BUILDING MATERIAL & CONSTRUCTION TECHNOLOGY

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DIPLOMA

LECTURE NOTES ON

BUILDING MATERIAL & CONSTRUCTION TECHNOLOGY

BY

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STONE

Topics to be covered

Classification of rock, uses of stone, natural bed of stone
Qualities of good building stone,
Dressing of stone
Characteristics of different types of stone and their uses

Classification of rock, uses of stone, natural bed of stone

Building stones are obtained from rocks occurring in nature and classified in three ways.

A. Geological classification
B. Physical classification
C. Chemical classification

Geological Classification:

According to this classification, the rocks are of the following types.

a. **Igneous rocks:** Rocks that are formed by cooling of Magana (molten or pasty rocky material) are known as igneous rocks.
   Eg: Granite, Basalt and Dolerite etc.
b. **Sedimentary rocks:** these rocks are formed by the deposition of products of weathering on the pre-existing rocks.
   Examples: gravel, sandstone, limestone, gypsum, lignite etc.
c. **Metamorphic rocks.** These rocks are formed by the change in character of the pre-existing rocks. Igneous as well as sedimentary rocks are changed in character when they are subject to great heat and pressure. Known as metamorphism.
   Examples: Quartzite, Schist, Slate, Marble and Gneisses.

Physical Classification:

This classification based on general structure of rocks. According to this, the rocks are classified into three types

a. **Stratified Rocks:** These rocks possess planes of stratification or cleavage and such rocks can be easily split along these planes
   Ex: sedimentary rocks
b. **Unstratified rocks:** The structure may be crystalline granular or compact granular.
   Examples: Igneous rocks and Sedimentary rocks affected by movements of the earth.
c. **Foliated Rocks:** These rocks have a tendency to split up in a definite direction only.
   Ex: Metamorphic rocks.

Chemical Classification:

According to this classification rocks are classified into three types.

a. **Siliceous rocks:** In these rocks, silica is predominates. The rocks are hard; durable and not easily effected by weathering agencies.
   Ex: Granite, Quartzite, etc.
b. **Argillaceous Rocks:** In these rocks, clay predominates. The rocks may be dense and compact or may be soft.
   Ex: slates, Laterites etc.

c. **Calcareous rocks:** In these rocks, calcium carbonate predominates. The durability to these rocks will depend upon the constituents present in surrounding atmosphere.
Ex: Lime Stone, marble etc.

Uses of stones:

The stones are used in the construction of buildings from the ancient times and most of the ancient temples and forts of our country were built with stone. Even at present they form a basic material for cement concrete and bricks.

Following are the various uses to which stones are employed

1. **Structure**: Stones are used for foundations, walls, columns, lintels, arches, roofs, floors, damp proof course etc.
2. **Face works.** Stones are adopted to give massive appearance to the structure. Wall are of bricks and facing is done in stones of desired shades. This is known as composite masonry.
3. **Paving stones:** These are used to cover floor of building of various types such as residential, commercial, industrial etc. They are also adopted to form paving of roads, foot paths etc.
4. **Basic material**: Stones are disintegrated and converted to form a basic material for cement concrete, murum of roads, calcareous cements, artificial stones, hallow blocks etc.
5. **Miscellaneous**: Stones are also used for
    i. Ballast for railways
    ii. Flux in blast furnace
    iii. Blocks in the construction of bridges, piers, abutments, retaining walls, light houses, dams etc.

Natural bed of stone

a. **definition**

The building stones are obtained from rocks. These rocks have a distinct plane of division along which stones can easily be split. This plane is known as natural bed of stone and it thus indicates the plane or bed on which the sedimentary stone was originally deposited.

b. **Importance**

The direction of natural bed of all sedimentary stones should be perpendicular or nearly so to the direction of pressure. Such an arrangement gives maximum strength to the stone work.

The natural bed of stones can be detected by pouring water and examining the direction of layers. The stones break easily along these natural bed.

i. Arches : Stones are placed with their natural beds radial
ii. Cornices, String Courses: Stones should be placed with direction of natural beds as vertical. For corner stones, stones without natural beds should be adopted
iii. Wall : Stones should be placed with the direction of their natural beds horizontal

Qualities of a good building stone:

The following are the qualities or requirements of a good building stone.

1. **Crushing strength:** For a good building stone, the crushing strength should be greater than l000kg per cm^2.
2. **Appearance**: Good building stone should be a uniform colour, and free from clay holes, spots of other colour bands etc capable of preserving the colour for long time.
3. **Durability**: A good building stone should be durable. The factors like heat and cold alternative wet and dry, dissolved gases in rain, high wind velocity etc affect the durability.
4. **Fracture**: For good building stone its fracture should be sharp, even and clear.
5. **Hardness**: The hardness greater than 17, treated as hard used in road works. It is between 14 to 17, medium hardness, less 14 said be poor hardness.
6. **Percentage wear**: For a good building stone, the percentage wear should be equal to or less than 3 percent.
7. **Resistance to fire**: A good building stone be fire proof. Sandstone, Argillaceous stone resists fire quite well
8. **Specific gravity**: For a good building stone the specific gravity should be greater than 8.7 or so.
9. **Texture**: A good building stone should have compact fine crystalline structure should be free from cavities, cracks or patches of stuff or loose material.
10. **Water absorption:** For a good building stone, the percentage absorption by weight after 24 hours should not exceed 0.60.
11. **Seasoning**: Stones should be well seasoned before putting into use. A period of about 6 to 12 months is considered to be sufficient for proper seasoning.
12. **Toughness Index**: Impact test, the value of toughness less than 13 – Not tough, between 13 and 19 – Moderate, greater than 19- high

Dressing of stone

The stones, after being quarried, are to be cut into suitable sizes and with suitable surfaces. This process is known as the dressing of stones

It is carried out for the following purposes

- To get the desired appearance from stone work
- To make the transport from quarry easy and economical
- To suit to the requirement of stone masonry
- To take advantage of local men near quarry who are trained for such type of work

Following are the varieties of finishes obtained by the dressing of stones

a. **Axed finish** : when the surfaces of hard stones such as granite are dressed by means of an axe is called axed finish

b. **Boasted or droved finish** : in this type of finish, the boaster is used to make non-continuous parallel marks on the stone surface

c. **Chisel-draughted margins** : in order to obtained uniform joint in stone work, the margins are placed which may be either squared or pitched or chamfered.

d. **Circular finish** : in this type of finish, the surface of the stone is made round or circular as in case of a column

e. **Moulded finish** : the surface of the stone can be moulded in any desired shape so as to improve the appearance of the work

f. **Hammer-dressed finish** : in this type of finish the stones are made roughly square or rectangular by means of a Waller's hammer

g. **Plain finish** : in this type of finish, the surface of the stone is made approximately smooth with a saw or with a chisel

h. **Polish finish** : the surface of the stones such as marbles, granites etc. can be policed either with hand or with machine

i. **Punched machine** : on the stone surface, the depressions are made by using a punch. The surface of the stones takes the form of a series of hollows and ridges

j. **Rubbed finish** : this type of finish is obtained by rubbing a piece of stone with the surface or by rubbing the surface with the help of a suitable machine

k. **Scabbling finish** : in this type of finish, the irregular projections are removed with a scabbling hammer.

l. **Sunk finish** : this finish is obtained by sinking the surface below the original level in the form of wide grooves, chamfers, inclined surfaces

Characteristics of different type of stone and their uses

In order to ensure suitable selection of stone of particular work, one must be conversant with its composition, characteristics, uses and place of availability.

1. **Granite**
   a. Igneous rock
   b. Composed of quart, felspar and mica and minerals
   c. Available in grey, green, brown and pink and red
   d. Hard and durable
   e. High resistance to weathering
   f. The texture varies with its quality
   g. Specify gravity 2.7 and compressive strength 700 to 1300 kg/cm^2
   h. Used for ornamental, road metal, railway ballast, aggregate for concrete; for construction of bridges, piers and marine works etc.
2. **Basalt**
   a. Igneous rock
   b. It is compact, hard and heavy
   c. Available in red, yellow grey, blue and greenish black colour
   d. Specific gravity is 3 and compressive strength varies 1530 to 1890 kg/cm^2.
   e. Used for ornamental, rail road ballast, aggregates for concrete etc.
3. **Sand Stone:**
   a. Sedimentary rock
   b. It is available in variety of formations fine grained, coarse grained compact or porous
   c. Available in white, green, blue, black, red and yellow.
   d. Specific gravity 2.65 to 2.95

e. Compressive strength is 650kgs / cm^2
f. Used for ashlar works

4. **Lime Stone:**
   a. Sedimentary rock: It is available in a variety of forms which differ from one another in colour Compaction, texture, hardness and durable
   b. Compact lime stone
   c. Granular lime stone
   d. Magnesia lime stone
   e. Kanker lime stone
   f. Used for paving, road metal, etc
5. **Marble**
   a. Metamorphic rock
   b. Available in white, blue, green, yellow black and red colour
   c. High compactness
   d. Suitable for decorative works, wall lining columns, pile, table slabs, hearths, tiled floors, steps of stair case etc.
6. **Slate:**
   a. Metamorphic rock
   b. Non-absorbent, compact fine grained and produce metallic ringing sound when struck
   c. Available in black, dark blue, grey, reddish brown etc.
   d. Used for providing damp proof course, paving dados etc.
7. **Chalk**
   a. Sedimentary rock
   b. Pure white lime stone, soft and easy to form powder
   c. Suitable for preparing glazier's putty as colouring material in manufacture of Portland cement
8. **Kankar**
   a. Sedimentary rock
   b. Impure lime stone
   c. Used as road metal, manufacture of hydraulic lime
9. **Murum**
   a. Metamorphic rock
   b. Decomposed laterite, deep brown or red in color
   c. Used as blindage for metal road, for fancy paths and garden walls
10. **Quartzite**
    a. Metamorphic rock
    b. Hard, brittle, crystalline and compact
    c. Difficult to work and dress
    d. Suitable for construction of retaining walls, road metals, concrete aggregate, pitching, rubble masonry, facing of buildings

Bricks

Brick earth – its composition

Brick making – Preparation of brick earth, Moulding, Drying, Burning in kilns (continuous Process)

Classification of bricks, size of traditional and modular bricks, qualities of good building bricks

Brick earth – its composition

Composition – Following are the constituents of good brick earth.

a. **Alumina**: - It is the chief constituent of every kind of clay. A good brick earth should contain 20 to 30 percent of alumina. This constituent imparts plasticity to earth so that it can be moulded. If alumina is present in excess, raw bricks shrink and warp during drying and burning.
b. **Silica**-A good brick earth should contain about 50 to 60 percent of silica. Silica exists in clay either as free or combined form. As free sand, it is mechanically mixed with clay and in combined form; it exists in chemical composition with alumina. Presence of silica prevents cracking, shrinking and warping of raw bricks. It thus imparts uniform shape to the bricks. Durability of bricks depends on the proper proportion of silica in brick earth. Excess of silica destroys the cohesion between particles and bricks become brittle.
c. **Lime** – A small quantity of lime is desirable in finely powdered state to prevent shrinkage of raw bricks. Excess of lime causes the brick to melt and hence, its shape is lost due to the splitting of bricks.
d. **Oxide of iron**- A small quantity of oxide of Iron to the extent of 5 to 6 percent is desirable in good brick to impart red colour to bricks. Excess of oxide of iron makes the bricks dark blue or blackish.
e. **Magnesia**- A small quantity of magnesia in brick earth imparts yellow tint to bricks, and decreases shrinkage. But excess of magnesia leads to the decay of bricks.

