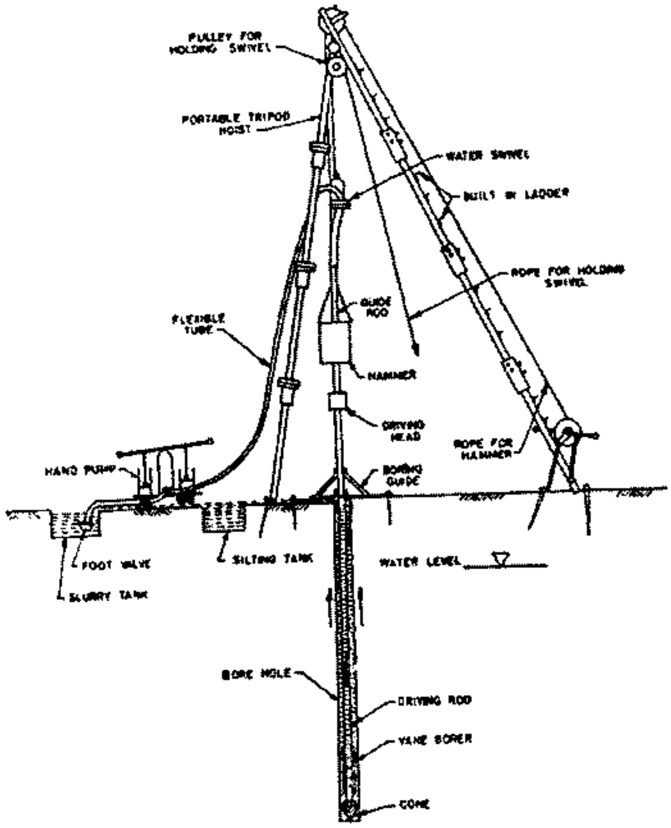


Wet DCPT:

- 62.5 mm (65 mm cone has been withdrawn as per IS code) with bentonite slurry at the end of drill rod with arrangement for drilling mud to flow through the cone as shown in fig.
- Conducted to eliminate frictional resistance on the drilling rods.



CONE ASSEMBLY



A TYPICAL SET UP FOR DYNAMIC CONE PENETRATION TEST

As Per IS : 4968 (Part I &II) – 1976, the correlation between cone penetration value (Ncbr) and standard penetration value (N)

When the 62.5mm cone is driven up to 9m (without bentonite slurry)

Ncbr = 1.5 N – for depth upto 4m

Ncbr = 1.75 N – for depth between 4 – 9m

When the 62.5mm cone is penetrated by circulating slurry.

Ncbr = N

- For medium to fine sands, the above relationships have been developed by the central Building Research Institute, Roorkee.
- These relationships, when utilized, shall be used with caution.

PRESSUREMETER:

- Was first conceived, designed, constructed and used by Menard (1957) of France.
- One of the in-situ methods for estimating ultimate bearing capacity and settlement of footings in both cohesive and cohesion less soils.
- Gives valuable information for the design of foundation by testing at 1m interval.

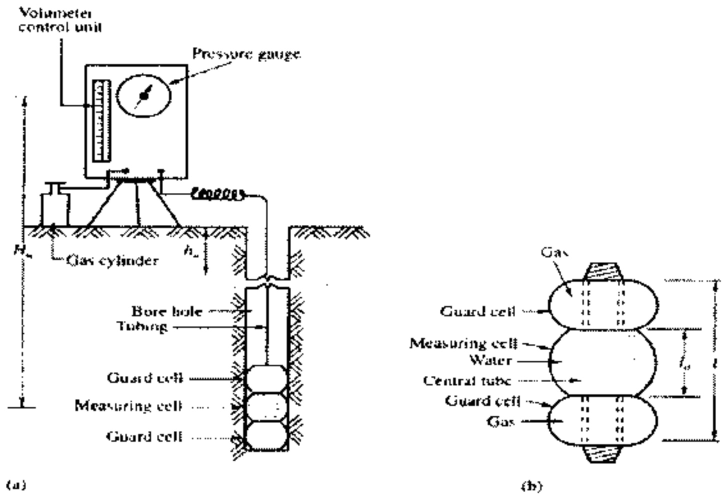
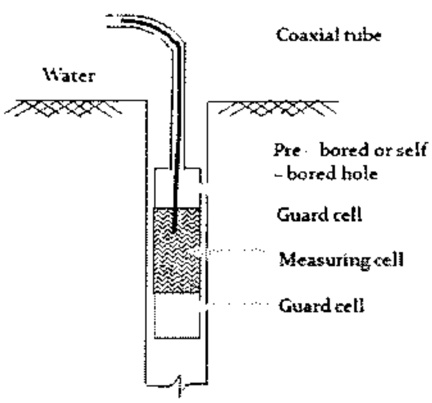
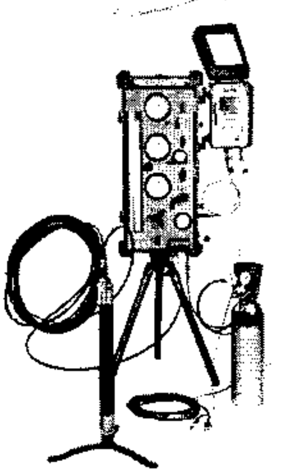


Figure 9.19 Components of Menard pressuremeter

Pressuremeter





- Consists of three parts, namely, the probe, the control unit and the tubing.
- Consists of a probe with three cells.
- The top and bottom ones are guard cells and the middle one is measuring cell.
- Probe cells can be expanded by either liquid or gas.

- ❖ The guard cells are expanded to reduce end condition effect on the measuring cells which has a volume (V_c) of 535 cm^3.

Types:

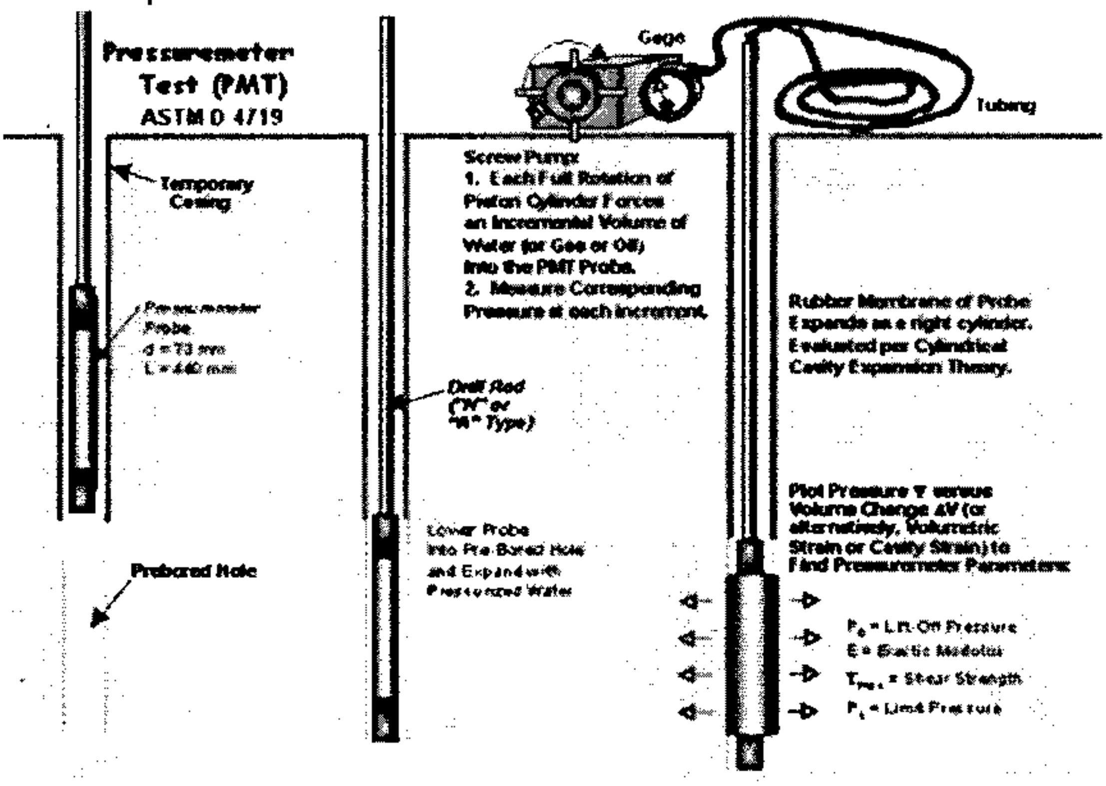
- ❖ Pre bored or Menard Pressure meter
- ❖ Self bored pressure meter

Menard Pressuremeter:

- ➢ Pressure meter test involves
- ❖ Drilling of a hole
- ❖ Lowering the probe into the hole and clamping it at the desired elevation
- ➢ Conducting the test.
- ❖ Diameter of bore hole, D_h, in which test is to be conducted shall satisfy the condition
- ❖ $1.03*D_p < Dh < 1.20*D_p$
- ❖ Where, D_p= Dia. Of probe under deflated condition.

Experimental Procedure:

- ❖ The measuring cell volume (V_c) is measured and the probe is inserted into the borehole.
- ❖ Pressure is applied in increments and the new volume cell is measured.
- ❖ The process is continued until the soil fails until the pressure limit pressure is reached.
- ❖ The soil is considered to be failed when total volume of the expanded cavity (v) is about twice the volume of the original cavity.
- ❖ After the completion of test, the probe is deflated and advanced for testing at another depth.
- ❖ After the pressure verses volume curve is plotted with the notations which are used in the computations of various factors.



OA-Zone I- Reloading
AB- Zone II- Pseudo Elastic
BC- Zone III- Plastic

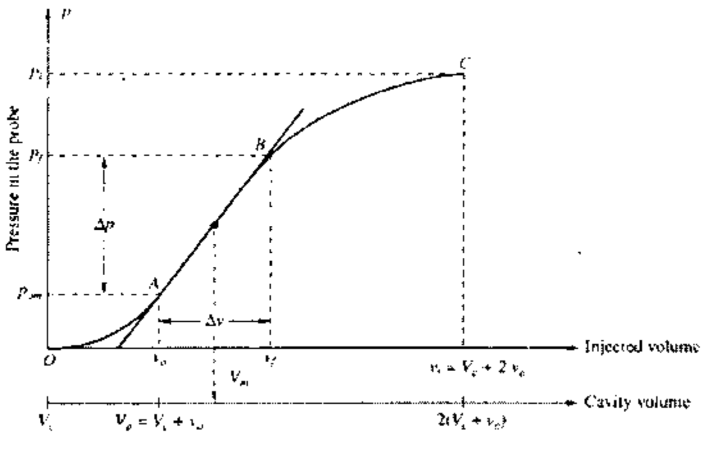


Figure 9.20 A typical corrected pressuremeter curve

Graph Interpretation

- Zone I- Represents reloading portion during which the soil around the borehole is pushed back into the initial state (i.e., the state it was before drilling)
- The pressure P_0 represent the initial total horizontal stress.
- Zone II – Represents a **pseudo elastic zone** in which the cell volume verses cell pressure is practically **linear.**
- Zone III – The **plastic zone** P_L represents the limit pressure
- Po=Lift off pressure , P_f= Yield pressure , P_l= Limit pressure

Correlations from PMT:

- Shear modulus 'G'
- Undrained shear strength 'Cu'
- Coefficient of earth pressure at rest 'Ko'

$$E_p = 2G_s(1+\mu) = 2(1+\mu)V_m\frac{\Delta p}{\Delta v}$$

where G_s is the shear modulus.

Pressuremeter Modulus:

$$V = V_m = V_c + \frac{v_0 + v_f}{2},$$

μ_s = Poisson's ratio, assumed as 0.33

$$E_m = 2.66V_m\frac{\Delta p}{\Delta v} \tag{9.25}$$

The following empirical relationship has been established from the results obtained from pressuremeter tests. Undrained shear strength c_u as a function of the limit pressure $\overline{p}_l$ may be expressed as

$$c_u = \frac{\overline{p}_l}{9} \tag{9.26}$$

where $\overline{p}_l = p_l - p_{oh}$ and p_{oh} = total horizontal earth pressure for the at rest condition.

Amar and Jézéquel (1972) have suggested another equation of the form

$$c_u = \frac{\overline{p}_l}{10} + 25 \text{ kPa} \tag{9.27}$$

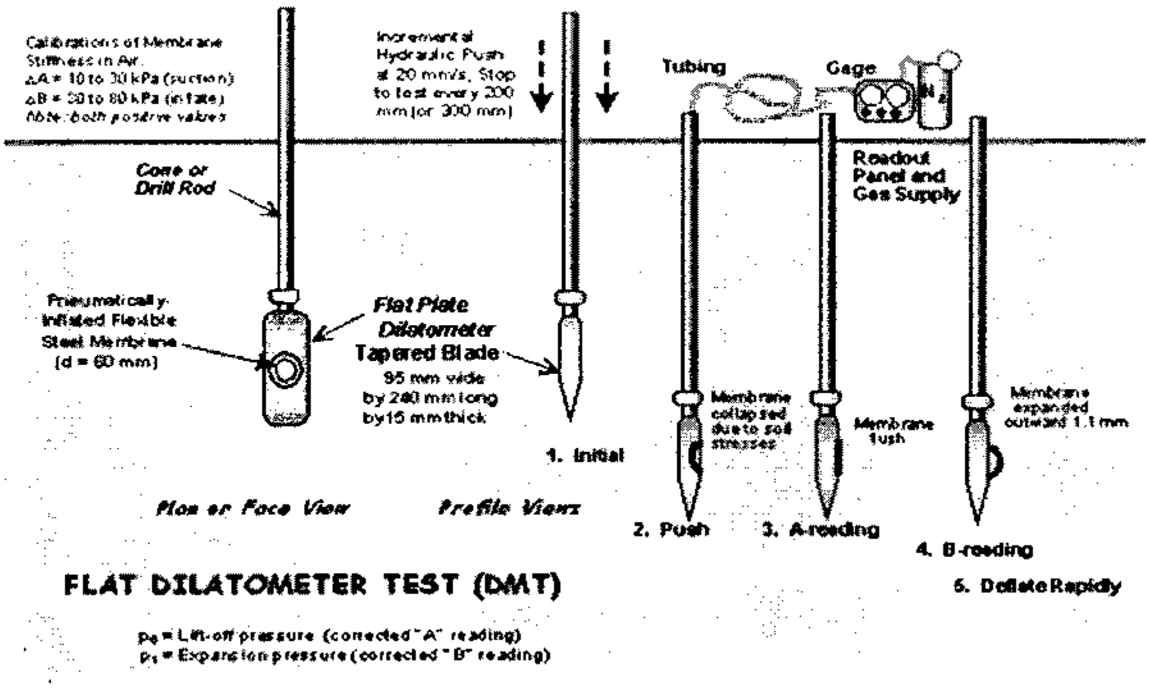
where both $\overline{p}_l$ and c_u are in kPa.

Application:

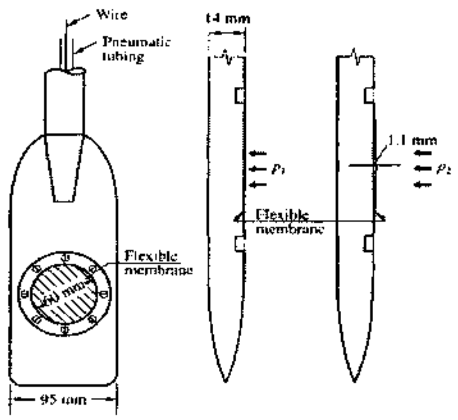
For the calculation of:

- ❖ Bearing capacity of shallow and deep foundations
- ❖ Settlement of all types of foundations
- ❖ Deformation of laterally loaded piles and sheet piles
- ❖ Resistance of anchors

DIALATOMETER TEST INTRODUCED BMARCHETTI (1980) IN ITALY:



- ❖ Is an in-situ testing device developed by Marchetti (1980's) in Italy
- ❖ A penetration device that include a lateral expansion arrangement after penetration. The test therefore combine many of the features contained in cone penetration test and pressure-meter test
- ❖ Has been extensively used for reliable, economical and rapid in-situ determination of Geotechnical parameters
- ❖ Flat plate dilatometer consists of stainless steel blade with a flat circular expandable membrane of 60mm diameter on one side of the stainless plate
- ❖ Size of plate is 220mmX95mmX14mm
- ❖ Dilatometer is attached to a string of drill rods and pressed into the ground
- ❖ A signal, combination of gas and electric line, extends through the drill rods and down to the blade from the surface control and pressure readout box.



Procedure:

1. The probe is positioned at the required level. Nitrogen gas is pumped into the probe. When the membrane is just flush with the side of the surface, a pressure reading is taken which is called the lift-off pressure. This pressure is called p1
2. The probe pressure is increased until the membrane expands by an amount Δl= 1.1 mm. The corrected pressure is *p2*.
3. The next step is to decrease the pressure until the membrane returns to the lift off position. This corrected reading is *p 3*.

The details of the calculation lead to the following equations:

1. Material index, $I_D = \dfrac{p_2 - p_1}{p_2 - u}$

2. The lateral stress index, $K_D = \dfrac{p_1 - u}{p'_o}$

3. The dilatometer modulus, $E_D = 34.7(p_2 - p_1)$ kN/m^2

where, p'_o = effective overburden pressure = $\gamma' z$

u = pore water pressure equal to static water level pressure

γ' = effective unit weight of soil

z = depth of probe level from ground surface

The lateral stress index K_D is related to K_0 (the coefficient of earth pressure for the at-rest condition) and to *OCR* (overconsolidation ratio).

Marchetti (1980) has correlated several soil properties as follows

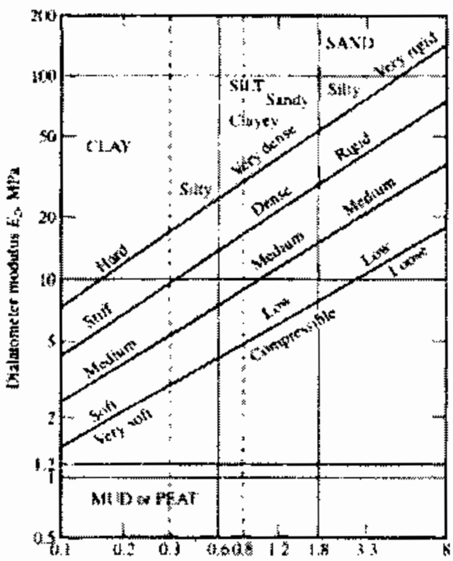
$$E_s = (1 - \mu^2)E_D$$

$$K_0 = \left(\frac{K_D}{1.5}\right)^{0.47} - 0.6$$

$$OCR = (0.5K_D)^{1.6}$$

$$\left(\frac{c_u}{p'_o}\right)_{oc} = \left(\frac{c_u}{p'_o}\right)_{nc} \times (0.5K_D)^{1.25}$$

where E_s is the modulus of elasticity



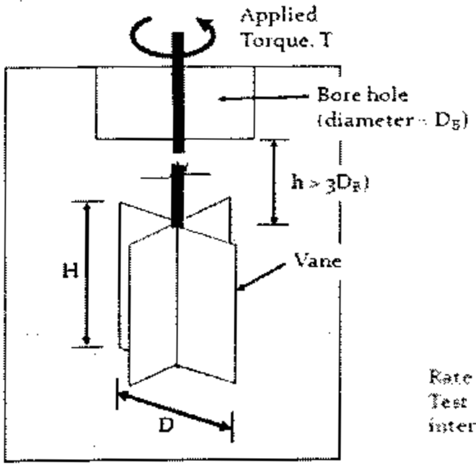
where E_s is the modulus of elasticity

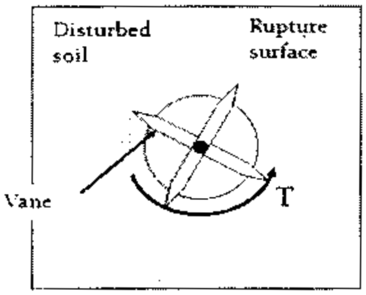
The soil classification as developed by Schmertmann (1986) is given in Fig. 9.22. I_D is related with E_D in the development of the profile.

VANE SHEAR TEST:

Vane shear test

This is one of the most versatile and widely used devices used for investigating undrained shear strength (C_u) and sensitivity of soft clays





PLAN VIEW

Rate of rotation : 6° – 12° per minute
Test can be conducted at 0.5 m vertical intervals

Let the force applied = P eccentricity (lever arm) = x units

Turning moment = Px

The surface resisting the turning is the cylindrical surface of the soil and the two end faces of the cylinder.

Therefore,

$$\text{resisting moment} = (2\pi r \times L \times c_u \times r + 2\pi r^2 \times c_u \times 0.67r) = 2\pi r^2 c_u(L + 0.67r)$$

where r = radius of the cylinder and c_u the undrained shear strength.

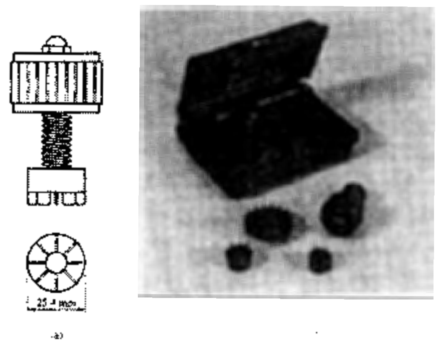
At failure the resisting moment of the cylinder of soil is equal to the turning moment applied at the torsion head.

Therefore, $Px = 2\pi r^2 c_u(L + 0.67r)$

$$c_u = \frac{Px}{2\pi r^2(L + 0.67r)}$$

Torvane

Torvane, a modification of the vane, is convenient for investigating the strength of clays in the walls of test pits in the field or for rapid scanning of the strength of tube or split spoon samples.



GROUND WATER TABLE LEVEL:

- Groundwater conditions and the potential for groundwater seepage are fundamental factors in virtually all geotechnical analyses and design studies. Accordingly, the evaluation of groundwater conditions is a basic element of almost all geotechnical investigation programs. Groundwater investigations are of two types as follows:

- Determination of groundwater levels and pressures.
- Measurement of the permeability of the subsurface materials.
- Ground water table can be measured by different methods as Hvorselev Method, Piezometer method, sounding method, by dropping cable or measuring tape etc.

Rising water table method (Hvorselev):

$$\left.\begin{aligned} H_0 &= \frac{h_1^2}{h_1 - h_2} \\ H_2 &= \frac{h_2^2}{h_1 - h_2} \\ H_3 &= \frac{h_3^2}{h_2 - h_3} \end{aligned}\right\}, \text{etc.}$$

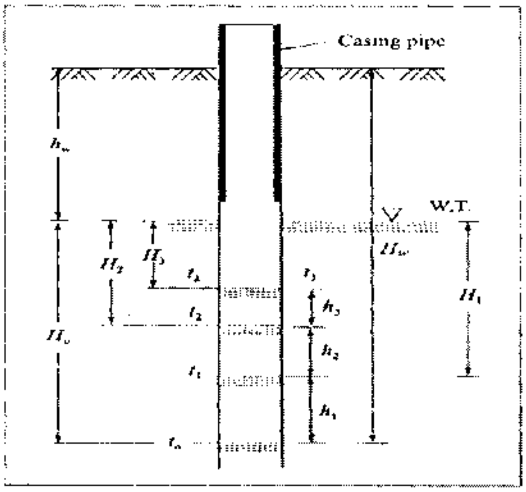


Fig. 3.29 Water table level location by rising water level method

Let the corresponding depths of water table level below the ground surface be h_{w1}, h_{w2}, h_{w3}, etc.
Now, we have

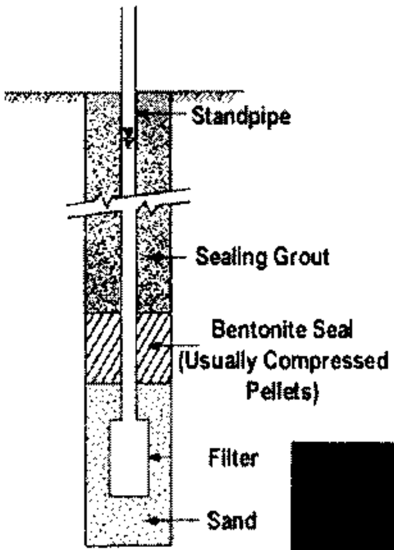
$$h_{w1} = H_w - H_o$$

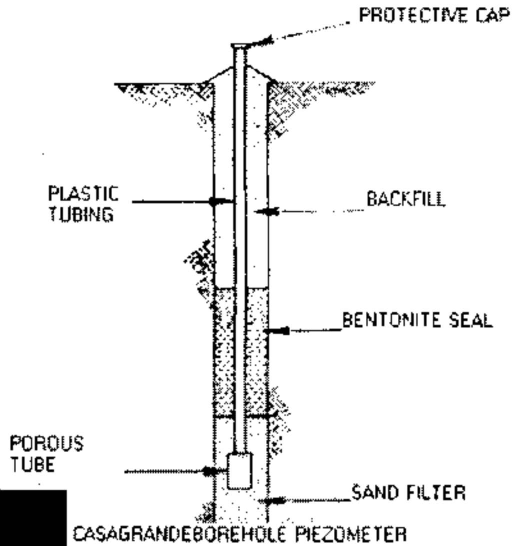
$$h_{w2} = H_w - (h_1 + h_2) - H_2$$

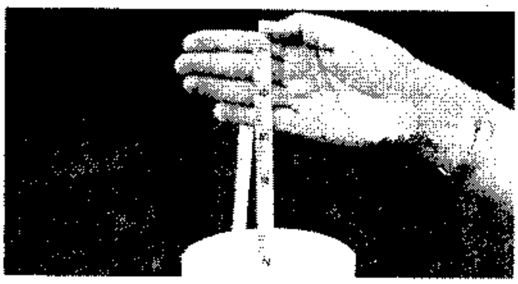
$$h_{w3} = H_w - (h_1 + h_2 + h_3) - H_3$$

where, H_w is the depth of water level in the casing from the ground surface at the start of the test.
Normally, $h_{w1} = h_{w2} = h_{w3} = h_w$; if not an average value gives h_w.

Cassagrande Piezometer Method







Ch-3.Lateral Earth Pressure Theories and Retaining Walls

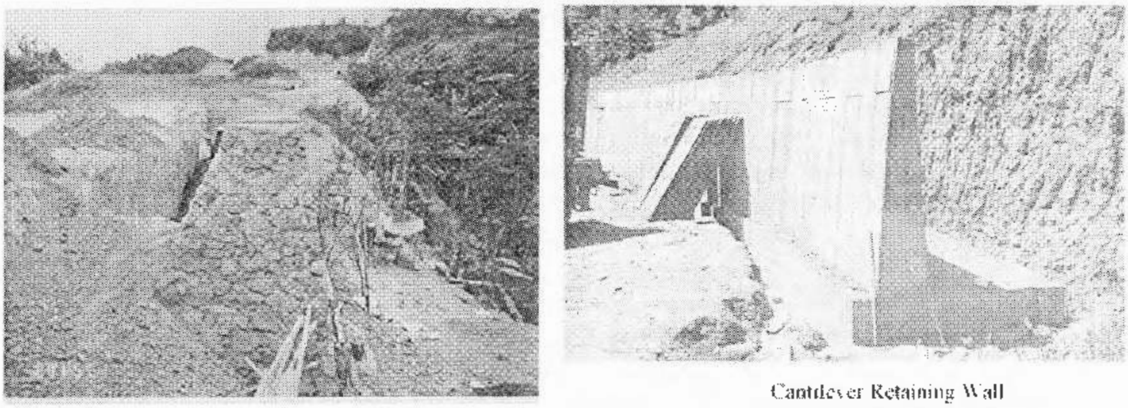
3.1. Introduction

A soil mass is stable when the slope of the surface of the soil mass is flatter than the safe slope. At some locations, due to limitation of space, it's not possible to provide flat slope & the soil is to be retained at a slope steeper than the safe one. In such cases, a retaining structure is required to provide lateral support to the soil mass.

A retaining structure is a permanent or temporary structure which is used for providing lateral support to the soil mass or other materials. Some of the examples of retaining structures used in soil & foundation engineering are: **Retaining wall, Sheet piles, Anchored Bulkheads, Sheeting & Basement wall,** etc. In the absence of a retaining structure, the soil on the higher side would have a tendency to slide and may not remain stable.

The design of the retaining structure requires the determination of the magnitude & line of action of the lateral earth pressure. The magnitude of the lateral earth pressure depends upon a number of factors, such as the mode of movement of the wall, the flexibility of the wall, the properties of the soil, the drainage conditions. For convenience, the retaining wall is assumed to be rigid & the soil structure interaction effect is neglected which arises due to the flexibility of the wall.

The pressure or force exerted by soil on any boundary is called the earth pressure. When the earth pressure acts on the side (back or face) of a retaining wall, it is known as the **Lateral earth pressure.** The magnitude of the lateral earth pressure depends upon the movement of the retaining wall relative to the backfill & upon the nature of the soil.



Cantilever Retaining Wall

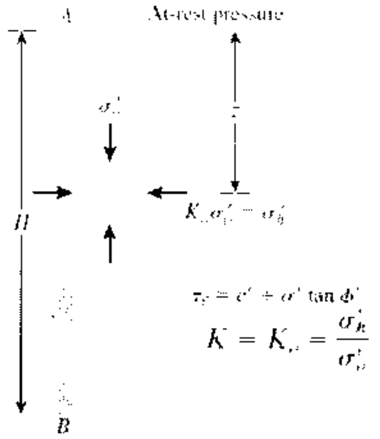
Figure- Gravity and Cantilever retaining wall under construction

3.2 Effect of wall movement on Earth Pressure

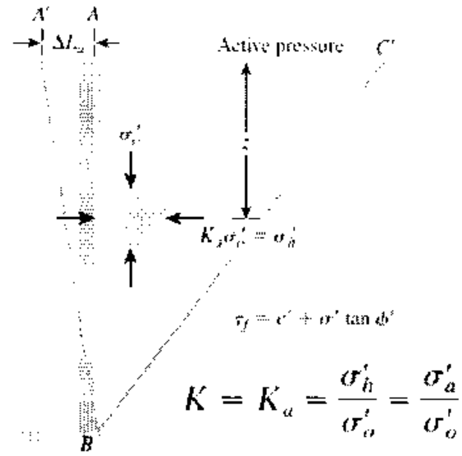
The mass is bounded by a frictionless wall of height AB. A soil element located at a depth z is subjected to a vertical effective pressure, and a horizontal effective pressure, There are no shear stresses on the vertical and horizontal planes of the soil element. Let us define the ratio of to as a nondimensional quantity *K*, or

$$K = \frac{\sigma'_h}{\sigma'_o}$$

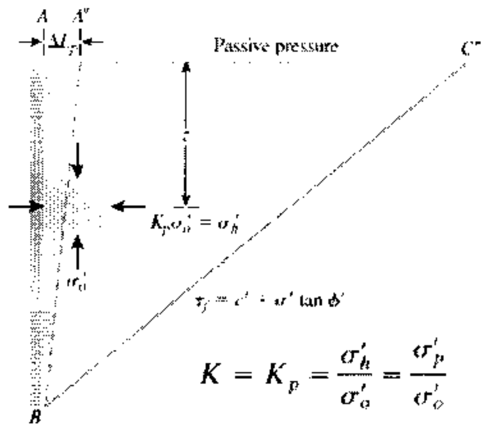
Case I - Coefficient of earth pressure at rest (K_0):
When the wall is rigid and unyielding, the soil mass is in a state of rest and there are no deformations and displacements. The earth pressure corresponding to this state is called the earth pressure at rest.



Case II- Active earth pressure coefficient (K_a):
If the wall rotates about its toe, thus moving away from the backfill, the soil mass expands, resulting in a decrease of the earth pressure. When the wall moves away from the backfill, a portion of the backfill located next to the retaining wall tends to break away from the rest of the soil mass and tends to move downwards and outwards relative to the wall. Since the shearing resistance is mobilized in directions away from the wall, there is a resultant decrease in earth pressure which continues until at a certain amount of displacement, failure will occur in the backfill and slip surfaces will be developed. At this stage the entire shearing resistance has been mobilized. The force acting on the wall at this stage does not decrease any more beyond this point even with further wall movement. This force is called the active earth pressure.



Case III- Passive earth pressure coefficient (K_p):
On the other hand, if the wall is pushed towards the backfill, the soil is com pressed and the soil offers resistance to this movement by virtue of its shearing resistance. Since the shearing resistance builds up in directions towards the wall, the earth pressure gradually increases. If this fore reaches a value which the backfill cannot withstand, failure again ensues and slip surface develop. The pressure reaches a maximum value when the entire shearing resistance has been mobilized and does not increase any more with further wall movement. This pressure is called the passive earth pressure.



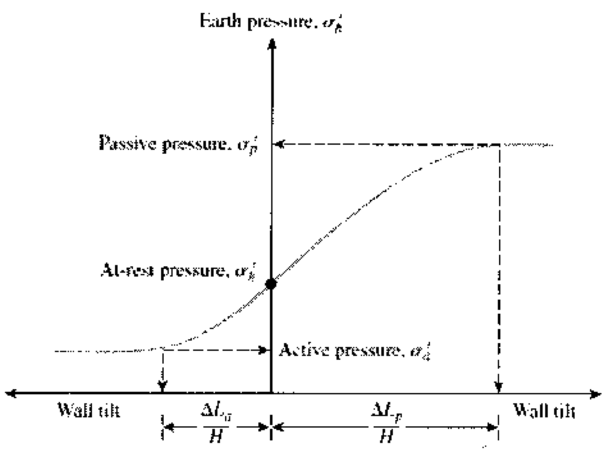


Fig. Variation of Magnitude of Lateral Earth Pressure with Wall Tilt

3.3 Earth Pressure at Rest

The value of K_0 depends upon the relative density of the sand and the process by which the deposit was formed. If this process does not involve artificial tamping the value of K_0 ranges from about 0.40 for loose sand to 0.6 for dense sand. Tamping the layers may increase it to 0.8.

The value of K_0 may also be obtained on the basis of elastic theory. If a cylindrical sample of soil is acted upon by vertical stress σ_v, and horizontal stress σ_h, the lateral strain ε_1 may be expressed as

$$\varepsilon_1 = \frac{1}{E}\left[\sigma_h - \mu(\sigma_h + \sigma_v)\right]$$

where E = Young's modulus, μ = Poisson's ratio.