The ingredients like, lime, iron pyrites, alkalies, pebbles, organic matter should not present in good brick earth

Brick making – Preparation of brick earth, Moulding, Drying, Burning in kilns (continuous Process)

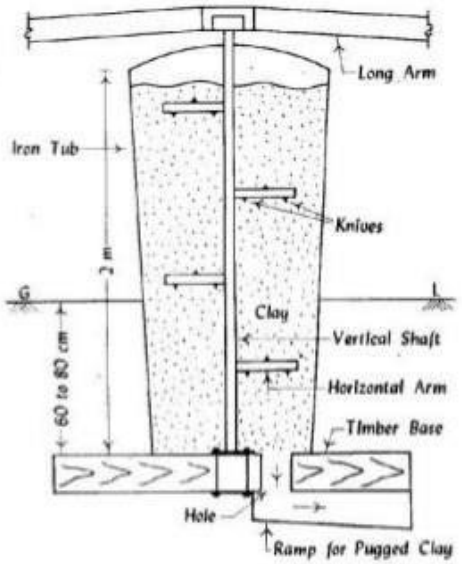
In the process of manufacturing bricks, the following operations are involved

1. Preparation of clay
2. Moulding
3. Drying
4. Burning

1. **Preparation of clay :-** The preparation of clay involves following operations
   a. **Unsoiling** :- Top layer of 20cm depth is removed as it contain impurities.

b. **Digging**: - Clay dug out from ground is spread on level ground about 60cm to 120cm heaps.
c. **Cleaning**:-Stones, pebbles, vegetable matter etc removed and converted into powder form.
d. **Weathering**:- Clay is exposed to atmosphere from few weeks to full season.
e. **Blending**:- Clay is made loose and any ingredient to be added to it is spread out at top and turning it up and down in vertical direction**.**
f. **Tempering**:- Clay is brought to a proper degree of hardness, then water is added to clay and whole mass is kneaded or pressed under the feet of men or cattle for large scale, tempering is usually done in pug mill as shown in the fig

Tempering



Pug mill

Process:- Clay with water is placed in pug mill from the top. When the vertical staff is rotated by using electric pair, steam or diesel or turned by pair of bullocks. Clay is thoroughly mixed up by the actions of horizontal arms and knives when clay has been sufficiently pugged, hole at the bottom of tub, is opened cut and the pugged earth is taken out from ramp for the next operation of moulding.

2. **Moulding**: Clay, which is prepared form pug mill, is sent for the next operation of moulding. Following are the two ways of moulding.

a. **Hand Moulding:** Moulds are rectangular boxes of wood or steel, which are open at top and bottom. Steel moulds are more durable and used for manufacturing bricks on large scale as shown in fig. Bricks prepared by hand moulding are of two types.

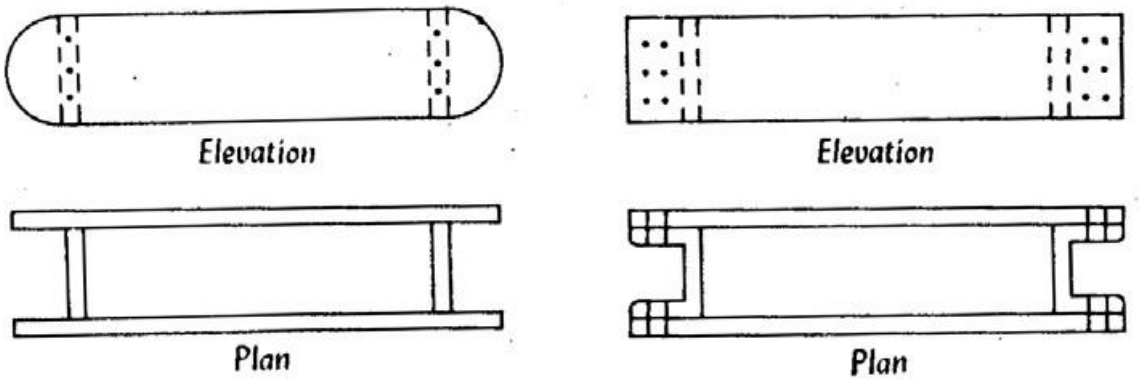


Fig : wooden mould and steel mould

i. Ground moulded bricks
ii. Table moulded bricks

i. Ground moulded bricks: ground is first made level and fine sand is sprinkled over it. Mould is dipped in water and placed over the ground to fill the clay. Extra clay is removed by wooden or metal strike after the mould is filled by pressing or forcing, mould is then lifted up and raw brick is left on the ground. Mould is then dipped in water every time lower faces of ground moulded bricks are rough and it is not possible to place frog on such bricks.
Ground moulded bricks of better quality and with frogs on their surface are made by using a pair of pallet boards and a wooden block.

ii. Table-moulded bricks: Process of moulding these bricks is just similar to ground bricks on a table of size about 2m x 1m.

b. Machine moulding: This method proves to be economical when bricks in huge quantity are to be manufactured at the same spot. It is also helpful for moulding hard and string clay. These machines are broadly classified in two categories

i. Plastic clay machines
ii. Dry clay machines

i. Plastic clay machines: This machine containing rectangular opening of size equal to length and width of a brick. Pugged clay is placed in the machine and as it comes out through the opening, it is cut into strips by wires fixed in frames, so these bricks are called wire cut bricks.

ii. Dry clay machines: In these machines, strong clay is first converted into powder form and then water is added to form a stiff plastic paste. Such paste is placed in mould and pressed by machine to form hard and well shaped bricks. These bricks are heavier than ordinary hand moulded bricks. They carry distinct frogs and exhibit uniform texture.

3. Drying: The damp bricks, if burnt, are likely to be cracked and distored. Hence moulded bricks are dried before thay are taken for the next operation of burning. Bricks are laid along and across the stock in alternate layers. The drying of brick is by the following means.

i. Artificial drying – drying by tunnels usually 120^{o}C about 1 to 3 days
ii. Circulation of air- Stacks are arranged in such a way that sufficient air space is left between them for free circulation of air.
iii. Drying yard- special yards should be prepared slightly higher level prevent the accumulation of rain water.
iv. Period for drying – usually about 3 to 10 days to bricks to become dry.

v. **Screens** – screens are necessary, may be provided to avoid direct exposure to wind or sun.

4. Burning: This is very important operation in the manufacturing of bricks to impart hardness, strength and makes them dense and durable. Burning of bricks is done either in clamps or in kilns.

Clamps are temporary structures and they are adopted to manufacture bricks on small scale.

Kilns are permanent structures and they are adopted to manufacture bricks on a large scale.

Kilns: A kiln is a large oven, which is used to burnt bricks and of two types

i. Intermittent kilns
ii. Continuous kilns

i. Intermittent kilns: These are intermittent in operation, which means that they are loaded, fired, cooled and unloaded. They are classified in two types

a. Intermittent up-draught kilns
b. Intermittent down-draught kilns

ii. Continuous kilns:

These kilns are continuous in operations. This means that loading, firing, cooling and unloading are carried out simultaneously in these kilns. There are three types of continuous kilns.

a. Bull's trench kiln
b. Hoffman's kiln
c. Tunnel kiln

a. Bull's trench kiln: This kiln may be of rectangular, circular or oval shape in the plan as shown in fig. It is constructed in a trench excavated in ground either fully underground partially projecting above ground openings is provided in the outer walls to act as flue holes.

Dampers are in the form of iron plates and they are used to divide the kilns in suitable sections and most widely used kiln in India.

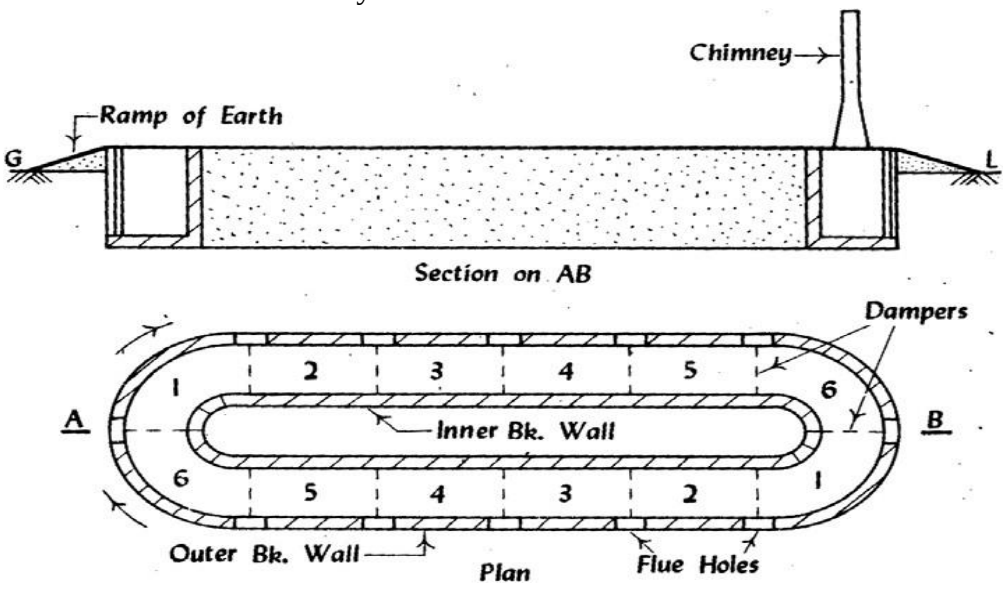


Fig ; bulls trench kiln

The bricks are arranged in such a way that flues are formed. Fuel is placed in flues and it is ignited through flue holes after covering top surface with earth and ashes to prevent the escape of heat usually two movable iron chimneys are employed to form draught. These chimneys are placed in advance of section being fired. Hence, hot gases leaving the chimney warm up the bricks in next section. Each section requires about one day to burn. The tentative arrangement for different sections may be as follows

Section 1 – loading
Section 2 – empty
Section 3 – unloading
Section 4 – cooling
Section 5 – Burning
Section 6 – Heating

b. Haffman's kiln: this kiln is constructed over ground and hence, it is sometimes known as flame kiln. Its shape is circular to plan and it is divided into a number of compartments or chambers. A permanent roof is provided; the kiln can even function during rainy season. Fig shows plan and section of Hoffman's kiln with 12 chambers

Chamber 1 – loading
Chamber 2 to 5 – drying and pre-heating
Chambers 6 and 7 – burning
Chambers 8 to 11 – cooling
Chamber 12 – unloading

The initial cost in stalling this kiln is high, the following advantages

i. Good quality of bricks are produced.
ii. It is possible to regulate heat inside the chambers through fuel holes.
iii. Supply of bricks is continuous and regular.
iv. There is considerable saving in fuel due to pre heating of raw bricks by flue gases

Such a kiln will manufacture about 25000 bricks daily or about 8 to 9 million bricks annually.

c. Tunnel kiln: This type of kiln is in the form of tunnel, which may be straight, circular or oval in the plan. Raw bricks are placed in trolleys which are then moved from one end to the other end of tunnel. Raw bricks get dried and pre-heated as they approach zone of fire. In zone of fire, bricks are burnt to the required degree and they are then pushed forward for cooling. When bricks are sufficiently cooled, they are unloaded. This kiln proves to be economical when the bricks are manufactures on a large scale. As temperature is under control, uniform bricks of better quality are produced.

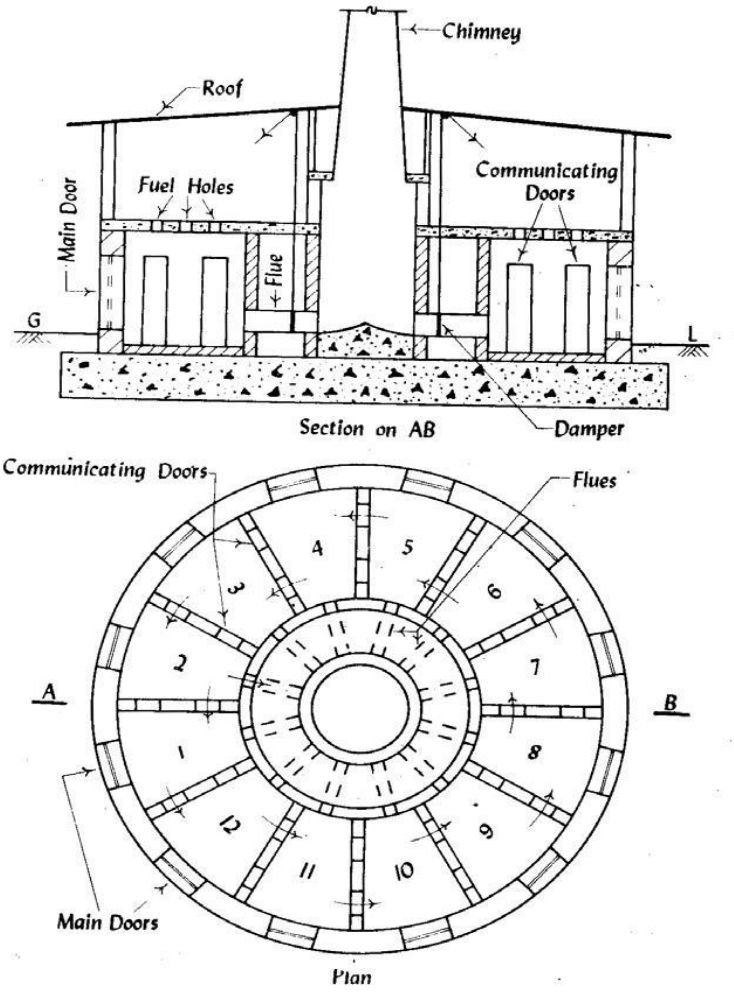


Fig : Hoffman's kiln

Classification of bricks, size of traditional and modular bricks, qualities of good building bricks

Classification:

Bricks can broadly be divided into two categories.

i. Unburnt or sundried bricks
ii. Burnt bricks

i. Unburnt or Sun dried bricks- Un-burn or sun dried with the help of heat received from sun after the process of moulding. These bricks can only be used in the constructions of temporary and cheap structures. Such bricks should not be used at places exposed to heavy rains.

ii. Burnt Bricks: The bricks used in construction works are burnt bricks and they are classified into the following four categories.

a. **First Class bricks:** These bricks are table moulded and of standard shape. The surface and edges of the bricks are sharp, square, smooth and straight. They comply all the qualities of good bricks and used for superior work of permanent nature.

b. **Second class bricks:** These bricks are ground moulded and they are burnt in kilns. The surface of bricks is somewhat rough and shape is also slightly irregular. These bricks are commonly used at places where brick work is to be provided with a coat of plaster.
c. **Third class bricks:** These bricks are ground moulded and they burnt in clamps. These bricks are not hard and they have rough surfaces with irregular and distorted edges. These bricks give dull sound when struck together. They are used for unimportant and temporary structures and at places where rainfall is not heavy.
d. **Fourth class bricks:** These are over burnt bricks with irregular shape and dark colour. These bricks are used as aggregate for concrete in foundation, floors, roads, etc because of the fact that the over burnt bricks have compacted structure and hence, they are sometimes found stronger than even first class bricks.

Size of traditional and modular bricks

The bricks are prepared in various sizes. The custom in the locality is the governing factor for deciding the size of a brick. Such bricks which are not standardised are known as the traditional bricks.

Brick of standard size 190 mm x 90 mm x 90 mm is recommended by the BIS. With mortar thickness, the size of such a brick becomes 200 mm x 100 mm x 100 mm and it is known as the nominal size of the modular brick.

Qualities of good building bricks

- Bricks should be table moulded, well burnt in kilns, copper coloured, free from cracks and with sharp and square edges.
- Bricks should be uniform shape and should be of standard size.
- Bricks should give clear ringing sound when struck each other.
- Bricks when broken should show a bright homogeneous and compact structure free from voids.
- Bricks should not absorb water more than 20 percent by weight for first class bricks and 22 percent by weight for second class bricks, when soaked in cold water for a period of 24 hours.
- Bricks should be sufficiently hard no impression, should be left on brick surface, when it is scratched with finger nail.
- Bricks should be low thermal conductivity and they should be sound proof.
- Bricks should not break when dropped flat on hard ground from a height of about one meter.
- Bricks, when soaked in water for 24hours, should not show deposits of white salts when allowed to dry in shade.
- No brick should have crushing strength below 55kg/cm^2

Cement, Mortar and Concrete

3.1.Cement: Types of cements, Properties of cements, Manufacturing of cement.

3.2.Importance and application of blended cement with fly ash and blast furnace slag.

3.3.Mortar: Definition and types of mortar.

Sources and classification of sand, Bulking of sand.

Use of gravel, morrum and fly ash as different building material.

Concrete: Definition and composition- Water cement ratio- Workability, mechanical properties and grading of aggregates, mixing, placing, compacting and curing of concrete.

Cement: Types of cements, Properties of cements, Manufacturing of cement

Cement in its broadest term means any substance which acts as a binding agent for materials. Natural cement (Roman Cement) is obtained by burning and crushing the stones containing clay, carbonates of lime and some amount of carbonate of magnesia. The clay content in such stones is about 20 to 40 percent. Natural cement resembles very closely eminent hydraulic lime. It is not strong as artificial cement, so it has limited use in practice.

Artificial cement is obtained by burning at very high temperature a mixture of calcareous and argillaceous materials in correct proportion. Calcined product is known as clinker. A small quantity of gypsum is added to clinker and it is then pulverized into very fine powder is known as cement. The common variety of artificial cement is known as normal setting cement or ordinary cement or Portland cement.

Types of cements

In addition to ordinary cement, the following are the other varieties of cement.

a. **Acid Resistance Cement**: This is consists of acid resistance aggregates such as quartz, quartzite's, etc, additive such as sodium fluro silicate (Na_2SiO_6) and aqueous solution of sodium silicate. This is used for acid resistant and heat resistant coating of installations of chemical Industry. By adding 0.5 percent of linseed oil or 2 percent of ceresit, its resistance to water is increased and known as acid water resistant cement.
b. **Blast Furnace Cement:** For this cement slag as obtained from blast furnace in the manufacture of pig iron and it contains basic elements of cement, namely alumina, lime and silica. The properties of this cement are more or less the same as those of ordinary cement and prove to be economical as the slag, which is waste product, is used in its manufacture.
c. **Coloured Cement:** Cement of desired colour may be obtained by intimately mixing mineral pigments with ordinary cement. The amount of colouring may vary from 5 to 10 percent and strength of cement is affected if it is exceeds 10 percent. Chromium oxide gives brown, red or yellow for different proportions. Coloured cements are used for finishing of floors, external surfaces, artificial marble, windows etc.

d. Expanding Cement : This type of cement is produced by adding an expanding medium like sulpho – aluminate and a stabilizing agent to ordinary cement. Hence this cement expands where as other cement shrinks. Expanding cement is used for the construction of water retaining structures and also for repairing the damaged concrete surfaces.

e. High alumina Cement: This cement is produced by grinding clinkers formed by calcining bauxite and lime. The total content should not be less than 32 percent and the ratio by weight of alumina to lime should be between 0.85 and 1.30.

Advantages

- Initial setting time is about $3\frac{1}{2}$ hours therefore, allows more time for mixing and placing operations.
- It can stand high temperatures.
- It evolves great heat during setting therefore not affected by frost.
- It resists the action of acids in a better way.
- It lets quickly and attains higher ultimate strength.

Disadvantages:

- It is costly.
- It cannot be used in mass construction as it evolves great heat and as it sets soon.
- Extreme care is to taken to see that it does not come in contact with even traces of lime or ordinary cement.

f. Hydrophobic Cement: This type of cement contains admixtures, which decreases the wetting ability of cement grains. The usual hydrophobic admixtures are acidol, napthenesoap, oxidized petrolatum etc. When hydrophobic cement is used, the fine pores in concrete are uniformly distributed and thus the frost resistance and the water resistance of such concrete are considerably increased.

g. Low Heat Cement: Considerable heat is produced during the setting action of cement. In order to reduce the amount of heat, this type of cement is used. It contains lower percentage of tricalcium aluminates C_3A and higher percentage of dicalcium silicate C_2S. This type of cement is used for mass concrete works because it processes less compressive strength.

h. Pozzolana Cement: Pozzolana is a volcanic powder and the percentage should be between 10 to 30.

Advantages

- It attains compressive strength with age.
- It can resist action of sulphates.
- It evolves less heat during setting.
- It imparts higher degree of water tightness.
- It imparts plasticity and workability to mortar and concrete prepared from it.
- It offers great resistance to expansion.
- It possesses higher tensile strength.

Disadvantages:

- Compressive strength in early days is less.
- It possesses less resistance to erosion and weathering action.

i. **Quick Setting Cement:** This cement is prepared by adding a small percentage aluminum sulphate which reduce the percentage of gypsum or retarder for setting action and accelerating the setting action of cement. As this cement hardens in less than 30 minutes, mixing and placing operations should be completed. This cement is used to lay concrete under static water or running water.

j. **Rapid Hardening cement:** This cement has same initial and final setting times as that of ordinary cement. But it attains high strength in early days due to
   - Burning at high temperature.
   - Increased lime content in cement composition.
   - Very fine grinding.

   Advantages:
   - Construction work may be carried out speedily.
   - Formwork of concrete can be removed earlier.
   - It is light in weight.
   - It is not damaged easily.
   - This cement requires short period of curing.
   - Use of this cement allows higher permissible stresses in the design.
   - Structural member constructed with this cement may be loaded earlier.

k. **Sulphate Resisting Cement:** In this cement percentage of tricalcium aluminates is kept below 5 to 6 percent and it results in the increase in resisting power against sulphate. This cement is used for structure which are likely to be damaged by sever alkaline condition such as canal linings, culverts, siphons etc.

l. **White Cement:** This is a variety of ordinary cement and it is prepared form such raw materials which are practically free from colouring oxides of Iron, manganese or chromium. For burning of this cement, oil fuel is used instead of coal. It is used for floor finish; plaster work, ornamental works etc.

Properties of cements

Following are the physical, mechanical and chemical properties of cement.

A. Physical properties of cement

Following are the important physical properties of a good cement which primarily depends upon its chemical composition, thoroughness of burning and fineness of grinding

1. It gives strength to the masonry.
2. It is an excellent binding material.
3. It is easily workable.
4. It offers good resistance to the moisture.
5. It possesses a good plasticity.
6. It stiffens or hardens early.
7. A thin paste of cement with water should feel sticky between the fingers.
8. A cement thrown in water should sink and should not float on the surface.
9. The particles should have uniformity of fineness. Specific surface area should not be less than 2250 cm^2/gm.
10. The initial setting time for ordinary Portland cement is about 30 minutes.

11. The final setting time for ordinary Portland cement is about 10 hours.
12. The expansion of cement after heating and cooling the mould, should not exceed 10mm.

B. Mechanical properties of cement

1. The compressive strength at the end of 3 days should not be less than 11.5 N/mm^2 and that at the end of 7 days should not be less than 17.5 N/mm^2.
2. The tensile strength at the end of 3 days should not be less than 2 N/mm^2 and that at the end of 7 days should not be less than 2.50 N/mm^2.

C. Chemical properties of cement

1. The ratio of percentage of alumina to iron oxide should not be less than 0.66.
2. The ratio of percentage of lime to alumina, iron oxide and silica, known as lime saturation factor (LSF) should not be less than 0.66 and should not be more than 1.02.
3. Total loss on ignition should not be more than 4 percent.
4. Total sulphur content should not be more than 2.75 percent.
5. Weight of insoluble residue should not be more than 1.5 percent.
6. Weight of magnesia should not exceed 5 percent.