The lateral strain $\varepsilon_l = 0$ when the earth is in the at rest condition. For this condition, we may write

$$\frac{1}{E}\left[\sigma_h - \mu(\sigma_h + \sigma_v)\right] = 0 \quad \text{or} \quad \frac{\sigma_h}{\sigma_v} = \frac{\mu}{1-\mu}$$

$$\text{or} \quad \sigma_h = \left(\frac{\mu}{1-\mu}\right)\sigma_v = K_0\sigma_v = K_0\gamma z$$

$$\text{where} \quad \frac{\mu}{1-\mu} = K_0, \quad \sigma_v = \gamma z$$

According to Jaky (1944), a good approximation for K_0 is given by Eq. $K_0 = 1 - \sin\phi$
Which fits the most experimental data

Sherif, Fang, and Sherif (1984),

$$K_o = (1 - \sin\phi) + \left[\frac{\gamma_d}{\gamma_{d(\min)}} - 1\right]5.5$$

where γ_d = actual compacted dry unit weight of the sand behind the wall
$\gamma_{d(\min)}$ = dry unit weight of the sand in the loosest state

Mayne and Kulhawy (1982)

$$K_o = (1 - \sin\phi')(OCR)^{\sin\phi'}$$

For fine-grained, normally consolidated soils, Massarsch (1979)

$$K_o = 0.44 + 0.42\left[\frac{PI(\%)}{100}\right]$$

Other Coorelations are:

$K_0 = (1 - \sin\phi')$ (Jaky, 1944)

$K_0 = 0.9\,(1 - \sin\phi')$ (Fraser, 1957)

$K_0 = 0.19 + 0.233 \log I_p$ (Kenney, 1959)

$K_0 = [1 + (2/3)\sin\phi']\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$ (Kezdi, 1962)

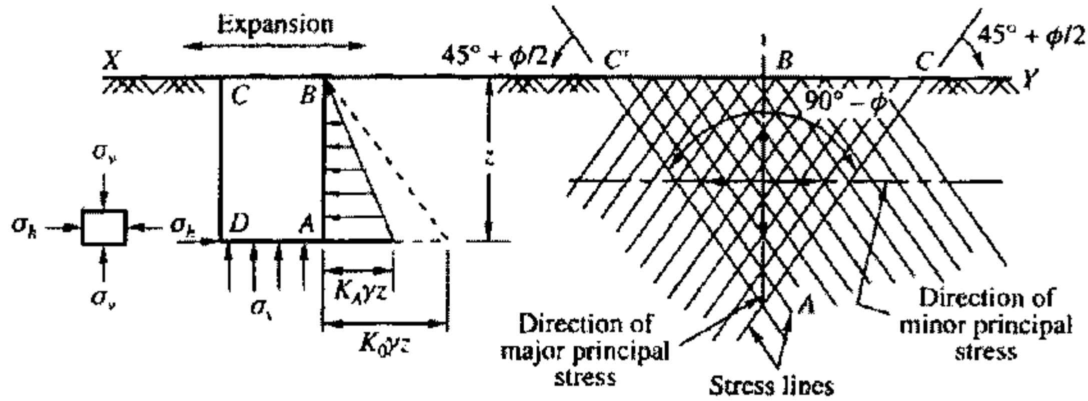
$K_0 = (0.95 - \sin\phi')$ (Brooker and Ireland, 1965)

3.4. Classical Earth Pressure Theories

Rankine's Theory

Assumptions made:

1. The back fill soil is isotropic, homogeneous and is cohesionless.
2. The soil is in a state of plastic equilibrium during active and passive earth pressure conditions.
3. The rupture surface is a planar surface which is obtained by considering the plastic equilibrium of the soil.
4. The backfill surface is horizontal.
5. The back of the wall is vertical.
6. The back of the wall is smooth.

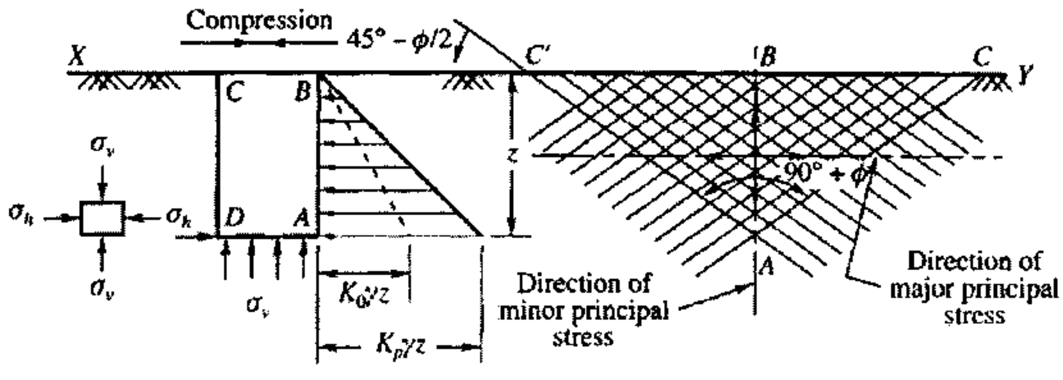


(a) Active state

Let XY in above Fig. (a) Active state, represent the horizontal surface of a semi-infinite mass of cohesionless soil with a unit weight g. The soil is in an initial state of elastic equilibrium. Consider a prismatic block ABCD. The depth of the block is z and the cross-sectional area of the block is unity. Since the element is symmetrical with respect to a vertical plane, the normal stress on the base AD is

$$\sigma_v = \gamma z$$

If the wall yields away from the soil, then the soil in the back expands thereby decreasing the horizontal stress. If the yield is large enough, the lateral stress decreases to a minimum value in which Mohr's circle drawn touches the failure envelope. This is known as state of plastic equilibrium and the soil mass is said to be in active Rankine state.



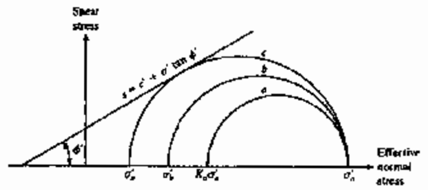
(b) Passive state

If we imagine that the entire mass is subjected to horizontal deformation, such deformation is a plane deformation. Every vertical section through the mass represents a plane of symmetry for the entire mass. Therefore, the shear stresses on vertical and horizontal sides of the prism are equal to zero.

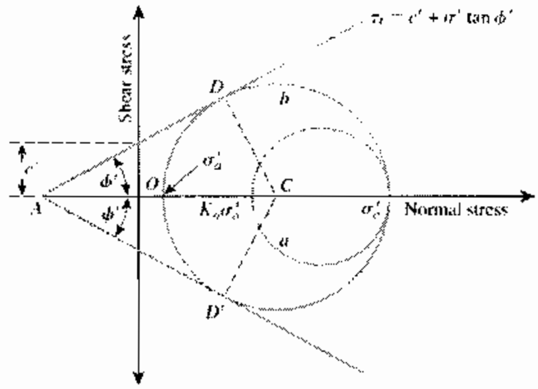
Due to the stretching, the pressure on vertical sides *AB* and *CD* of the prism decreases until the conditions of *plastic equilibrium* are satisfied, while the pressure on the base *AD* remains unchanged. Any further stretching merely causes a plastic flow without changing the state of stress. The transition from the state *of plastic equilibrium* to the state *of plastic flow* represents the failure of the mass. Since the weight of the mass assists in producing an expansion in a horizontal direction, the subsequent failure is called *active failure.*

If, on the other hand, the mass of soil is compressed, as shown in Fig. (b) Passive state, in a horizontal direction, the pressure on vertical sides *AB* and CD of the prism increases while the pressure on its base remains unchanged at *yz*. Since the lateral compression of the soil is resisted by the weight of the soil, the subsequent failure by plastic flow is called *a passive failure.*

Rankine's Theory of Active Pressure



The stress condition in the soil element can berepresented by the Mohr's circle in an adjacent figure. However, if the wall is allowed to move away from the soil mass gradually, the horizontal principal stress will decrease.



Ultimately a state will be reached when the stress condition in the soil element can be represented by the Mohr's circle b, the state of plastic equilibrium and failure of the soil will occur. This situation represents Rankine's active state, and the effective pressure on the vertical plane is Rankine's active earth pressure. We can derive in terms of σ_a in terms of γ, z, *c (cohesion)*, and ϕ from the figure

$$\sin\phi' = \frac{CD}{AC} = \frac{CD}{AO + OC}$$

$$CD = \text{radius of the failure circle} = \frac{\sigma'_o - \sigma'_a}{2}$$

From figure, $AO = c'\cot\phi'$, $OC = \dfrac{\sigma'_o + \sigma'_a}{2}$

$$\sin\phi' = \frac{\dfrac{\sigma'_o - \sigma'_a}{2}}{c'\cot\phi' + \dfrac{\sigma'_o + \sigma'_a}{2}}$$

$$c'\cos\phi' + \frac{\sigma'_o + \sigma'_a}{2}\sin\phi' = \frac{\sigma'_o - \sigma'_a}{2}$$

$$\sigma'_a = \sigma'_o\frac{1 - \sin\phi'}{1 + \sin\phi'} - 2c'\frac{\cos\phi'}{1 + \sin\phi'}$$

$$\sigma'_o = \text{vertical effective overburden pressure} = \gamma z$$

$$\frac{1 - \sin\phi'}{1 + \sin\phi'} = \tan^2\left(45 - \frac{\phi'}{2}\right) \text{ and } \frac{\cos\phi'}{1 + \sin\phi'} = \tan\left(45 - \frac{\phi'}{2}\right)$$

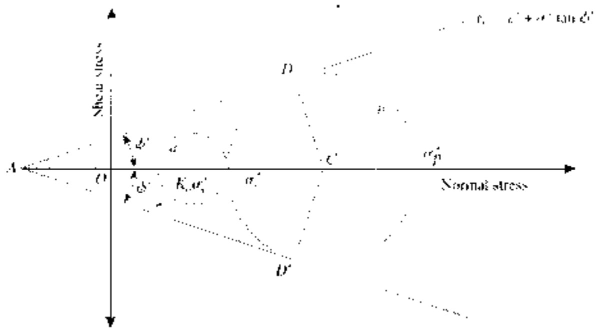
Therefore,

$$\sigma'_a = \gamma z\tan^2\left(45 - \frac{\phi'}{2}\right) - 2c'\tan\left(45 - \frac{\phi'}{2}\right)$$

In the case when c'=0, the relationship will be

$$\sigma'_a = \sigma'_o\tan^2\left(45 - \frac{\phi'}{2}\right)$$

Rankine's Theory of Passive Pressure (Do yourself)

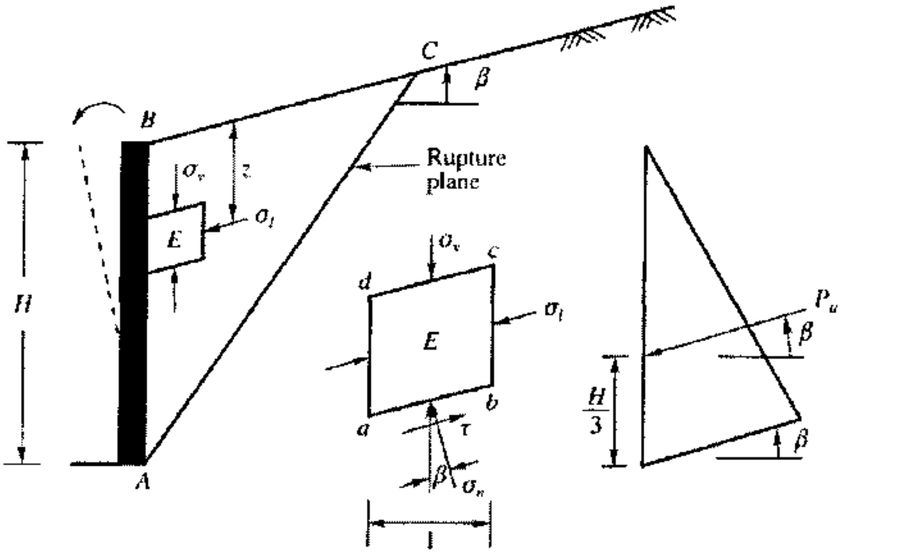


$$\sigma'_p = \sigma'_o \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

$$= \gamma z \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

Active case for cohesionless soil with sloping backfill surface

Figure a, b and c, below shows a smooth vertical wall with a sloping backfill of cohesionless soil. As in the case of a horizontal backfill, the active state of plastic equilibrium can be developed in the backfill by rotating the wall about A away from the backfill. Let AC be the rupture line and the soil within the wedge *ABC* be in an active state of plastic equilibrium.



(a) Retaining wall (b) Pressure distribution

Consider a rhombic element *E* within the plastic zone *ABC* which is shown to a larger scale outside. The base of the element is parallel to the backfill surface which is inclined at an angle β to the horizontal. The horizontal width of the element is taken as unity.

$$\sigma_v = \frac{\gamma . z}{1/\cos\beta} = \gamma . z \cos\beta$$

Let σ_v = the vertical stress acting on an elemental length $ab = \gamma z \cos\beta$

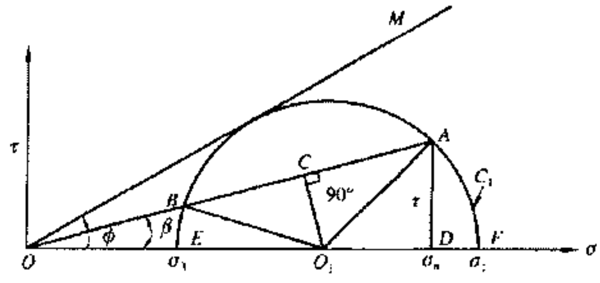
σ_l = the lateral pressure acting on vertical surface *bc* of the element

The vertical stress σ_v can be resolved into components σ_n the normal stress and τ the shear stress on surface *ab* of element *E*. We may now write

$$\sigma_n = \sigma_v \cos\beta = \gamma z \cos\beta \cos\beta = \gamma z \cos^2\beta$$

$$\tau = \sigma_v \sin\beta = \gamma z \cos\beta \sin\beta$$

A Mohr diagram can be drawn as shown in Fig. (c). Here, length $OA = \gamma z \cos\beta$ makes



(c) Mohr diagram

an angle β with the σ-axis. $OD = \sigma_v = \gamma z\cos^2\beta$ and $AD = \tau = \gamma z\cos\beta\sin\beta$. OM is the Mohr envelope making an angle ϕ with the σ-axis. Now Mohr circle C_1 can be drawn passing through point A and at the same time tangential to envelope OM. This circle cuts line OA at point B and the σ-axis at E and F.

The following relationships can be expressed with reference to the Mohr diagram.

$$BC = CA = \frac{\sigma_1 + \sigma_3}{2}\sqrt{\sin^2\phi - \sin^2\beta}$$

$$\sigma_v = OA = OC + CA = \frac{\sigma_1 + \sigma_3}{2}\cos\beta + \frac{\sigma_1 + \sigma_3}{2}\sqrt{\sin^2\phi - \sin^2\beta}$$

$$\sigma_l = p_a = OC - BC = \frac{\sigma_1 + \sigma_3}{2}\cos\beta - \frac{\sigma_1 + \sigma_3}{2}\sqrt{\sin^2\phi - \sin^2\beta}$$

Now we have (after simplification)

$$\frac{\sigma_l}{\sigma_v} = \frac{p_a}{\gamma z\cos\beta} = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$\text{or} \quad p_a = \gamma z\cos\beta \times \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} = \gamma z K_A$$

$$\text{where,} \quad K_A = \cos\beta \times \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

is called as *the coefficient of earth pressure* for the active state or the active earth pressure coefficient.

Sometimes it is also referred as conjugate ratio.

Sloping Surface-Passive Earth Pressure (Do yourself)

An equation for P_p for a sloping backfill surface can be developed in the same way as for an active case. The equation for P_p may be expressed as

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

$$\text{where,} \quad K_p = \cos\beta \times \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

P_p acts at a height $H/3$ above point A and parallel to the sloping surface.

COULOMB'S EARTH PRESSURE THEORY

More than 200 years ago, Coulomb (1776) presented a theory for active and passive earth pressures against retaining walls. In this theory, Coulomb assumed that the failure surface is a plane. The *wall friction* was taken into consideration. The following sections discuss the general principles of the derivation of Coulomb's earth-pressure theory for a cohesionless backfill (shear strength defined by the equation $\tau_f = \sigma' \tan \phi'$).,

Thus the assumptions made by Coulomb can be summarized as:

1. The soil is isotropic and homogeneous
2. The rupture surface is a plane surface
3. The failure wedge is a rigid body
4. The pressure surface is a plane surface
5. There is wall friction on the pressure surface
6. Failure is two-dimensional and
7. The soil is cohesionless and the backfill surface can be inclined.

Let AB (Figure 13.22a) be the back face of a retaining wall supporting a granular soil; the surface of which is constantly sloping at an angle α with the horizontal. BC is a trial failure surface. In the stability consideration of the probable failure wedge ABC, the following forces are involved (per unit length of the wall):

1. W—the weight of the soil wedge.
2. F—the resultant of the shear and normal forces on the surface of failure, BC. This is inclined at an angle of ϕ' to the normal drawn to the plane BC.
3. P_a—the active force per unit length of the wall. The direction of P_a is inclined at an angle δ' to the normal drawn to the face of the wall that supports the soil. δ' is the angle of friction between the soil and the wall.

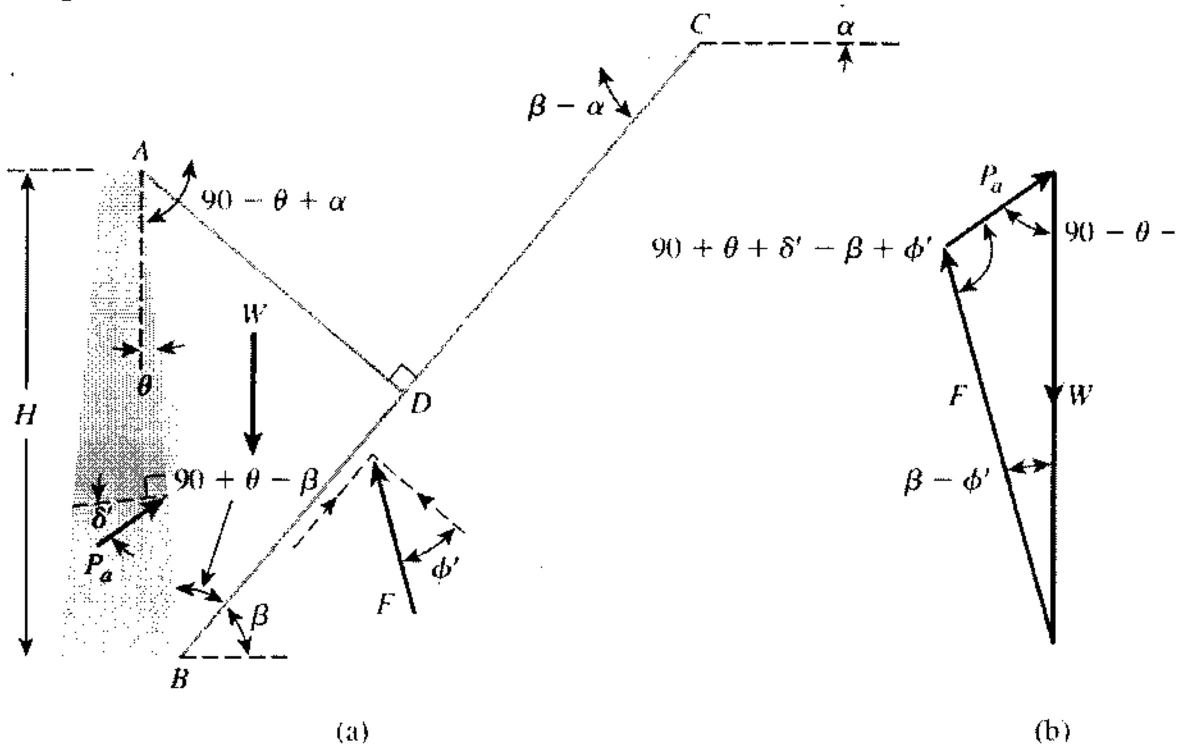


Figure 13.22 Coulomb's active pressure: (a) trial failure wedge; (b) force polygon

The force triangle for the wedge is shown in Figure 13.22b. From the law of sines, we have

$$\frac{W}{\sin(90 + \theta + \delta' - \beta + \phi')} = \frac{P_a}{\sin(\beta - \phi')}$$

or

$$P_a = \frac{\sin(\beta - \phi')}{\sin(90 + \theta + \delta' - \beta + \phi')}W$$

The preceding equation can be written in the form

$$P_a = \frac{1}{2}\gamma H^2\left[\frac{\cos(\theta - \beta)\cos(\theta - \alpha)\sin(\beta - \phi')}{\cos^2\theta\sin(\beta - \alpha)\sin(90 + \theta + \delta' - \beta + \phi')}\right]$$

where γ = unit weight of the backfill. The values of γ, H, θ, α, ϕ', and δ' are constants, and β is the only variable. To determine the critical value of β for maximum P_a, we have

$$\frac{dP_a}{d\beta} = 0$$

After solving

$$P_a = \tfrac{1}{2}K_a\gamma H^2$$

where K_a is Coulomb's active earth-pressure coefficient and is given by

$$K_a = \frac{\cos^2(\phi' - \theta)}{\cos^2\theta\cos(\delta' + \theta)\left[1 + \sqrt{\dfrac{\sin(\delta' + \phi')\sin(\phi' - \alpha)}{\cos(\delta' + \theta)\cos(\theta - \alpha)}}\right]^2}$$

Similarly, the Coulomb's passive earth pressure coefficient is given by

$$K_p = \frac{\cos^2(\phi' + \theta)}{\cos^2\theta\cos(\delta' - \theta)\left[1 - \sqrt{\dfrac{\sin(\phi' + \delta')\sin(\phi' + \alpha)}{\cos(\delta' - \theta)\cos(\alpha - \theta)}}\right]^2}$$

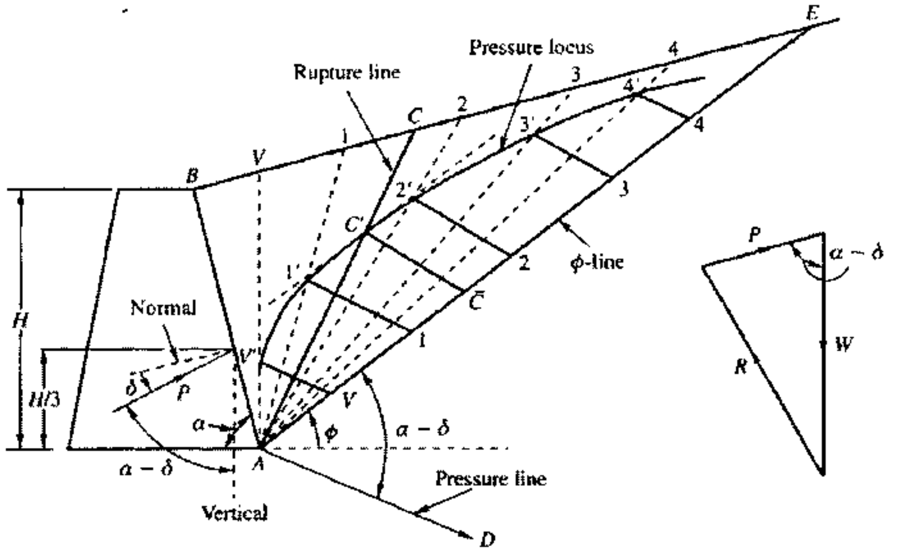
3.5 Yielding of Wall of Limited Height

We learned in the preceding discussion that sufficient movement of a frictionless wall extending to an infinite depth is necessary to achieve a state of plastic equilibrium. However, **the distribution of lateral pressure against a wall of limited height is influenced very much by the manner in which the wall actually yields.** In **most retaining walls of limited height, movement may occur by simple translation or, more frequently, by rotation about the bottom.**

3.6 Graphical solution for coulomb's earth pressure

ACTIVE PRESSURE BY CULMANN'S METHOD FOR COHESIONLESS SOILS

Culmann's (1875) method is the same as the trial wedge method. In Culmann's method, the force polygons are constructed directly on the ϕ-line AE taking AE as the load line. The procedure is as follows:

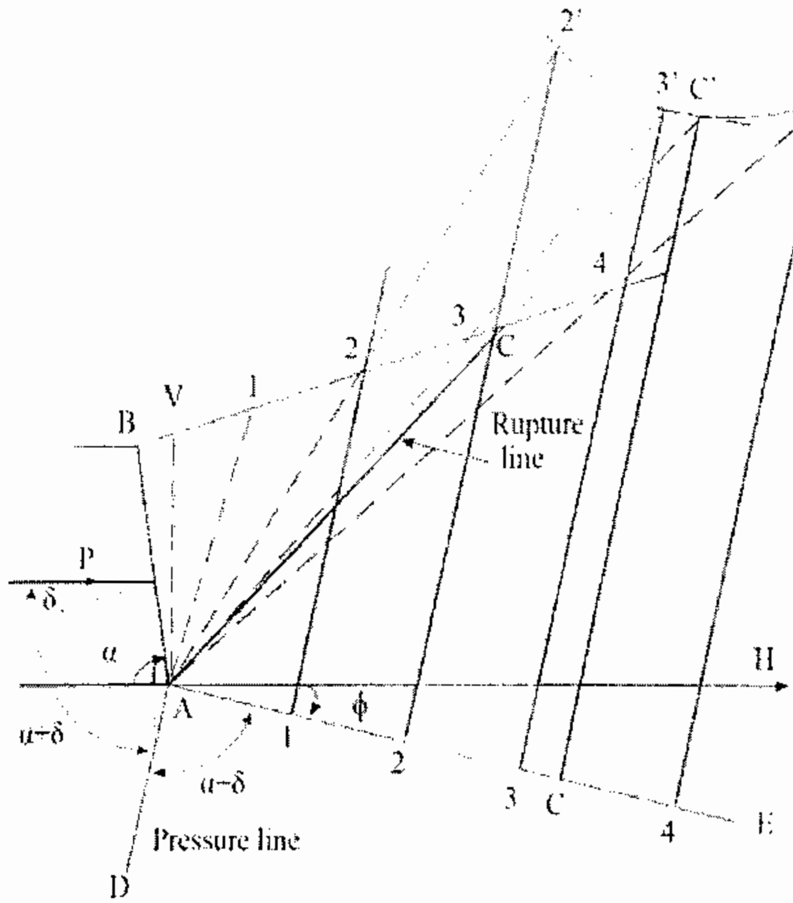


1. Draw ϕ -line AE at an angle ϕ to the horizontal.
2. Lay off on AE distances, AV, $A1$, A2, A3, etc. to a suitable scale to represent the weights of wedges ABV, $AB1$, $AB2$, $AB3$, etc. respectively.
3. Draw lines parallel to AD from points V, 1, 2, 3 to intersect assumed rupture lines AV, A1, A2, A3 at points V', $1'$, $2'$, $3'$, etc. respectively.
4. Join points V', $1'$, $2'$, $3'$, etc. by a smooth curve which is the pressure locus.
5. Select point C' on the pressure locus such that the tangent to the curve at this point is parallel to the ϕ-line AE.
6. Draw $C'\bar{C}$ parallel to the pressure line AD. The magnitude of $C'\bar{C}$ in its natural units gives the active pressure P_a.
7. Join AC' and produce to meet the surface of the backfill at C. AC is the rupture line.

For Passive case (Practice yourself)

- Draw ϕ-line AE at an angle ϕ below the horizontal
- Lay off on AE distances A_2, A_3, A_4 etc to a suitable scale to represent the weight of wedges AB_2, AB_3, AB_4, and so on
- Lay off AD at an angle equal to (α+ δ) to the line AE. The line AD is called pressure line.
- Draw lines parallel to AD from points, 2, 3, 4 etc to intersect the weight vectors A_2, A_3, A_4 at points 2', 3', 4' etc respectively.
- Join points, 2', 3', 4' etc by a smooth curve which is the pressure locus.
- Select the point C' on pressure locus curve such that the line tangent to the curve is parallel to ϕ-line AE.
- Draw CC' parallel to the pressure line AD. The magnitude of CC' in its natural units gives the passive pressure P_p.

Join AC'. The line cuts the surface of the backfill at C. The line AC is the rupture line.



Other graphical Methods are

Rebhann's Method

3.7 Trial wedge method for earth pressure

As previously noted, the trial wedge and Culmann procedures are identical except for orientation of the force polygon. The trial wedge also has an advantage over the Culmann solution since one can have cohesion as a soil parameter. Figure 11-14 illustrates the general procedure, which may be outlined as follows:

1. Draw the wall and ground surface to a scale that is as large as possible and compute the depth of the tension crack as

$$h_t = \frac{2c}{\gamma\sqrt{K_a}}$$

   This value of h_t is then plotted at sufficient points to establish the tension-crack profile (dashed line BD_1D_2D of Fig. 11-14a).
2. Lay off trial wedges as $AB'E_1D_1$, $AB'E_2D_2, \ldots,$ and compute the weight of the corresponding wedges as $w_1, w_2, \ldots, w_n$. With a tension crack it may be preferable to compute the weights as the sum of the tension block plus the weight of the triangle (as in Ex. 11-6).
3. Compute C_w and C_s (note that C_w is a constant) and lay off C_w as indicated in Fig. 11-14b to the wall slope and to the appropriate force scale. As a tension crack can form along the wall, the length AB (and not AB') should be used to compute C_w. Also draw the weight vectors $\mathbf{w}_1, \mathbf{w}_2, \ldots, \mathbf{w}_n$ along the line OY. Note that the slopes are transferred from the wedge to the force polygon.
4. From the terminus of C_w lay off C_s at the slope of the assumed trial failure wedges.
5. Through points $w_1, w_2, \ldots, w_n$ established in step 3, lay off a vector P_a to the correct slope. Note that the slope of P_a (or P_p) is constant.
6. Through the terminus of C_s lay off the vector R to the appropriate slope. The slope is at the angle ϕ to a perpendicular to the assumed failure surfaces $AD_1, AD_2, AD_3, \ldots$.
7. The intersection of the R and P_a vectors establishes a locus of points, through which a smooth curve is drawn.
8. Draw a tangent to the curve obtained in step 7, parallel to the weight vector, and draw the vector P_a through the point of tangency. As with the Culmann solution, several maximum values may be obtained. The largest possible value of P_a is the design value.

The slope of the R vector (step 6 preceding) can be established conveniently (Fig. 11-14c) as follows:

1. To some radius r draw the arc GJ from the vertical line AF in Fig. 11-14a.
2. Draw a horizontal line AO and lay off the angle ϕ as shown. With the same r used in step 1, draw arc OJ using A as the center.
3. Then AG is the slope of the vector R to failure plane AF.
4. Now lay off arcs GH, HI, IJ in Fig. 11-14c to the same arc length used in step 1.
5. The slopes of lines AH, AI, and AJ of Fig. 11-14c are the corresponding slopes of the vector R to failure surface $AD_1, AD_2, \ldots$.

In cohesionless materials the values C_w and C_s are zero, and the trial wedge solution is the same as the Culmann solution except for the orientation of the force polygon.

Example of Trial Wedge Method

Calculate $h_t = \frac{2c}{\gamma\sqrt{K_a}} = \frac{20}{19.0 \times 0.70} = 1.50$ m

Calculate cohesion C_i:

$C_w = 4.6(10) = 46$ kN
$C_1 = 5.7(10) = 57$ kN
$C_2 = 6.0(10) = 60$ kN
$C_3 = 6.4(10) = 64$ kN
$C_4 = 7.0(10) = 70$ kN
$C_5 = 7.6(10) = 76$ kN
$C_6 = 8.35(10) = 84$ kN
$C_7 = 9.1(10) = 91$ kN

W_i = Triangle crack block + Triangle i			$+W_{i-1}$	$\Sigma = W$
$W_1 = 1.2 \times 34.2 \times 19$	$+ \frac{1}{2}(5.7 \times 0.97)(19) =$	86.7	0	86.7
$W_2 = 1 \times 28.5 \times 19$	$+ \frac{1}{2}(6.0 \times 0.93)(19) =$	81.5	86.7	168.2
$W_3 = 28.5$	$+ \frac{1}{2}(6.4 \times 0.86)(19) =$	80.8	168.2	249.0
$W_{4L} = 28.5$	$+ \frac{1}{2}(7.0 \times 0.80)(19) =$	81.7	249.0	330.7
$W_{4R} =$	35.0 =		330.7	365.7
$W_5 = 28.5$	$+ \frac{1}{2}(7.6 \times 0.72)(19) =$	80.5	365.7	446.2
$W_{6L} = 28.5$	$+ \frac{1}{2}(8.4 \times 0.67)(19) =$	82.0	446.2	528.2
$W_{6R} =$	40 =		528.2	568.2
$W_7 = 1 \times 28.5 \times 19$	$+ \frac{1}{2}(9.1 \times 0.61)(19) =$	81.2	568.2	649.4

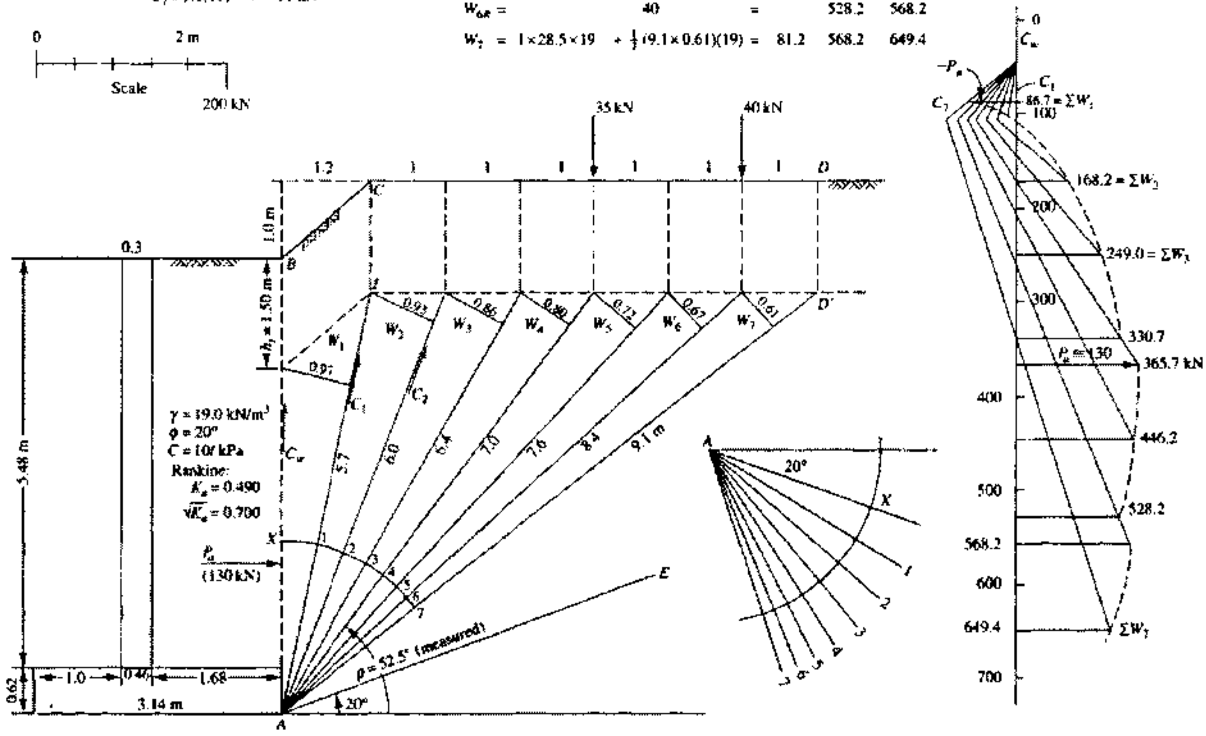
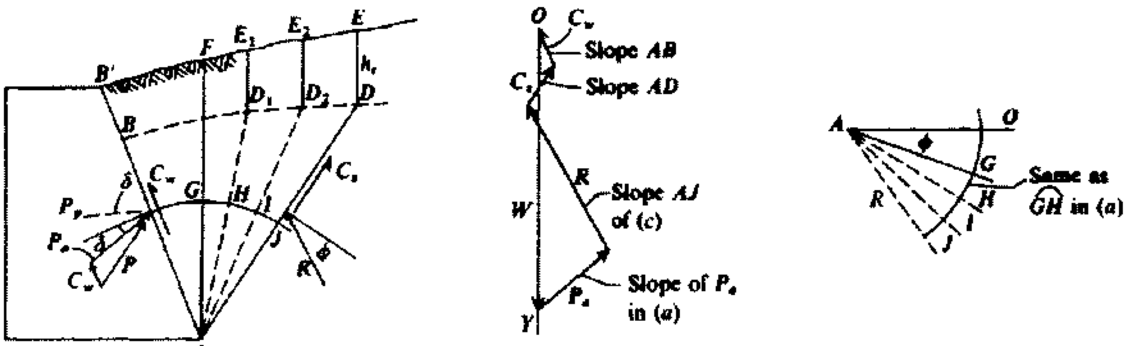


Figure E11-6



(*a*) Forces acting on a trial wedge *AB'ED*

(*b*) Forces acting on *AB'ED* formed into the force polygon

(*c*) Rapid method of establishing the slope of *R*

C_w = *AB*·cohesion (direction and magnitude known)
C_s = *AD*·cohesion (direction and magnitude known)
W = weight of trial wedge (direction and magnitude known)

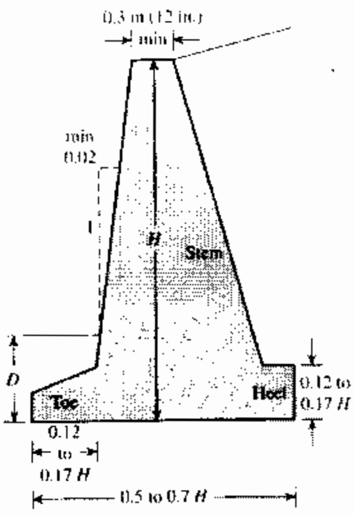
R = known in direction
P_a = known in direction

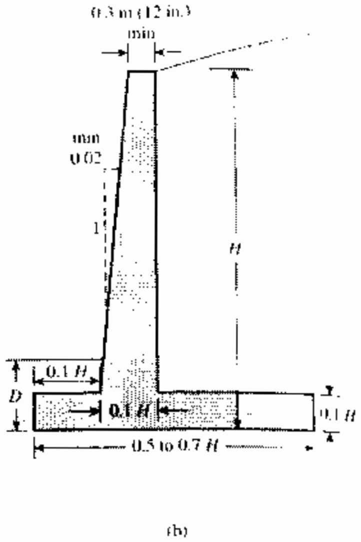
Figure 11-14 The trial wedge active force solution. For passive force slope of P_p is shown; slope R changes, C_s, C_w reverse directions.