Manufacturing of cement

Following three distinct operations are involved in the manufacture of normal setting or ordinary or Portland cement

1. Mixing of raw material
2. Burning
3. Grinding

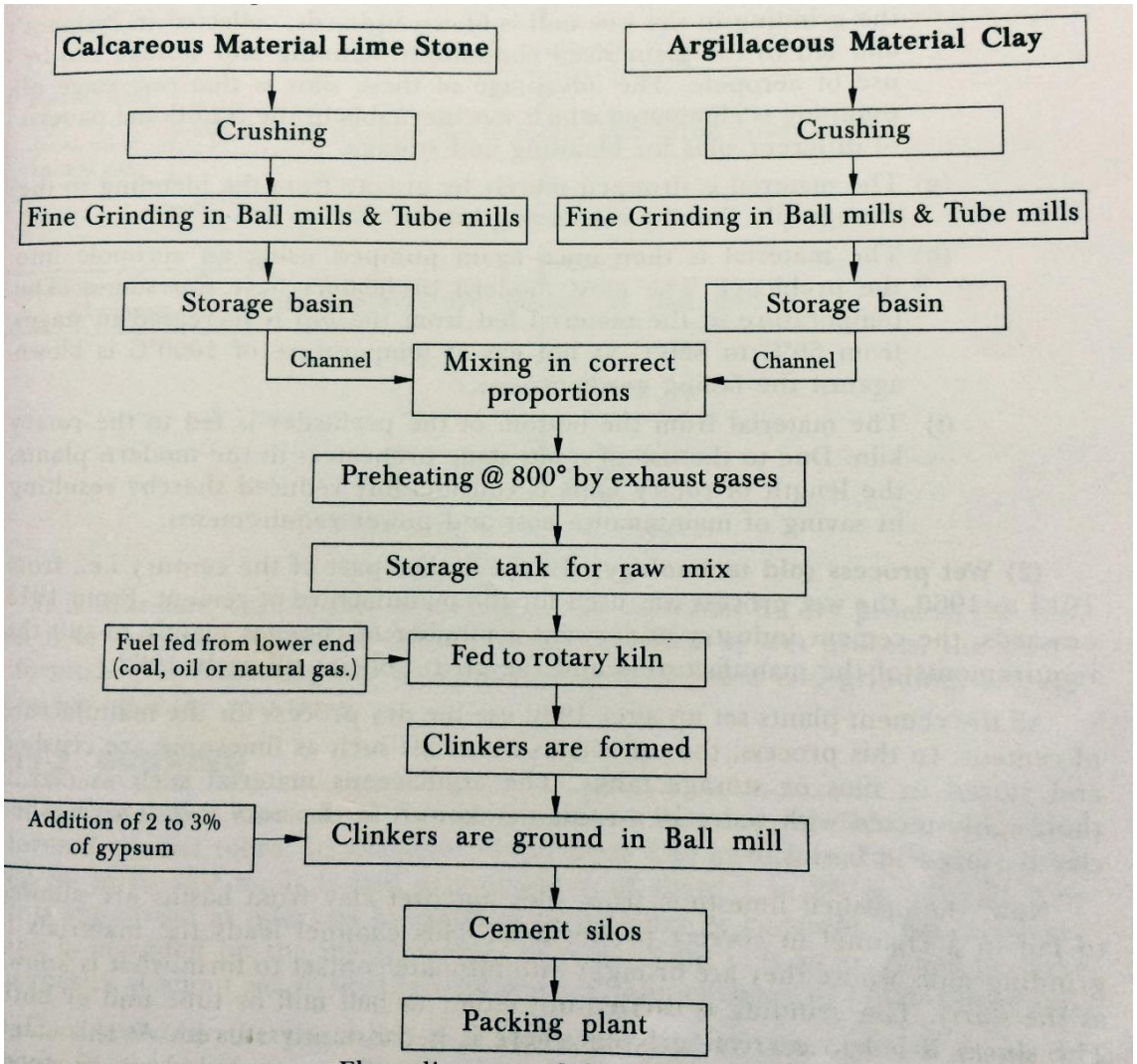
1. Mixing of raw material

The raw materials such as lime stone or chalk and shale or clay may be mixed either in dry condition or wet condition, accordingly known as the dry process or the wet process of mixing.

a. Dry process

The both calcareous and argillaceous raw materials are firstly crushed in the gyratory crushers to get 2.5cm size pieces separately. The crushed materials are again grinded to get fine particles in ball mills or tube mills.

Each finely grinded material is stored in hopper after screening. Now these powdered minerals are mixed in required proportion to get dry raw mix which is then stored in silos and kept ready to be sent into rotary kiln. Fig shows the flow diagram of raw materials by dry process.



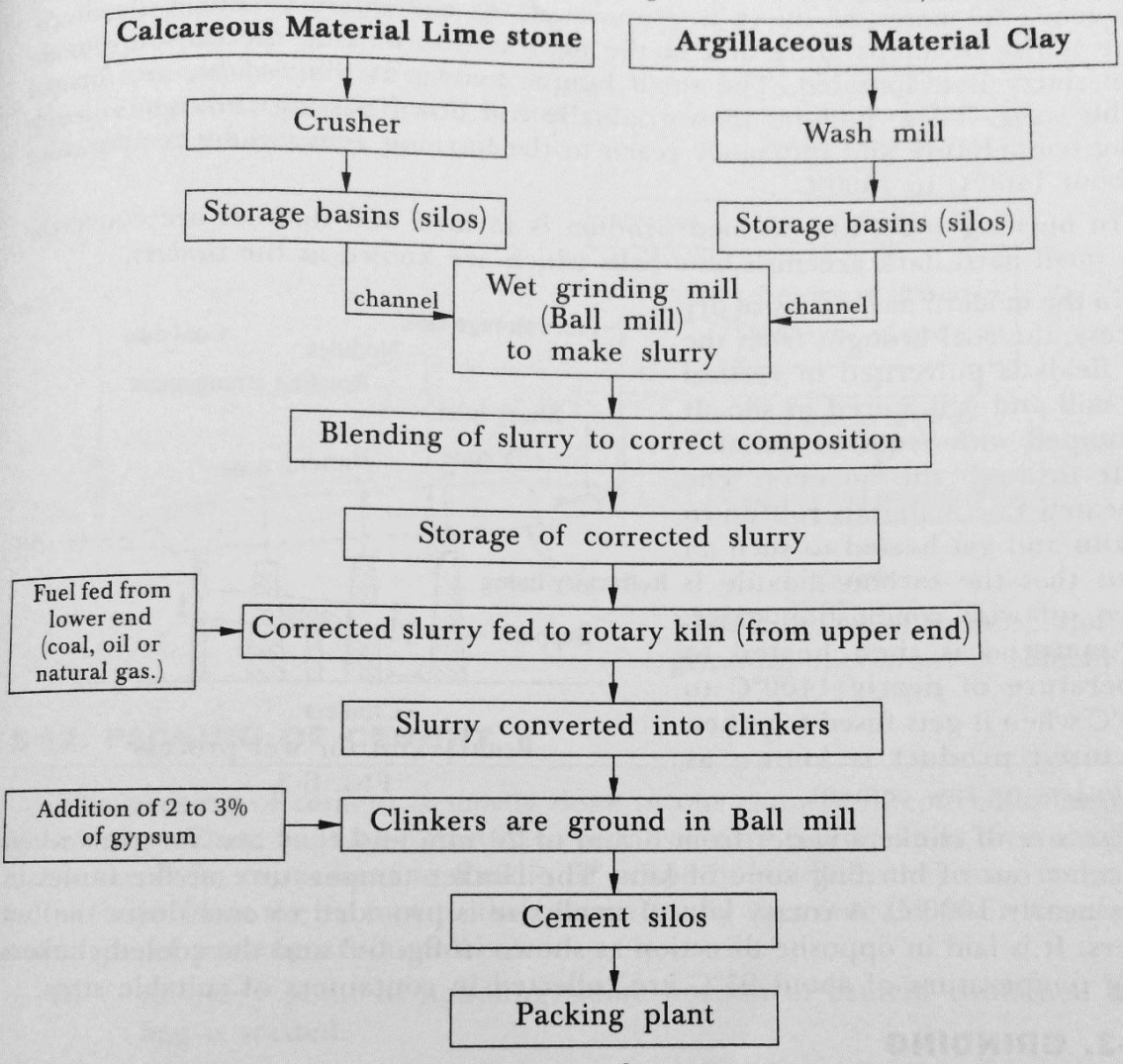
Flow diagram of dry process
FIG. 6-1

b. Wet process

The raw materials are firstly crushed and made into powdered form and stored in silos. The clay is then washed in washing mills to remove adhering organic matters found in clay. The powdered limestone and water washed clay are sent to flow in the channels and transferred to grinding mills where they are completely mixed and the paste is formed, i.e., known as slurry.

The corrected slurry is stored in storage tanks and kept ready to serve as feed for rotary kiln.

Fig. 6-2 shows the flow diagram of mixing of raw materials by the wet process.



Flow diagram of wet process

FIG. 6-2

2. Burning

Corrected slurry is feed to rotary kiln, which is about 2.5 to 3 m in diameter and 90 to 120 m in length and temperature arrangement is up to 1400-1500 °C. At this temperature slurry losses moisture and forms into small lumps, after that changes to clinkers. Clinkers are cooled in another inclined tube similar to kiln but of lesser length.

3. Grinding

Grinding is the final process where clinker is grinded, it is first cooled down to atmospheric temperature. Grinding of clinker is done in large tube mills. After proper grinding gypsum (Calcium sulphate $CaSO_4$) in the ratio of 01-04 % is added for controlling the setting time of cement.

Finally, fine ground cement is stored in storage tanks from where it is drawn for packing.

Importance and application of blended cement with fly ash and blast furnace slag.

Blended cement are produced by intergrinding portland cement clinker with other specified materials or by blending portland cement with the other materials, or a combination of intergrinding and blending. Binary blended cements contain two constituent materials, while ternary blended cements contain three constituent materials Fly ash is a fine powder that is a byproduct of burning pulverized coal in electric generation power plants. Fly ash is a pozzolana, a substance containing aluminous and siliceous material that forms cement in the presence of water. When mixed with lime and water, fly ash forms a compound similar to Portland cement. This makes fly ash suitable as a prime material in blended cement. When used in concrete mixes, fly ash improves the strength and segregation of the concrete and makes it easier to pump.

Blast furnace slag is a nonmetallic coproduct produced in the production of iron, iron ore, iron scrap, and fluxes. It consists primarily of silicates, aluminosilicates, and calcium-alumina-silicates.

- It reduces water demand and therefore water-cement ratio can be reduced.
- It improves workability for the same water content.
- The blended cements are finer as compared to OPC, therefore the permeability of concrete is less. This results into improved durability

Energy saving: Blended cements are obtained by adding mineral admixtures with OPC. The energy, which would have otherwise been utilized for production of OPC, is thus saved. This saving is to the tune of 0.8 to 1.2 MWH/ton of cement.

Conservation of natural resources: The used mineral admixtures are the waste products of thermal and steel plants. By using these products, we are conserving the precious minerals like lime stone, clay and silica etc.

Pollution control: By reducing the production of cement, pollution is also controlled as cement is an energy intensive product. It has been estimated that 7% of total pollution is only due to cement production which can proportionately be reduced if more blended cement is used.

Mortar: Definition and types of mortar

The term mortar is used to indicate a paste prepared by adding required quantity of water to a mixture of binding material like cement or Lime and fine aggregates like sand. The two components of mortar namely the binding material and fine aggregates are sometimes referred to as matrix. The durability, quality and strength of mortar will mainly depends on quantity and quality of the matrix. The combined effect of the two components of mortar is that the mass is able to bind the bricks or stones firmly.

Types of Mortar

The mortar are classified on the bases of the following

1. Bulk density
2. Kinds of binding material
3. Nature of application
4. Special mortars

1. **Bulk density:**
   According to bulk density of mortar in dry state, the mortars are two types
   a. **Heavy mortars** bulk density is more than 1500kg/m^3 and prepared from heavy quartz.
   b. **Lightweight mortars** – bulk density is less than 1500kg/m^3 and prepared from light porous sands.
2. **Kinds of binding Material**
   According to the kinds of binding material, several factors such as expected working conditions, hardening temperature, moisture conditions, etc should be considered. The mortars are classified into four categories.
   a. **Lime Mortar -** in this mortar, lime is used as binding material. Lime may be fate lime or Hydraulic lime. Fat lime mortar 1:2 to 1:3 and hydraulic lime mortar may be 1:2 by volume.
   b. **Cement mortar:** In this mortar, cement is used as binding material. Depending upon the strength required and importance of work, the proportion of cement to sand varies from 1:2 to 1:6 or more.
   c. **Gauged Mortar or composite mortar:** The process of adding cement to lime mortar to improve the quality of lime mortar is known as gauging. It makes lime mortar economical, strong and dense. The usual proportion of cement to lime by volume is about 1:6 to 1:8.
   d. **Gypsum mortar:** These mortars are prepared from gypsum binding material such as building gypsum and anhydrite binding materials.
3. **Nature of Application:**
   According to the nature of application, the mortars are classified into two categories.
   a. **Brick laying mortars:** Mortars for brick laying are intended to be used for brick works and walls. Depending up on the working conditions and type of construction, the composition of masonry mortars with respect to the kind of binding materials is decided.
   b. **Finishing Mortars:** these mortars include common plastering work and mortars for developing architectural or ornamental effects. Generally cement or lime is used as binding material.
4. **Special Mortars:**
   a. **Fire resistant mortar-** This mortar is prepared by adding 1:2 ratio of aluminous cement with crushed powder of fire bricks used for fire brick lining furnaces, fire places, ovens etc.
   b. **Light weight mortar –** This mortar is prepared by adding sawdust, wood powder to lime or cement mortar for sound proof and heat proof construction.
   c. **Packing Mortar –** To pack of oil wells, special mortars possessing the properties of high homogeneity, water resistance, predetermined setting time, ability to form solid water proof plugs in cracks and voids of rocks, resistance to subsoil water pressure etc. have to be formed with cement sand, cement loam and cement sand loam mortars.
   d. **Sound absorbing mortars:** To reduce the noise level, sound absorbing mortars with Portland cement, lime, gypsum, slag Portland cement etc as the binding materials employed in its composition. The aggregates re selected from lightweight porous material such as pumice, cinders etc.

e. **X-ray shielding mortar:** This type of mortar is used for providing the plastering coat to walls and celling of x-ray cabinets. This is heavy mortar with bulk density over 2200kg/m^3 is used. The aggregates are obtained from heavy rock and suitable admixture are added to enhance protective property of such a mortar.

Sources and classification of sand, Bulking of sand

Sand is an important building material used in the preparation of mortar, concrete, etc.

Sources of Sand:

Sand particles consist of small grains of silica ($Si0_2$). It is formed by the decomposition of sand stones due to various effects of weather. The following are the natural sources of sand.

a. **Pit Sand:** This sand is found as deposits in soil and it is obtained by forming pits to a depth of about 1m to 2m from ground level. Pit sand consists of sharp angular grains, which are free from salts for making mortar, clean pit sand free from organic and clay should only be used.
b. **River Sand:** This sand is obtained from beds of rivers. River sand consists of fine rounded grains. Colour of river sand is almost white. As the river sand is usually available in clean condition, it is widely used for all purposes.
c. **Sea Sand:** This sand is obtained from sea shores. Sea sand consists of rounded grains in light brown colour. Sea sand consists of salts which attract the moisture from the atmosphere and causes dampness, efflorescence and disintegration of work. Due to all such reasons, sea sand is not recommendable for engineering works. However be used as a local material after being thoroughly washed to remove the salts.

Classification of sand

According to the size of grains, the sand is classified as fine, coarse and gravelly.

- The sand passing through a screen with clear openings of 1.5875 mm is known as the fine sand. It is mainly used for plastering.
- The sand passing through a screen with clear openings of 3.175 mm is known as the coarse sand. It is mainly used for masonry works.
- The sand passing through a screen with clear openings of 7.62 mm is known as the gravelly sand. It is mainly used for concrete work.

Bulking of Sand:

The presence of moisture in sand increases the volume of sand. This is due to fact that moisture causes film of water around the sand particles which result in the increase of volume of sand. For a moisture content of 5 to 8 percent, the increase in volume may be about 20 to 40 percent, depending upon the grading of sand. The finer the material, the more will be the increase in volume for a given moisture content. This phenomenon is known as bulking of sand.

Use of gravel, morrum and fly ash as different building material

Gravel as building material

Gravel is widely used for industrial and personal projects. It is an integral component of concrete and outdoor design alike. It is used to build homes and highways as well as make concrete, mortar and plaster. Gravel can be both decorative and functional. It can be used as mulch in a home garden or as a method of erosion control. Many industries use this material in their day-to-day functions. Industries that are involved with commercial or private construction often use gravel as an ingredient for building materials.

Morrum

Morrum is used for construction material for building, raw material for construction of dam, raw material for road construction, paving etc. Morrum can be frequently used as sub-base materials. They are good material for building huts and paths, as they can be compacted easily to form hard surfaces.

Fly Ash

It is a by-product in coal based thermal plants. Its particles can fly in ordinary air. At one time it was considered a nuisance but now it is used as a useful material in manufacturing bricks, for stabilizing soil and to improve workability.

Fly ash can be used as prime material in many cement-based products, such as poured concrete, concrete block, and brick. One of the most common uses of fly ash is in Portland cement concrete pavement or PCC pavement. Road construction projects using PCC can use a great deal of concrete, and substituting fly ash provides significant economic benefits. Fly ash has also been used as embankment and mine fill. It can be utilised for manufacturing bricks. Fly ash can be utilised for construction of embankment and roads.

Concrete: Definition and composition- Water cement ratio- Workability, mechanical properties and grading of aggregates, mixing, placing, compacting and curing of concrete.

Concrete Definition

Cement concrete is a mixture of cement, sand, pebbles or crushed rock and water. When placed in the skeleton of forms and allowed to cure, becomes hard like a stone.

Concrete composition

The main constituents of concrete are

a. **Cement / Lime:** Before introduction of ordinary Portland cement, lime was used as cementing material. At present most of the cement concrete works in the building construction is done with ordinary Portland cement. But other special varieties of cement such as rapid hardening cement, high alumina cement are used under certain circumstances. The cement should comply with all standard specifications.
b. **Fine Aggregates:** The material, which is passed through 4.7625mm B.S.test sieve, is termed as fine aggregates. Usually natural river sand is used as fine aggregates. But places where natural sand is not available economically, finely crushed stone may be used as fine aggregates.
c. **Coarse Aggregates**: The material retained on 4.7625mm size B.S.test sieve is termed as coarse aggregates. Broken stone is generally used as coarse aggregates. For thin slabs, and walls, the maximum size of coarse aggregates should be limited to one third the thickness of the concrete section.
d. **Water**: Water to be used in the concrete work should have the following properties.
    - It should be free from oils.
    - It should be free from acids or alkalies.
    - It should be free from Iron, Vegetables matter or other substance, which is likely to have adverse effect on concrete.
    - It should be fit for drinking purpose.

Water cement ratio

The water in concrete has to perform the following two function

i. The water enters into chemical action with cement and this action causes the setting and hardening of concrete.
ii. The water lubricates the aggregates and it facilitates the passage of cement through voids of aggregate. Water makes the concrete workable.

It is found theoretically that water requires for these two functions is about 0.5 to 0.6 times the weight of cement. This ratio of the amount of water to the amount of cement by weight is termed as the water cement ratio and the strength and quality of concrete primarily depends upon this ratio.

Workability

Workability of Concrete is a broad and subjective term describing how easily freshly mixed concrete can be mixed, placed, consolidated, and finished with minimal loss of homogeneity.
The important facts in connection with workability are as follows

- If more water is added to attain the required degree of workmanship, it results into concrete of low strength and poor durability.
- The workability of concrete is also affected mainly by water content, water cement ratio and aggregate cement ratio.
- It is also affected by the grading, shape, texture and maximum size of the coarse aggregate to be used in the mixture.

Mechanical properties

Performance of concrete is evaluated from mechanical properties which include shrinkage and creep, compressive strength, tensile strength, flexural strength, and modulus of elasticity. But compressive strength of concrete is the most important characteristic and it is generally assumed that an improvement in concrete compressive strength will improve its mechanical properties.

Compressive strength

Characteristic strength of concrete is the strength of concrete specimens casted and tested as per given code of practice and cured for a period of 28 days; 95% of tested cubes should not have a value less than this value.
The compressive strength of concrete is calculated by applying load in cube of size 150 mm and dividing the maximum applied load with the cube area.

Grading of aggregates

In order to obtain concrete of denser quality, the fine and coarse aggregates are properly graded. The grading of fine aggregates is expressed in terms of BIS test sieve nos. 480, 240, 120, 60, 30 and 15.
The grading of fine aggregates has a marked effect on the uniformity, workability and finishing qualities of concrete. Table shows the grading limits for fine aggregates.

	Percentage by weight passing through sieve	
BIS sieve	Natural or crushed gravel sand	Crushed stone sand
No. 480	95-100	90-100
No. 240	70-95	60-90
No. 120	45-85	40-80
No. 60	25-60	20-50
No. 30	5-30	5-30
No. 15	0-10	0-15

The grading of coarse aggregates may be varied through wider limits than those of sand without appreciable effect on the workability of concrete.

Mixing

The process of rolling, folding and spreading of particles is known as the mixing of concrete.
There are two types of concrete mixing

1. Hand mixing
2. Machine mixing

1. **Hand Mixing:** For hand mixing, the materials are stacked on a water tight platform, which may be either of wood, brick or steel.
   The materials should be thoroughly mixed, at least three times, in dry condition before water is added. The prepared mix should be consumed in 30 minutes after adding water. The mixing by hand is allowed in case of small works or unimportant works where small quantity of concrete is required. It is advised to add 10% extra cement to guard against the possibility of inadequate mixing by this method.
2. **Machine mixing:** For machine mixing, all the materials of concrete including water, are collected in a revolving drum and then the drum is rotated for a certain period. The resulting mix is then taken out of the drum.

Placing

Concrete is placed on form works. The form works should be cleaned and properly oiled. If concrete is to be placed for foundation, the soil bed should be compacted well and is made free from loose soil.
Concrete should be dropped on its final position as closely as possible. If it is dropped from a height, the coarse aggregates fall early and then mortar matrix. This segregation results into weaker concrete.

Compacting

In the process of placing concrete, air is entrapped. The entrapped air reduces the strength of concrete up to 30%. Hence it is necessary to remove this entrapped air. This is achieved by compacting the concrete after placing it in its final position. Compaction can be carried out either by hand or with the help of vibrators.

a. ***Hand Compaction:*** In this method concrete is compacted by ramming, tamping, spading or by slicing with tools. In intricate portions a pointed steel rod of 16 mm diameter and about a metre long is used for poking the concrete.
b. ***Compaction by Vibrators:*** Concrete can be compacted by using high frequency vibrators. Vibration reduces the friction between the particles and set the motion of particles. As a result entrapped air is removed and the concrete is compacted. The use of vibrators reduces the compaction time. When vibrators are used for compaction, water cement ratio can be less, which also help in improving the strength of concrete. Vibration should be stopped as soon as cement paste is seen on the surface of concrete. Over vibration is not good for the concrete.