3.8. Proportioning of retaining walls

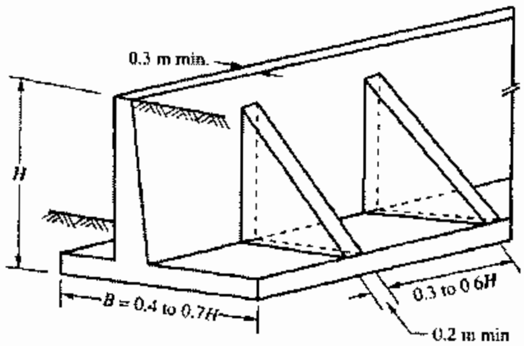
Based on practical experience, retaining walls can be proportioned initially which may be checked for stability subsequently.

- Proportioning is an assumptions which allow the engineer to check trial sections of the walls for stability.
- If the stability checks yield undesirable results, the sections can be changed and rechecked.
- The top of the stem of any retaining wall should not be less than about 0.3m for proper placement of concrete.





- The top of the stem of any retaining wall should not be less than about 0.3m for proper placement of concrete.
- The depth to the bottom of the base slab should be a minimum of 0.6m
- The bottom of the base slab should be positioned below the seasonal frost line.
- The counterfort slabs may be 0.3m thick and spaced at center to center distances of 0.3H to 0.7H and width of 0.4 to 0.7H.

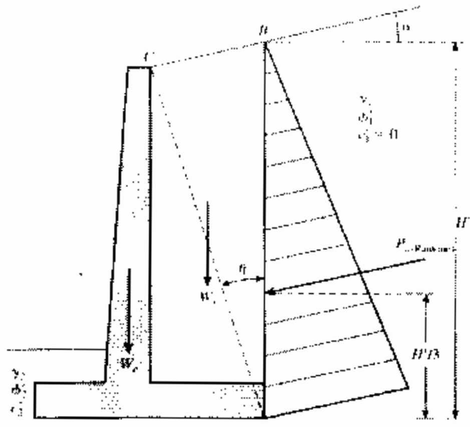


Types	RCC Crib	Dry Masonry	Bauded Masonry	Cement Masonry	Gabion Masonry	Reinforced Earth
Schematic						
Top width (m)	1.2	0.6–1.0	0.6–1.0	0.5–1.0	1.0	4.0 or 0.7–0.8H
Base width	0.4–0.6H	0.5-0.7H	0.6–0.65H	0.5–0.65H	0.6–0.75H	4.0 or 0.7–0.8H
Front batter (V:H)	4:1	3:1	varies	10:1	6:1–4:1	3:1
Back batter (V:H)	4:1	vertical	vertical	varies	varies	3:1
Foundation dip (V:H)	1:4	1:3	1:3	1:10–1:6	1:6–1:4	horizontal
Foundation depth (m)	0.5–1.0	0.5	0.5–1.0	0.5–1.0	0.5	0.5
Height range (m)	4.0–12.0	1.0–4.0	4.0–8.0	1.0–10.0	1.0–6.0	3.0–12.0
Fill slope angle (°)	< 30°	< 30°	< 20°	35°-60°	35°-60°	< 35°

Source: Adapted from MREH

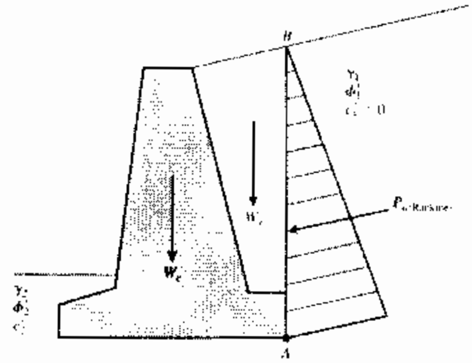
3.9 Stability of retaining walls

The stability of retaining walls should be checked for the following conditions:



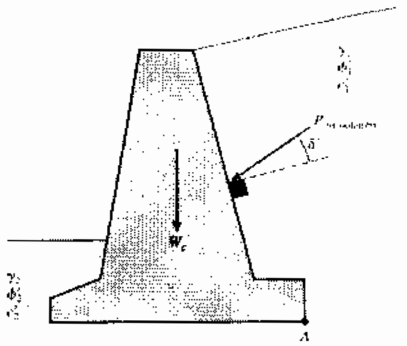
- The assumption for the development of Rankines active earth pressure along the soil face AB is theoretically correct if the shear zone bounded by the line AC is not obstructed by the stem of the wall.
- A similar type of analysis may be used for gravity walls, however Coulomb's earth pressure theory also may be used.

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

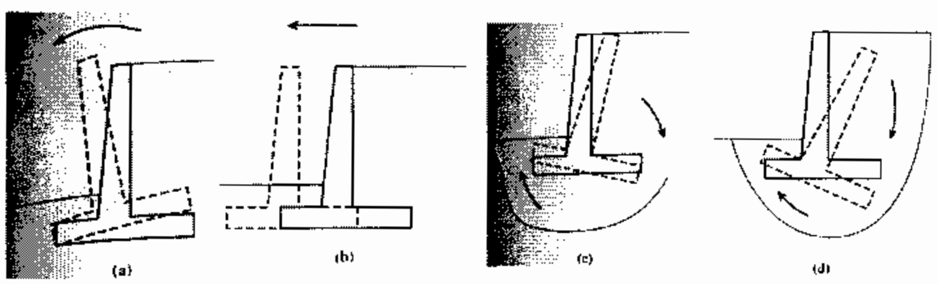


- While using Rankine theory for the Wall's stability, the active and passive forces, the weight of the soil above the heel and the concrete weight, all should be taken into consideration.
- If Coulomb's theory is used, the only forces to be considered are active and passive forces and the weight of the wall.
- If Coulomb's theory is used , it will be necessary to know the range of the wall friction angle with various types of backfill material.

Backfill material	Range of δ' (deg)
Gravel	27–30
Coarse sand	20–28
Fine sand	15–25
Stiff clay	15–20
Silty clay	12–16



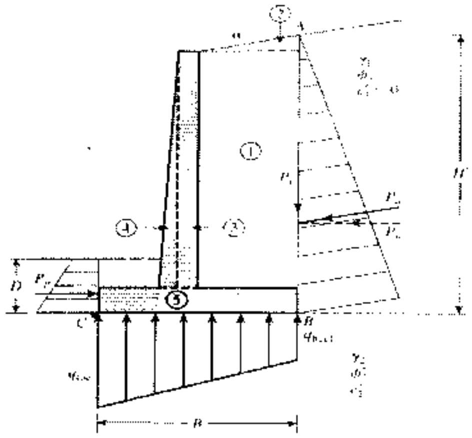
Stability of retaining wall must be checked a) by overturning, b) by sliding, c) by bearing capacity failure and d) by deep seated failure



1. **Check for overturning**

Due to the lateral force, there is possibility of overturning of retaining wall w.r.t. the toe of the wall. So the force causing the overturning of wall is the moment due to the horizontal force which is anticlockwise in nature and the force resisting this action is the clockwise moment taking toe as the point of rotation. Mathematically,

The factor of safety against overturning about point (Toe) may be expressed as



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

Where,

ΣMO=sum of the moments of forces tending to overturn about point C (Toe)

ΣMR=sum of the moments of forces tending to resist overturning about point C (Toe)

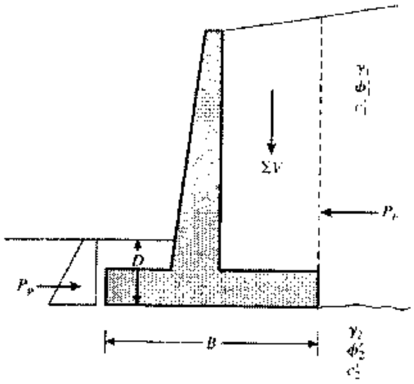
$$\Sigma M_o = P_h\left(\frac{H'}{3}\right) \quad \text{where } P_h = P_a \cos\alpha$$

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

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

2. **Check for sliding**

The force causing sliding is the horizontal component of lateral earth pressure acting on the back of the wall and the forces resisting the sliding action are the passive force generated in the front (which is generally neglected for the safe design),the friction on the base of the wall and the adhesion on the base. Mathematically



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

where

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

ΣF_d = sum of the horizontal driving forces

The shear strength of the soil immediately below the base slab may be represented as

$$s = \sigma' \tan\delta' + c'_a$$

where

δ' = angle of friction between the soil and the base slab

c'_a = adhesion between the soil and the base slab

The maximum resisting force that can be derived from the soil per unit length of the wall along the bottom of the base slab is

$$R' = s(\text{area of cross section}) = s(B \times 1) = B\sigma' \tan\delta' + Bc'_a$$

$$B\sigma' = \text{sum of the vertical force} = \Sigma V$$

$$R' = (\Sigma V)\tan\delta' + Bc'_a$$

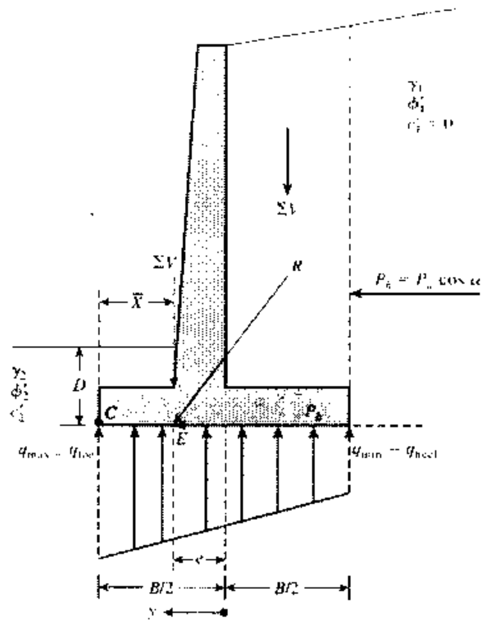
In the above figure the passive pressure P_p is also horizontal resisting force

$$\Sigma F_{R'} = (\Sigma V)\tan\delta' + Bc'_a + P_p$$

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

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

Note: Factor of safety against sliding is generally taken as 1.5.



3. **Check for bearing capacity failure**

The stress induced due to the loading including weight of retaining wall itself at the base level soil shouldn't be greater than the allowable bearing capacity of the soil. For simplicity, the variation of soil pressure is assumed to vary linearly.

$$\mathbf{R = \Sigma V + P_h}$$

- The net moment of these forces about point C

$$M_{net} = \Sigma M_R - \Sigma M_o$$

$$\overline{CE} = \bar{X} = \frac{M_{net}}{\Sigma V}$$

$$e = \frac{B}{2} - \overline{CE}$$

- The pressure distribution under the base slab may be determined by using principles from the mechanics of materials

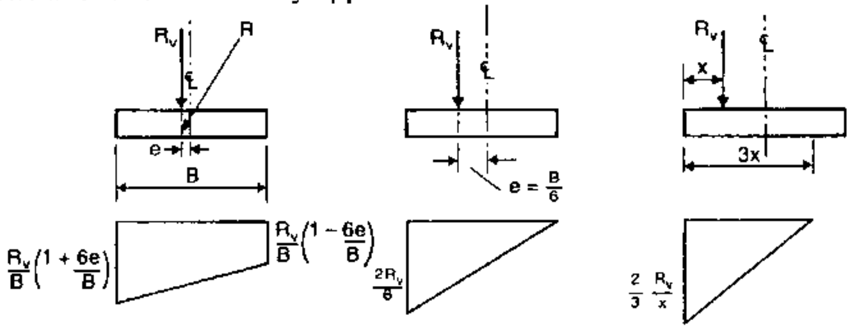
where
$$q = \frac{\Sigma V}{A} \pm \frac{M_{net}y}{I}$$

M_{net} = moment = $(\Sigma V)e$

I = moment of inertia per unit length of the base section $= \frac{1}{12}(1)(B^3)$

$$q_{max} = q_{toe} = \frac{\Sigma V}{(B)(1)} + \frac{e(\Sigma V)\frac{B}{2}}{\left(\frac{1}{12}\right)(B^3)} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right) \qquad q_{min} = q_{heel} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right)$$

The above formulae only applies when ΣV or Rv is within the middle third



(a) Within middle third (b) On middle third (c) Outside middle third

When R_v or ΣV lies on middle third then, $e = \frac{B}{6}$ Maximum pressure $= \frac{2R_v}{B}$, Minimum pressure = 0

When R_v or ΣV lies outside the middle third then, Maximum pressure $= \frac{2R_v}{3x}$; Minimum pressure = 0

and the maximum soil reaction (q_{s-max}) is given $q_{s-max} = \dfrac{2V}{3B\left(\frac{B}{2} - e\right)}$ by,

$$FOS\ against\ bearing\ capacity = \frac{q_{na}}{q_{max}}$$

NOTE: Factor of safety against bearing capacity failure ≥ 3.0

4. **Check for tension**

If the eccentricity is greater than B/6, where B is the width of the retaining wall, then the minimum soil pressure is negative that mean tension. But we know that soil cannot bear any tension, the contact area between wall base and soil decreases thus leading to failure of wall system.

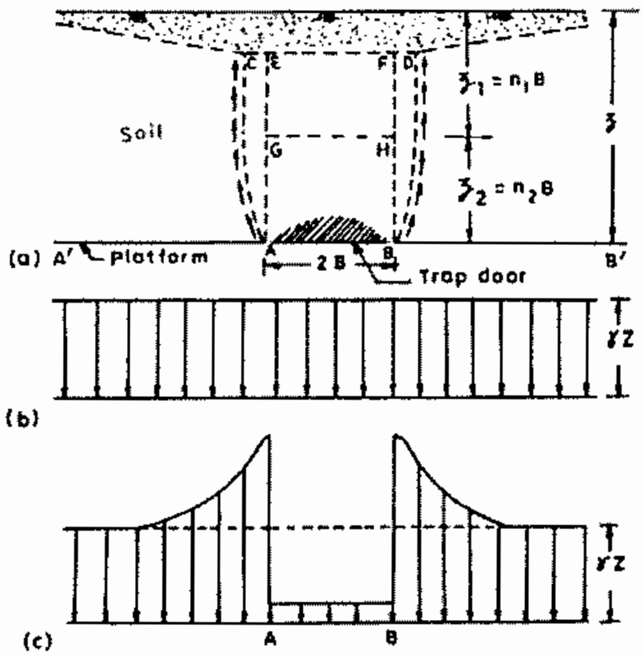
Hence for safety,

$$\text{Eccentricity (e)} \leq B/6$$

Chapter : 4 Arching in Soils and Braced Cuts

4.1.Arching in soils

In supported soil moss, when certain port of the soil moss yields, then the soil adjoining the yielding part also gets displaced from its original position. The deformation of the parted soil is resisted by mobilization of shearing resistance along the zones of contact between the yielding and non-yielding portions of the soil As the direction of mobilization of shear strength is opposed to the direction of deformation of the yielding soil, there is a reduction in pressure on the yielding part of the support and a consequent increase in the pressure of the adjoining stationary parts. This phenomenon of the transfer of pressure from the yielding part of a soil mass to the non-yielding part of the mass is referred to as **arching**.



Terzaghi's Trap Door Experiment

Arching in soil can be demonstrated from trap door experiment of Terzaghi a.s shown above. It is clear tha.t if certain mass of soil tends to move downward (the sol mass withing AE and BF in the figure) there is resistance (shearing resistance) in the upward direction offered by adjacent soil which get mobilized along the boundaries AC and BD.
The pressure which was earlier acting on the trap door AB, is now transferred from the yielding mass onto the sides. This transfer of pressure onto the stationary adjoining mass is called arching.

The two essential components of arching effect to exists are relative movement within soil and shear strength available for mobilisation.

Theories of Soil Arching

Terzaghi (1942) classified the available theories of arching into three main groups. Theories under the first group considered conditions of equilibrium of sand immediately above the yielding strip without satisfying compatibility conditions at greater distances from the strip. Theories under the second group are based on the unjustifiable assumption that the entire mass of sand above the yielding strip is in a state of plastic equilibrium. The theories under the third group assume vertical sliding surfaces passing through the outer edges of the yielding strip and the pressure on the yielding strip is taken as the difference between the weight of sand above the strip and full frictional resistance along the vertical sections.

The values of vertical pressure on a strip, obtained by different theories are, however, different. It is difficult to say conclusively which theory is better. However theories under the third group (assuming vertical sliding surface) are simplest. Cain's theory (1961) is one of them which falls in this category.

CAIN'S THEORY

Cain's theory makes the following assumptions:

(a) The soil is homogeneous, isotropic and semi-infinite.

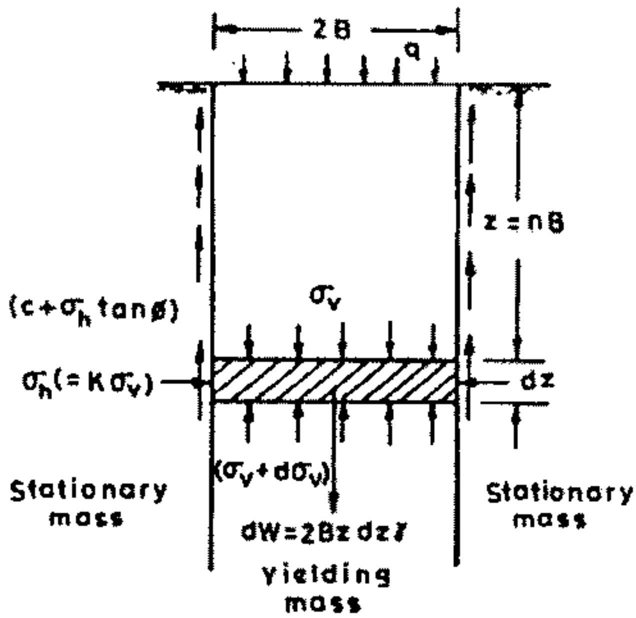
(b) The shearing resistance of the soil is governed by the equation,
$s = c + \sigma \tan \phi$.

(c) The sliding surfaces are vertical and pass through the outer edges of the yielding strip. The actual surfaces of sliding may resemble logarithmic spirals (*AC* and *BD* of Fig. 13.1) such that the depression of the surface at top is wider than the support.

(d) Full shearing resistance is mobilised on the vertical sliding surfaces.

(e) The ratio of vertical to the horizontal pressure is constant, say *K*.

(f) The problem is two-dimensional.



Forces acting on an elemental strip

The general equation developed by Cain's theory is

$$\sigma_v = qe^{-K\tan\phi\, z/B} - \frac{B}{K\tan\phi}(\gamma - c/B)\left(1 - e^{K\tan\phi\, z/B}\right)$$

The above equation can be used for three different cases

(a) $q = 0, c > 0$

$$\sigma_v = \frac{B}{K\tan\phi}(\gamma - c/B)\left(1 - e^{-K\tan\phi\, z/B}\right)$$

(b) $q > 0, c = 0$

$$\sigma_v = \frac{B\gamma}{K\tan\phi}\left(1 - e^{-k\tan\phi z/B}\right) + qe^{-K\tan\phi\, z/B}$$

(c) $q = 0, c = 0$

$$\sigma_v = \frac{B\gamma}{K\tan\phi}\left(1 - e^{-K\tan\phi\, z/B}\right)$$

4.2. Braced excavations

Shallow excavations can be made without supporting the surrounding material if there is adequate space to establish slopes at which the material can stand. The steepest slopes that can be used in a given locality are best determined by experience. Many building sites extend to the edges of the property lines. Under these circumstances, the sides of the excavation have to be made vertical and must usually be supported by bracings.

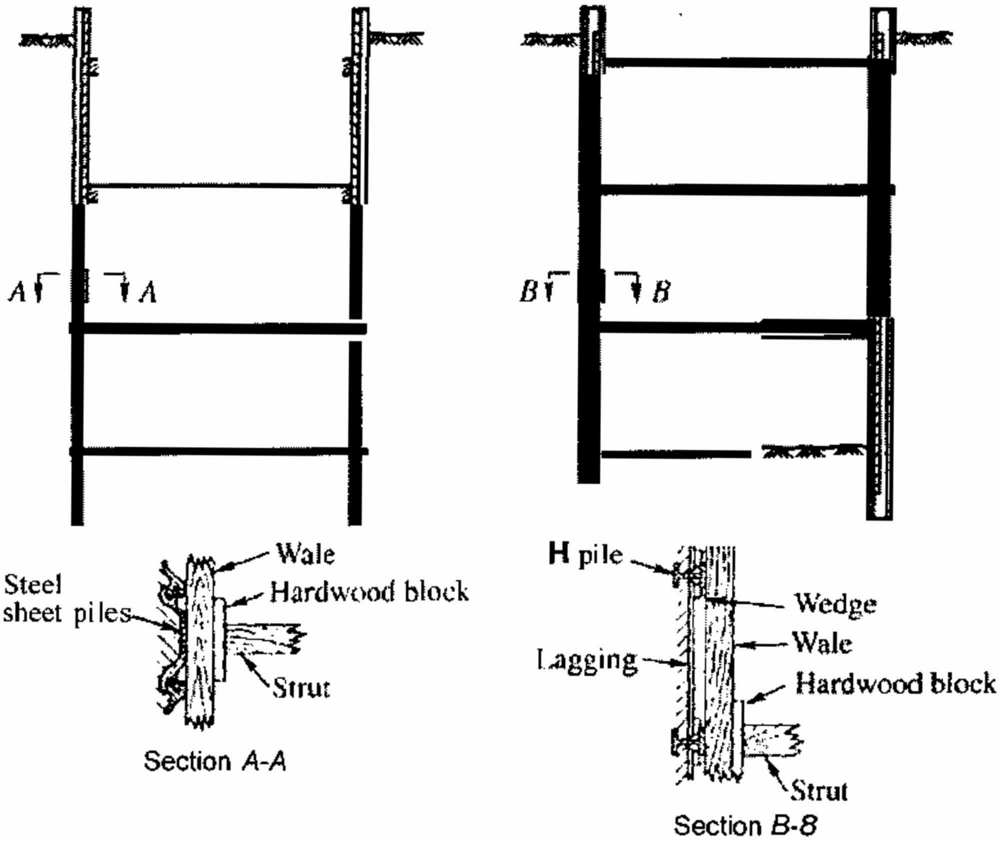


Figure:2 Bracing system

Common methods of bracing the sides are shown in Fig. 2 The practice is to drive vertical timber planks known as sheeting along the sides ofthe excavation. The sheeting is held in place by means of horizontal beams called wales that in turn are commonly supported by horizontal struts extending from side to side of the excavation. The struts are usually of timber for widths not exceeding about 2m. For greater widths metal pipes called trench braces are commonly used.

Uses of braced cut:

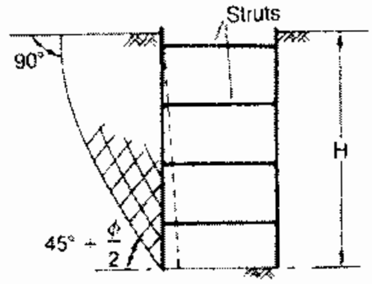
- for deep, narrow excavations
- pipelines
- service cuts

Steps followed in sheet piling:

i. drive in piling
ii. excavate first ponion
iii. install wales and top struts
iv. excavate next portion
v. install next wales and struts
vi. excavate next portion
vii. install next wales and struts
viii. excavate last portion

4.3.Earth pressure against bracings in cuts

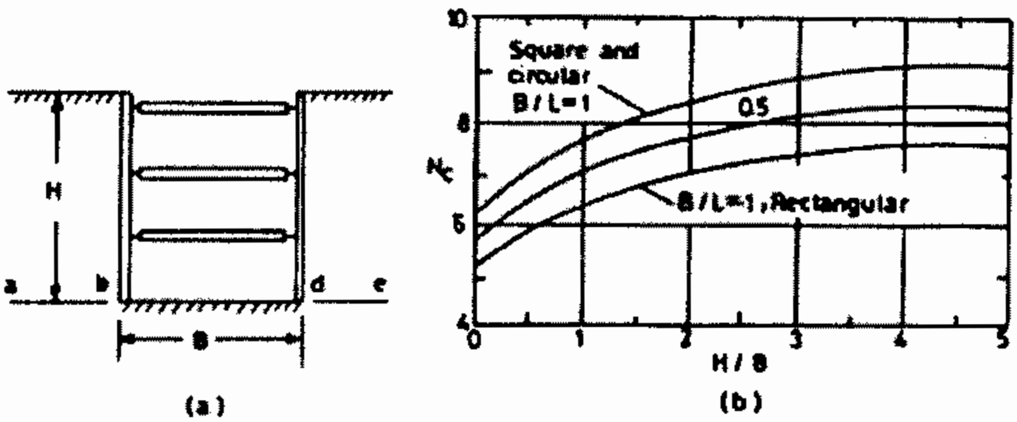
In cuts, when the first row of strut is placed, the depth of excavation is so small that for all practical problems, the original state of the stress is unaltered I.e. at rest condition. Thus the first row of struts is in position before any yielding of soil mass can take place. As the excavation proceeds to the level of the second row of struts, the horizontal yielding of the soil near the ground surface is prevented by the rigidity of the first set of struts. However the lateral pressure of the soil outside the cut acts on the sheet piles, resulting in their inward yield by rotating about a line at the level of the uppermost row of struts. Thus the placement of second row of struts is preceded by the horizontal yield as shown in the figure. As the depth of cutting increases, the yielding also increases.



From this discussion, it is clear that the yielding at the top of the strut level is insignificant to produce Rankine active condition. The soil at the bottom portion is only in the plastic condition to produce the Rankine active condition. Hence, the Rankine theory cannot be applied in the analysis of braced cut.

4.4.Heave of the bottom of cut in soft clays

The bottom of the cuts in soft clays are likely to foil by heaving. as the weight of the blocks of clay adjoining the cut tend to displace the underlying soft clay towards the excavation. The surcharge at the level **ab** and **de** in figure below is equal to the weight of clay above these levels. These strips **ab** and **de** act as footings.



Thus if the bearing capacity of the clay at the base is exceeded, the bottom of the cut fails by heaving. As the ultimate bearing capacity of day for ϕ_u=0 condition is equill to c_uN_c, the safety factor against the heaving is given by;

$$F = \frac{c_u N_c}{\gamma H} \qquad F_s = \frac{5.7c_u}{\gamma H}$$

This tendency of the bottom of cut to heave reduces if the depth of sheet pile extends below the bottom of the cut This is because of the higher stiffness of the sheet pile and this depends upon the location of the hard soil below the depth of cutting;

4.5. Strut load

The bracing for a cut are composed of several components and if failure of one unit occurs then the failure of whole system occurs. The important member of bracing is strut and the estimation of the strut load is important. If a strut fails by overstressing then the load carried by this strut is shared by remaining struts which consequently fails by overloading. Thus the size of strut should be so selected that it can take maximum estimated load on any strut, which type of design is known as conservative design. The load taken by strut is the function of;

- The deformation condition
- The forces with which the wedges are driven
- The time-elapse between the excavation of cut and the installation of supports.

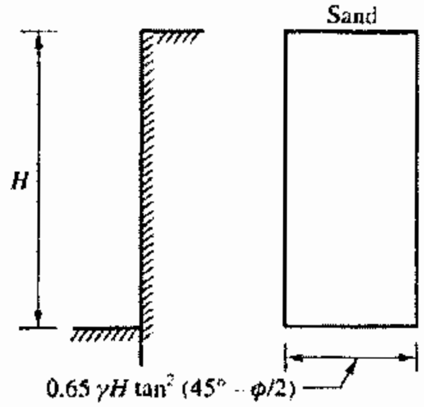
For simplification the forces of the strut is calculated using the procedure given by Terzaghi and Peck as follows:

- It is assumed that the load on each strut is equal to the total earth pressure acting on the sheeting over a rectangular area extending horizontally half the distance to the next vertical row of struts on either side and also vertically half the distance between the horizontal sets of struts.
- The earth pressure is assumed to be uniformly distributed over the rectangular area.
- For purposes of calculations, the bottom of the cut is assumed to be a strut.

Due to the continuity of the sheeting and the assumption regarding the pressure near the bottom of the cut, the computed earth pressure diagram and real distribution may differ significantly. Also, in cohesionless soil, the earth pressure at the ground surface must be zero. So, the pressure diagram is worked out from the measured strut load which is known as apparent earth pressure diagram.

4.6. Deep cuts in sand

Measurement from different cities of world by different researchers it is recommended to use Rankine solution taking the maximum pressure of $.65K\gamma H$ for the entire pressure diagram for deep cuts in sand as shown below in the figure.



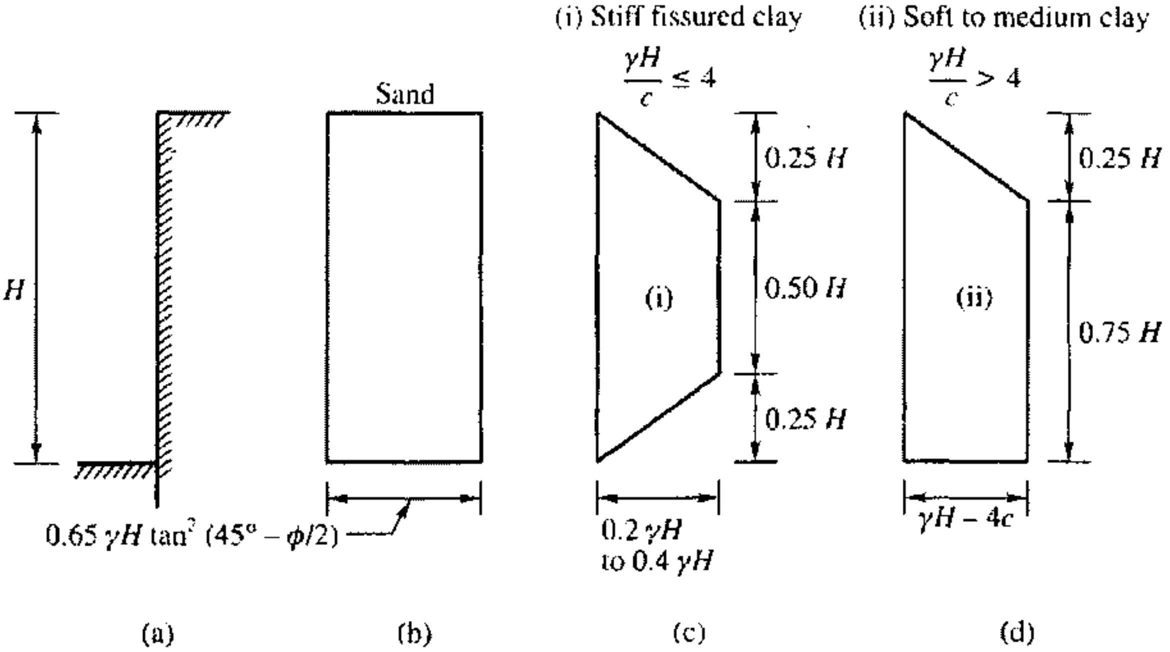
4.7. Deep cut in saturated, soft to medium days

From measurements of strut loads in cuts in medium to soft clays in different cities of world with excavation made rapidly, the condition ϕ_u=0 applies to a clay soil and only undrained shear strength c_u is used in the analysis. If there is no possibility of failure by bottom heave, failure by buckling of struts remains the principal type of failure to be possible.

From experience, when $\gamma H/c_u$, is less than 6, movements of bracing system and heave of tile clay are small. If $\gamma H/c_u$, approaches about 8, the movements of even well-designed bracing system becomes very large. At $\gamma H/c_u$, exceeding 8, the bracing is likely to collapse because of large inward movements of clay outside the embedded portion of the sheet piles and excessive upward heave of clay beneath the excavation.

For the case of clay soil when the magnitude of $\gamma H/c_u \leq 4$,the apparent pressure diagram shown if figure (c) is used and the magnitude of peak earth pressure is taken as average of 0.3γH.

In case the value of $\gamma H/c_u > 4$, the apparent pressure diagram of figure (d) is used.



Figures - Apparent pressure diagram for calculating loads in struts of braced cuts: (a) sketch of wall of cut. (b) diagram for cuts in drv or moist sand. (c) diagram for clays if $\gamma H/c_u \leq 4$ (d) diagram for clays if $\gamma H/c_u > 4$. where c is the average undrained shearing strength of the soil (Peck. 19691

Example

A long trench is excavated in medium dense sand for the foundation of a multistorey building. The sides of the trench are supported with sheet pile walls fixed in place by struts and wales as shown in Fig. The soil properties are:

$\gamma = 18.5$ kN/m^3, $c = 0$ and $\phi = 38°$

Determine: (a) The pressure distribution on the walls with respect to depth.

(b) Strut loads. The struts are placed horizontally at distances $L = 4$ m center to center.

(c) The maximum bending moment for determining the pile wall section.

(d) The maximum bending moments for determining the section of the wales.