The following types of vibrators are commonly used in concreting:

i. Needle or immersion vibrators
ii. Surface vibrators
iii. Form or shutter vibrators
iv. Vibrating tables.

Needle vibrators are used in concreting beams and columns. Surface vibrators and form vibrators are useful in concreting slabs. Vibrating tables are useful in preparing precast concrete elements.

Curing of concrete

Curing may be defined as the process of maintaining satisfactory moisture and temperature conditions for freshly placed concrete for some specified time for proper hardening of concrete. Curing in the early ages of concrete is more important. Curing for 14 days is very important. Better to continue it for 7 to 14 days more. If curing is not done properly, the strength of concrete reduces. Cracks develop due to shrinkage. The durability of concrete structure reduces.

The following curing methods are employed:

a. Spraying of water
b. Covering the surface with wet gunny bags, straw etc.
c. Ponding
d. Steam curing and
e. Application of curing compounds.

a. Spraying of water: Walls, columns, plastered surfaces are cured by sprinkling water.

b. Wet covering the surface: Columns and other vertical surfaces may be cured by covering the surfaces with wet gunny bags or straw.

c. Ponding: The horizontal surfaces like slab and floors are cured by stagnating the water to a height of 25 to 50 mm by providing temporary small hunds with mortar.

d. Steam curing: In the manufacture of pre-fabricated concrete units steam is passed over the units kept in closed chambers. It accelerates curing process, resulting into the reduction of curing period.

e. Application of curing compounds: Compounds like calcium chloride may be applied on the curing surface. The compound shows affinity to the moisture and retains it on the surface. It keeps the concrete surface wet for a long time.

Other Construction Materials

4.1 Timber: Classification and Structure of timber.
Seasoning of timber – Importance.
Characteristics of good timber.
Clay products and refractory materials – Definition and Classification.
Properties and uses of refractory materials- tiles, terracotta, porcelain glazing.
Iron and Steel: Uses of cast iron, wrought iron, mild steel and tor steel

Timber: Classification and Structure of timber.

Timber denotes wood, which is suitable for building or carpentry or various other engineering purposes like for construction of doors, windows, roofs, partitions, beams, posts, cupboards, shelves etc.

Classification of trees

Depending upon their mode of growth trees may be divided in the following two categories

(i) **Endogeneous trees –** These trees grow inwards and fibrous mass is seen in their longitudinal sections. Timber from these trees has very limited engineering applications Ex: bamboo, cane, palm etc.

(ii) **Exogeneous trees:** These increases in bulk by growing outwards and used for engineering purposes. Exogeneous trees are further sub divided into two groups
a) conifers
b) deciduous

a. Conifers or evergreen trees: These trees having pointed, needle like or scale like leaves and yield soft wood.

b. Deciduous trees: The trees having flat broad leaves and leaves of those trees fall in autumn and new ones appear in spring season. Timber for engineering purpose is mostly derived from deciduous trees. These trees yield hard wood.
Ex: ash, beach, oak, sal, teak, shishum and walnut

Structure of timber

From the visibility aspect, the structure of a tree can be divided into two categories
1. Macro structure
2. Micro structure

I. **Macro structure:** The structure of wood visible to the naked eye or at a small magnification is called macro structure. Fig shows the macro structure of exogenous tree.

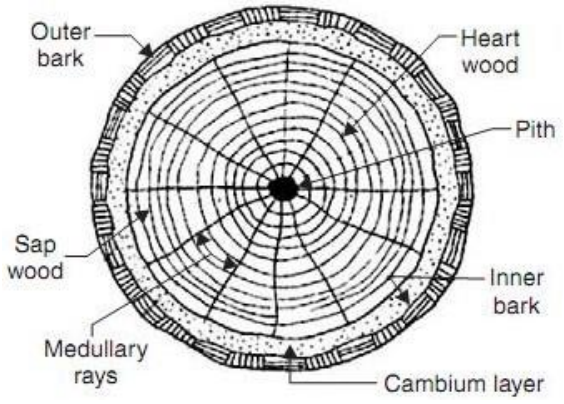


Fig : Cross-section of an exogenous tree

Pith: The innermost central portion or core of the tree is called pith or medulla
Heart wood: The inner annual rings surrounding the pith is known as heart wood. It imparts rigidity to tree
Sap wood: The cuter annual rings between heart wood and cambium layer is known as sap wood
Cambium layer: Thin layer of sap between sap wood and inner bark is known as cambium layer
Inner bark: The inner skin or layer covering the cambium layer is known as inner bark
Outer Bark: The outer skin or cover of the tree is known as outer bark
Medullary rays: The thin radial fibres extending from pith to cambium layer are known as medullary rays

II Micro structure: The structure of wood apparent only at great magnifications is called micro structure under micro scope, it becomes evident that the wood consists of living and dead cells of various sizes and shapes.

Seasoning of timber – Importance.

Seasoning of timber is the process of drying timber or removing moisture or sap, present in a freshly felled timber, under more or less control condition.
Freshly felled timber contains a large quantity of moisture roughly from 100 to 200 % , based on dry weight of wood. A well seasoned piece of wood may contain about 10 to 12 % moisture and will be in equilibrium with atmospheric humidity of a particular place.
The importance of seasoning of timber is

- To check/minimise the tendency of timber to shrink, warp and split.
- To increase strength, durability and electrical resisting power of the timber.
- To make timber safe from attack of fungi and insect.
- To reduce the weight of transport purposes, handling and there by reduction in cost.
- To make timber fit for receiving treatment of paints, preservatives, varnishes etc.
- To make timber suitable for gluing.
- To make timber easily workable and to facilitate operations during conversion.

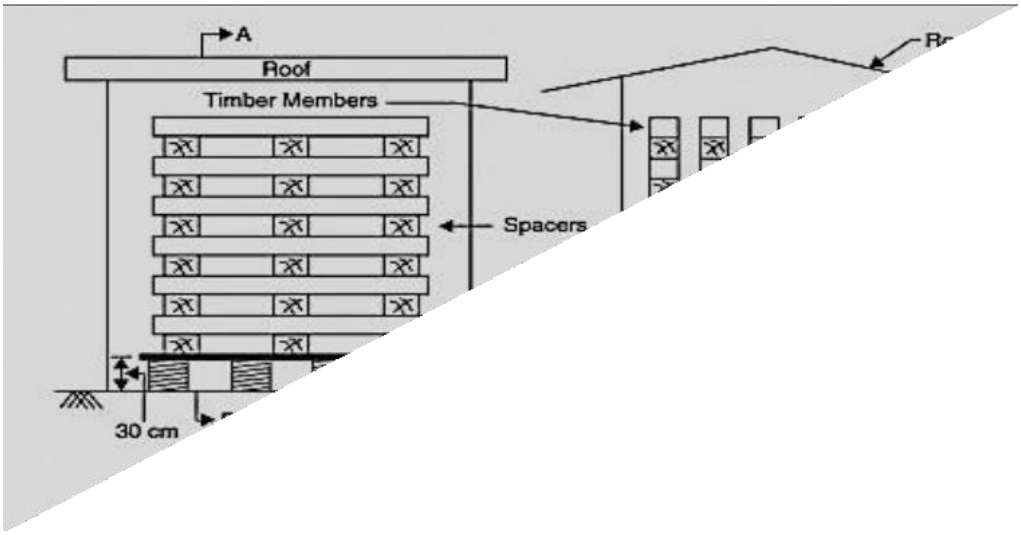
Methods of seasoning of timber

Seasoning of timber may be carried out in two ways

i. Natural seasoning
ii. Artificial seasoning

Natural seasoning

It may be air seasoning or water seasoning. Air seasoning is carried out in a shed with a platform. On about 300 mm high platform timber balks are stacked as shown in Fig.
Care is taken to see that there is proper air circulation around each timber balk. Over a period, in a natural process moisture content reduces. A well-seasoned timber contains only 15% moisture. This is a slow but a good process of seasoning. Water seasoning is carried out on the banks of rivers. The thicker end of the timber is kept pointing upstream side. After a period of 2 to 4 weeks the timber is taken out. During this period sap contained in the timber is washed out to a great extent. Then timber is stalked in a shed with free air circulation.



Artificial Seasoning: In this method timber is seasoned in a chamber with regulated heat, controlled humidity and proper air circulation. Seasoning can be completed in 4 to 5 days only. The different methods of seasoning are:

a. Boiling
b. Kiln seasoning
c. Chemical seasoning
d. Electrical seasoning.

Boiling: In this method timber is immersed in water and then water is boiled for 3 to 4 hours. Then it is dried slowly. Instead of boiling water hot steam may be circulated on timber. The process of seasoning is fast, but costly.

Kiln Seasoning: Kiln is an airtight chamber. Timber to be seasoned is placed inside it. Then fully saturated air with a temperature 35°C to 38°C is forced in the kiln. The heat gradually reaches inside timber. Then relative humidity is gradually reduced and temperature is increased, and maintained till desired degree of moisture content is achieved. The kiln used may be stationary or progressive. In progressive kiln the carriages carrying timber travel from one end of kiln to other end gradually. The hot air is supplied from the discharging end so that temperature increase is gradual from charging end to discharging end. This method is used for seasoning on a larger scale.

Chemical Seasoning: In this method, the timber is immersed in a solution of suitable salt. Then the timber is dried in a kiln. The preliminary treatment by chemical seasoning ensures uniform seasoning of outer and inner parts of timber.

Electrical seasoning: In this method, the use is made of high frequency alternating currents. The timber, when it is green, offers less resistance to the flow of electric current. The resistance increases as the wood dries internally which also results in the production of heat. This is the most rapid method of seasoning and uneconomical.

Characteristics of good timber.

The principal characteristics of timber of concern are strength, durability and finished appearance.

1. Narrow annual rings, closer the rings greater is the strength.
2. Compact medullary rays.
3. Dark colour.
4. Uniform texture.
5. Sweet smell and a shining fresh cut surface.

6. When struck sonorous sound is produced.
7. Free from the defects in timber.
8. Heavy weight.
9. No woolliness at fresh cut surface.
10. It should have regular annular ring etc.

4.3 Clay products and refractory materials – Definition and Classification.

Other Construction Materials

Timber: Classification and Structure of timber.
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2. Micro structure

I. **Macro structure:** The structure of wood visible to the naked eye or at a small magnification is called macro structure. Fig shows the macro structure of exogenous tree.

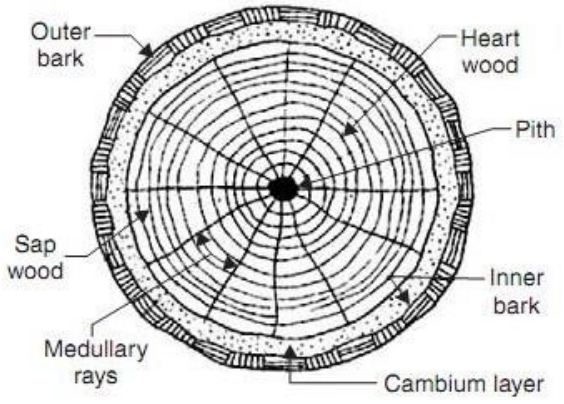


Fig : Cross-section of an exogenous tree

Pith: The innermost central portion or core of the tree is called pith or medulla
Heart wood: The inner annual rings surrounding the pith is known as heart wood. It imparts rigidity to tree
Sap wood: The cuter annual rings between heart wood and cambium layer is known as sap wood
Cambium layer: Thin layer of sap between sap wood and inner bark is known as cambium layer
Inner bark: The inner skin or layer covering the cambium layer is known as inner bark
Outer Bark: The outer skin or cover of the tree is known as outer bark
Medullary rays: The thin radial fibres extending from pith to cambium layer are known as medullary rays

II Micro structure: The structure of wood apparent only at great magnifications is called micro structure under micro scope, it becomes evident that the wood consists of living and dead cells of various sizes and shapes.