Solution

(a) For a braced cut in sand use the apparent pressure envelope given in equation for p_a is

$$p_a = 0.65\ \gamma \text{H}\ K_A = 0.65 \times 18.5 \times 8 \tan^2(45 - 38/2) = 23 \text{ kN/m}^2$$

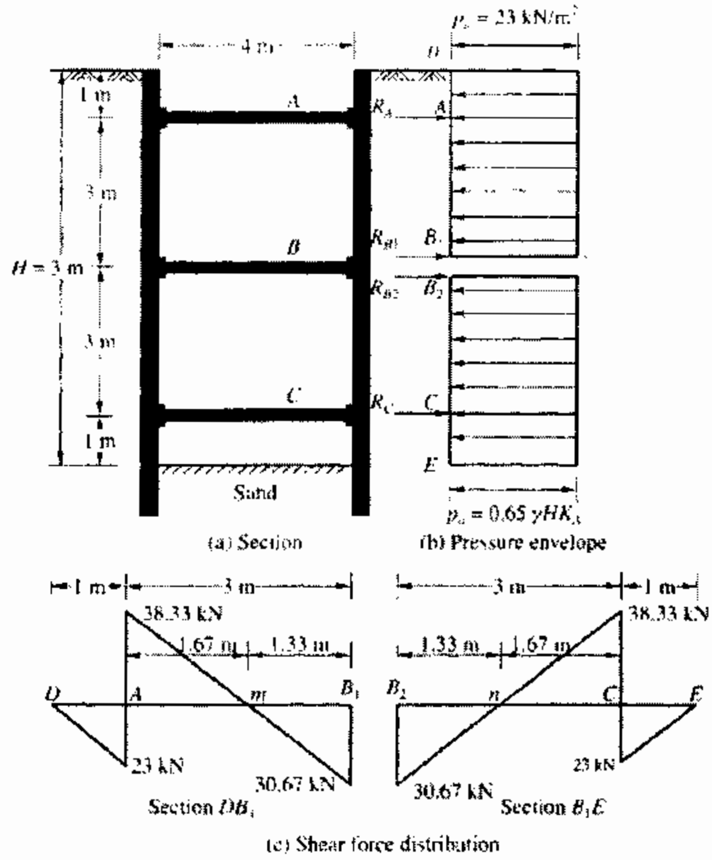


Figure - EX 20.17

(b) Strut loads

The reactions at the ends of struts A, B and C are represented by R_A, R_B and R_c respectively

For reaction R_A, take moments about B

$$R_A \times 3 = 4 \times 23 \times \frac{4}{2} \text{ or } R_A = \frac{184}{3} = 61.33 \text{ kN}$$

$R_{B1} = 23 \times 4 - 61.33 = 30.67$ kN

Due to the symmetry of the load distribution,

$R_{B1} = R_{B2} = 30.67$ kN, and $R_A = R_C = 61.33$ kN.

Now the strut loads are (for $L = 4$ m)

Strut A, $P_A = 61.33 \times 4 \approx 245$ kN

Strut B, $P_B = (R_{B1} + R_{B2}) \times 4 = 61.34 \times 4 = 245$ kN

Strut C, $P_C = 245$ kN

(c) Moment of the pile wall section

To determine moments at different points it is necessary to draw a diagram showing the shear force distribution.

Consider sections DB_1 and B_2E of the wall in Fig. Ex. 20.17(b). The distribution of the shear forces are shown in Fig. 20.17(c) along with the points of zero shear.

The moments at different points may be determined as follows

$$M_A = \frac{1}{2} \times 1 \times 23 = 11.5 \text{ kN- m}$$

$$M_C = \frac{1}{2} \times 1 \times 23 = 11.5 \text{ kN- m}$$

$$M_m = \frac{1}{2} \times 1.33 \times 30.67 = 20.4 \text{ kN- m}$$

$$M_n = \frac{1}{2} \times 1.33 \times 30.67 = 20.4 \text{ kN- m}$$

The maximum moment M_{max} = 20.4 kN-m. A suitable section of sheet pile can be determined as per standard practice.

(d) Maximum moment for wales

The bending moment equation for wales is

$$M_{max} = \frac{RL^2}{8}$$

where R = maximum strut load = 245 kN

L = spacing of struts = 4 m

$$M_{max} = \frac{245 \times 4^2}{8} = 490 \text{ kN-m}$$

A suitable section for the wales can be determined as per standard practice.

Example 20.18

Fig. Ex. 20.18a gives the section of a long braced cut. The sides are supported by steel sheet pile walls with struts and wales. The soil excavated at the site is stiff clay with the following properties

c = 800 lb/ft^2, $\phi = 0$, γ = 115 lb/ft^3

Determine: (a) The earth pressure distribution envelope.

(b) Strut loads.

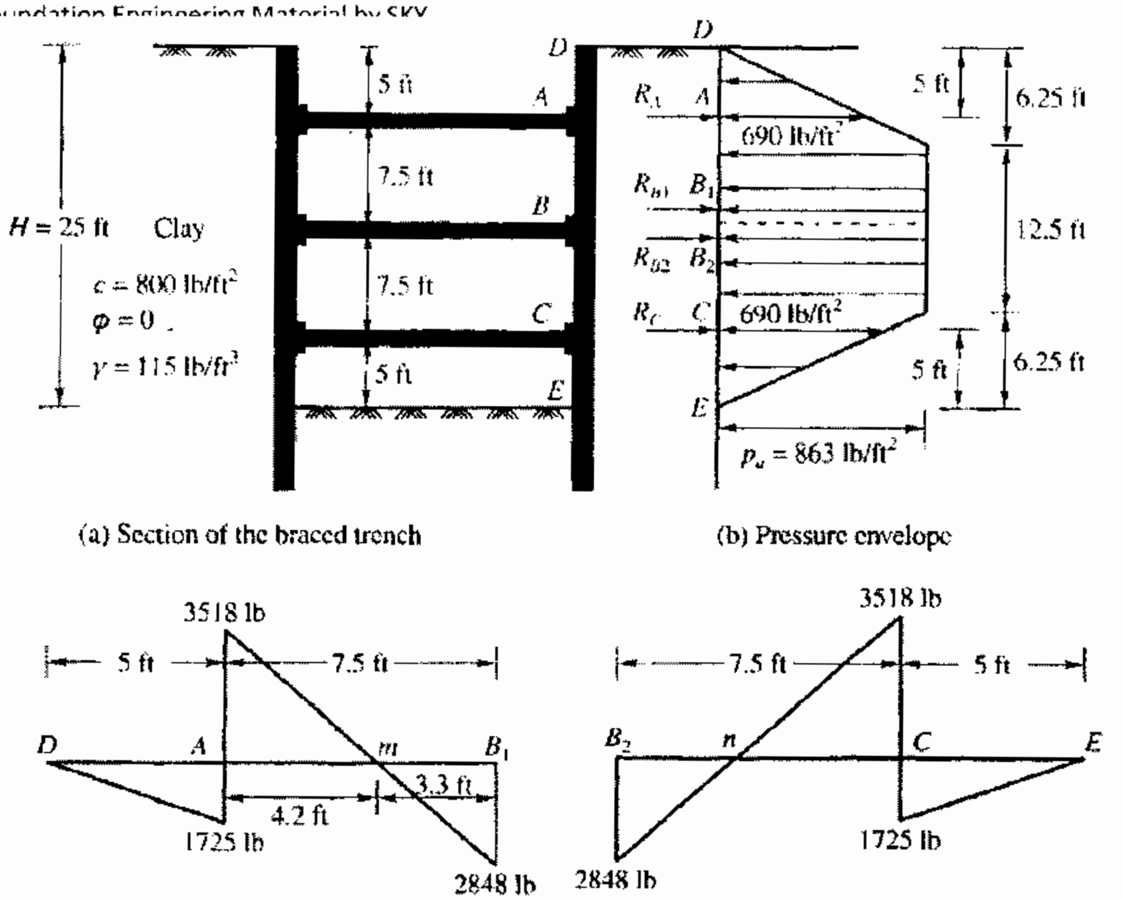
(c) The maximum moment of the sheet pile section.

The struts are placed 12 ft apart center to center horizontally.

Solution

(a) The stability number N_s

$$N_s = \frac{\gamma H}{c} = \frac{115 \times 25}{800} = 3.6 < 4$$



(a) Section of the braced trench

(b) Pressure envelope

(c) Shear force diagram

Figure Ex. 20.18

The soil is stiff fissured clay. As such the pressure envelope shown in Fig. 20.28(c) is applicable. Assume $p_a = 0.3\,\gamma H$

$p_a = 0.3 \times 115 \times 25 = 863$ lb/ft²

The pressure envelope is drawn as shown in Fig. Ex. 20.18(b).

(b) Strut loads

Taking moments about the strut head B_1 (B)

$$R_A \times 7.5 = \frac{1}{2} 863 \times 6.25\left(\frac{6.25}{3} + 6.25\right) + 863 \times \frac{(6.25)^2}{2}$$

$= 22.47 \times 10^3 + 16.85 \times 10^3 = 39.32 \times 10^3$

$R_A = 5243$ lb/ft

$$R_{B1} = \frac{1}{2} \times 863 \times 6.25 + 863 \times 6.25 - 5243 = 2848 \text{ lb/ft}$$

Due to symmetry

$R_A = R_C = 5243$ lb/ft

$R_{B2} = R_{B1} = 2848$ lb/ft

Strut loads are:

$P_A = 5243 \times 12 = 62{,}916$ lb $= 62.92$ kips

$P_B = 2 \times 2848 \times 12 = 68{,}352$ lb $= 68.35$ kips

$P_C = 62.92$ kips

(c) Moments

The shear force diagram is shown in Fig. 20.18c for sections DB_1 and B_2E

Moment at $A = \frac{1}{2} \times 5 \times 690 \times \frac{5}{3} = 2{,}875$ lb-ft/ft of wall

Moment at $m = 2848 \times 3.3 - 863 \times 3.3 \times \frac{3.3}{2} = 4699$ lb-ft/ft

Because of symmetrical loading

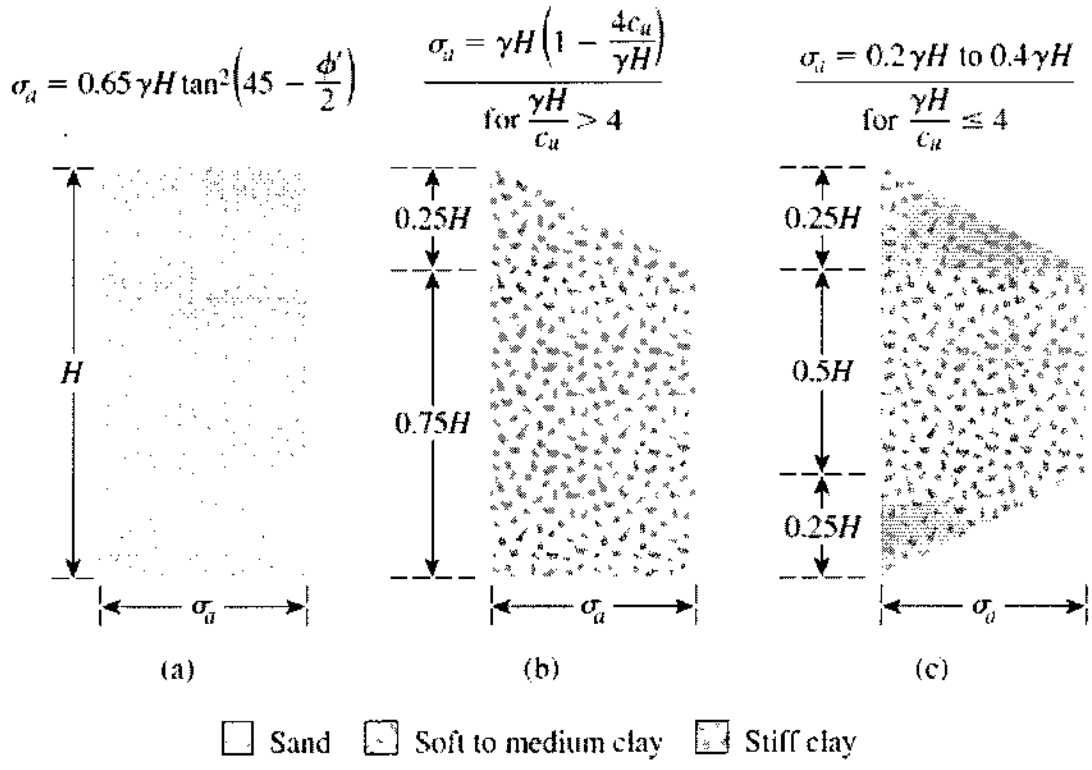
Moment at A = Moment at C = 2875 lb-ft/ft of wall

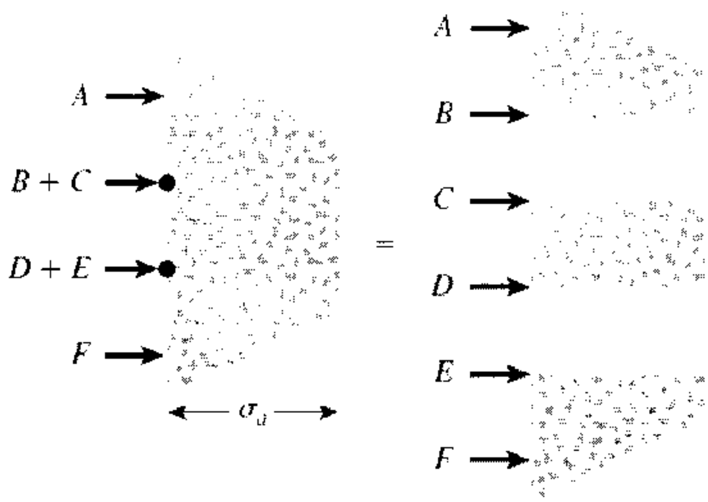
Moment at m = Moment at n = 4699 lb-ft/ft of wall

Hence, the maximum moment = 4699 lb-ft/ft of wall.

The section modulus and the required sheet pile section can be determined in the usual way.

Example from B M Das.





Example

A 7-m-deep braced cut in sand is shown in Figure 14.13. In the plan, the struts are placed at $s = 2$ m center to center. Using Peck's empirical pressure diagram, calculate the design strut loads.

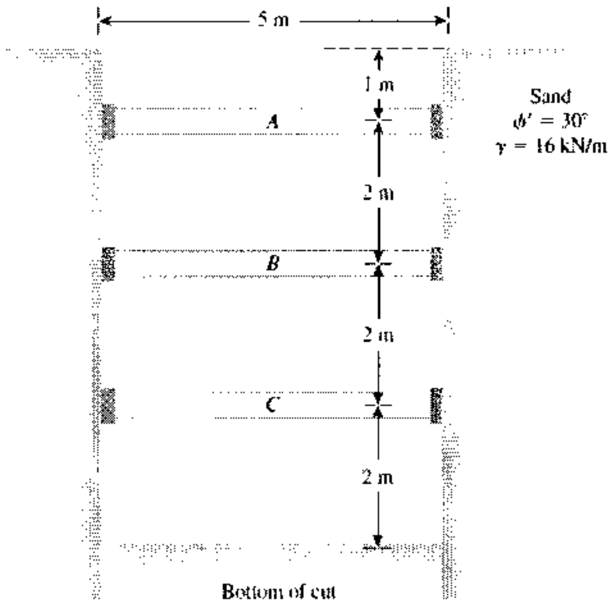


Figure 14.13 Braced cut in sand

Solution

Refer to Figure 14.11a. For the lateral earth pressure diagram,

$$\sigma_a = 0.65\gamma H \tan^2\left(45 - \frac{\phi'}{2}\right) = (0.65)(16)(7)\tan^2\left(45 - \frac{30}{2}\right) = 24.27\text{kN/m}^2$$

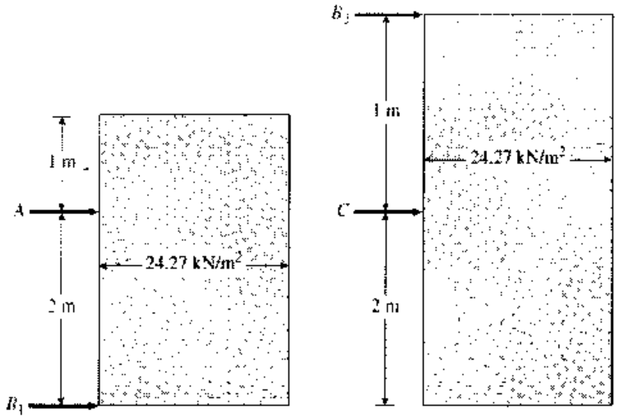


Fig. 14.14 Calculation of strut loads from pressure envelope

Assume that the sheeting is hinged at strut level B. Now refer to the diagram in Figure 14.14 We need to find reactions at A, B_1, B_2, and C. Taking the moment about B_1, we have

$$2A = (24.27)(3)\left(\frac{3}{2}\right); \quad A = 54.61\,\text{kN/m}$$

Hence.

$$B_1 = (24.27)(3) - 54.61 = 18.2\,\text{kN/m}$$

Again, taking the moment about B_2, we have

$$2C = (24.27)(4)\left(\frac{4}{2}\right)$$

$$C = 97.08\,\text{kN/m}$$

So,

$$B_2 = (24.27)(4) - 97.08 = 0$$

The strut loads are as follows:

At level A: $(A)(s) = (54.61)(2) = \mathbf{109.22\ kN}$

At level B: $(B_1 + B_2)(s) = (18.2 + 0)(2) = \mathbf{36.4\ kN}$

At level C: $(C)(s) = (97.08)(2) = \mathbf{194.16\ kN}$

Chapter – 6 Bearing Capacity and Settlement of Shallow Foundation

6.1. Introduction

Bearing capacity is the power of foundation soil to hold the forces from the superstructure without undergoing shear failure or excessive settlement. Foundation soil is that portion of ground which is subjected to additional stresses when foundation and superstructure are constructed on the ground.

Factors influencing Bearing Capacity:

Bearing capacity of soil depends on many factors. The following are some important ones.

1. Type of soil
2. Unit weight of soil
3. Surcharge load
4. Depth of foundation
5. Mode of failure
6. Size of footing
7. Shape of footing
8. Depth of water table
9. Eccentricity in footing load
10. Inclination of footing load
11. Inclination of ground
12. Inclination of base of foundation

The following are a few important terminologies related to bearing capacity of soil.

6.2. Basic definition and their relationship

❖ **Ultimate bearing capacity (q_u):**
The ultimate bearing capacity is the gross pressure at the base of the foundation at which soil fails inshear.

❖ **Net ultimate Bearing Capacity (q_{nu}):**
It is the net increase in pressure at the base of foundation that cause shear failure of the soil.

$q_{nu} = q_u - \gamma D_f$ (overburden pressure)

❖ **Net Safe Bearing Capacity (q_{ns}) :** It is the net soil pressure which can be safely applied to the soil considering only shear failure.

Thus, $q_{ns}=q_{nu}/FOS$ *(Factor of Safety usually taken as 2 to 3)*

❖ **Gross Safe Bearing Capacity (q_s) :**
It is the maximum pressure which the soil can carry safely without shear failure.

$q_s = q_{nu} / FOS + \gamma Df$

❖ **Net Safe Settlement Pressure (q_{np}) :**
It is the net pressure which the soil can carry without exceeding allowable settlement.

Net Allowable Bearing Pressure (q_{na}):
It is the net bearing pressure which can be used for design of foundation.

Thus,

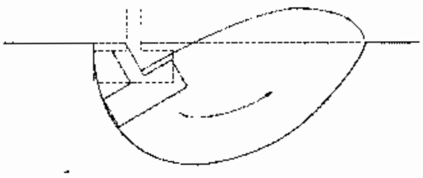
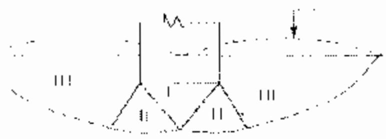
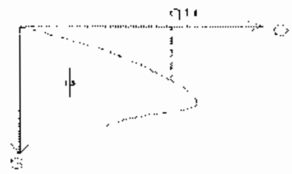
$q_{na} = q_{ns}$; if $q_{np} > q_{ns}$

$q_{na} = q_{np}$; if $qns > q_{np}$

It is also known as Allowable Soil Pressure (ASP).

6.3. Principle mode of soil failure or Types of shear failure:

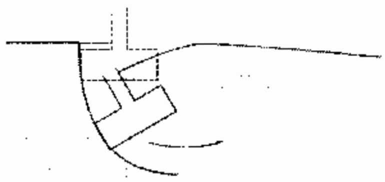
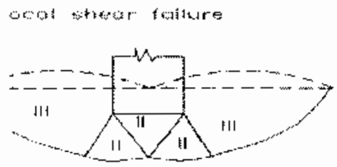
➢General shear failure: For Dense sand.

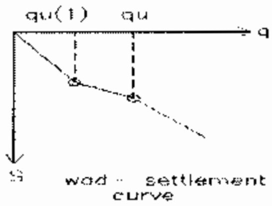




Strip footing resting on surface   Load –settlement curve of dense sand or stiff clay

- ❖ The load - Settlement curve in case of footing resting on surface of dense sand or stiff clays shows pronounced peak & failure occurs at very small stain.
- ❖ A loaded base on such soils sinks or tilts suddenly in to the ground showing a surface heave of adjoining soil
- ❖ The shearing strength is fully mobilized all along the slip surface & hence failure planes are well defined.
- ❖ The failure occurs at very small vertical strains accompanied by large lateral strains.
- ❖ $I_D > 65$,N>35, $\Phi > 36^0$, $e < 0.55$

➢Local shear failure: For Medium compaction soil.

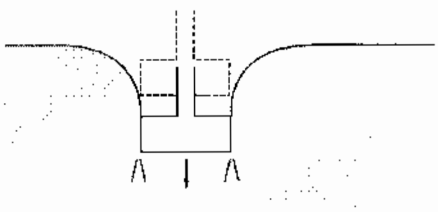
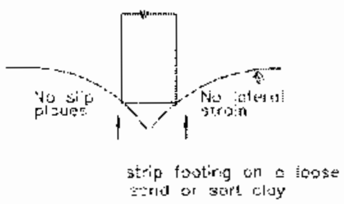


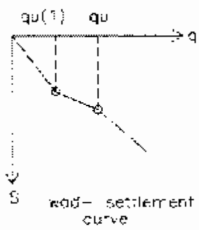
Strip footing resting on surface   Load –settlement curve of Medium sand or Medium clay when load is

equal to a certain value qu(1),

- ❖ The foundation movement is accompanied by sudden jerks.
- ❖ The failure surface gradually extends out wards from the foundation.
- ❖ The failure starts at localized spot beneath the foundation & migrates out ward part by part gradually leading to ultimate failure.
- ❖ The shear strength of soil is not fully mobilized along planes & hence failure planes are not defined clearly.
- ❖ The failure occur at large vertical strain & very small lateral strains.
- ❖ I_D = 15 to 65 , N=10 to 30 , Φ <30, e>0.75

➢ Punching shear failure: For loose soil



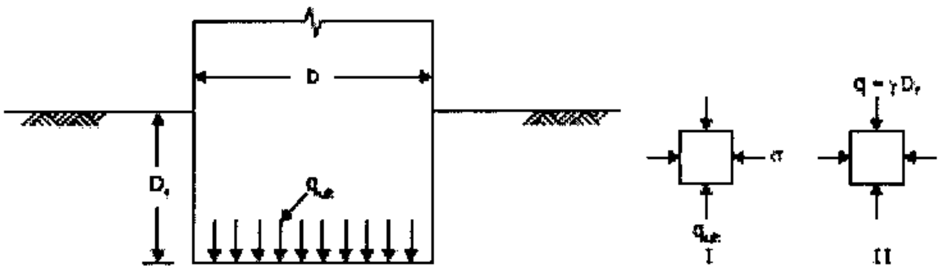
The loaded base sinks into soil like a punch.

The failure surface does not extend up to the ground surface.
No heave is observed. Large vertical strains are involved with practically no lateral deformation.
Failure planes are difficult to locate.

6.4 Bearing Capacity by classical Earth pressure theory of

The classical earth pressure theory assumes that on exceeding a certain stress condition. rupture surfaces are formed in the soil mass. The stress developed upon the formation of the rupture surfaces is treated as the ultimate bearing capacity of the soil.
The bearing capacity may be determined from the relation between the principal stresses at failure. The pertinent methods are those of Rankine, Pauker and Bell.

Rankine's Method
This method, based on Rankine's earth pressure theory, is too approximate and conservative for practical use. However, it is given just as a matter of academic interest.
Rankine uses the relationship between principal stresses at limiting equilibrium conditions of soil elements, one located just beneath the footing and the other just outside it as shown in Fig below.



Rankine's method for bearing capacity of a footing

In element *I*, just beneath the footing, at the base level of the foundation, the applied pressure *q*ult is the major principal stress; under its influence, the soil adjacent to the element tends get pushed out, creating active conditions. The active pressure is σ on the vertical faces to the element. From the relationship between the principal stresses at limiting equilibrium relating to the active state, we have:

$$\sigma = q_{\text{ult}} \cdot K_A = q_{\text{ult}} \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right)$$

In element *II*, just outside the footing, at the base level of the foundation, the tendency of the soil adjacent to the element is to compress, creating passive conditions. The pressure σ on the vertical faces of the element will thus be the passive resistance. This will thus be the major principal stress and the corresponding minor principal stress is $q(= \gamma Df)$, the vertical stress caused by the weight of a soil column on it, or the surcharge dut to the depth of the foundation. From the relationship between the principal stresses at limiting equilibrium relating to the passive state, we have,

$$\sigma = q \cdot K_p = \gamma D_f \cdot K_p = \gamma D_f \left(\frac{1 + \sin \phi}{1 - \sin \phi} \right)$$

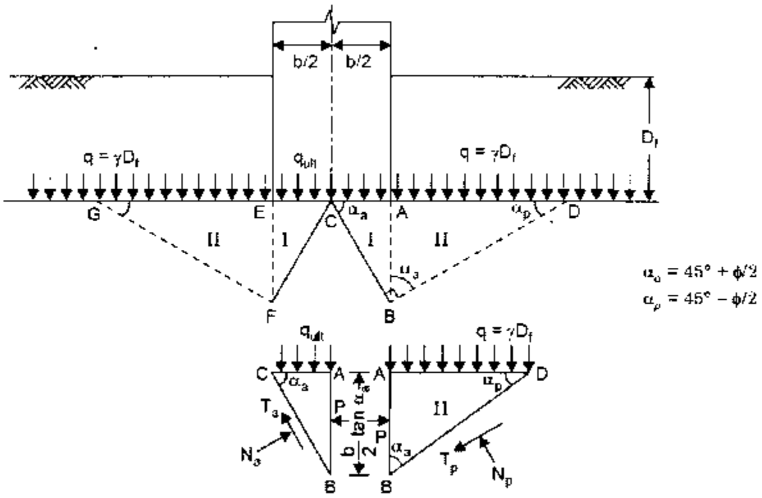
The two values of σ may be equated to get a relationship for q_{ult}:

$$q_{\text{ult}} = \gamma D_f \left(\frac{1 + \sin \phi}{1 - \sin \phi} \right)^2 \quad \text{or,} \quad D_f = \frac{q}{\gamma} \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right)^2$$

This gives the bearing capacity of the footing. It does not appear to take into account the size of the footing. Further the bearing capacity reduces to zero for $D_f = 0$ or for a footing founded at the surface. This is contrary to facts.

An alternative approach based on Rankine's earth pressure theory which takes into account the size *b* of the footing is as follows:

It is assumed that rupture in the soil takes place along *CBD* and *CFG* symmetrically. The failure zones are made of two wedges as shown. It is sufficient to consider the equilibrium of one half.
Wedge *I* is Rankine's active wedge, pushed downwards by *q*ult on *CA*; consequently the vertical face *AB* will be pushed outward.
Wedge *II* is Rankine's passive wedge. The pressure *P* on face *AB* of wedge *I* will be the same as that which acts on face *AB* of wedge *II*; consequently, the soil wedge *II* is pushed up. The surcharge, $q = \gamma D_f$, due to the depth of footing resists this.



Rankine's method taking into account the size of the footing

From wedge *II*,

$$\overline{AB} = \frac{b}{2}\tan\alpha_a = \frac{b}{2}\tan(45° + \phi/2) = \frac{b}{2}\sqrt{N_\phi}$$

$$P = \frac{1}{2}\cdot\gamma\frac{b^2}{4}\cdot N_\phi^2 + \gamma D_f\cdot\frac{b}{2}N_\phi^{3/2}$$

from Rankine's theory for the case with surcharge. From Wedge *I*, similarly,

$$P = \frac{1}{2}\cdot\gamma\cdot\frac{b^2}{4}\cdot\frac{N_\phi}{N_\phi} + q_{ult}\cdot\frac{b}{2}\sqrt{N_\phi}\cdot\frac{1}{N_\phi} = \frac{1}{2}\gamma\cdot\frac{b^2}{4} + q_{ult}\cdot\frac{b}{2}\cdot\frac{1}{\sqrt{N_\phi}}$$

Equating the two values of P, we get

$$q_{ult} = \frac{1}{2}\cdot\gamma\cdot\frac{b}{2}\sqrt{N_\phi}\,(N_\phi^2 - 1) + \gamma D_f N_\phi^2$$

This is written as $q_{ult} = \frac{1}{2}\gamma\cdot bN_\gamma + \gamma D_f N_q^2$

where $$N_\gamma = \frac{1}{2}\sqrt{N_\phi}\,(N_\phi^2 - 1)$$

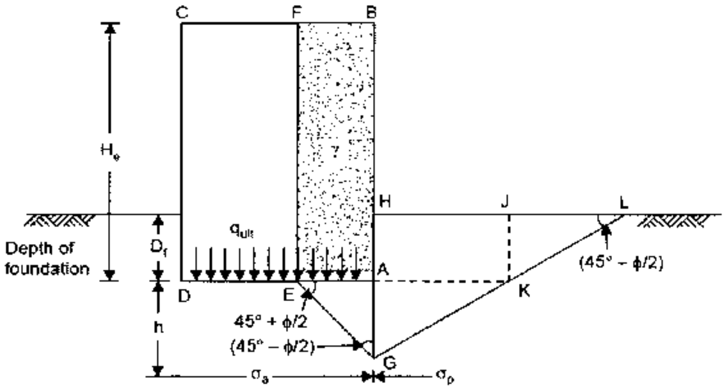
and $$N_q = N_\phi^2,$$

Both are known as "bearing capacity factors".

6.5 Pauker and Bell's bearing capacity theory of failure

Pauker's Theory

Colonel Pauker, a Russian military engineer, is credited to have derived one of the oldest formulae for the bearing capacity of a foundation in cohesionless soil and the minimum depth of foundation. He was supposed to have used his formula in the 1850's during the construction of fortifications and sea-batteries for the Czarist Naval base of Kronstadt (Pauker, 1889— reported by Jumikis, 1962). His theory was once very popular and was extensively used in Czarist Russia, before the revolution. The theory is set out below as (Figure):



Pauker's method of determination of bearing capacity

Pauker considered the equilibrium of a point say, G, in the soil mass underneath the base of the footing, as shown, at a depth h below the base, the depth of foundation being D_f below the ground surface. The strip foundation is assumed to transmit a pressure of q_{ult} to the soil at its base.

The classical earth pressure theory for an ideal soil is used under the following assumptions:

(*i*) The soil is cohesionless.

(*ii*) The contact pressure, q_{ult}, is replaced by an equivalent height, H_e, of soil of unit weight, γ, the same as that of the foundation soil:

$$H_e = \frac{q_{ult}}{\gamma}$$

(*iii*) At imminent failure, it is assumed that a part $AEFB$, obtained by drawing GE at $(45° - \phi/2)$ with respect to GA (G being chosen vertically below A), tears off from the rest of the soil mass.

(*iv*) Under the influence of the weight of the equivalent layer of height He, the soil to the left of the vertical section GA tends to be pushed out, inducing active earth pressure on GA.

(*v*) The soil to be right of GA tends to get compressed, thus offering passive earth resistance against the active pressure.

(*vi*) The equilibrium condition at G is determined by that of soil prisms GEA and $GHJK$. The friction of the soil on the imaginary vertical section, GA, is ignored. In other words, the earth pressures act normal to GA, *i.e.*, horizontally.

(*vii*) If sliding of soil from underneath the footing is to be avoided, the condition stated by Pauker is $\sigma_p \geq \sigma_a$

By Rankine's earth pressure theory,

$$\sigma_p = \gamma\,(Df + h)\tan^2(45° + \phi/2)$$
$$\sigma_a = \gamma\,(He + h)\tan^2(45° - \phi/2)$$

where ϕ is the angle of internal friction of the soil. And from above relationship the equations reduces to

$$\frac{(D_f + h)}{(H_e + h)} \geq \tan^4(45° - \phi/2)$$

[by dividing by $(H_e + h)\tan^2(45° + \phi/2)$ and noting that

$$\frac{\tan^2(45° - \phi/2)}{\tan^2(45° + \phi/2)} = \tan^4(45° - \phi/2).]$$

The most dangerous point G is that for which $\frac{(D_f + h)}{(H_e + h)}$ is a minimum.

By inspection, one can see that this is minimum when $h = 0$; that is to say, the critical point is A itself.

Thus,

$$\frac{D_f}{H_c} \geq \tan^4(45° - \phi/2)$$

This is known as Pauker's equation and is written as:

$$D_f = H_c \tan^4(45° - \phi/2)$$

or, noting,

$$H_c = \frac{q_{ult}}{\gamma}, D_f = \frac{q_{ult}}{\gamma} \cdot \tan^4(45° - \phi/2)$$

This may be written in the following form also:

$$q_{ult} = \gamma D_f \tan^4(45° + \phi/2)$$

In the first form it may be used to determine the minimum depth of foundation and in the second, to determine the ultimate bearing capacity.

Bell's Theory

Bell (1915) modified Pauker-Rankine formula to be applicable for cohesive soils; both friction and cohesion are considered in this equation. With reference to Rankine's method (explained above with figure), from, the stresses, on element *I*,

$$\sigma = q_{ult} \tan^2(45° - \phi/2) \quad 2c \tan(45° - \phi/2)$$

or

$$\sigma = \frac{q_{ult}}{N_\phi} - \frac{2c}{\sqrt{N_\phi}}$$

with the usual notation, $N_\phi = \tan^2(45° + \phi/2)$.

This is from the relationship between the principal stresses in the active Rankine state of plastic equilibrium.

From the stresses on element *II*,

$$\sigma = \gamma D_f \tan^2(45° + \phi/2) + 2c \tan(45° + \phi/2)$$

or

$$\sigma = \gamma D_f N_\phi + 2c\sqrt{N_\phi}$$

Equating the two values of σ for equilibrium, we have:

$$q_{ult} = \gamma D_f \tan^4(45° + \phi/2) + 2c \tan(45° + \phi/2)[1 + \tan^2(45° + \phi/2)]$$

or

$$q_{ult} = \gamma D_f N_\phi^2 + 2c\sqrt{N_\phi}(1 + N_\phi)$$

This is Bell's equation for the ultimate bearing capacity of a $c - \phi$ soil at a depth D_f. If $c = 0$, this reduces to

$$\sigma = \gamma D_f \tan^2(45° + \phi/2)$$

For pure clay, with $\phi = 0$, Bell's equation reduces to $q_{ult} = \gamma D_f + 4c$

If D_f is also zero, $q_{ult} = 4c$. This value of considered to be too conservative.

The limitation of Bell's equation that the size of the foundation is not considered may be overcome as in the case of Rankine's equation by considering soil wedges instead of elements.

6.6 Prandtl's theory of failure

Prandtl analysed the plastic failure in metals when punched by hard metal punchers (Prandtl, 1920). This analysis has been adapted to soil when loaded to shear failure by a relatively rigid foundation (Prandtl, 1921). The bearing capacity of a long strip footing on the ground surface may be determined by this theory, illustrated in figure below.