Seasoning of timber – Importance.

Seasoning of timber is the process of drying timber or removing moisture or sap, present in a freshly felled timber, under more or less control condition.
Freshly felled timber contains a large quantity of moisture roughly from 100 to 200 % , based on dry weight of wood. A well seasoned piece of wood may contain about 10 to 12 % moisture and will be in equilibrium with atmospheric humidity of a particular place.
The importance of seasoning of timber is

- To check/minimise the tendency of timber to shrink, warp and split.
- To increase strength, durability and electrical resisting power of the timber.
- To make timber safe from attack of fungi and insect.
- To reduce the weight of transport purposes, handling and there by reduction in cost.
- To make timber fit for receiving treatment of paints, preservatives, varnishes etc.
- To make timber suitable for gluing.
- To make timber easily workable and to facilitate operations during conversion.

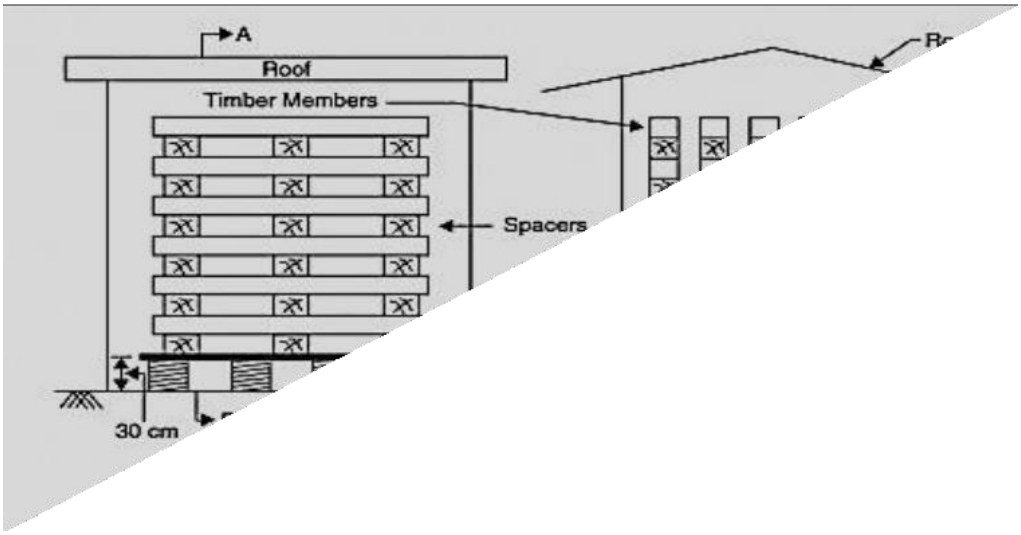
Methods of seasoning of timber

Seasoning of timber may be carried out in two ways

i. Natural seasoning
ii. Artificial seasoning

Natural seasoning

It may be air seasoning or water seasoning. Air seasoning is carried out in a shed with a platform. On about 300 mm high platform timber balks are stacked as shown in Fig.
Care is taken to see that there is proper air circulation around each timber balk. Over a period, in a natural process moisture content reduces. A well-seasoned timber contains only 15% moisture. This is a slow but a good process of seasoning. Water seasoning is carried out on the banks of rivers. The thicker end of the timber is kept pointing upstream side. After a period of 2 to 4 weeks the timber is taken out. During this period sap contained in the timber is washed out to a great extent. Then timber is stalked in a shed with free air circulation.



Artificial Seasoning: In this method timber is seasoned in a chamber with regulated heat, controlled humidity and proper air circulation. Seasoning can be completed in 4 to 5 days only. The different methods of seasoning are:

a. Boiling
b. Kiln seasoning
c. Chemical seasoning
d. Electrical seasoning.

Boiling: In this method timber is immersed in water and then water is boiled for 3 to 4 hours. Then it is dried slowly. Instead of boiling water hot steam may be circulated on timber. The process of seasoning is fast, but costly.

Kiln Seasoning: Kiln is an airtight chamber. Timber to be seasoned is placed inside it. Then fully saturated air with a temperature 35°C to 38°C is forced in the kiln. The heat gradually reaches inside timber. Then relative humidity is gradually reduced and temperature is increased, and maintained till desired degree of moisture content is achieved. The kiln used may be stationary or progressive. In progressive kiln the carriages carrying timber travel from one end of kiln to other end gradually. The hot air is supplied from the discharging end so that temperature increase is gradual from charging end to discharging end. This method is used for seasoning on a larger scale.

Chemical Seasoning: In this method, the timber is immersed in a solution of suitable salt. Then the timber is dried in a kiln. The preliminary treatment by chemical seasoning ensures uniform seasoning of outer and inner parts of timber.

Electrical seasoning: In this method, the use is made of high frequency alternating currents. The timber, when it is green, offers less resistance to the flow of electric current. The resistance increases as the wood dries internally which also results in the production of heat. This is the most rapid method of seasoning and uneconomical.

Characteristics of good timber.

The principal characteristics of timber of concern are strength, durability and finished appearance.

1. Narrow annual rings, closer the rings greater is the strength.
2. Compact medullary rays.
3. Dark colour.
4. Uniform texture.
5. Sweet smell and a shining fresh cut surface.

6. When struck sonorous sound is produced.
7. Free from the defects in timber.
8. Heavy weight.
9. No woolliness at fresh cut surface.
10. It should have regular annular ring etc.

Clay products and refractory materials – Definition and Classification.

Clay products are one of the most important classes of structural materials. The raw materials used in their manufacture are clay blended with quartz, sand, chamatte (refractory clay burned at 1000–1400°C and crushed), slag, sawdust and pulverized coal. Structural clay products or building ceramics are basically fabricated by moulding, drying and burning a clay mass. Higher the bulk specific gravity, the stronger is the clay product.

According to the method of manufacture and structure, bricks, tiles, pipes, terracotta, earthenwares, stonewares, porcelain, and majolica are well recognized and employed in building construction.

REFRACTORIES

These materials are capable of withstanding high temperature in different industrial processes. In addition to the thermal endurance, a refractory should be able to resist the chemical action of the material being heated and withstand the mechanical load. They have high dimensional and chemical stability and do not lose their physical shape and chemical composition. Refractories confine the heat and prevent the heat loss to the atmosphere from the outside walls of furnaces.

Classification

0n the Basis of Chemical Behaviour

- **Acid refractories**: Combine readily with bases. Their chief constituent is silica: quartz, sand, ganister and silica bricks.
- **Basic refractories: C**onsist mainly of basic oxides: magnesite and dolomite.
- **Neutral refractories: C**onsist of materials which do not combine with either basic or acidic oxides: silicon carbide, chromite and carbon.

On the basis of resistance to temperature

- **Low quality** used in the manufacture of fire-bricks, as lining material for furnaces. The melting point of such materials is more than 1580 ^{o}C.
- **High quality** stable even at high temperature and they are used in the construction of modern aeroplanes such as jets, rockets etc. These materials are composed of either pure clay or metals or combination of clay and metals.

Properties and uses of refractory materials- tiles, terracotta, porcelain glazing.

TILES

Tiles are thin slabs of low melting clays used for various purposes in engineering constructions. These give a very pleasing appearance and good service properties. Roofing tiles, flooring tiles, wall tiles and partition tiles are some of the examples.

Following are the characteristics of a good tile:

i. It should be free from any cracks, flaws or bends.
ii. It should be regular in shape and size.
iii. It should be sound, hard and durable.
iv. It should be well burnt.
v. It should give a clear ringing sound when struck with hand or with one another or with light hammer.

vi. It should fit in properly, when placed in position.
vii. It should give an even and compact structure when seen on its broken surface.
viii. It should possess uniform colour.

Tiles are widely used for flooring in kitchen, bathrooms, parking lots, rooftops and also used as tabletops for dining rooms, old factory floors, garages, gyms and sports complexes, schools and shops etc.

TERRACOTTA

The terra means earth and cotta means baked. Hence the terra-cotta means the baked earth. It is thus a type of earthenware or porous pottery made from local clays and glazed with glazes containing galena. It is soft enough to be scratched by a knife.

Following are the properties of terra-cotta:

i. It is strong and durable material.
ii. It is available in different colours.
iii. It is cheaper than ordinary finely dressed stones.
iv. It is easily cleaned.
v. It is easily moulded in desired shapes.
vi. It is fire-proof and can therefore be conveniently used with R.C.C. work.
vii. It is light in weight.
viii. It is not affected by atmospheric agencies and acids and is capable of withstanding weathering actions better than most kinds of stone.
ix. It is twisted due to unequal shrinkage in drying and burning.

The hollow terra-cotta blocks are used for various ornamental purposes such as facing work, arches, cornices, casing for columns, etc. It is adopted for all sorts of ornamental work. It is used as a decorative material in place of stones for ornamental parts of buildings such as cornices, string courses, sills, copings, bases of pillars, fire places, etc.

PORCELAIN

The term porcelain is used to indicate fine earthenware which is white, thin and semi-transparent. Since the colour of porcelain is white, it is also referred to as the whiteware.

The clay of sufficient purity and possessing high degree of tenacity and plasticity is used in preparing porcelains. It is hard, brittle and non-porous. It is prepared from clay, felspar, quartz and minerals. The constituents are finely ground and then they are thoroughly mixed in liquid state. The mixture is given the desired shape and it is burnt at high temperature.

The various types of porcelains are available and they are adopted for various uses such as sanitarywares, electric insulators, storage vessels, reactor chambers, crucibles, etc. The porcelains are of two types, namely low voltage porcelain and high voltage porcelain. The low voltage porcelain is prepared by dry process and it is mainly used for switch block, insulating tubes, lamp sockets, etc. If some quantity of alumina or silicate of magnesia is added, it can resist high temperature to a certain extent.

GLAZING

It is the process of forming some transparent film over the surfaces of bricks, tiles, earthenware or stoneware to improve upon their appearance and also to protect them from the action of weather, sewage, chemicals or other destroying agents.

A glaze is a glassy coat of thickness about 0.1 to 0.2 mm applied on the surface of an item and then fused into place by burning at high temperature.

Glazing should be such that it does not craze when the article is exposed to any change of temperature.

Glazing may be transparent like glass or opaque like enamels. For obtaining coloured glazes, the oxides and salts of various metals or special refractory colouring agents are added.

Transparent Glazing

Transparent glazing is of two types

1. Salt glazing
2. Lead glazing.

Salt glazing

- In this method, common salt (sodium chloride) is thrown into the kiln, containing the articles to be glazed at the close of the burning operation when the temperature reaches to about 1300 °C. The salt vapourizes and attaches itself chemically to all exposed surfaces of the clay products, thus forming a thin layer of transparent glass over the surfaces. The quantity of salt and throwing at proper time should be done with extreme care.
- This method is useful for sanitary pipes and chemical stonewares.

Lead glazing

- Lead glazing is preferred to salt glazing for getting articles of better quality.
- In this method, hot article to be glazed is dipped in a bath containing oxide of lead and tin when particles of these materials adhere to the article dipped. The article is then withdrawn and reburnt when high temperature of the finance melts it forming a film of glass over the exposed surface.
- In this method of glazing, the glaze does not penetrate into the body of the ware and as a matter of fact, it can be easily detached from the surface of the ware.

Uses: This method is used for terracolta, earthenwares and fire-clay wares.