The assumptions in Prandtl's theory are:

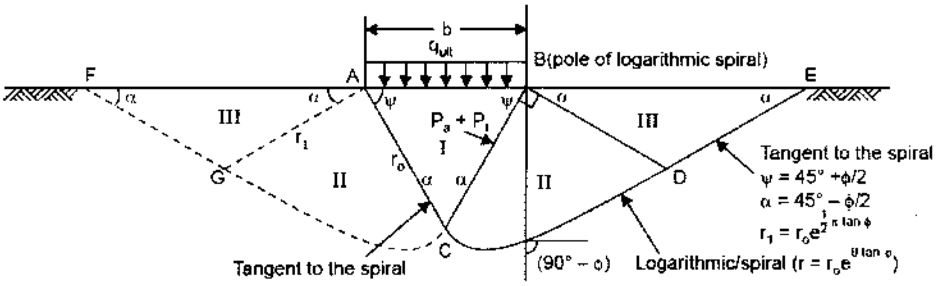
(i) The soil is homogeneous, isotropic and weightless.

(ii) The Mohr-Coulomb equation for failure envelope $\tau = c + \sigma \tan \phi$ is valid for the soil, as shown in Fig. (*b*).

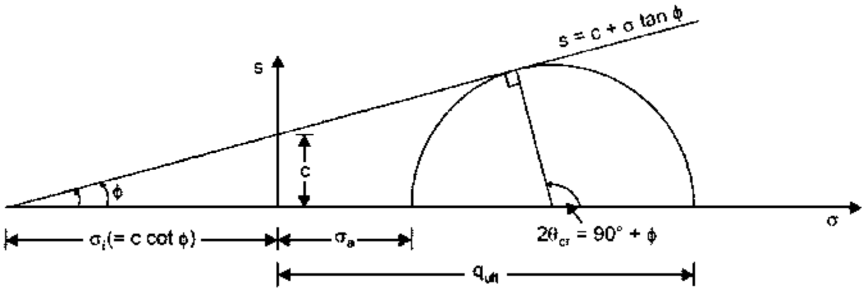
(iii) Wedges *I* and *III* act as rigid bodies. The zones in Sectors *II* deform plastically. In the plastic zones all radius vectors or planes through *A* and *B* are failure planes and the curved boundary is a logarithmic spiral.

(iv) Wedge *I* is elastically pushed down, tending to push zones *III* upward and outward, which is resisted by the passive resistance of soil in these zones.

(v) The stress in the elastic zone *I* is transmitted hydrostatically in all directions.



(a) Prandtl's system



(b) Mohr's circle for active zone

Prandtl's method of determining bearing capacity of a $c - \phi$ soil

It may be noted that the section is symmetrical up to the point of failure, with an equal chance of failure occurring to either side. (That is why the section to one side, say to the left, is shown by dashed lines). The equilibrium of the plastic sector is considered by Prandtl.

Let BC be r_0. The equation to a logarithmic spiral is:

$$r = r_0 \, e^{\theta \tan \phi} \text{ where } \theta \text{ is the spiral angle.}$$

$$\text{Then } BD = r_0 \, e^{(\pi/2) \tan \phi}, \text{ since } \angle CBD = 90° = \pi/2 \text{ rad.}$$

From the Mohr's circle for $c - \phi$ soil, Fig. (*b*), the normal stress corresponding to the cohesion intercept is:

$$\sigma_i = c \cot \phi$$

This is termed the 'initial stress', which acts normally to BC in view of assumption (*v*); also q_{ult}, the applied pressure is assumed to be transferred normally on to BC. Thus the force on BC is

$$(\sigma_i + q_{ult})\ \overrightarrow{BC} \text{ or } r_0(\sigma_i + q_{ult})$$

Moment, M_0, of this force about B is

$$r_0\,(\sigma_i + q_{ult}) \times \frac{r_0}{2}$$

Substituting for σ_i,

$$M_0 = \frac{r_0^2}{2}\ (c \cot \phi + q_{ult}), \text{ counterclockwise}$$

The passive resistance P_p on the face BD is given by

$$P_p = \sigma_i \cdot N_\phi \cdot \overrightarrow{BD}$$

where $N_\phi = \tan^2 (45° + \phi/2) = \dfrac{1+\sin\phi}{1-\sin\phi}$

This is because σ_i, due to cohesion alone is transmitted by the wedge BDE. Its moment about B, M_r, is,

$$M_r = P_p \cdot \frac{\overrightarrow{BD}}{2} = \sigma_i N_\phi \cdot \frac{(\overrightarrow{BD})^2}{2} = \cot\phi \cdot N_\phi \cdot \frac{1}{2}\, r_0^2\, e^{\pi \tan\phi}$$

For equilibrium of the plastic zone, equating M_0 and M_r, and rearranging,

$$q_{ult} = c \cot\phi\ (N_\phi \cdot e^{\pi \tan\phi} - 1)$$

This is Prandtl's expression for ultimate bearing capacity of a $c - \phi$ soil. Apparently this leads one to the conclusion that if $c = 0$, $q_{ult} = 0$. This is ridiculous since it is well known that even cohesionless soils have bearing capcity. This anomaly arises chiefly owing to the assumption that the soil is weightless. This was later rectified by Terzaghi and Taylor.

For purely cohesive soils, $\phi = 0$ and the logarithmic spiral becomes a circle and Prandtl's analysis for this special case leads to an indeterminate quantity. But, by applying L' Hospital's rule, for taking limit one finds that

$$q_{ult} = (\pi + 2)c = 5.14c$$

Discussion of Prandtl's Theory

(*i*) Prandtl's theory is based on an assumed compound rupture surface, consisting of an arc of a logarithmic spiral and tangents to the spiral.

(*ii*) It is developed for a smooth and long strip footing, resting on the ground surface.

(*iii*) Prandtl's compound rupture surface corresponds fairly well with the mode of failure along curvilinear rupture surfaces observed from experiments. In fact, for $\phi = 0°$, Prandtl's rupture surface agrees very closely with Fellenius' rupture surface (Taylor, 1948).

(*iv*) Although the theory is developed for a $c - \phi$ soil, the original Prandtl expression for bearing capacity reduces to zero when $c = 0$, contradicting common observations in reality. This anomaly arises from the fact that the weight of the soil wedge directly beneath the base of the footing is ignored in Prandtl's analysis.

This anomaly is sought to be rectified by the Terzagthi/Taylor correction.

(*v*) For a purely cohesive soil, $\phi = 0$, and Prandtl's equation, at first glance, leads to an indeterminate quantity; however this difficulty is overcome by the mathematical technique of evaluating a limit under such circumstances.

Then, for $\phi = 0$, $q_{ult} = (2 + \pi)c = 5.14\ c$

(*vi*) Prandtl's expression, as originally derived, does not include the size of the footing.

6.7 Terzaghi's method of determining bearing capacity of soil

Terzaghi's method is, in fact, an extension and improved modification of Pandtl's (Terzaghi, 1943). Terzaghi considered the base of the footing to be rough, which is nearer facts, and that it is located at a depth Df below the ground surface ($Df \leq b$, where b is the width of the footing).

Assumption for Terzaghi's theory:

- ❖The foundation is considered to be shallow if $(D_f \leq B)$; in recent studies the foundation is considered to be shallow if $(D_f / B \leq 4)$. Otherwise it is considered to be deep foundation.
- ❖Foundation is considered to be strip if $(B / L \to 0.00)$.
- ❖The soil from ground surface to the bottom of the foundation is replaced by stress $q = \gamma D_f$
- ❖Soil is homogeneous and Isotropic.
- ❖ The shear strength of soil is represented by Mohr Coulombs Criteria.
- ❖The footing is of strip footing type with rough base. It is essentially a two dimensional plane strain problem.
- ❖Elastic zone has straight boundaries inclined at an angle equal to ϕ to the horizontal.
- ❖Failure zone is not extended above, beyond the base of the footing. Shear resistance of soil above the base of footing is neglected.
- ❖Method of superposition is valid.
- ❖Passive pressure force has three components (P_{PC} produced by cohesion, P_{Pq} produced by surcharge and $P_{p\gamma}$ produced by weight of shear zone).
- ❖Effect of water table is neglected.
- ❖Footing carries concentric and vertical loads.
- ❖Footing and ground are horizontal.
- ❖Limit equilibrium is reached simultaneously at all points. Complete shear failure is mobilized at all points at the same time.
- ❖The properties of foundation soil do not change during the shear failure

Limitations:

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

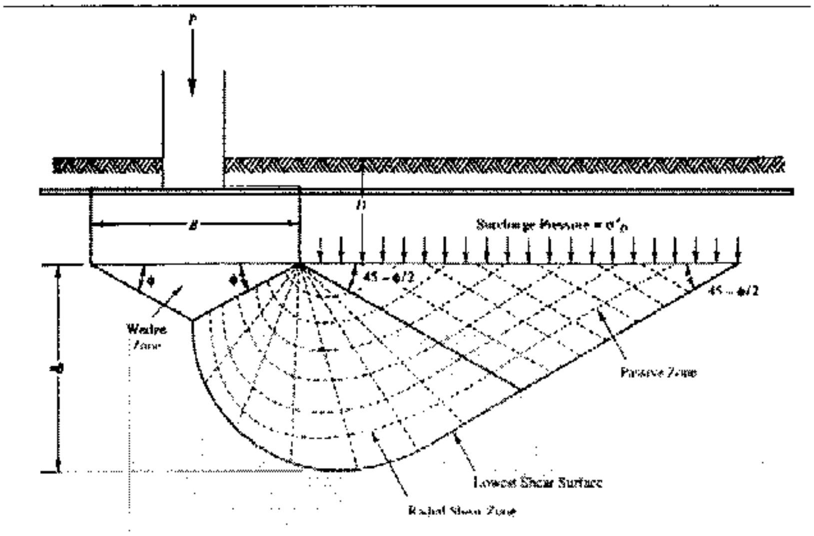
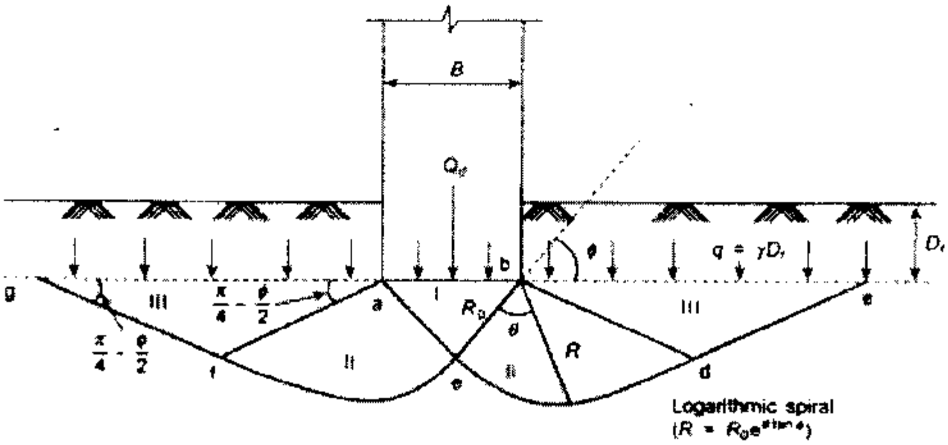
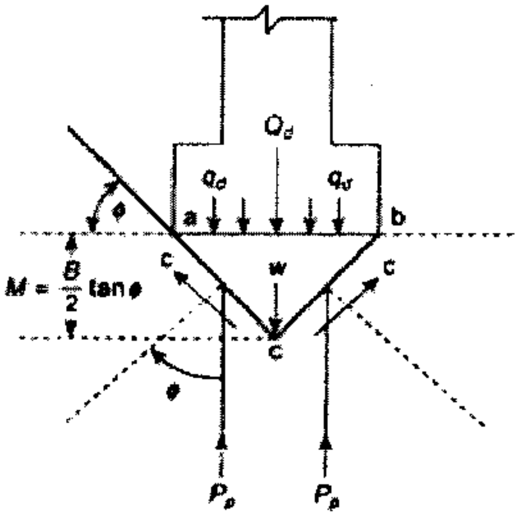


Fig : Terzaghi's concept of Footing with five distinct failure zones in foundation soil.

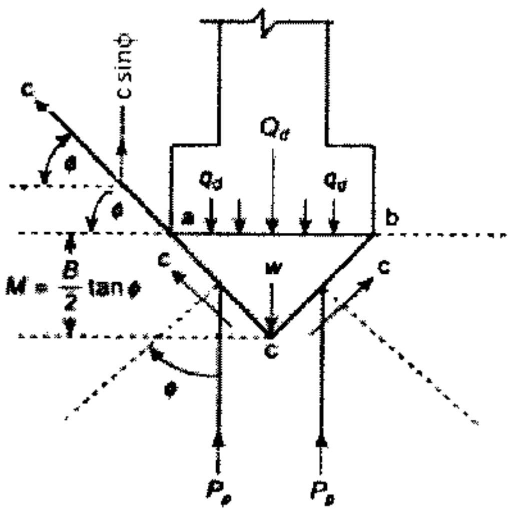
A strip footing of width B gradually compresses the foundation soil underneathdue to the vertical load from superstructure. Let qf be the final load at which thefoundation soil experiences failure due to the mobilization of plastic equilibrium.The foundation soil fails along the composite failure surface and the region is divided in to five zones. Zone 1 which is elastic, two numbers of Zone 2 which are the zones of radial shear and two zones of Zone 3 which are the zones oflinear shear. Considering horizontal force equilibrium and incorporatingempirical relation,the equation for ultimate bearing capacity is obtained as follows.



(a) Failure surface and zone



(b) Forces on elastic wedge



(b) Forces on elastic wedge

The failure zones do not extend above the horizontal plane passing through base of footing

- The failure occurs when the down ward pressure exerted by loads on the soil adjoining the inclined surfaces on soil wedge is equal to upward pressure.
- Downward forces are due to the load ($=q_u \times B$) & the weight of soil wedge ($=\frac{1}{2} \times \gamma \times B \times B/2 \tan\emptyset = \frac{1}{4} \gamma B^2 \tan\emptyset'$) [$q_u = q_d$]
- Upward forces are the vertical components of resultant passive pressure (Pp) & the cohesion (c') acting along the both inclined surfaces. The vertical component of c' will be c'sinϕ'.

For equilibrium:

Σ **Fv = 0**

$$\frac{1}{4} \gamma B^2 \tan\emptyset + q_u \times B = 2Pp + 2c' \times Li \sin\emptyset'$$

where Li = length of inclined surface **cb** or **ca**
(**cb**=**ca** = B/2 cosø')

Therefore,

$q_u \times B \quad 2Pp + Bc' \tan\phi - \frac{1}{4}\gamma B^2 \tan\phi'$ ....(i)

The resultant passive pressure (Pp) on the surface cb & ca constitutes three components i.e. $(Pp)_r$, $(Pp)_c$ & $(Pp)_q$.

Thus,

$Pp = (Pp)_r - (Pp)_c + (Pp)_q$......................................ii

Then equation (i) becomes

$q_u \times B = 2[(Pp)_r + (Pp)_c - (Pp)_q] + Bc' \tan\phi - \frac{1}{4}\gamma B^2 \tan\phi'$

Substituting; $2(Pp)r - \frac{1}{4}rB^2 \tan\phi' = B \times \frac{1}{2}\gamma BNr$ iii

$2(Pp)q - B \times \gamma D Nq$ iv

& $2(Pp)c + Bc' \tan\phi' = B \times c' Nc;$ v

We get,

$qu = c' Nc + \gamma Df Nq + 0.5 \gamma B N\gamma$........................vi

This is Terzaghi's Bearing capacity equation for determining ultimate bearing capacity of strip footing. Where Nc, Nq & Nr are Terzaghi's bearing capacity factors & depends on angle of shearing resistance (ϕ).

6.8 Effect of water table on bearing Capacity

The basic theory of bearing capacity is derived by assuming the water table tobe at great depth below and not interfering with the foundation. However, the presence of water table at foundation depth affects the strength of soil. Further, the unit weight of soil to be considered in the presence of water table is submerged density and not dry density. Hence, the reduction coefficients $R_{W1}$and R_{W2} are used in second and third terms of bearing capacity equation toconsider the effects of water table.

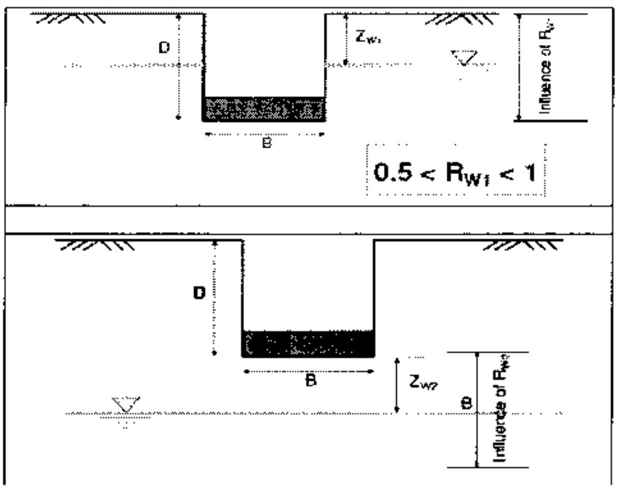


Fig : Effect of water table on bearing capacity

Ultimate bearing capacity with the effect of water table is given by,

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

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

where Z_{w1} is the depth of water table from ground level

1. $0.5 < R_{w1} < 1$
2. When water table is at the ground level ($Z_{w1} = 0$), $R_{w1} = 0.5$
3. When water table is at the base of foundation ($Z_{w1} = D$), $R_{w1} = 1$
4. At any other intermediate level, R_{w1} lies between 0.5 and 1

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

where Z_{w2} is the depth of water table from foundation level.

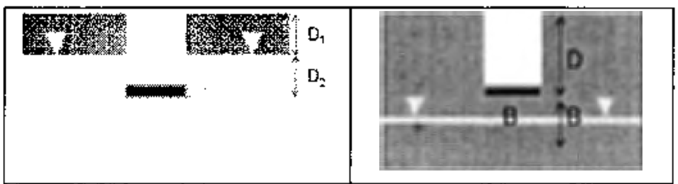
1. $0.5 < R_{w2} < 1$
2. When water table is at the base of foundation ($Z_{w2} = 0$), $R_{w2} = 0.5$
3. When water table is at a depth B and beyond from the base of foundation ($Z_{w2} >= B$), $R_{w2} = 1$
4. At any other intermediate level, R_{w2} lies between 0.5 and 1

Density of soil :

In geotechnical engineering, one deals with several densities such as dry density, bulk density, saturated density and submerged density. There will always be a doubt in the students mind as to which density to use in a particular case. In case of Bearing capacity problems, the following methodology may be adopted.

1. Always use dry density as it does not change with season and it is always smaller than bulk or saturated density.
2. If only one density is specified in the problem, assume it as dry density and use.
3. If the water table correction is to be applied, use saturated density in stead of dry density. On portions above the water table, use dry density.
4. If water table is somewhere in between, use equivalent density as follows. In the case shown in Fig. γ_{eq} should be used for the second term and γ_{sat} for the third term. In the case shown in Fig. γ_d should be used for second term and γ_{eq} for the third term

$$\gamma_{eq} = \frac{\gamma_1 D_1 + \gamma_2 D_2}{D_1 + D_2}$$



(a) Water table above base	(b) Water table below base

Figure: Evaluation of equivalent density

6.9 . Extension of Terzaghi's bearing capacity Theory

For General shear failure:

Type of foundation	Ultimate bearing capacity q_u
Strip Footing	$q_u = cN_c + qN_q + \frac{1}{2}\gamma BN_\gamma$
Square footing	$q_u = 1.3cN_c + qN_q + 0.4\gamma BN_\gamma$
Circular footing	$q_u = 1.3cN_c + qN_q + 0.3\gamma BN_\gamma$

C : Cohesive.

$q = \gamma D_f$

B: Foundation width (Diameter if circular).

N_c, N_q, N_γ : Bearing capacity factors given from table as function of angle of friction ϕ.

Rectangle footing:

$$q_f = (1 + 0.3\frac{B}{L})cN_c + \gamma DN_q + (1 - 0.2\frac{B}{L})0.5\gamma BN_\gamma$$

For Local shear failure:

Type of foundation	Ultimate bearing capacity q_u
Strip Footing	$q_u = \frac{2}{3}cN_c' + qN_q' + \frac{1}{2}\gamma BN_\gamma'$
Square footing	$q_u = 0.867cN_c' + qN_q' + 0.4\gamma BN_\gamma'$
Circular footing	$q_u = 0.867cN_c' + qN_q' + 0.3\gamma BN_\gamma'$

N_c', N_q', N_γ' : Factors for bearing capacity given from table or from table but replace Φ'by Φ:

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

6.10. Recent bearing capacity Theory

Meyerhof's equations (General bearing capacity equation):

Terzagi equations neglect:

✓Rectangular footings.

✓Inclination of loads.

✓Shear strength of soil above the foundation.

Meyerhof's equation takes in consideration these variables:

$$q_u = c'\lambda_{cs}\lambda_{cd}\lambda_{ci}N_c + q\lambda_{qs}\lambda_{qd}\lambda_{qi}N_q + \tfrac{1}{2}\lambda_{\gamma s}\lambda_{\gamma d}\lambda_{\gamma i}\gamma BN_\gamma$$

where λ_{cs}, λ_{qs}, and $\lambda_{\gamma s}$ = shape factors

λ_{cd}, λ_{qd}, and $\lambda_{\gamma d}$ = depth factors

λ_{ci}, λ_{qi}, and $\lambda_{\gamma i}$ = inclination factors

N_c, N_q, N_γ : (From bearing capacity factor of Meyerhof's Table)

Skempton's equation for clay without inclination:

$$q_u = 5c\left(1+0.2\frac{D_f}{B}\right)\left(1+0.2\frac{B}{L}\right)$$

Vesic's equation (Consider compressibility of soil):

$$q_u = cN_cF_{cs}F_{cd}F_{cc} + qN_qF_{qs}F_{qd}F_{qc} + 0.5\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma c}$$

N_c, N_q, N_γ : Table

$F_{cc}, F_{qc}, F_{\gamma c} \Rightarrow$ Soil Compressibility factors.

Brinch Hansen's Bearing Capacity equation:

As mentioned in previous section, bearing capacity depends on many factors and Terzaghi's bearing capacity equation does not take in to consideration all the factors. Brinch Hansen and several other researchers have provided a comprehensive equation for the determination bearing capacity called Generalised Bearing Capacity equation considering the almost all the factors mentioned above. The equation for ultimate bearing capacity is as follows from the comprehensive theory

$$q_f = cN_cs_cd_ci_c + qN_qs_qd_qi_q + 0.5\gamma BN_\gamma s_\gamma d_\gamma i_\gamma$$

Here, the bearing capacity factors are given by the following expressions which depend on ϕ.

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

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

$$N_\gamma = 1.5(N_q - 1)\tan\phi$$

Equations are available for shape factors (s_c, s_q, s_γ), depth factors (d_c, d_q, d_γ) and load inclination factors (i_c, i_q, i_γ). The effects of these factors is to reduce the bearing capacity.

Factors	Meyerhof	Hansen	Vesic
s_c	$1+0.2N_\phi \frac{B}{L}$	$1+\frac{N_q}{N_c}\frac{B}{L}$	
s_q	$1+0.1N_\phi \frac{B}{L}$ for $\phi>10^\circ$	$1+\frac{B}{L}\tan\phi$	
s_γ	$s_\gamma = s_q$ for $\phi>10^\circ$ $s_\gamma = s_q = 1$ for $\phi=0$	$1-0.4\frac{B}{L}$	The shape and depth factors of Vesic are the same as those of Hansen.
d_c	$1+0.2\sqrt{N_\phi}\frac{D_f}{B}$	$1+0.4\frac{D_f}{B}$	
d_q	$1+0.1\sqrt{N_\phi}\frac{D_f}{B}$ for $\phi>10^\circ$	$1+2\tan\phi(1-\sin\phi)^2\frac{D_f}{B}$	
d_γ	$d_\gamma = d_q$ for $\phi>10^\circ$ $d_\gamma = d_q = 1$ for $\phi=0$	1 for all ϕ	
		Note; Vesic's s and d factors = Hansen's s and d factors	
i_c	$\left(1-\frac{\alpha^\circ}{90}\right)^2$ for any ϕ	$i_q-\frac{1-i_q}{N_q-1}$ for $\phi>0$	Same as Hansen for $\phi>0$
		$0.5\left(1-\frac{Q_h}{A_f c_a}\right)^{\frac{1}{2}}$ for $\phi=0$	$1-\frac{mQ_h}{A_f c_a N_c}$
i_q	$i_q = i_c$ for any ϕ	$\left(1-\frac{0.5Q_h}{Q_u+A_f c_a\cot\phi}\right)^5$	$\left(1-\frac{Q_h}{Q_u+A_f c_a\cot\phi}\right)^m$
i_γ	$\left(1-\frac{\alpha^\circ}{\phi^\circ}\right)^2$ for $\phi>0$ $i_\gamma=0$ for $\phi=0$	$\left(1-\frac{0.7Q_h}{Q_u+A_f c_a\cot\phi}\right)^5$	$\left(1-\frac{Q_h}{Q_u+A_f c_a\cot\phi}\right)^{m+1}$

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

$N_\gamma = 1.5(N_q - 1)\tan\phi$ (Hansen)

$N_\gamma = 2(N_q + 1)\tan\phi$ (Vesic)

6.11 Bearing capacity from in- situ tests (Plate load Test)

Field Tests are performed in the field. You have understood the advantages offield tests over laboratory tests for obtaining the desired property of soil. Thebiggest advantages are that there is no need to extract soil sample and theconditions during testing are identical to the actual situation.

Major advantages of field tests are

· Sampling not required

· Soil disturbance minimum

Major disadvantages of field tests are

· Laborious

· Time consuming

· Heavy equipment to be carried to field

· Short duration behavior

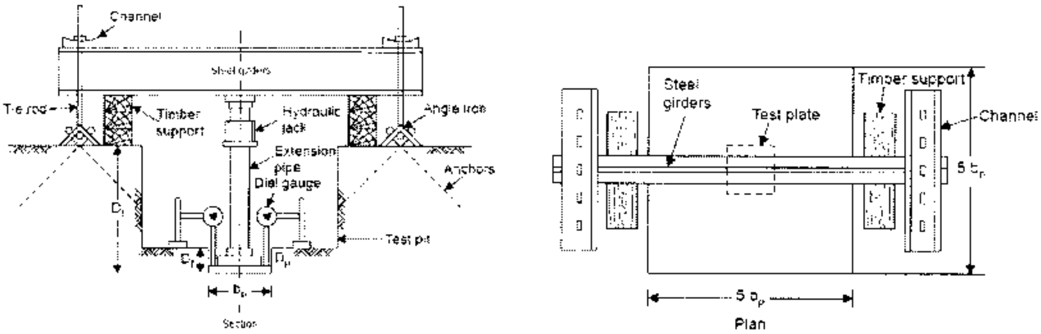


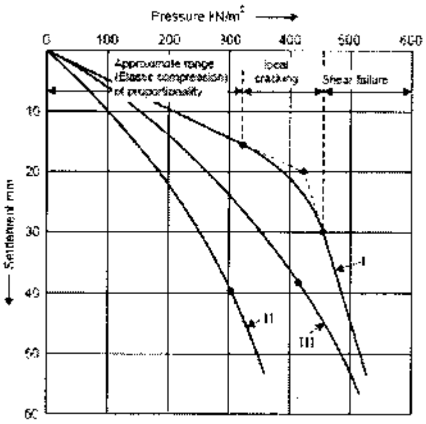
Figure: Typical set up for Plate Load test assembly

1. It is a field test for the determination of bearing capacity and settlement characteristics of ground in field at the foundation level.
2. The test involves preparing a test pit up to the desired foundation level.
3. A rigid steel plate, round or square in shape, 300 mm to 750 mm in size, 25 mm thick acts as model footing.
4. Dial gauges, at least 2, of required accuracy (0.002 mm) are placed on plate on plate at corners to measure the vertical deflection.
5. Loading is provided either as gravity loading or as reaction loading. For smaller loads gravity loading is acceptable where sand bags apply the load.
6. In reaction loading, a reaction truss or beam is anchored to the ground. A hydraulic jack applies the reaction load.
7. At every applied load, the plate settles gradually. The dial gauge readings are recorded after the settlement reduces to least count of gauge (0.002 mm) & average settlement of 2 or more gauges is recorded.
8. Load Vs settlement graph is plotted as shown. Load (P) is plotted on the horizontal scale and settlement (s) is plotted on the vertical scale.
9. Red curve indicates the general shear failure & the blue one indicates the local or punching shear failure.
10. The maximum load at which the shear failure occurs gives the ultimate bearing capacity of soil.

Reference can be made to IS 1888 - 1982.

The failure point is obtained as the point corresponding to the intersection of the initial and final tangents.

The value of q_{ult} here is given by $\frac{1}{2}\gamma b_p N_\gamma$.



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

where S = settlement of the proposed foundation (mm),
S_p = settlement of the test plate (mm), (same units)
b = size of the proposed foundation (m), and
b_p = size of the test plate (m).
This is applicable for sands.

However, the relationship is simpler for clays, since the modulus value E_s for clays is reasonably constant.

$$\frac{S}{S_p}=\frac{b}{b_p}$$

Equation may be put in a slightly simplified form as follows:

$$S=S_p\left[\frac{2b}{b+0.3}\right]^2$$

where S_p = Settlement of a test plate of 300 mm square size,
and S = Settlement of a footing of width b.

The advantages of Plate Load Test are

1. It provides the allowable bearing pressure at the location considering both shear failure and settlement.
2. Being a field test, there is no requirement of extracting soil samples.
3. The loading techniques and other arrangements for field testing are identical to the actual conditions in the field.
4. It is a fast method of estimating ABP(Allowable bearing Presssure) and Pressure– Settlement behaviour of ground.

The disadvantages of Plate Load Test are

1. The test results reflect the behaviour of soil below the plate (for a distance of ~2Bp), not that of actual footing which is generally very large.
2. It is essentially a short duration test. Hence, it does not reflect the long term consolidation settlement of clayey soil.
3. Size effect is pronounced in granular soil. Correction for size effect is essential in such soils.
4. It is a cumbersome procedure to carry equipment, apply huge load and carry out testing for several days in the tough field environment.

Bearing capacity from SPT tests based on settlement:

Bearing capacity of Sand based on settlement, by Bowels

$$q_{net}(\text{kN/m}^2)=\frac{N_{60}}{0.05}F_d\left[\frac{S_e(\text{mm})}{25}\right]\quad(\text{for } B\leq 1.22\text{m})$$

$$F_d=\text{depth factor}=1+0.33\left(\frac{D_f}{B}\right)\leq 1.33$$

$$q_{net}(\text{kN/m}^2)=\frac{N_{60}}{0.08}\left(\frac{B+0.3}{B}\right)^2F_d\left[\frac{S_e(\text{mm})}{25}\right]\quad(\text{for } B>1.22\text{m})$$

where B = foundation width (m)
S_e = settlement

Some empirical relationships are:

Teng (1969) has proposed the following equation for the graphical relationship of Terzaghi and Peck for a settlement of 25 mm

$$q_{na} = 34.3\,(N - 3)\left(\frac{b + 0.3}{2b}\right)^2 R_\gamma \,.\, R_d$$

where q_{na} = net allowable soil pressure in kN/m^2 for a settlement of 25 mm,

N = Standard penetration value corrected for overburden pressure and other applicable factors,

b = width of footing in metres.

R_γ = correction factor for location of water table.

and R_d = Depth factor $(= 1 + D_f/b) \leq 2$, where D_f = depth of footing in metres.

The modified equation of Teng is as follows:

$$q_{na} = 51.45(N - 3)\left(\frac{b + 0.3}{2b}\right)^2 R_\gamma \,.\, R_d$$

The notation is the same

Meyerhof (1956) has proposed slightly different equations for a settlement of 25 mm, but these yield almost the same results as Teng's equation:

$$q_{na} = 12.25\, NR_\gamma \,.\, R_d\,, \text{ for } b \leq 1.2 \text{ m}$$

$$q_{na} = 8.17\, N\left(\frac{b + 0.3}{b}\right) .\, R_\gamma \,.\, R_d\,, \text{ for } b > 1.2 \text{ m}$$

The notation is the same as those of Eqs. 14.119 and 14.120

Modified equation of Meyerhof is as follows:

$$q_{na} = 18.36\, NR_\gamma \,.\, R_d\,, \text{ for } b \leq 1.2 \text{ m}$$

$$q_{na} = 12.25\, N\left(\frac{b + 0.3}{b}\right) R_\gamma \,.\, R_d\,, \text{ for } b > 1.2 \text{ m}$$

The modified equations of Teng and Meyerhof are based on the recommendation of Bowles (1968).

The I.S. code of practice gives for a settlement of 40 mm; but, it does not consider the depth effect.

Teng (1969) also gives the following equations for bearing capacity of sands based on the criterion of shear failure:

$$q_{net\,ult} = 1/6[3N^2 . b\, R_\gamma + 5(100 + N^2)D_f\, R_q]$$

(for continuous footings)

$$q_{net\,ult} = 1/6[2N^2\, b\, R_\gamma + 6\,(100 + N^2)D_f \,.\, R_q]$$

(for square or circular footings)

Here again,

$q_{net\,ult}$ = net ultimate soil pressure in kN/m^2.,

N = Standard penetration value, after applying the necessary corrections,

b = width of continuous footing (side, if square, and diameter, if circular in metres),

D_f = depth of footing in metres, and

R_γ and R_q = correction factors for the position of the ground water table,

With a factor of safety of 3, the net safe bearing capacity q_{ns}, is given by

$$q_{ns} = \frac{1}{18}\,[3N^2 b\, R_\gamma + 5(100 + N^2)D_f R_q] - \frac{2}{3}\,\gamma\;\; D_f$$

(for continuous footings)

$$q_{ns} = \frac{1}{18}\,[2N^2 b\, R_\gamma + 6(100 + N^2)D_f R_q] - \frac{2}{3}\,\gamma \,.\, D_f$$

(for square or circular footings)

In some books, Teng's equation is in different form as: (Verify required)

From bearing capacity considerations,

for very long and strip footings:

$$q_{ult-net} = \frac{1}{62}\left[3N^2BR'_w + 5(100 + N^2)DR_w\right]$$

for square and circular footings:

$$q_{ult-net} = \frac{1}{31}\left[N^2BR'_w + 3(100 + N^2)DR_w\right]$$

From settlement considerations the equations for safe bearing pressure are as follows

$$q_{safe-nr} = 3.5(N - 3)\left(\frac{B + 0.3}{2B}\right)^2 R'_w C_D \quad \text{for } S_p = 2.5 \text{ cm}$$

$$= 1.4(N - 3)\left(\frac{B + 0.3}{2B}\right)^2 R'_w C_D S_p$$

for a specified permissible settlement of S_p in cm

6.12 Types of Settlement and their relationship:

1. The downward movement of a building structure due to consolidation of soil beneath the foundation.
2. The sinking of solid particles of aggregate in fresh concrete or mortar after its placement and before its initial set.

Types of settlement (In structure)

a.Angular Distortion

b.Tilt

c. Different settlement

Computation of settlement is not required for light structures and computation settlement is necessary for heavy structures and can be calculated in several methods.

Settlement in soil:

Immediate Settlement: Occurs immediately after the construction. This is computed using elasticity theory (Important for Granular soil)

Primary Consolidation: Due to gradual dissipation of pore pressure induced by external loading and consequently expulsion of water from the soil mass, hence volume change. (Important for Inorganic clays)

Secondary Consolidation: Occurs at constant effective stress with volume change due to rearrangement of particles. (Important for Organic soils)

- Settlement under Loads (Components of settlement):
    - S_e - Elastic or Immediate Settlement
    - S_c - Consolidation Settlement
    - S_s - Secondary Settlement

$S = S_e + S_c + S_s$

- Net elastic settlement of flexible footing

$$S_e = q_n B \frac{(1-\mu^2)}{E_s} I_f$$

S_e = elastic settlement

B = width of foundation,

E_s = modulus of elasticity of soil,

μ = Poisson's ratio,

q_n = net foundation pressure,

I_f = influence factor.

Influence factor as per Bowels

	I_f (average values)	
Shape	Flexible footing	Rigid footing
Circle	0.85	0.88
Square	0.95	0.82
Rectangle	1.20	1.06
L/B = 1.5	1.20	1.06
2.0	1.31	1.20
5.0	1.83	1.70
10.0	2.25	2.10
100.0	2.96	3.40

- Elastic Settlement of Rigid Footing

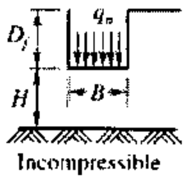
  $S_{ef} = C_r d_f S_e$

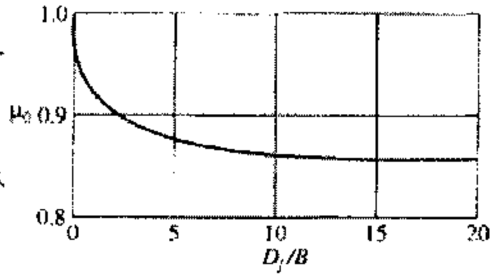
  Where,

  S_{ef} = Final elastic settlement
  C_r = Rigidity Factor (Highly rigid footing, Cr = 0.8)
  D_f = Depth factor
  S_e = Settlement for a surface flexible footing

Janbu, Bjerrum and Kjaernsli's method of determining elastic settlement under undrained conditions

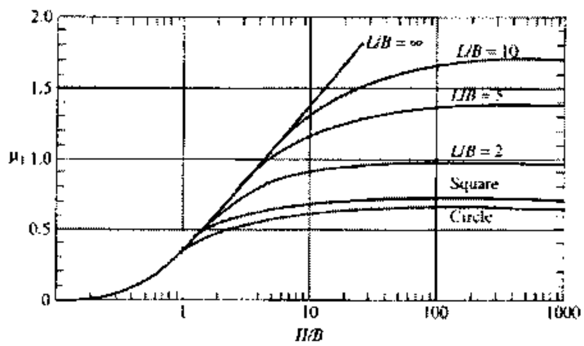
$$S_e = \frac{\mu_0 \mu_1 q_n B}{E_s}$$





- Consolidation and secondary Settlement (studied in Soil Mechanics- Refer Arora)

$$S_c = H\frac{C_c}{1+e_0}\log\frac{p_0+\Delta p}{p_0}$$



6.13 Permissible settlement and allowable bearing pressure :

It is amount of vertical displacement in which settlenment of structure falls in permissible limit.The permissible settlement of spread and mat foundation generally taken 25mm and 40mm according to Terzaghi .The allowable bearing capacity or pressure is governed by allowable settlement. It is correspond value of permissible settlement. Computation of allowable bearing pressure is made according to the procedure for doing plate load test and penetration test.

6.14 Steps involved in the proportion of footings:

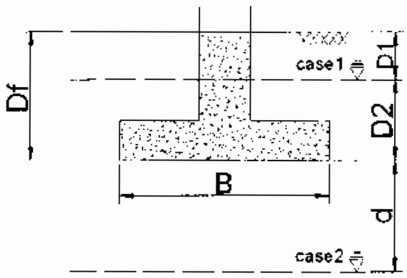
To reduce the differential settlement due to variations of loads proportion of all footings is desirable for uniform settlement.

Steps for proportioning of spread footing – (From Arora Book)

1. Determine dead load including weight of footing.
2. Detrmine the footing subjected to maximum load.
3. Compute the ratio of live load to dead load for each footing .
4. Identify the governing footing having maximum live load to dead load ratio is the governing footing.
5. Find the area of governing footing (Ag).
6. Area of governing Footing
   = DL+LL / Allowable bearing capacity of soil

7. Determine the design service load for all the footing.
8. Determine the design bearing capacity (q_d) of all the footings except the governig footing.
9. Design bearing capacity
   = service load of governing footing $/A_g$
10. Determine area under the other footing (A).
11. A = Service load of that loading $/q_d$

Effect of water table in bearing capacity equations:



Case I) Water table is located at depth D_1 so that $0 \le D_1 \le D_f$:

$$q = \gamma D_1 + \gamma' D_2$$

$$\gamma = \gamma' = \gamma_{sat} - \gamma_w$$

Case II) Water table is located at depth d below the foundation so that $0 \le d \le B$:

$$q = \gamma D_f$$

$$\gamma = \bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$$

Case III) Water table is located at depth d below the foundation so that $d > B$:
No changes in equations.

Factor of safety:

Ultimate bearing capacity

$q_u \Rightarrow$ Gross ultimate bearing capacity

$((q_u)_{net} = q_u - q) \Rightarrow$ Net ultimate bearing capacity

$q_{all} \Rightarrow$ Gross allowable bearing capacity

$(q_{all})_{net} \Rightarrow$ Net allowable bearing capacity

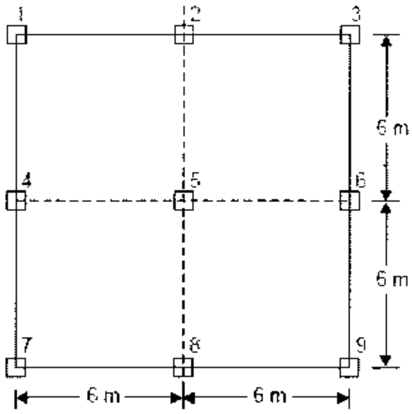
$Q_u \Rightarrow$ *Gross* Ultimate load.

$$\Rightarrow q_{all} = \frac{q_u}{FS}$$

$$\Rightarrow (q_{all})_{net} = \frac{(q_u)_{net}}{FS} = \frac{q_u - q}{FS} = q_{all} - \frac{q}{FS}$$

FS = (3 – 4) for bearing capacity Factor of safety with respect to shear:

Example : A building is supported on nine columns as shown in Fig. below and column loads are indicated. Determine the required areas of the column footings:



Column No.	1	2	3	4	5	6	7	8	9
Dead Load (kN)	180	360	240	300	600	360	180	360	210
Max. Live Load (kN)	180	400	210	300	720	360	120	300	180

At the selected depth of 1.5 m the allowable bearing capacity is 270 kN/m2. $\gamma = 20$ kN/m³.

Solution:

Dead load plus maximum live load, maximum live load to dead load ratio, reduced live load and dead load plus reduced live load are all determined and tabulated for all the columns (A reduction factor of 50% is used for *LL*).

Column No.	1	2	3	4	5	6	7	8	9
Dead Load (kN)	180	360	240	300	600	360	180	360	210
Max. LL (kN)	180	400	210	300	720	360	120	300	180
DL + Max. LL (kN)	360	760	450	600	1320	720	300	660	390
Max. LL/DL	1.00	1.11	0.88	1.00	1.20	1.00	0.67	0.83	0.86
Reduced LL (kN)	90	200	105	150	360	180	60	150	90
DL + Reduced LL (kN)	270	560	345	450	960	540	240	510	300

Column No. 5 has the maximum LL to DL ratio of 1.20 and hence it governs the design.

Assuming the thickness of the footing as 1 m,

allowable soil pressure corrected for the weight of the footing $= (270 - 1 \times 20) = 250$ kN/m²

$\therefore$ Area of footing for column No. 5 $= \dfrac{1320}{250} = 5.28$ m²

Reduced Load for this column $= 960$ kN

$$\text{Reduced allowable pressure} = \frac{\text{Reduced load}}{\text{Area}} + \text{Weight of footing}$$

$$= \frac{960}{5.28} + 20 = 182 + 20 = \mathbf{200\ kN/m^2}$$

The footing sizes will be obtained by dividing the reduced loads, for each column by the corrected reduced allowable pressure of $\dfrac{960}{5.28}$ or 182 kN/m².

The results are tabulated below:

Column No.	1	2	3	4	5	6	7	8	9
Reduced Load (kN)	270	560	345	450	960	540	240	510	300
Corrected reduced soil pressure (kN/m^2)	182	182	182	182	182	182	182	182	182
Required area (m^2)	1.49	3.07	1.90	2.48	5.28	2.97	1.32	2.80	1.65
Size of footing (m^2)	**1.25**	**1.75**	**1.40**	**1.60**	**2.30**	**1.75**	**1.20**	**1.70**	**1.30**

The thickness of the footing may be varied somewhat with loading. This will somewhat alter the reduced allowable pressures for different footings. The areas of the footings will get increased slightly. However, this refinement is ignored in tabulating the sizes of the square footings.

The structural design of the footings may now be made.

8 PILE FOUNDATION

8.1 Introduction

- It is a slender member structural member having it's cross sectional dimensions very smaller than length.
- Deep foundation which transfers loads to great depths .

Necessity of pile foundation

- When strata below the ground surface is highly compressive or very weak to support the load transmitted by the structure .
- To reduce the differential settlement.
- To transfer the load below the active zones i.e shrinks or swells and through deep strata to firm strata .
- On ill conditions soils such as washout , erosion and scour etc.
- When the foundation subjected to uplift .

8.2 Types and uses of Piles

According to material used

a. Steel piles : Thick pipes or steel sections
b. Concrete piles : Precast or cast in situ
c. Timber piles: Made up of trunk of tree after proper trimming .Timber should be straight ,sound and free from defects . .
d. Composite piles: made up of two materials (i.e lower portion steel and upper portion concrete).

According to made of transfer of loads

1. End bearing pile
2. Friction pile
3. Combined end bearing and friction

According to method of installation

i. Driven piles
ii. Driven and cast in situ pile
iii. Bored and cast in situ pile
iv. Screwed pile
v. Jacked pile

According to displacement of pile

1. Displacement pile
2. Non – Displacement pile

According to based on use

i. Load bearing pile
ii. Tension Pile
iii. Compaction pile
iv. Sheet pile
v. Anchor piles

Uses of pile

i. To transfer loads to strong / less compressible strata
ii. To compact loose grannular soil
iii. To provide foundation below the scour depth
iv. To carry horizontal and vertical faces from abutments and retaining walls
v. To carry up lifts forces

8.3 Construction of Piles

The construction of a pile foundation involves two steps, namely the installation of piles and the making of pile caps. The second step is relatively simple and is similar to the construction of footings.

Installation of piles would depend upon whether they are driven or cast-in-place. Some details regarding the equipment required to install piles by driving them into soil have already been given. Water jetting is used to assist penetration of the piles.

Cast-in-place piles are mostly concrete piles of standard types such as the Raymond pile and the Franki pile, so called after the piling firms which standardized their construction.

Damage due to improper driving may be avoided if driving is stopped when the penetration reaches the desired resistance.

Some degree of tolerance in alignment has to be permitted since piles can never be driven absolutely vertical and true to position.

A pile may be considered defective if it is damaged by driving or is driven out of position, is bent or bowed along its length. A defective pile must be withdrawn and replaced by another pile. It may by left in place and another pile may be driven adjacent to it.

Pile driving may induce subsidence, heave, compaction, and disturbance of the surrounding soil. These effects are to be carefully studied so as to understand their bearing on the capacity of the pile.

The method of installing a pile at a site depends upon the type of pile. The equipment required for this purpose varies. The following types of piles are normally considered for the purpose of installation

1. Driven piles

The piles that come under this category are,

a. Timber piles,
b. Steel piles, H-section and pipe piles,
c. Precast concrete or prestressed concrete piles, either solid or hollow sections.

2. Driven *cast-in-situ* piles

This involves driving of a steel tube to the required depth with the end closed by a detachable conical tip. The tube is next concreted and the shell is simultaneously withdrawn. In some cases the shell will not be withdrawn.

3. Bored *cast-in-situ* piles

Boring is done either by auguring or by percussion drilling. After boring is completed, the bore is concreted with or without reinforcement.

8.4 Selection of Pile

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

1. Length of pile in relation to the load and type of soil
2. Character of structure
3. Availability of materials
4. Type of loading
5. Factors causing deterioration

6. Ease of maintenance
7. Estimated costs of types of piles, taking into account the initial cost, life expectancy and cost of maintenance
8. Availability of funds

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

8.5 Types of Foundation to suit subsoil conditions (Gopal Ranjan and ASR Rao)

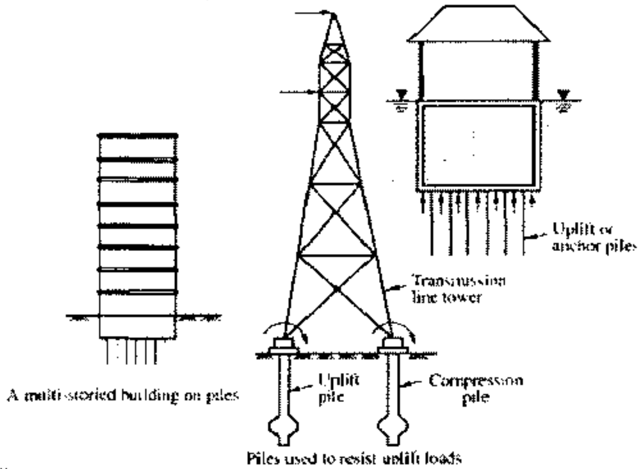


Figure (a) Principles of floating foundation; and a typical rigid raft foundation

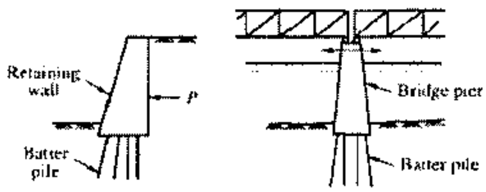


Figure (b) Piles used to resist lateral loads

8.6 Pile Driving Formula

8.7 Static Pile Load Formula

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

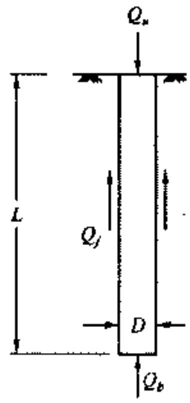
1. By the use of static bearing capacity equations.
2. By the use of SPT and CPT values.
3. By field load tests.
4. By dynamic method.

1. By the use of static bearing capacity equations.

Total ultimate load capacity of a pile foundation is it's sum of base or resistance and shaft resistance.

$Q_u = Q_b + Q_s + W$

$= q_b A_b + q_s A_s$



Where; Q_u = load at failure applied to the pile,
Q_b = base resistance,
Q_s = shaft (skin friction) resistance,
W = wieight of pile
A_b= Area of Base
A_s= Area of Shaft

For single Pile

In sandy soil shaft resistance is given by friction component only. Hence the ultimate load carrying capacity of pile in sand is given by

$$Q_b = A_b(cN_c + \sigma_{vb}N_q + 0.5\gamma bN_\gamma)$$

$$Q_s = q_s A_s = f_s A_s = \int_0^L P\tau_a dz$$

where

$$\tau_a = c_a + \sigma_n \tan\phi_a$$

τ_a = pile-soil shear strength
c_a = adhesion
σ_n = normal stress between pile and soil
ϕ_a = angle of friction between pile and soil

Normal (vertical load) can be given as

$$\sigma_n = K_s\,\sigma_v$$

Where,

K_s = coefficient of lateral pressure
σ_{vb} = *Vertical stress in the soil at pile base*

Thus,

$$\tau_a = c_a + \sigma_v K_s \tan\phi_a$$

$$A_s f_s = \int_0^L P\tau_a dz$$

$$= \int_0^L P(c_a + \sigma_v K_s \tan\phi_a)dz$$

Where, P=Pile peremeter , L= Length of pile

Thus the final general equation will be

$$Q_u = \int_0^L P(c_a + \sigma_v K_s \tan\phi_a)\,dz + A_b(cN_c + \sigma_{vb}N_q + 0.5\gamma bN_\gamma) - W$$

Weight of pile is normally neglected in the case of end bearing pile and considered in the case of friction piles in practical use.

Piles in Clay

If the clay is saturated, the undrained angle of friction ϕ_u=0, and fa may also be taken as zero, In addition Nq=1 and $N\gamma$=0 for ϕ=0 so the general equation reduces to

$$Q_u = \int_0^L Pc_a dz + A_b(c_u N_c + \sigma_{vb}) - W$$

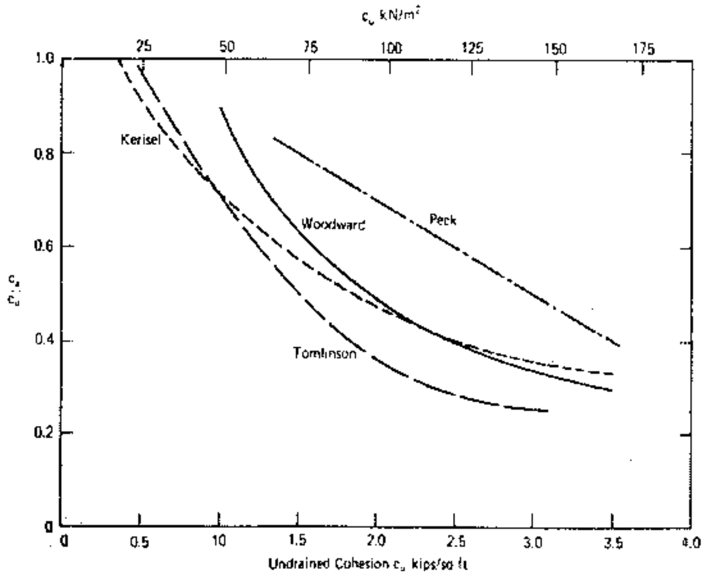
Where,

c_u = undrained cohesion of soil at level of pile base

c_a= undrained pile-soil adhesion

Further simplification is possible in many cases, since for piles without an enlarged base, $A_b\sigma_{vb} \approx W$, in which case,

$$Q_u = \int_0^L Pc_a dz + A_b c_u N_c$$



For driven piles, typical relationships between ca/cu and cu, based on the summary provided by McClelland (1974), are shown in the figure below. It is generally agreed that for soft clays (cu< or = 24 kPa), ca/cu is 1 or more for different cases (driven piles in stiff clays).

Piles in Sand

For the case of piles in sand, if the pile-soil adhesion ca and term *cNc* are taken as zero, and the term $0.5\gamma bN\gamma$ is neglected as being small in relation to the term involving *Nq*, the ultimate load capacity of a single pile in sand may be expressed as follows:

$$Q_u = \int_0^L P\sigma_v' K_s \tan\phi_a' . dz + A_b \sigma_{vb}' N_q - W$$

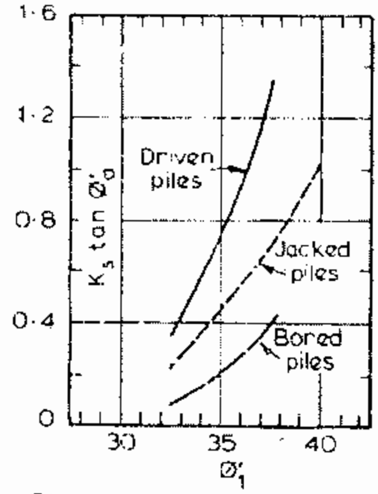
Where,

σ_v' = effective vertical stress along shaft

σ_{vb}'= effective vertical stress at level of pile base

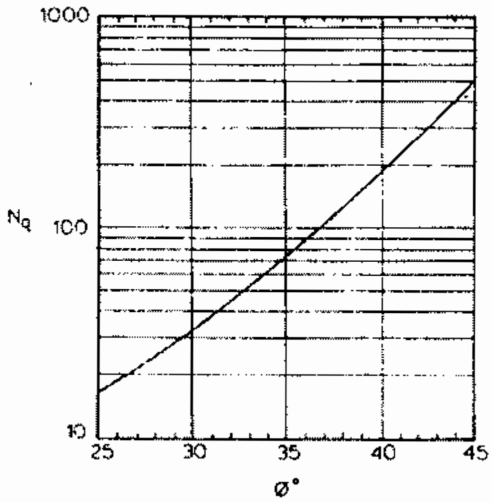
The values of ϕ_a' is taken as $0.75\phi'$

Values of $K_s \tan \phi_a'$ Base on Meyerhof (1976)

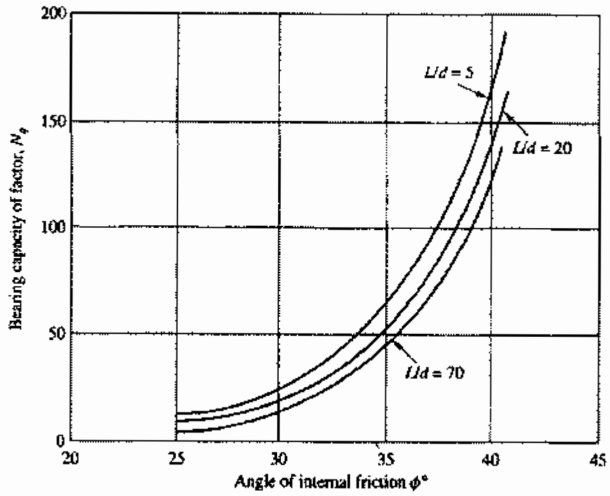


where ϕ_1' = angle of internal friction prior to installation of pile

The bearing capaicyt factor Nq and f is based on those derived by Berezantzev et al. (1961) and the values appear to fit the available test data best. The solutions given by Berzantzev et al. indicate only a small effect of relative embedment depth L/d, and represents an average of this small range. The curves given by meryerhof (1967) show a large effect of L/d, however the curve given by Berzantzev also lies near the middle of Meyerhof's range.



Relationship between N_q and ϕ (after Berezantzev et al., 1961)



Berezantsev's bearing capacity factor, N_q (after Tomlinson, 1986)

S. No.	Material of pile	Consistency of clay	Cohesion (kN/m^2)	Adhesion factor
1.	Wood and concrete	Soft	0–35	0.90 to 1.00
		Medium	35–70	0.60 to 0.90
		Stiff	70–140	0.45 to 0.60
2.	steel	Soft	0–35	0.45 to 1.00
		Medium	35–70	0.10 to 0.50
		Stiff	70–140	0.50

Adhesion factors for piles in clay (Tomlinson, 1969)

The adhesion c_a may be expressed as

$$c_a = \alpha . c$$

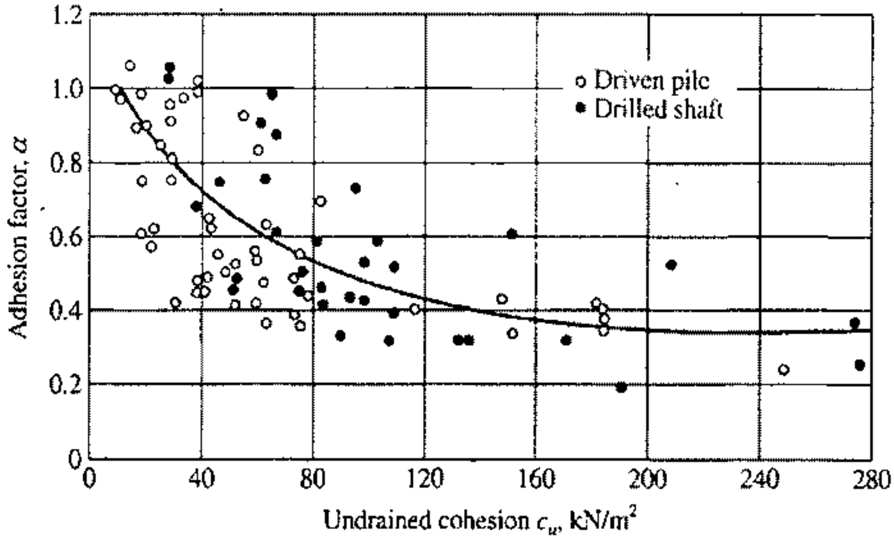


Figure- Adhesion factor for piles with penetration lengths less than 50 m
(Data from Dennis and Olson 1983 Stas and Kulhawy, 1984)

For a layered system in sand:

$Q_u = \sigma_v (Nq - 1)A_b + P\Sigma\, \sigma_v' K \tan\delta\, \Delta t$

Where,

σv = effective overburden pressure at the base level of pile.

σ_v' =effective overburden pressure (taken average) over the embedded depth in a particular layer of soil.

Ks = Lateral earth pressure coefficient.

Ab = Base area of the pile.

A_S = Shaft area of the pile. =P*Δt

P = Perimeter of the pile.

Δt = Thickness of the layer

δ = Angle of internal frictional of pile.

Nc, Nq = Bearing capacity factors for cohesion and surcharge

Nc =9, for deep foundation and

Nq can be found from the charts of bearing capacity factors

Single Pile in Clay

In clayey soil shaft resistance is due to adhesion of surrounding soil with the pile shaft. So the ultimate load carrying capacity of pile in clay is given by,

$Qu = Q_b + Q_f = c_b\, N_c\, A_b + \alpha\; c_u\, A_s$

For a layered soil:

$Qu = c_b\, N_c\, A_b + p\;\; \Sigma\, \alpha\;\; c_u\, P\;\; \Delta t$

Where,

c_b= undrained shear strength of clay at base of pile

c_u= Average undrained shear strength of clay at side of pile..

Nc= bearing capacity factor. = 9 for pile foundations.

Ab = Base of the pile.

α = Adhesion factor which can be found from following figure

A_S= Shaft area of the pile.

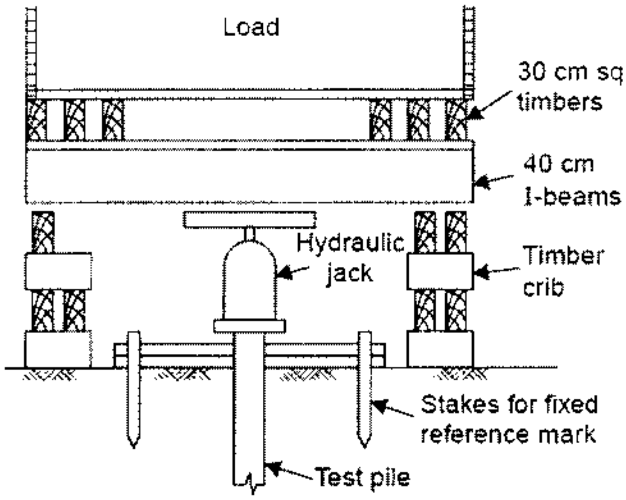
P= perimeter of the pile.

Δt = thickness of the layer clay.

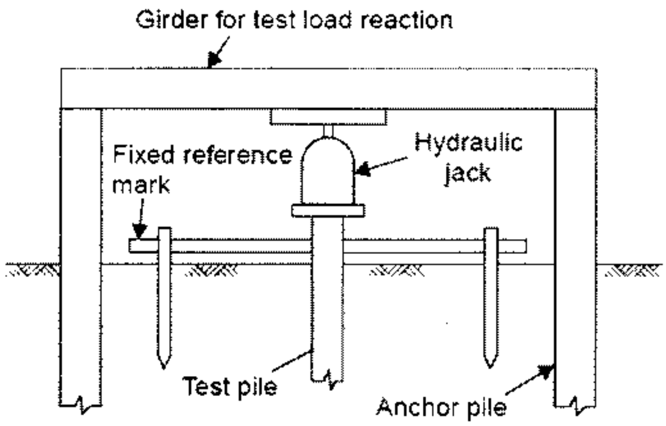
8.8 Load Test on Piles

Load test on a pile is one of the best methods of determining the load-carrying capacity of a pile. It may be conducted on a driven pile or cast-in-situ pile, on a working pile or a test pile, and on a single pile or a group of piles. A working pile is one which forms part of the foundation, while a test pile is one which is used primarily to check estimated capacities (as predetermined by other methods).

The test should be conducted only after a lapse of a few weeks in clays and at least a few days in sands, in order that the results obtained be more meaningful for design.



(a) Weighted platform for jacking reaction



(b) Anchor piles and girder for reaction

Fig. Typical pile load test arrangements

Load may be applied by using a hydraulic jack against a supported platform (above Fig. *a*), or against a reaction girder secured to anchor piles (above Fig. *b*). Sometimes a proving ring is preferred for better accuracy in obtaining the load. Instead of reaction loading, gravity loading may also be used; but the former is given better uniformity in loading. Measurement for pile settlement is related to a fixed reference mark. The support for the reference mark has to be located outside the zone that could be affected by pile movements.

The most common procedure is the test in which the load is maintained slowly.

- About five to eight equal increments are used until the load reaches about double the design value.
- Time-settlement data are recorded for each load increment.
- Each increment is maintained until the rate of settlement becomes a value less than 0.25 mm per hour.
- The final load is maintained for 24 hours.

Another procedure is the constant-strain rate method. In this method,

- The load is increased such that the settlement occurs at a predetermined rate such as 0.5 mm per minute.
- This test is considerably faster than the other approach.

The load-settlement curve is obtained from the data. Often the definition of 'failureload' is arbitrary. It may be taken when a predetermined amount of settlement has occurred or where the load-settlement plot is no longer a straight line. If the ultimate load could be found, a suitable factor of safety—2 to 3—may be used to determine the allowable load.

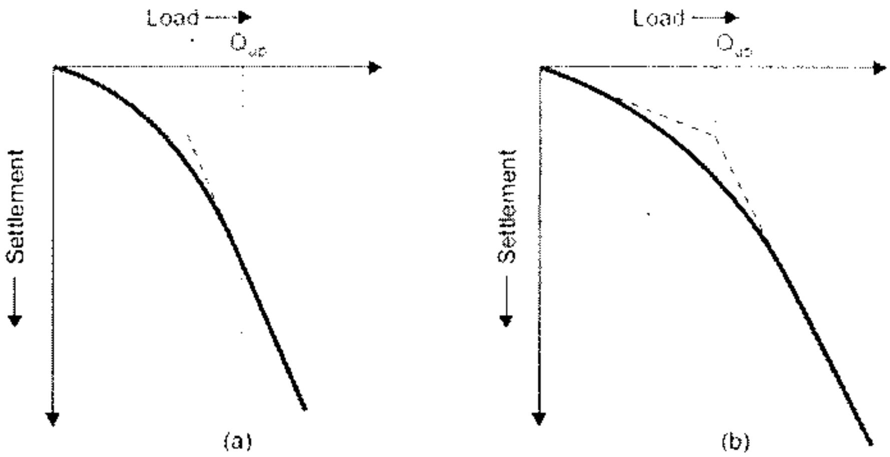


Fig. Determination of ultimate load from load-settlement curve for a pile

The ultimate load may be determined as the abscissa of the point where the curved part of the load-settlement curve changes to a steep straight line (Fig. *a*). Alternatively, the ultimate load is the abscissa of the point of intersection of initial and final tangents of the load settlement curve (Fig. *b*).

Another method in use for the slow test is to plot both load and settlement values on logarithmic scale. The results typically plot as two straight lines (Fig. *c*). The intersection of the straight lines is taken as failure load for design purposes although this may not be the actual load at which failure occurs.

The allowable load on a single pile may be obtained as one of the following [I.S: 2911 (Part I)-1974]:

1. 50% of the ultimate load at which the total settlement is equal to one-tenth the diameter of the pile.

2. Two-thirds of the load which causes a total settlement of 12 mm.

4. Two-thirds of the load which causes a net (plastic) settlement of 6 mm (total settlement minus elastic settlement).

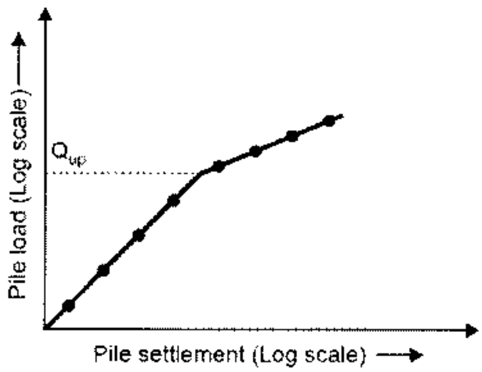


Fig. *C* Log-Log of pile load-settlement curve and determination of failure

8.9 Dynamic Pile Formulae

Pile capacity from pile driving Formuale

The relationship between dynamic resistance and pile during driving and static load carrying capacity of piles are called pile driving formula.

1. Engineering News Record (ENR) formula
2. Janbu's formula
3. Hiley formula

Engineering News Record (ENR) formula:

The 'Engineering News' formula (Wellington, 1886) was derived from observations of driving of timber piles in sand with a drop hammer. The general form of this equation is as follows:

$$Q_u = \frac{W_h H}{(s + C)}$$

Where

s = final penetration (set) per blow. It is taken as average penetration per blow for the last 5 blows or 20 blows depending on whether the hammer is a drop hammer or steam hammer,

C = empirical constant (representing the temporary elastic compression of the helmet, pile and soil)

A factor of safety of 6 was introduced to make up for any inaccuracies arising from the use of arbitrary values for the constant, while arriving at the allowable load on the pile.

$$Q_a = \frac{W_h \cdot H}{6(s + C)}$$

The value of C (in cm) is taken as 2.5 for drop hammer, and 0.25 for steam hammer.

$$Q_a = \frac{500\, W_h \cdot H}{3(s + 25)} \quad \text{for drop hammer}$$

$$Q_a = \frac{500\, W_h \cdot H}{3(s + 2.5)} \quad \text{for steam hammer}$$

$$Q_a = \frac{(W_h + ap) \cdot H}{6(s + 2.5)} \quad \text{For double acting steam hammers}$$

where

W_h = weight of hammer (newtons),

a = effective area of piston (mm^2),

p = mean effective steam pressure (N/mm^2),

H = height of fall of hammer (metres)

s = final penetration of pile per blow (mm), and

Q_a= allowable load on the pile (kN).

(Note: This equation has mixed units).

ENR Formula

The earliest pile driving formula assumes that for a given hammer blow, the resistance increases in an elastic manner as the pile is displaced, remains constant for further displacement, and finally falls to zero as the pile rebounds. Equating the energy supplied to the work done, the following formula was obtained.

$$\underline{WH = Q_u(S + C)}$$

where
- W = **weight of hammer (ton),**
- H = **fall of hammer (ft),**
- S = **penetration per blow (in),**
- R = **pile resistance (ton), and**
- C = **constant which accounts for elastic settlement of pile-soil system (1.0 in for drop hammer and 0.1 in for single acting steam hammer)**

Equation has subsequently been revised to a more generalized form,

$$Q_u = \frac{EWH}{S + C}\,\frac{W + n^2 W_p}{W + W_p}$$

where
- E = **hammer efficiency (0.7–0.9)**
- C = **0.1 in (if S and H are in inches)**
- W = **weight of hammer (ton)**
- W_p = **weight of pile (ton)**
- n = **coefficient of restitution (0.4–0.5)**

Janbu's Formula (Janbu, 1953)

$$Q_u = \frac{EH}{K_u S}$$

where

$$K_u = C_d\left(1 + \sqrt{1 + \frac{\lambda}{C_d}}\right);$$

$$C_d = 0.75 + 0.14\left(\frac{W_p}{W}\right), \text{ and}$$

$$\lambda = \frac{EHL}{A_p E_p S^2}$$

Where,

$E = \eta W$ =Hammer efficiency
η = efficiency factor
= 0.7 ,for good driving condition
= 0.5 ,for average driving condition
= 0.4 ,for difficult or bad driving condition
H= Height of fall
W =Weight of Hammer
W_p = weight of pile
L = Length of pile
A_p = Cross –sectional of Pile
E_p=Modulus of elasticity of pile
S = final set

Hiley Formula

$$Q_u = \frac{\eta WH}{S + C/2}\,\frac{W + n^2 W_p}{W + W_p}$$

Here, Q_u, W, H, n, S, and W_p have the same meaning as in Eq. generalized ENR form

η = efficiency of hammer blow (0.75–1.0)

C = a factor which accounts for energy losses due to elastic compression of pile, C_1, elastic compression of the head assembly, C_2, and elastic compression of the soil, C_3, that is,

$$C = C_1 + C_2 + C_3$$

The approximate values of C_1, C_2, and C_3 to be used in Hiley formula for concrete piles are given in Table 9.2.

Table 9.2 Values of C_1, C_2, C_3 in Hiley formula

C_1 = 0.075–0.10 in. for hard driving		
$C_2 = R_u L/AE_p$,	where	R_u = ultimate test load,
		L = Length of pile,
		A = Cross sectional area of pile, and
		E_p = Elastic modulus of pile material.
C_3 = 0.1 in (0 for hard soil and 0.2 for resilient soil)		

8.10 Pile Capacity from In-situ Tests

Meyerhoff suggested following relation for ultimate bearing capacity of pile:

- Displacement pile

  $Q_u = 400NA_b + 2N'A_s$

- H- pile

  $Q_u = 400NA_b + N'A_s$

- Bored pile:

  $Q_u = 133NA_b + 0.67\,N'A_s$

N= SPT value below pile tip,N'= Avg SPT along the pile shaft, A_b = base area , A_s= Shaft area and Q_u = Ultimate load .

Allowable Load $Q_a = Q_u/Fos$, Fos = 4

BC based on CPT result

Vandeer- Veen 's method for pile in cohesion less soil

$Q_u = Q_b + Q_s$

Base resistance

$Q_b = A_b.q_p'$

A_b = Base area of pile

q_p'= avg cone resistance over depth 4D (look curve from RKP book)

Shaft Resistance :

Meryerhoff suggested following values

Large displacement pile ,$Q_s = A_s\,q'_c/2$

Small displacement pile ,$Q_s = A_s\,q'_c/4$

$Q_a = (Q_b+Q_s)/2.5$

Pile Group subjected to eccentric vertical loading

For symmetrical pile system and pile cap being rigid and very thick ,when the total vertical load centrally placed be be Q_g and the number of piles be n, then load to be transmitted to each pile ,

$Qi = Q_g/n$

But for a centrally loaded pile cap.

- $Q_i = \{(Q_g/n) \pm (M_y{*}x/\sum x^2) \pm (M_x{*}y/\sum y^2)\}$

  Where,

  $M_x = Q_g * e_y$ =Moment about X- axis

  $M_y = Q_g * e_x$ = Moment about Y- axis

  Q_i = Load transmitted to particular pile

Q_g= Total Vertical load acting on the pile cap centrally

n = No. of pile in group

e_x, e_y = Eccentricity along Xand Y direction

$\sum x^2$ = Summation of squares of distance of all piles from Y-axis

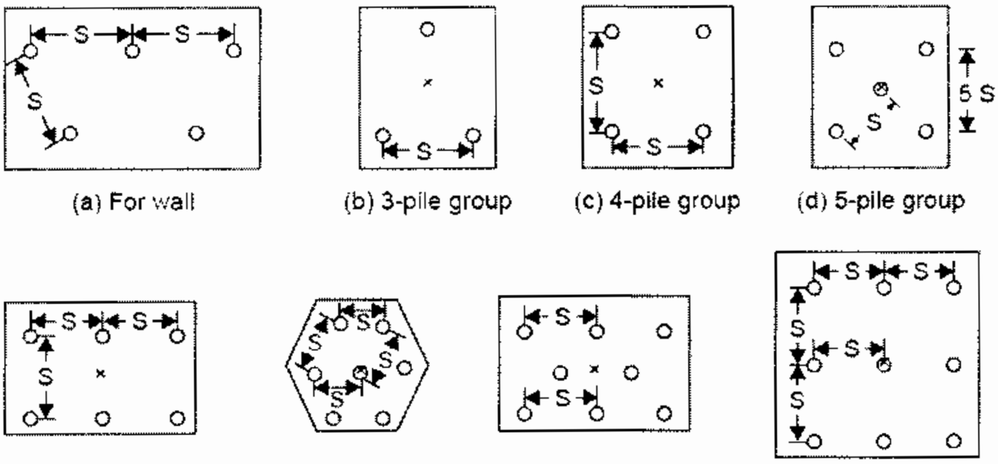
$\sum y^2$= Summation of squares of distance of all piles from X-axis

8.11 Group Action of Piles

Group action of piles:
A structure is never founded on a single pile. Piles are ordinarily closely spaced beneath structures; consequently, the action of the entire pile group must be considered. This is particularly important when purely friction piles are used.

The bearing capacity of a pile group is not necessarily the capacity of the individual pile multiplied by the number of piles in the group; the phenomenon by virtue of which this discrepancy occurs is known as 'Group action of piles'.

Number of Piles and Spacing
Point-bearing piles may be more closely spaced than friction piles. The minimum spacing of piles is usually specified in building codes.
The spacing may vary from **2*d* to 6*d*** for straight uniform **cylindrical piles**, ***d*** being the diameter of the pile.
For **friction piles**, the recommended minimum spacing is ***3d***.
For **point-bearing piles** passing through relatively compressible strata, the minimum spacing is **2.5*d*** when the piles rest in compact sand or gravel; this should be **3.5*d*** when the piles rest in stiff clay.
The minimum spacing may be **2*d*** for compaction piles.



(a) For wall (b) 3-pile group (c) 4-pile group (d) 5-pile group

(e) 6-pile group (f) 7-pile group (g) 8-pile group (h) 9-pile group

When piles are used in groups then the capacity of pile group does not necessarily become equal to single pile capacity multiplied by number of piles.
When piles are applied in loose sand then sand gets compacted and the bearing capacity of the group becomes greater than summation of individual pile capacity on the other hand when piles are inserted into loose clay it can lead to group failure or individual pile failure depending upon the spacing of the piles.

Hence summation of individual pile capacity becomes greater than group capacity in cohesive soil. Spacing should be determined based on economical and practical considerations.

Group Efficiency of Pile Groups
Group efficiency of a pile group is the ratio of total load carried by a pile group to the summation of individual pile load capacities.

Mathematically, it can be written as,

$$E_f = \{Q_u(g) / n^* Q_u(s)\} * 100\%$$

Where; E_f = Group efficiency ratio.
$Q_u(g)$ = Ultimate bearing capacity of pile group
$Q_u(s)$ = Ultimate bearing capacity of single pile

Converse-Labarre formula

$$\eta_g = 1 - \frac{\phi}{90}\left[\frac{m(n-1)+n(m-1)}{mn}\right]$$

where η_g = efficiency of pile group.

$\phi = \tan^{-1}\frac{d}{s}$ in degrees, d and s being the diameter and spacing of piles,

m = number of rows of piles, and
n = number of piles in a row (interchangeable)

- Other formulae **Fled's rule** is sometimes applied for determining pile efficiency in which efficiency decreases by 1/16 for each adjacent pile.

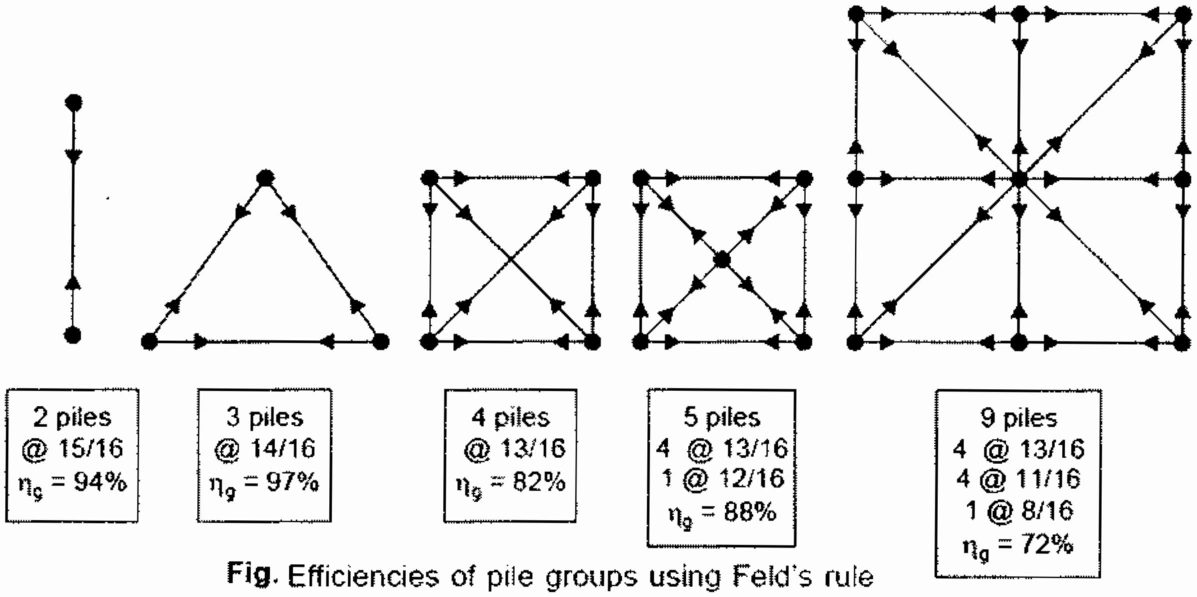


Fig. Efficiencies of pile groups using Feld's rule

Pile Group in Cohesionless soil.

- For piles driven in loose sand and gravel, the soil around 3 times diameter gets compacted and hence pile group acts as a block together as pier foundation having larger base area contained by the piles. So in pile group efficiency increases and may reach greater than unity but for design purpose efficiency is generally taken as unity.

  So, we can write, $Q_{gu} = n^*Q_u$
  Where, n = the number of piles in the group.

- When piles rest on compressible soils such as silts or clay then above statement does not hold well because stress transferred to compressible soil from pile group can result in over stressing or excessive consolidation. The carrying capacity of pile in such cases is governed by the shear failure and compressibility of soil criteria rather than group efficiency.

Pile Groups in Cohesive Soils

- Due to the possibility of remolding and lifting up of pile due to upheaval of soil in driven piles, so usually bored piles are preferred in cohesive soils. The piles should be driven from centre and then towards edges and should be kept at greater distances apart.
- From various experiments performed on piles in cohesive soils when loaded, it shows that it may fail individually (individual failure) or as a block (block failure). At closer spacing around 2-3 times diameter of piles, usually acts as a group and fails in group known as group failure. For piles kept at distance apart i.e. larger spacing (greater than 8 times diameter of piles), piles in pile group fails individually known as individual failure.

$$Q_{gu} = c_b * N_c * A_g + P_g * L * c_u * \alpha$$

Where,

c_b = cohesive strength of clay base of the pile group,
c_u = average cohesive strength of clay around the group,
L = Length of pile,
P_g = Perimeter of pile group,
A_g = Base area of group,
N_c = Bearing capacity factor, which may be assumed as 9 for deep foundation

Bearing capacity for individual failure:

$Q_{gu} = n * Q_u$

Where; n = Number of piles in the group,
Q_u = Bearing capacity of an individual pile.

Terzaghi and Peck recommends the value of load capacity of group is taken as smaller value given by above equations.

8.12 Negative skin Friction

When filled up soil stars consolidating under its own overburden pressure it develops a drag on the surface of a pile is called negative skin friction .It may also occurs when the fill is placed over the peat or soft clay strata .

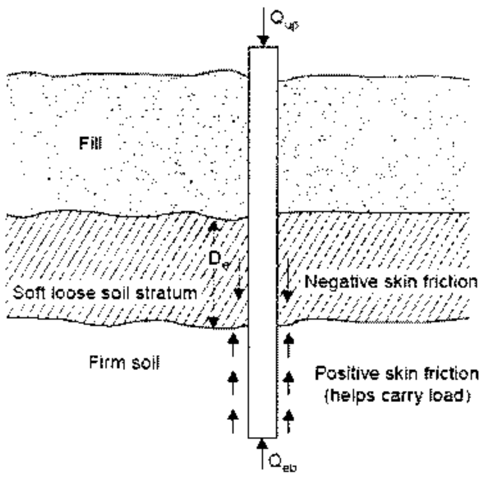


Fig. Negative skin friction on a pile

Here, net ultimate load carrying capacity of pile is decreased

$Q_u' = Q_u - Q_{nsf}$,
Q_{nsf} = negative skin friction

For cohesive soil:

$Q_{nsf} = C_u' * A_s'$
A_s'= shaft area of pile subjected to a negative skin friction

For Cohesionless Soil:

$Q_{nsf} = k\ \sigma_v'\ K \tan\delta * A_s'$
A_s'= shaft area of pile subjected to negative skin friction
$\sigma_v' = (0+ \gamma * L_c)/2$

Negative skin friction for pile group

Q_{ns} (g) = n * Q_{ns} (s)

Q_{ns} (g)= C_u'*A_s' (g) + γ* L_c A_b(g)

The greater value from above equation is taken as negative skin friction for pile group.

A_g(s) = perimeter area of pile group subjected to negative skin friction .

A_b(g) = area of group at base

L_c = length of soil unde the negative friction

Remedial measures for negative skin friction

1. By providing small area of cross section of pile shaft
2. By driving pile inside casing and space between pile and casing be filled with a viscous material
3. By coating a pile with bitumen

8.13 Laterally Loaded Piles

Piles and pile groups may be subjected to vertical loads, lateral loads or a combination of both. If the lateral loads act at an elevation considerably higher than the base of the foundation, there will be significant moments acting on it.

Vertical piles can resist lateral forces to a certain extent depending on the strength and stiffness of the pile and the soil . According to **IS 2911-1985**, permissible lateral load of a vertical pile is 2 – 5% of the permissible vertical load. For greater horizontal load, additional reinforcement is to be provided in the pile or raker piles may be used.

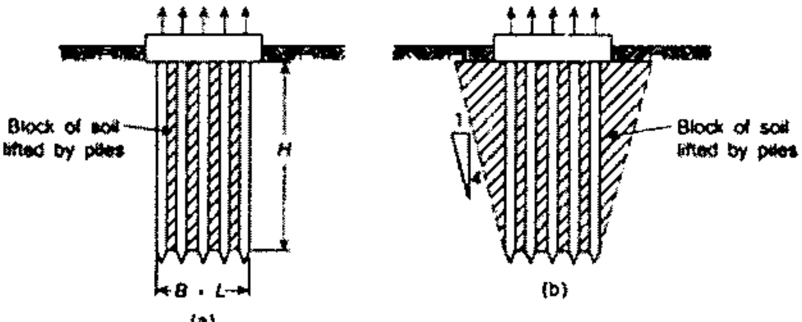
Extensive theoretical and experimental studies have been made on laterally loaded piles by Reese and Matlock (1960), Palmer and Brown (1954), and Murthy (1964). Most of these are based on the concept of coefficient of sub grade reaction, which is the pressure required to cause unit deflection.

8.14 Piles Subjected to Uplift Loads

A straight shaft pile, when subjected to uplift forces, derives its ultimate capacity from frictional resistance of the pile which can be determined in the same way as indicated for piles under compression. However for cyclic loading, skin friction may be reduced by the degradation of soil strength at the pile – soil interface under repetitive load. In particular, for sandy soils, reduction in uplift capacity to 50% of the ultimate skin friction has been reported. For cohesive soils, 30 – 50% reduction of uplift capacity in short augured piles has been observed.

As a general rule, a factor of safety of 3 – 4 on the frictional resistance calculated for compression may be applied to determine the uplift capacity of piles. However it should be noted that an upward movement of only 0.5 – 1% of the pile diameter is required to mobilize the peak frictional resistance.

For cohesionless soil, an assumed spread of 1:4 from the pile tip to the ground and the weight of soil block enclosed within the group give the frictional resistance. The submerged weight of the soil below the ground water table should be taken.



Uplift resistance of pile group

For cohesive soils, the uplift resistance of the block may be obtained by summing up the undrained shearing resistance around the periphery of the block and the weight of the soil enclosed by the group as

$$Q_u = 2(L + B)D_f c_u + W$$

where L = length of the pile group,
B = width of the pile group,
D_f = depth of the pile group,
c_u = average undrained shear strength of the clay, and
W = weight of the soil enclosed within the block.

A safety factor of 3 should be used to determine the safe uplift capacity of the group.

Q.1 A square group of 9 piles was driven into soft clay extending to a large depth. The diameter and length of the piles were 30 cm and 9 m respectively. If the unconfined compression strength of the clay is 90 kN/m2, and the pile spacing is 90 cm centre to centre, what is the capacity of the group? Assume a factor of safety of 2.5 and adhesion factor of 0.75.

Block failure:
Since, it is a square group, 3 rows of 3 piles each will be used.

$$Q_g = c' . N_c . A_g + c . P_g . L$$

Here, cohesion $c = c' = 45$ kN/m^2

N_c is taken as 9.

$$L = 9 \text{ m} \quad B = 2s + d = 2 \times 0.9 + 0.3 = 2.1 \text{ m}$$

$$P_g = 4B = 8.4 \text{ m}$$

$$A_g = B^2 = 2.1^2 \text{ m}^2 = 4.41 \text{ m}^2$$

Substituting,

$$Q_g = 45 \times 9 \times 4.41 + 45 \times 8.4 \times 9$$

$$= \mathbf{5{,}186\ kN.}$$

Individual pile failure:

$$Q_g = n[q_{cb} . A_b + f_s . A_s]$$

$$= n[cN_cA_b + \alpha . c . A_s]$$

$$= 9\left[45 \times 9 \times \frac{\pi}{4} \times 0.3^2 + 0.75 \times 45 \times \pi \times 0.3 \times 9\right]$$

$$= 2{,}835 \text{ kN}$$

In this case, individual pile failure governs the design. Allowable load on the pile group

$$= \frac{2{,}835}{2.5} = \mathbf{1{,}130\ kN.}$$

Q2. A 16-pile group has to be arranged in the form of a square in soft clay with uniform spacing. Neglecting end-bearing, determine the optimum value of the spacing of the piles in terms of the pile diameter, assuming a shear mobilization factor of 0.6.

At the optimum spacing, efficiency of the pile group is unity.

Let d and s be the diameter and spacing of the piles. Let L be their length.

Width of the block for a 16-pile square group.

$$B = 3s + d$$

Group capacity for block failure

$$= 4L(3s + d) \times c$$

where c is the unit cohesion of the soil.

Group capacity based on individual pile failure

$$= n[(0.6c)(\pi dL)]$$

$$= 16 \times 0.6\, \pi d\, Lc$$

Equating these two,

$$4Lc(3s + d) = 16 \times 0.6\, \pi d\, Lc$$

$$12s + 4d = 9.6\, \pi d$$

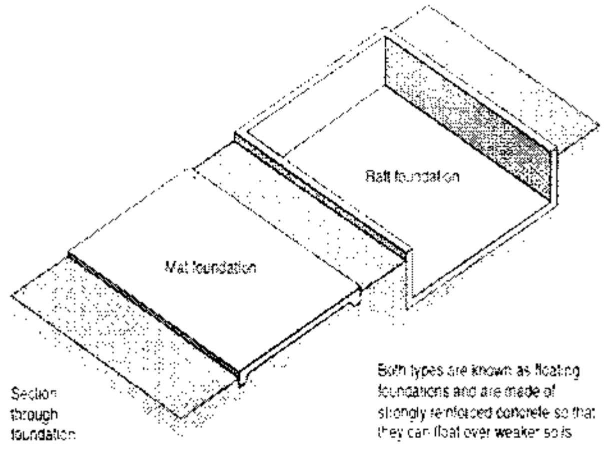
$$s = \frac{(9.6\pi - 4)}{12} d = 2.18d$$

$\therefore$ The optimum spacing is about **2.2 d.**

7.0 Mat / Raft Foundations

7.1 Introduction

A 'raft' or a 'mat' foundation is a combined footing which covers the entire area beneath of a structure and supports all the walls and columns.



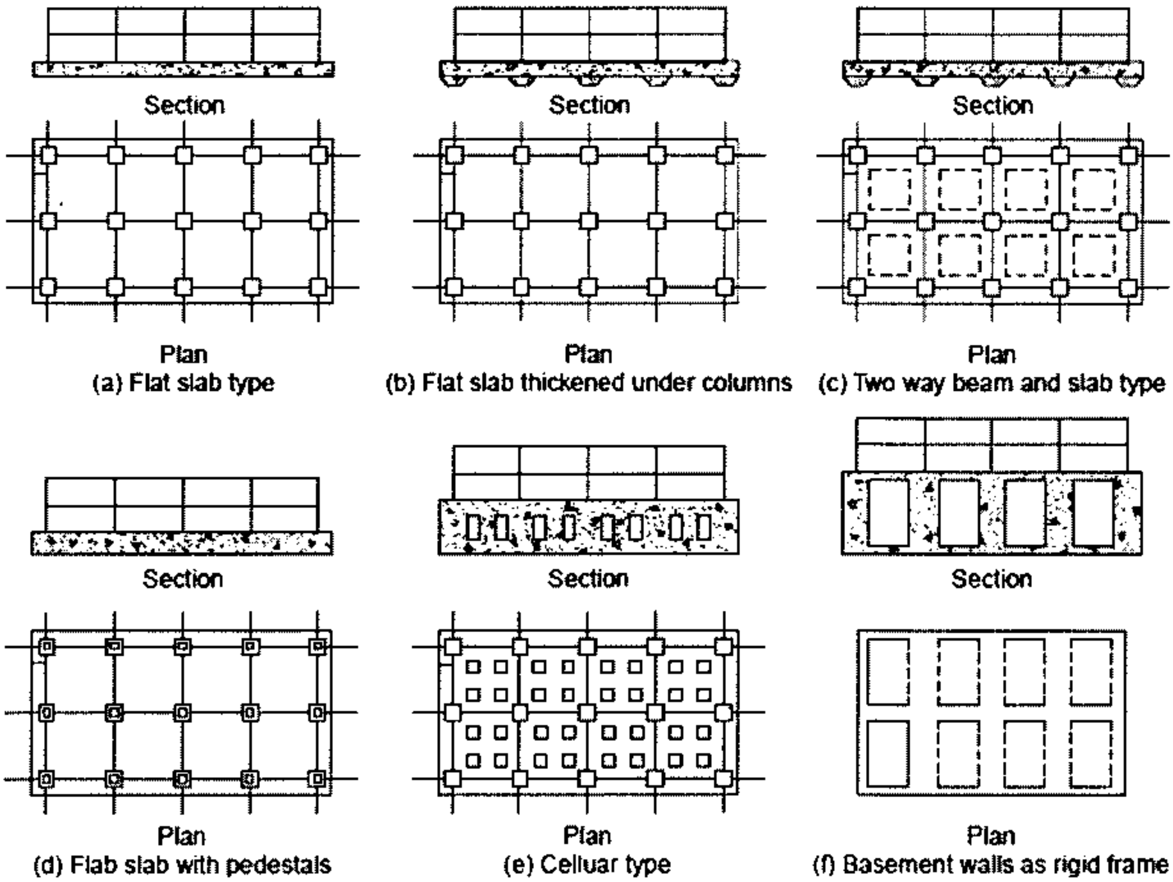
This type of foundation is most appropriate and suitable when the allowable soil pressure is low, or the loading heavy, and spread footings would cover more than one half the plan area. Also, when the soil contains lenses of compressible strata which are likely to cause considerable differential settlement,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

On occasions, the principle of floating foundation may be applied best in the case of raft foundations, in order to minimise settlments.

A mat foundation comes to be more economical than the individual footings when the total base area required for individual footings exceeds about one half of the area covered by the structure.

7.2 Common Types of Raft Foundations

Common types of raft foundations in use are illustrated in Fig.



Common types of raft foundations (Teng, 1976)

7.3 Bearing capacity and settlement of Mat foundation

Bearing Capacity of Rafts on Sands

- Since the bearing capacity of sand increases with the size of the foundation and since rafts are usually of large dimensions, a bearing capacity failure of raft on sand is practically ruled out.

- As a raft bridges over loose pockets and eliminates their influence, the differential settlements are much smaller than those of a footing under the same pressure. Hence, higher allowable soil pressures may be used for design of rafts on sands.

- Terzaghi and Peck (1948), as also Peck, Hanson and Thornborn (1974), recommend an increase of 100% over the value allowed for spread footings. The design charts developed for the bearing capacity from N-values for footings on sands may be used for this purpose. The effect of the location of water table is treated as in the case of footings.

- Allowable bearing capacity for **surface-loaded** footings with settlement limited to approximately 25 mm

$$q_a = \frac{N_{55}}{0.08}\left(\frac{B+0.3}{B}\right)^2\left(1+0.33\frac{D}{B}\right)$$

for: $0 \leq D \leq B$ and $B > 1.2$ m

The safe bearing capacity can be determined as per Teng's (1962)

- From shear failure criteria

$$qns = 0.22N^2 B W_\gamma + 0.67(100 + N^2) Df Wq$$

- The safe settlement pressure for a settlement of 25mm is given by,

$$qnp = 17.5\,(N - 3)\,W_\gamma \ \ kN/m^2$$

The equation is further modified as Teng's equations for safe settlement pressure are found conservative and Bowel's gave equations for a safe settlement of 25mm.

$$q_{np} = 12.2N((B + 0.3)/B)^2 R_d W\gamma$$ Where, $$Rd = 1 + 0.33\,(Df/B)$$

7.4 Compensated foundation

Bearing Capacity of Rafts on Clays

The net ultimate bearing capacity is divided by the factor of safety to obtain the net allowable soil pressure for a footing. The same principle is applicable to rafts on clay. Accordingly, the factor of safety, η, in terms of net soil pressure, is given by

$$\eta = \frac{cN_c}{(q - \gamma D_f)}$$

where, c = unit cohesion,
N_c = bearing capacity factor for cohesion,
q = gross soil pressure or contact pressure,
γ = unit weight of soil,
and D_f = depth of raft below ground surface.

In other words,

- Factor of safety against bearing capacity failure can be represented as,

$$qnu = Cu \cdot N_C$$

$$q_{nu}= c_u \cdot 5\left(1 + 0.2\frac{D_f}{B}\right)\left(1 - \frac{0.2B}{L}\right)$$

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

Or,

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

$$F = Cu \cdot 5(1 + 0.2\ Df/B)\ (1 - 0.2B/L)/(\frac{Q}{A} - \gamma\ Df)$$

IS 6403 recommends a minimum factor of safety 2.5, usually F is taken as 3.

- It is obvious that the factor of safety is very large for rafts established at such depths that γDf is nearly equal to q and it is obvious that the factor of safety is very large for rafts established at such depths that γDf is nearly equal to q.

  i.e,

  $q = Q/A = \gamma\ D_f$, implies $F = \infty$

  And $D_f = (Q/A\ \gamma)$
- The foundation satisfying above requirements is known to be 'fully compensated foundation' (Peck, Hanson and Thornburn, 1974), or floating foundation.

7.5 Analysis of mat foundation

Step 1: Calculate the column load

$Q = Q_1 + Q_2 + Q_3 + \ldots\ldots\ldots\ldots + Q_n$

Step 2: determine the pressure on the soil (q) below the mat at a point A,B,C,D…… by using the equation.

$$q = \frac{Q}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$$

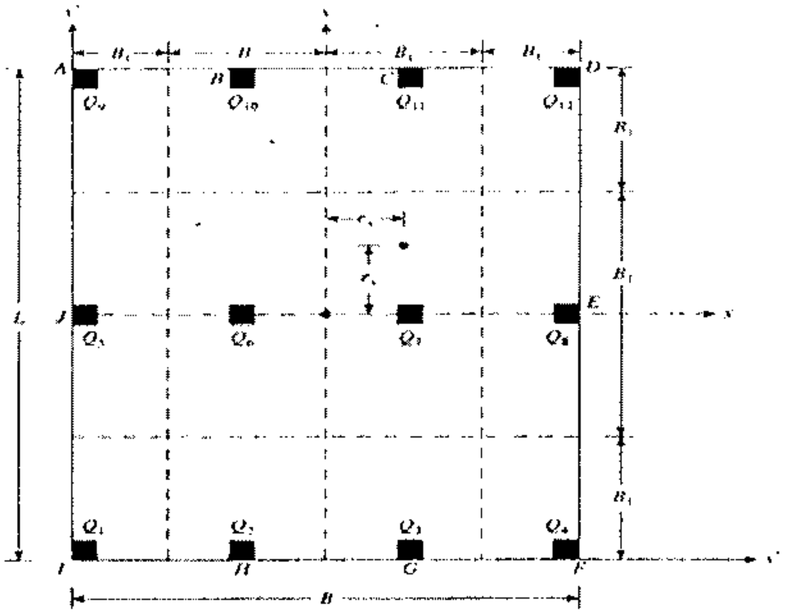
Where: A = B*L = Area of mat

$I_x = (1/12)\ BL^3$ = Moment of inertia about the x axis

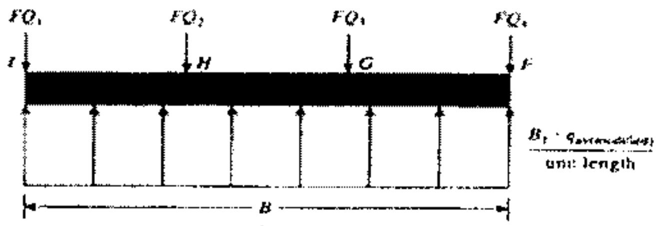
$I_y = (1/12)\ LB^3$ = Moment of inertia about the y axis

Mx = moment of the column loads about the x axis = $Q*e_y$

My = moment of the column loads about the y axis = $Q*e_x$



(a)



(b)

$$X' = \frac{Q_1x'_1 + Q_2x'_2 + Q_3x'_3 + \cdots}{Q}$$

and

$$e_x = X' - B/2$$

Similarly,

$$Y' = \frac{Q_1y'_1 + Q_2y'_2 + Q_3y'_3 + \cdots}{Q}$$

and

$$e_y = Y' - L/2$$

Step 3: Compare the values of the soil pressure determined in step 2 with net allowable soil pressure to check if $q \leq q_{all}$.

Step 4: Divide the mat into several strips in x and y directions, let the width of any strip be B_1.

Step 5: Draw the shear (V) and moment (M) diagram for each individual strip (in x and y direction). For example , take the bottom strip in the x direction of, its average soil pressure can be given as

$$q_{av}' = \frac{q_I+q_F}{2}$$

Where q_I and q_F = soil pressure at point I and F as determined from step2.

The total soil reaction is equal to $q_{av}B_1B$. Now obtain the total column load on the strip as $Q_1+Q_2+Q_3+Q_4$. The sum of the shear between the adjacent strips has not been taken into account. For this reason, the soil reaction and column load need to be adjusted, or

$$\text{Average load} = \frac{q_{av}B_1B+(Q_1+Q_2+Q_3+Q_4}{2}$$

Now, modified average soil reaction,

$$q_{av(modified)} = \frac{q_{av}(Average\ Load)}{q_{av}B_1B}$$

Step 5: Draw the shear (V) and moment (M) diagram for each individual strip (in x and y direction). For example , take the bottom strip in the x direction of, its average soil pressure can be given as

Also, the column load modification factor is

$$F = \text{Average Load} / (Q_1+Q_2+Q_3+Q_4)$$

So, the modification column loads are FQ_1,FQ_2,FQ_3 and FQ_4. This modified loading on the strip under consideration is shown in figure.

Step 6: Now, shear force and bending moment diagram for this strip can be drawn. This procedure can be repeated for all strips in x and y directions.

Step 7: Design the individual strips for the bending moment and shear force found in step 6. The raft is designed as an inverted floor supported at columns.

CHAPTER 9: Well Foundations

9.1 INTRODUCTION

A '*Caisson*' is a type of foundation of the shape of a box, built above ground level and sunk to the required depth as a single unit. This terminology is popular in the U.S.A., and is used to refer to a water-tight chamber employed for laying foundations under water as in lakes, rivers, seas, and oceans.

9.2 Types of caisson foundations:

Caissons are broadly classified into three types, based on the method of construction:
(*a*) Open Caissons
(*b*) Pneumatic Caissons
(*c*) Floating or box Caissons

(*a*) *Open Caissons:* These are of box-shape, open both at the top and the bottom during construction. The caisson is sunk into position, and upon reaching its final position, a concrete seal is placed at its bottom in water. Finally, the inside is pumped dry and filled with concrete.

(*b*) *Pneumatic Caissons:* These are of box-shape, closed at the top, with a working chamber at the bottom from which water is kept off with the aid of compressed air. Thus excavation is facilitated in the dry, and the Caisson sinks as excavation proceeds. Finally, the working chamber is filled with concrete, upon reaching the final location at the desired depth.

(*c*) *Floating or Box Caissons:* These are also of box-shape, closed at the bottom and open at the top. This type of Caisson is cast on land, launched in water, towed to the site, and sunk into position by filling it with sand, gravel, concrete, or water.

Timber, Steel, and Reinforced Concrete are the materials used to construct Caissons, depending upon the importance and magnitude of the job. Timber is much less used these days than steel and Reinforced Concrete. Steel Caissons are made of steel skin plate, internal steel frames, and Concrete fill, the last one being meant only to provide the necessary weight to aid in the sinking process, which is more continuous, and relatively faster when compared with Caissons built of reinforced Concrete.

Reinforced Concrete Caissons utilise concrete for the dual purpose of providing the necessary strength and the dead weight for sinking. These must be poured in convenient heights, called 'Lifts', cured for the mandatory period, and each lift sunk into position. This necessarily involves some loss of time, the time required for the sinking operation being much more than that for a Steel Caisson. However, Concrete Caissons prove to be much more economical than Steel ones for large and heavy jobs.

Generally speaking, a Caisson is advantageous compared to other types of deep foundations when one or more of the following conditions exist:

(*i*) The soil contains large boulders which obstruct the penetration of piles or placement of drilled piers ('drilled piers' are nothing but large diameter bored piles.)
(*ii*) A massive substructure is required to extend below the river bed to resist destructive forces due to scour and/or floating objects.
(*iii*) Large magnitudes of lateral forces are expected.

Caissons are mostly used as the foundation for bridge piers and abutments in lakes, rivers, and seas, breakwaters and other shore protection works, and large water-front structures such as pump houses, subjected to huge vertical and horizontal forces. Occasionally Caissons, especially Pneumatic Caissons, have been used as foundations for large and tall multi-storey buildings and other structures.

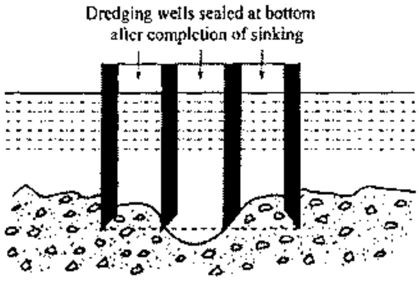


Fig. 12.1 Open caisson

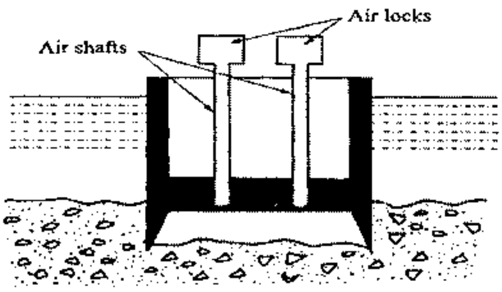


Fig. 12.2 Pneumatic caisson

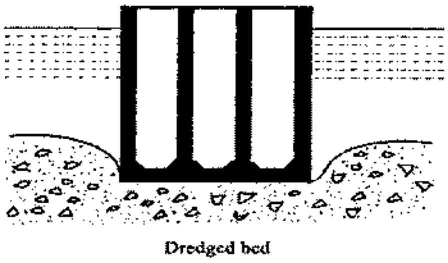
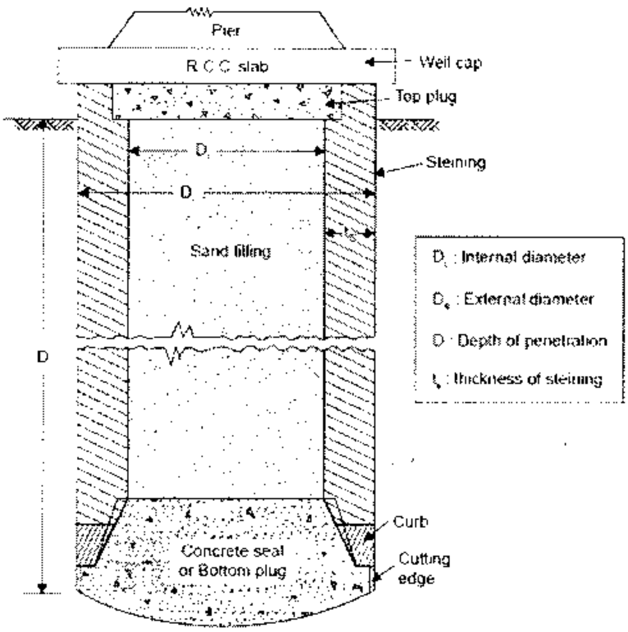


Fig. 12.3 Box caisson



9.3 Components of a Well Foundation

The elements of a well foundation are: (*i*) Cutting edge (*ii*) Curb (*iii*) Concrete seal or Bottom Plug (*iv*) Steining (*v*) Top Plug, and (*vi*) Well Cap. These shown in the sectional elevation of a typical well foundation of circular cross section.

Fig. 9.2: Components of well foundations

(*i*) *Cutting Edge:* The function of the cutting edge is to facilitate easy penetration or sinking into the soil to the desired depth. As it has to cut through the soil, it should be as sharp as possible, and strong enough to resist the high stresses to which it is subjected during the sinking process. Hence it usually consists of an angle iron with or without an additional plate of structural steel. It is similar to the sharp-edged cutting edge of a caisson shown in Fig. 9.2.

(*ii*) *Steining:* The steining forms the bulk of the well foundation and may be constructed with brick or stone masonry, or with plain or reinforced concrete occasionally. The thickness of the steining is made uniform throughout its depth. It is considered desirable to provide vertical reinforcements to take care of the tensile stresses which might occur when the well is suspended from top during any stage of sinking.

(*iii*) *Curb:* The well curb is a transition member between the sharp cutting edge and the thick steining. It is thus tapering in shape. It is usually made of reinforced concrete as it is subjected to severe stresses during the sinking process.

(*iv*) *Concrete Seal or Bottom Plug:* After the well foundation is sunk to the desired depth so as to rest on a firm stratum, a thick layer of concrete is provided at the bottom inside the well, generally under water. This layer is called the concrete seal or bottom plug, which serves as the base for the well foundation. This is primarily meant to distribute the loads on to a large area of the foundation, and hence may be omitted when the well is made to rest on hard rock.

(*v*) *Top Plug:* After the well foundation is sunk to the desired depth, the inside of the well is filled with sand either partly or fully, and a top layer of concrete is placed. This is known as 'top plug'. The sand filling serves to distribute the load more uniformly to the base of the well, to reduce the stresses in the steining, and to increase the stiffness of the well foundation. However, as this adds to the weight and load transmitted to the foundation stratum, the engineer has to consider the desirability or otherwise of providing the sand filling from the point of view of bearing power and settlement. The top plug of concrete serves to transmit the loads to the base in a uniform manner.

(*vi*) *Well Cap:* The well cap serves as a bearing pad to the superstructure, which may be a pier or an abutment. It distributes the superstructure load onto the well steining uniformly.

9.4 Shapes of well or caisson foundations:

Caissons are constructed with practically straight and vertical sides from top to bottom. The shape of a Caisson in plan may be Circular, Square, Rectangular, Octogonal, Twin-Circular, Twin-Rectangular, Twin-Hexagonal, Twin-Octogonal, or Double-D as shown in Fig. 9.1. Sometimes, the choice of shape of a Caisson is influenced by its size (for example, the shape is governed by the outline of the base of the superstructure, especially for large superstructures; smaller ones may, however, be made circular for convenience in sinking and achieving economy), and by the shape of the superstructures (for example, oblong shape may be preferred for the superstructure either to avoid restriction of flow or for convenience in navigation; or circular or pointed shape may be preferred on the upstream side to minimise the possible impact from large and heavy floating objects or ice floes). Twin-Circular, Twin- Rectangular, Twin Hexagonal, Twin-octogonal, and the Double-D types are used to support heavy loads from large bridge piers.

The size of a Caisson is governed by the following factors:

(*i*) *Size of Base:* The size of Caisson should be such that the Caisson has a minimum projection of 0.3 m all-round the base of the superstructure; this would help take care of a reasonable amount of inevitable tilting and misalignment.
(*ii*) *Bearing Pressure:* The area required is obviously governed by the allowable bearing pressure of the soil (dealt with in later subsections).
(*iii*) *Practical Minimum Size:* A minimum size of 2.5 m is considered necessary from the point of view of convenience in sinking and economy in construction; smaller sizes of Caisson frequently prove more expensive than other types of deep foundation.

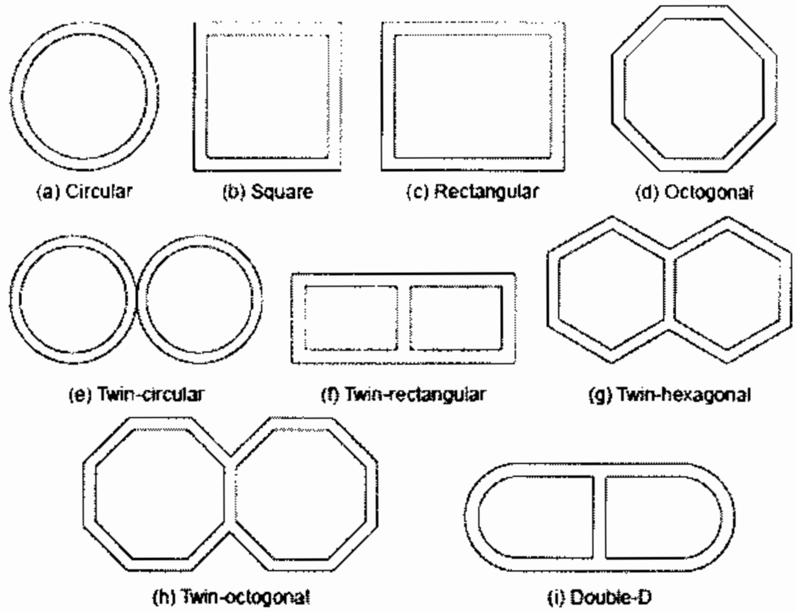


Fig. 9.1: Various shapes of well foundations

9.5 Depth of well foundation:

- In order to resist lateral forces acting on the well it should be placed at sufficient depth below maximum scour level
- Depth of well foundation is chosen by considering grip length and bearing capacity.
- The depth of bottom of well below max. Scour level is known as grip length.
- The max. and min. pressure developed on base should be within permissible limit.

The depth of scour can be ascertained by sounding near proposed site immediately after flood. (The max. scour would be greater than measured scour as design discharge is greater than flood discharge and there is increase in velocity due to obstruction to flow by construction of bridge.)

In case the first approach of taking soundings is not feasible, the second approach may be used and the normal depth of scour may be calculated by Lacey's formula:

$$d = 0.473(Q/f)^{1/3}$$

where d = normal scour depth, measured below high flood level (m),

Q = design discharge (m^3/s),

and f = Lacey's silt factor.

The silt factor may be calculated from the equation

$$f = 1.76\sqrt{d_m}$$

where d_m = mean size of the particle (mm).

The regime width of the waterway, w, can be computed as

$$w = 4.75Q^{1/2}$$

If the actual waterway, L, is less than the regime width, the actual scour depth, d', is given by

$$d' = d(w/L)^{0.61}$$

Recommended values of Lacey's silt factor, f, for particular particle sizes in the ranges of coarse silt to boulders are given in the Indian Standard Code of Practice:

IS:3955-1967-"Indian Standard Code of Practice for Design and Construction of Well Foundations".

Values of maximum scour depth as recommended by IRC (1966)* and IS:3955-1967 are given in Table 9.1

Table 9.1 Maximum scour depth recommended by IRC

S.No.	*River Section*	*Maximum Scour Depth*
1.	Straight reach	1.27 d'
2.	Moderate Bend	1.50 d'
3.	Severe Bend	1.75 d'
4.	Right-angled bend or at nose of Pier	2.00 d'
5.	Upstream nose of guide banks	2.75 d'
6.	Severe Swirls (IS:3955-1967 only)	2.50 d'

The grip length for wells of railway bridges is taken as 50% of maximum scour depth, generally, while for road bridges 30% of maximum scour depth is considered adequate. The base of the well is usually taken to a depth of 2.67 d' below the HFL.

According to IS:3955-1967, the depth should not be less than 1.33 times the maximum scour depth. The depth of the base of the well below the scour level is kept not less than 2 m for piers and abutments with arches, and 1.2 m for piers and abutments supporting other types of structures.

If inerodible stratum like rock is available at a shallow elevation, the foundation may be taken into it and securely bonded, or anchored to it if necessary.

9.6 Forces Acting on Well Foundations

The following forces should be considered in the design of a well foundation:

(1) *Dead Loads:* The weight of the superstructure and the self-weight of the well foundation constitute the dead loads.

(2) Live Loads: The live loads in the case of highway bridges are specified by IRC Standard specifications and code of practice for Road Bridges-Sec. II (1966). Live loads for railway bridges are specified in the Indian Railway Bridge Rules (1963) given by Research, Design, and Standards Organisation (RDSO), Lucknow of the Ministry of Railways, Govt. of India.

(3) Impact Loads: The live loads cause impact effect and it is considered in the design of pier cap and bridge seat on the abutment. Impact effect may be ignored for the elements of the well.

(4) Wing Loads: Wind loads on the live load, superstructure, and the part of substructure located above the water level are calculated based on IS:875-1964 "Indian Standard Code of Practice for Structural Safety of Buildings-Loading Standards". Wind Loads act on the exposed area laterally.

(5) Water Pressure: Water Pressure is due to the water current acting on the part of the substructure between the water level and the maximum scour level.
The intensity of Water Pressure on piers parallel to the direction of flow is given by

$$p = K.\ v^2$$

where p = Intensity of Water Pressure (N/m2),
v = Velocity of the water current (m/s),
and K = a constant, which depends upon the shape of the well (maximum 788 for square ended piers, and minimum 237 for piers with cut-waters and ease-waters).

v is taken to be the maximum at the free surface of flow and zero at the deepest scour level, the variation being assumed to be linear. The maximum value is taken to be 2 times the average value.

A transverse force of 20% of that parallel to the flow is assumed to allow for occasional obliquity of flow.

(6) Longitudinal Force: Longitudinal force occurs due to tractive and braking forces. These are transmitted to the substructure mainly through fixed bearings and through friction is movable bearings. According to IRC code, a longitudinal force of μW is taken on the free bearing, and the balance on the fixed bearing, where W is the total reaction and μ is the coefficient of friction.

(7) Earth Pressure: The earth pressure is calculated based on one of the classical earthpressure theories of Rankine and Coulomb. Passive earth resistance of the soil is taken into account for the stability of foundations below the scour level. The effect of the live load on the abutment on the earth pressure is considered by taking an equivalent height of surcharge.

(8) Centrifugal Force: A centrifugal force is taken to be transmitted through the bearings if the superstructure is curved in plan.

(9) Buoyancy Force: Buoyancy reduces the effective weight of the well. In masonry or concrete steining, 15% of the weight is taken as the buoyancy force to account for the porousness. When the well is founded on coarse sand, full buoyancy equal to the weight of the displaced volume of water is considered. For semiprevious foundations, appropriate reduction may be made based on the location of water table.

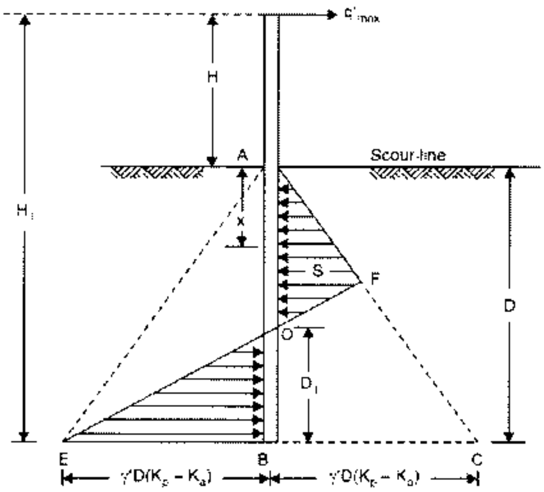
(10) Temperature Stresses: Longitudinal forces are induced owing to temperature changes. The movements due to temperature changes are partially restrained in girder bridges because of friction.

(11) Seismic Forces: These are to be considered in Seismic Zones. The force is taken is αW, where W is the weight of the component, and α is the seismic coefficient. The value of α depends upon the Zone and is given in IS: 1893-1975 "Indian Standard Criteria of Earthquake-Resistant Design of Structures". Its value ranges from 0.01 to 0.08. The Seismic Force acts through the centre of gravity of the component.

(12) Resultant Force: The magnitude, direction, and the point of application of all the applicable forces are found for the worst possible combination. The resultant can be imagined to be replaced by an equivalent vertical force W, and lateral forces, P and Q in the longitudinal and transverse directions of the pier, respectively. The action of Q will be more critical in the consideration of lateral stability of the well.

9.7 LATERAL STABILITY OF WELL FOUNDATIONS:

Terzaghi's (1943) Analysis:



Terzaghi's (Terzaghi, 1943) solution for free rigid bulkheads may be used for the approximate analysis of a well foundation. When a rigid bulkhead embedded in sand moves parallel to its original position, the sand on the front side and rear side are respectively transformed into passive and active states. Assuming that both the active pressure and the passive resistance are fully mobilised, the net pressure at any depth z below the ground surface is given by

$$p = \gamma z(Kp - Ka)$$

A free rigid bulkhead depends for its stability solely on the lateral resistance.

Let q'max be the horizontal force per unit length acting on the structure of total height $H1$ (as given in Fig.). The pressure distribution on both sides of the bulkhead at incipient failure may be represented as shown. The bulkhead rotates about the point 0 at a height of $D1$ above the base. As the soil around the well is usually submerged, the submerged unit weight is used.

Considering unit length, and applying $\Sigma H = 0$,

$$q'_{max} = \text{Area } ABC - \text{Area } FEC$$

$$= \frac{1}{2}\gamma' D^2 (K_p - K_a) - \frac{1}{2}(2\gamma' D)(K_p - K_a).D_1$$

$$= \frac{1}{2}\gamma' D(K_p - K_a)(D - 2D_1)$$

(Note: For convenience, the height to F is also taken as D_1 nearly)

Taking moments about the base,

$$q'_{max}H_1 = \frac{1}{2}\gamma' D^2 (K_p - K_a)\frac{D}{3} - \frac{1}{2}(2\gamma' D)(K_p - K_a)\frac{D_1^2}{3}$$

Substituting for q'_{max} from Eq. 19.27,

$$\frac{1}{2}\gamma' D(K_p - K_a)(D - 2D_1).H_1 = \frac{1}{2}\gamma' D^2 (K_p - K_a)\frac{D}{3} - \frac{1}{2}(2\gamma' D)(K_p - K_a)\frac{D_1^2}{3}$$

or
$$(D - 2D_1)H_1 = \frac{D^2}{3} - \frac{2D_1^2}{3}$$

or
$$D_1^2 - 3D_1H_1 + (1.5DH_1 - 0.5D^2) = 0$$

Solving for D_1,

$$2D_1 = 3H_1 \pm \sqrt{(3H_1)^2 - 2D(3H_1 - D)}$$

or
$$D_1 = \frac{1}{2}\left[3H_1 \pm \sqrt{9H_1^2 - 2D(3H_1 - D)}\right]$$

. The positive sign yields a value for D_1 greater than D, which is ridiculous.

Hence, rejecting the positive sign,

$$D_1 = \frac{1}{2}\left[3H_1 - \sqrt{9H_1^2 - 2D(3H_1 - D)}\right]$$

Substituting this value in Eq.1, q'_{max} can be computed. For K_p and K_a, Rankine values can be used. (It is interesting to note that K_p and K_a do not appear in the Eqs. of $D1$.

In this simplified analysis, the moments due to side friction and base reaction are neglected; the error is on the safe side, since this results in the under estimating of the stabilizing forces.

Heavy Wells

Wells are in general, heavy compared to bulkheads, with low ratios of length to lateral dimension. A heavy well is expected to rotate about its base, as observed in model experiments by several investigators; the force per unit length may be obtained by taking moments about the base.

$$q'_{max}H_1 = (1/2)\gamma'(K_p - K_a)D^2 \times \frac{D_1}{3}$$

or
$$q'_{max} = (1/6)\gamma'(K_p - K_a)\frac{D^3}{H_1}$$

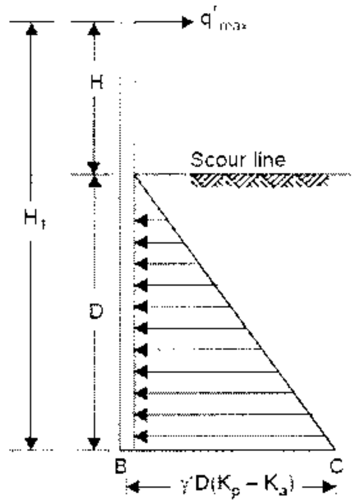


Fig. Heavy well

Effect of Surcharge

The effect of surcharge due to the weight of soil above the scour line can be considered in the analysis. The soil below the maximum scour line is subjected to a surcharge Z of the unscoured soil (Fig. below). The height Z may be taken as half the normal depth of scouring case it is not possible to ascertain it by actual measurement.

The pressure distribution is shown in the figure. The maximum pressure at the base is equal to $\gamma' (K_p - K_a)(D + Z)$.

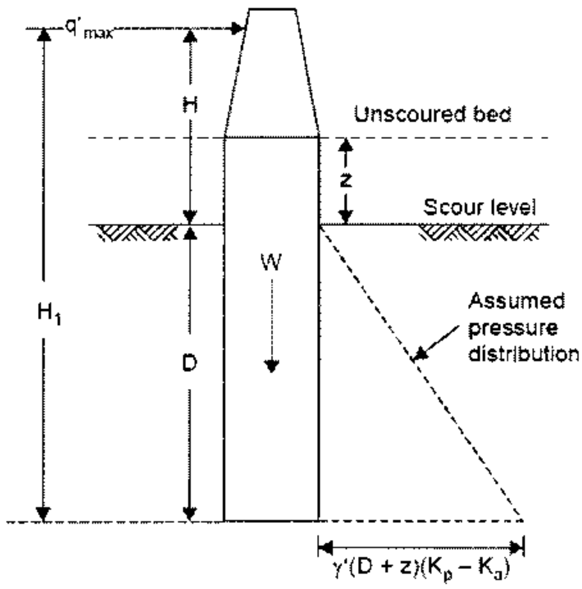


Fig. Effect of surcharge on wells

In this case q'_{max} is given by

$$q'_{max} = (1/6)\gamma(K_p - K_a).\frac{D^2(D+z)}{H_1}$$

Total Safe Lateral Load on a Well

Since the equations for q'_{max} by Terzaghi's analysis, Heavy wells and effect of surcharge give only the lateral load per unit length of well, this value should be multiplied by the length of the well, L, parallel to the water flow in order to obtain the total lateral load on the well. Since the bulkhead equations are derived based on the assumption that the length is very much larger than the width, in practice, the error is considered to be not appreciable if wells are rectangular in shape. A multiplying factor, less than unity, called the 'Shape Factor', has to be applied for circular wells. This factor is taken to be $\frac{\pi}{4}$.

However, the shape factor for circular wells with a diameter larger than 4.5m, is taken to be unity, as for rectangular wells. The safe later load, Qa, for the well would be got by applying K_p' in place of K_p in the relevant equation for q'_{max}. and multiplying by the length and the shape factor as applicable.

Base Pressures

If Q is the actual applied transverse (horizontal) load and Q_a is the allowable equivalent resisting force, the unbalanced force $(Q - Q_a)$ acting at a height H above the scour level would produce an overturning moment M_B about the base.

$$M_B = (Q - Q_a)(H + D)$$

The maximum and minimum pressures at the base will then be

$$q_{max} = \frac{W}{A_b} + \frac{M_B}{Z_b}$$

and

$$q_{min} = \frac{W}{A_b} - \frac{M_B}{Z_b}$$

where W = net vertical load on the base of the well, after making allowance for buoyancy and skin friction,

A_b = Area of the base of the well,

and Z_b = Section modulus of the base cross-section of the well.

The maximum pressure should not exceed the allowable soil pressure. The minimum pressure should not be negative, that is to say, it should not be tensile. It is the general practice not to give any relief due to skin friction while calculating the maximum pressure at the base in clays, but to consider it for calculating the minimum pressure, so that the worst conditions are taken into account in either case.

Maximum Moment in Steining

The maximum moment in the steining occurs at the point of zero shear. Referring Terzaghi's analysis the depth χ to the point of zero shear, S, is such that the applied force and force due to the mobilised earth pressure balance each other. With a factor of safety η, $K_p' = K_p/\eta$.

$$[1/2\gamma'(K_p' - K_a).\chi^2.L] = Q$$

or

$$x = \left[\frac{2Q}{\gamma'(K_p' - K_a).L}\right]^{1/2}$$

Taking moments about S,

$$M_{max} = Q(H + \chi) - (\text{Force due to pressure}).\chi/3$$

Taking the force due to pressure as being equal to Q,

$$M_{max} = Q(H + \chi) - Q(\chi/3)$$

or

$$M_{max} = Q.H + (2/3)Q.\chi$$

If the well rests on rock or on unyielding stratum, no rotation need be expected, and the moment developed is transmitted to the foundation bed, which withstands it.

9.8 Construction and sinking of well foundation:

(1) Construction of the Well Curb

If the river bed is dry, the cutting edge over which the well curb is to be built is placed at the correct position after excavating the bed for about 150 mm for seating. If there is water, with a depth upto 5 m, a sand island is created before placing the curb. The size of the island should be large enough to accommodate the well with adequate working space all round (Fig. 19.22). In case the depth of water is more than 5 m, it is more economical to build the curb on the bank and float it to the site.

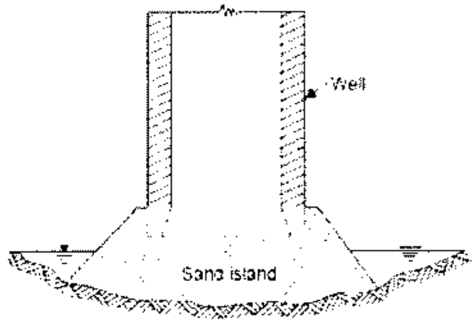


Fig. 19.22 Sand island for sinking a well

Wooden sleepers are usually inserted below the cutting edge at regular intervals to distribute the load evenly on the soil. The shuttering of the well curb is erected—the outer one with steel or wood and inner with brick masonry. The reinforcements for the curb are then placed in position, the vertical bars projecting about 2 m above the top of the curb. Concreting of the curb is done in continuous operation. After the curb is cured and allowed to cure for at least seven days, the shuttering may be removed as also the sleepers.

(2) Construction of Well Steining

The Steining is constructed with a height of 1.5 m at a time and sinking done after allowing at least 24 hours for setting. Once the well has acquired a grip of about 6 m into the ground, the steining can be raised 3 m at a time. The height of any lift is restricted such that the well does not lose stability.

(3) Sinking Process

The sinking process is commenced after the curb is cast and the first stage of steining is ready after curing. The material is excavated from inside manually or mechanically. Manual dredging is feasible when the depth of water inside the well is not more than 1 m. An automatic grab operated by diesel winches is used when the depth of water is more. Blasting with explosives is used when weak rock is encountered.

Additional loading, known as 'Kentledge' is used, if necessary. Kentledge is generally in the form of sand bags placed on a suitable platform on top of the well. Water jetting on the exterior face is applied in conjunction with kentledge. Pumping water from inside the well is also effective in sinking a well. But this should be resorted to only when the well has gone sufficiently deep into the ground, so as to avoid tilts and shifts. Also dewatering is not used after the well has sunk to about 10 m. 'Blow of sand' may occur if dewatering is resorted to in the early stage of sinking, inducing sudden tilting, and posing hazards to the workmen. Scrap gunny bags and grass boundles are placed round the periphery of the well to prevent sand blow.

Shifts and Tilts

The well should be sunk straight and vertical at the correct position. It is not an easy task to The well should be sunk straight and vertical at the correct position. It is not an easy task to achieve this objective in the field. Sometimes the well tilts onto one side or it shifts away from the desired position.

The following precautions may be taken to avoid tilts and shifts:

(*i*) The outer surface of the well curb and steining should be smooth.

(*ii*) The curb diameter should be kept 40 to 80 mm larger than the outer diameter of the steining, and the well should be symmetrically placed.

(*iii*) The cutting edge should be uniformly thick and sharp.

(*iv*) Dredging should be done uniformly on all sides and in all the pockets.

Tilts and shifts must be carefully noted and recorded. Correct measurement of tilt is an important observation in well sinking. It is difficult to specify permissible values for tilts and shifts. IS:3955-1967 recommends that tilt should be generally limited to 1 in 60. The shift should be restricted to one percent of the depth sunk. In case these limits are exceeded, suitable remedial measures are to be taken for rectification.

Remedial Measures for Rectification of Tilts and Shifts:

The following remedial measures may be taken to rectify tilts and shifts:

(1) *Regulation of Excavation:* The higher side is grabbed more be regulating the dredging. In the initial stages this may be all right. Otherwise, the well may be dewatered if possible, and open excavation may be carried out on the higher side [Fig. 19.23 (*a*)].

(2) *Eccentric Loading*: Eccentric placing of the kentledge may be resorted to provide greater sinking effort on the higher side. If necessary a platform with greater projection on the higher side may be constructed and used for this purpose. As the depth of sinking increases, heavier kentledge with greater eccentricity would be required to rectify tilt [Fig. 19.23 (b)].

(3) *Water Jetting*: If water jets are applied on the outer face of the well on the higher side, the friction is reduced on that side, and the tilt may get rectified [Fig. 19.23 (c)].

(4) *Excavation under the Cutting Edge*: If hard clay is encountered, open excavation is done under the cutting edge, if dewatering is possible; if not, divers may be employed to loosen the strata.

(5) *Insertion of Wood Sleeper under the Cutting Edge*: Wood sleepers may be inserted temporarily below the cutting edge on the lower side to avoid further tilt.

(6) *Pulling the Well*: In the early stages of sinking, pulling the well to the higher side by placing one or more steel ropes round the well, with vertical sleepers packed in between to distribute pressure over larger areas of well steining, is effective [Fig. 19.23 (d)].

(7) *Strutting the Well*: The well is strutted on its tilted side with suitable logs of wood to prevent further tilt. The well steining is provided with sleepers to distribute the load from the strut. The other end of the logs rest against a firm base having driven piles [Fig. 19.23 (e)].

(8) *Pushing the Well with Jacks*: Tilt can be rectified by pushing the well by suitably arranging mechanical or hydraulic jacks. In actual practice, a combination of two or more of these approaches may be applied successfully [Fig. 19.23 (f)].

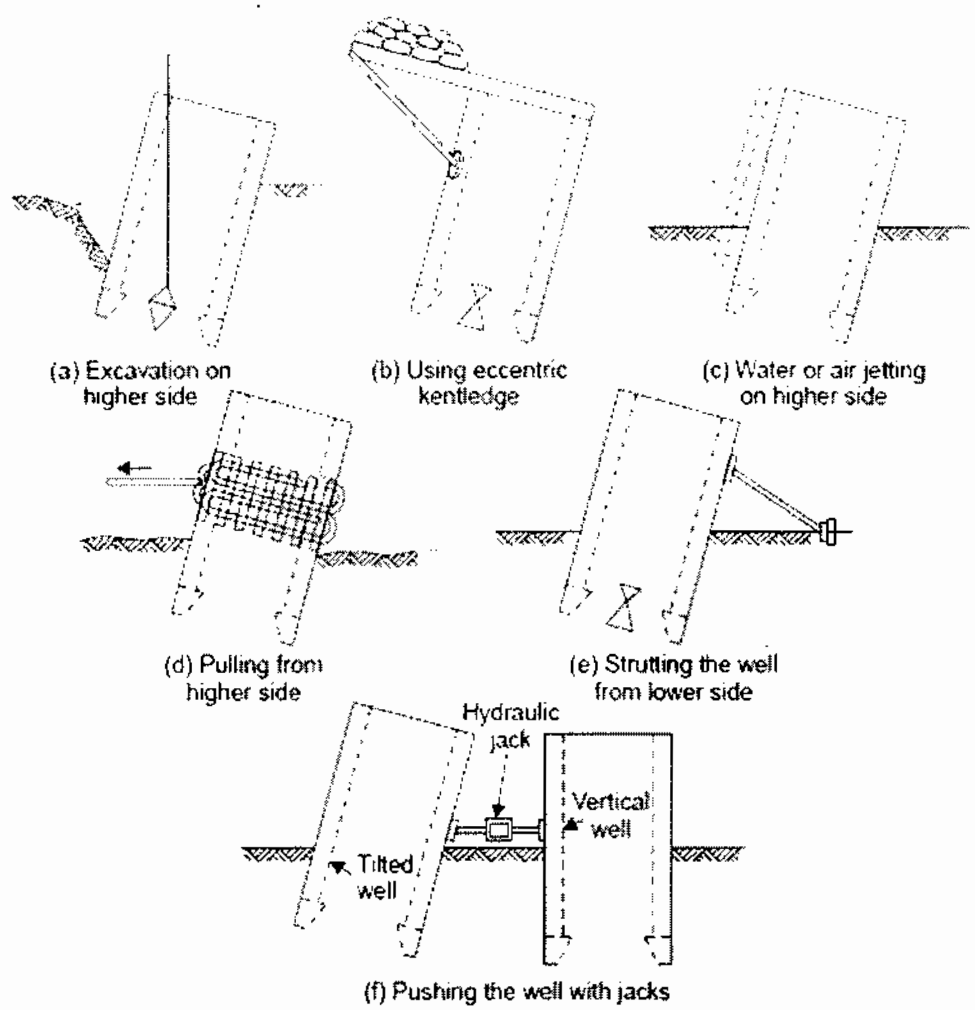


Fig. 19.23 Remedial measures for correction of tilt of wells

ILLUSTRATIVE EXAMPLES:

1. A cylindrical well of external diameter 6 m and internal diameter 4 m is sunk to a depth 16 m below the maximum scour level in a sand deposit. The well is subjected to a horizontal force of 1000 kN acting at a height of 8 m above the scour level. Determine the total allowable equivalent resisting force due to earth pressure, assuming that (*a*) the well rotates about a point above the base, and (*b*) the well rotates about the base. Assume $\gamma' = 10$ kN/m3, $\varphi = 30°$, and factor of safety against passive resistance = 2. Use Terzaghi's Approach:

2. A circular well has an external diameter of 7.5 m and is sunk into a sandy soilto a depth of 20 m below the maximum scour level. The resultant horizontal force is 1800 kN.The well is subjected to a moment of 36,000 kN.m about the maximum scour level due to thelateral force. Determine whether the well is safe against lateral forces, assuming the well torotate (*a*) about a point above the base, and (*b*) about the base, Assume $\gamma' = 10$ kN/m3, and $\varphi = 36°$.
Use Terzaghi's analysis, and a factor of safety of 2 against passive resistance.

Solution of Q.No. 1 is given below. And solve no. 2yourself.

$D = 16$ m $\qquad H = 8$ m $\qquad \phi = 30°$

$$\therefore \qquad K_a = \frac{(1-\sin 30°)}{(1+\sin 30°)} = \frac{1}{3}$$

$$K_p = \frac{(1+\sin 30°)}{(1-\sin 30°)} = 3$$

Total height above base + $H_1 = 16 + 8 = 24$

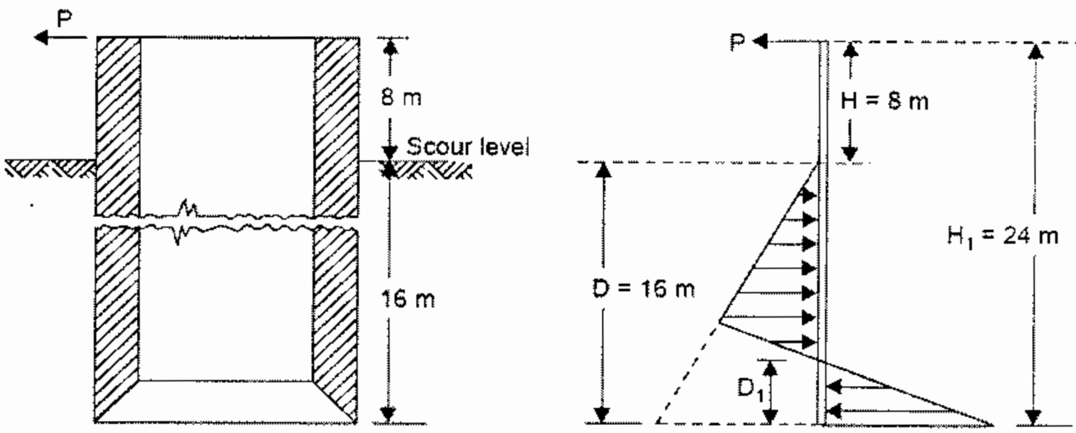
Modified passive pressure coefficient,

$$K_p' = \frac{K_p}{F.S} = \frac{3}{2} = 1.50$$

(*a*) Rotation about a point above the base:

From Eq. 19.30:

$$2D_1 = 3H_1 \pm \sqrt{9H_1^2 - 2D(3H_1 - D)}$$
$$= 3 \times 24 \pm \sqrt{9 \times 24^2 - 2 \times 16(3 \times 24 - 16)}$$
$$= 72 \pm \sqrt{5184 - 1792}$$



(a) Well with lateral load

(b) Pressure distribution

Fig. Cylindrical well with lateral load (Ex.)

$2\,D_1 = 72 - 58.24$ (rejecting the +ve sign, as it leads to a value for $D_1 > D$)

$= 13.76$

$D_1 = 6.88$ m

From Eq. 19.28,

$$q'_{max} = \frac{1}{2}\gamma' D(K_p' - K_a).(D - 2D_1)$$
$$= (1/2) \times 10 \times 16\{(3/2) - (1/3)\}(16 - 13.76)$$
$$= 209.1 \text{ kN/m}$$

Allowable Transverse load,

$$Q_a = q'_{max} \times L = 209 \times 6 = 1254 \text{ kN}$$

(The shape factor may be taken as unity since $D_e > 6$ m)

(*b*) Rotation about the base:

From Eq. 19.32,

$$q'_{max} = \frac{1}{6}\gamma'(K_p' - K_a)\frac{D^3}{H_1}$$

$$= (1/6) \times 10\{(3/2) - 1/3)\}.\frac{16^3}{24}$$

$$= 332 \text{ kN/m}$$

Hence the allowable lateral load

$$Q_a = q'_{max} \times L = 332 \times 6 = 1992 \text{ kN}$$

Since the actual horizontal load is given as 1000 kN, the well is safe against lateral load.