Opaque Glazing or Ship Glazing

- The glazing materials consist of quartz, felspar, boric oxides, oxide of tin, zinc, lead, china clay etc. A mixture of these materials is prepared and ground to fine paste (known as slip) in the presence of water.
- The clay product to be glazed is first dried and dipped into the slip (paste) and then fired into the furnace at a temperature of about 120°C.During the process of firing the composition of glaze gets completely vitrified and forms uniform glaze of desired colour over the surface of the burnt clay product.
- In this glazing process, burning and glazing of the clay products are simultaneously achieved

Uses: Opaque glazing is used where it is desired to give the whole or any part of the article a better appearance than that presented by the burnt material. Basins, dishes, cups, etc., are opaque glazed.

Iron and Steel: Uses of cast iron, wrought iron, mild steel and tor steel

PIG-IRON

The crude impure iron which is extracted from iron ores is known as the pig-iron and it forms the basic material for the manufacture of cast-iron, wrought-iron and steel.

CAST-IRON

The cast-iron is manufactured by re-melting pig-iron with coke and limestone. This re-melting is done in a furnace known as the cupola furnace. It is more or less same as the blast furnace, but it is smaller in size.

Following are the important uses of cast-iron:

- For making cisterns, water pipes, gas pipes and sewers, manhole covers and sanitary fittings.
- For making ornamental castings such as brackets, gates, lamp posts, spiral staircases, etc.
- For making parts of machinery which are not subject to heavy shocks.
- For manufacturing compression members like columns in buildings, bases of columns, etc.
- For preparing agricultural implements.
- For preparing rail chairs, carriage wheels, etc.

WROUGHT-IRON

The wrought-iron is almost pure iron and it hardly contains carbon more than 0.15 per cent or so. But the process of its manufacture is laborious and tedious.

USE

The wrought-iron is replaced at present to a very great extent by mild steel. It is therefore produced to a very small extent at present. It is used where a tough material is required.

The wrought-iron, at present, is used for rivets, chains, ornamental iron work, railway couplings, water and steam pipes, raw material for manufacturing steel, bolts and nuts, horse shoe bars, handrails, straps for timber roof trusses, boiler tubes, roofing sheets, armatures, electro-magnets, etc.

PART: B (CONSTRUCTIONS TECHNOLOGY)

Introduction

1.1.Buildings and classification of buildings based on occupancy.
1.2.Different components of a building.
1.3.Site investigation — objectives, site reconnaissance and explorations.

Buildings and classification of buildings based on occupancy

Buildings

It simply indicates anything that is built with walls and a roof. The term building in civil engineering parlance is used to mean a structure having various components like foundations, walls, columns, floors, roofs, doors, windows, ventilators, stairs, lifts, various types of surface finishes, etc.

As per National Building Code of India, buildings are classified into nine groups based on occupancy as follows:

Group A: Residential buildings.
Group B: Educational buildings.
Group C: Institutional buildings.
Group D: Assembly buildings.
Group E: Business buildings.
Group F: Mercantile buildings.
Group G: Industrial buildings.
Group H: Storage buildings.
Group I: Hazardous buildings.

Different components of a building.

A building has two basic parts:

i. Sub-structure or foundations, and
ii. Super-structure.

Sub-structure or Foundation is the lower portion of the building, usually located below the ground level, which transmits the loads of the super-structure to the supporting soil. A foundation is therefore that part of the structure which is in direct contact with the ground to which the loads are transmitted.

Super-structure is that part of the structure which is above ground level, and which serves the purpose of its intended use. A part of the super-structure, located between the ground level and the floor level is known as plinth.

Plinth is therefore defined as the portion of the structure between the surface of the surrounding ground and surface of the floor, immediately above the ground. The level of the floor is usually known as the plinth level. The built-up covered area measured at the floor level is known as plinth area.

A building has the following components:

1. Foundations.
2. Masonry units: walls and columns.
3. Floor structures.
4. Roof structures.

5. Doors, windows and other openings.
6. Vertical transportation structures, such as stairs, lifts, ramps etc.
7. Building finishes

Site investigation — objectives, site reconnaissance and explorations.

Since the foundations have to transfer the load to the sub-soil, surface conditions at any given site must be adequately explored to obtain information required for the design and construction of foundations.

Site investigation is done for the following purposes:

a. For New Structures
    1. The selection of type and depth of foundation.
    2. The determination of bearing capacity of the selected foundation.
    3. The prediction of settlement of the selected foundation.
    4. The determination of the ground water level.
    5. The evaluation of the earth pressure against walls, basements, abutments etc.
    6. The provision against constructional difficulties.
    7. The suitability of soil and degree of compaction of soil.
b. For Existing Structures
    1. The investigation of the safety of the structure.
    2. The prediction of settlement.
    3. The determination of remedial measures if the structure is unsafe or will suffer detrimental settlement.

SITE RECONNAISSANCE

An inspection of the site and study of topographical features is often helpful in setting useful information about the soil and ground water conditions and in deciding the future programme of exploration. On going over the site, a study of the following features may be useful: local topography, excavations, cuttings, quarries, escarpments evidence of erosion or landslides, fills, water level in wells and drainage pattern for the building site. If there has been an earlier use of the site, information should be gathered, in particular about the underground workings, if any, and about the location of fills and excavations.

SITE EXPLORATION

The object of the site exploration is to provide reliable, specific and detailed information about the soil and ground water conditions of the site which may be required for a safe and economic design of foundations. For this purpose, an exploration of the region likely to be affected by the proposed works should yield precise information about the following:

i. The order of occurrence and extent of soil and rock strata.
ii. The nature and engineering properties of the soil and rock formation, and
iii. The location of ground water and its variation.

FOUNDATIONS

2.1 Concept of foundation and its purpose

2.2 Types of foundations — shallow and deep

Shallow foundation-constructional details of : Spread foundations for walls, thumb rules fordepth and width of foundation and thickness of concrete block

Deep foundations: Pile foundations-their suitability, classification of piles based on materials,function and method of installation.

Concept of foundation and its purpose

Every structure consists of the following two parts:

i. Foundations; and
ii. Superstructures.

The lowest artificially prepared parts of the structures which are in direct contact with the ground and which transmit the loads of the structures to the ground are known as the foundations or substructures. The solid ground on which the foundations rest is called the foundation bed or foundation soil and it ultimately bears the load and interacts with the foundations of buildings. The lowermost portion of the foundation which is in direct contact with the sub-soil is called the footing.

Purposes of foundation

The foundations are provided for the following purposes.

- To distribute the total load coming on the structure on a larger area so as to bring down the intensity of load at its base below the safe bearing capacity of sub-soil;
- To support the structures;
- To give enough lateral stability to the structures against various disturbing horizontal forces such as wind, rain, earthquake, etc.;
- To prepare a level and hard surface for concreting and masonry work;
- To transmit the super-imposed loads through side friction and end bearing in case of deep foundations;
- To distribute the non-uniform load of the superstructure evenly to the sub-soil;
- To provide the structural safety against undermining or scouring due to animals, flood water, etc.;
- To prevent or minimize cracks due to movement of moisture in case of weak or poor soils; etc.

Types of foundations — shallow and deep

Foundations may be broadly classified into 2 types;

a) Shallow Foundations
b) Deep Foundation

According to Terzaghi, A foundation is shallow if its depth is equal to or less than its width. Foundation is said to be deep if its depth is greater than its width.

a) SHALLOW FOUNDATIONS

From the point of view of design, shallow foundations may be of the following types:

1. Spread footings.
2. Combined footings
3. Strap footings.
4. Mat foundation.

A brief description of these is given below.

1. Spread Footings

Spread footings are those which spread the super-imposed load of wall or column over a larger area. Spread footings support either a column or wall. Spread footings may be of the following kinds:

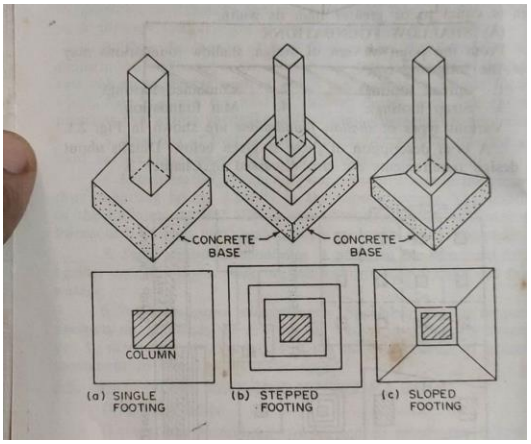
i. Single footing for a column
ii. Stepped footing for a column
iii. Sloped footing for a column
iv. Wall footing without step
v. Stepped footing for wall
vi. Grillage foundation

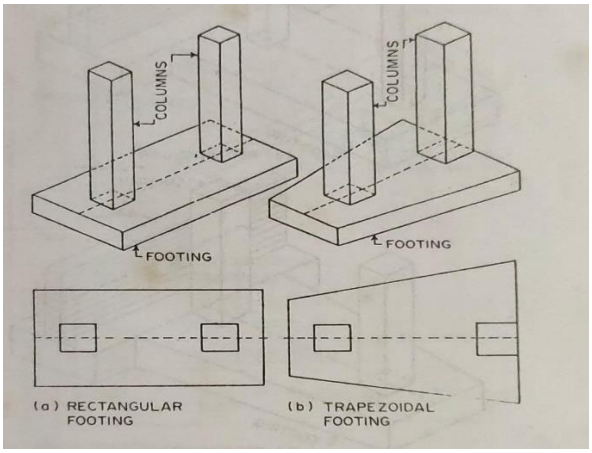
2. Combined Footings

A spread footing which supports two or more columns is termed as combined footing. The combined footings may be of the following kinds:

i. Rectangular combined footing
ii. Trapezoidal combined footing
iii. Combined column-wall footings

Combined footings are invariably constructed of reinforced concrete.



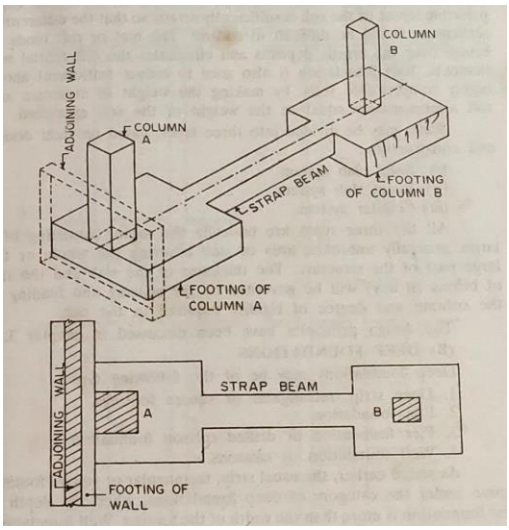


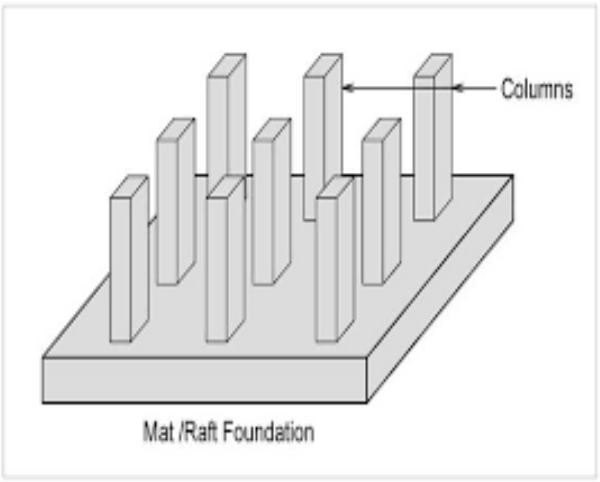
3. Strap Footings

If the independent footings of two columns are connected by a beam, it is called a strap footing. A strap footing may be used where the distance between the columns is so great that a combined trapezoidal footing becomes quite narrow, with high bending moments.

4. Mat Foundation (Raft Foundation)

A raft or mat is a combined footing that covers the entire area beneath a structure and supports all the walls and columns, when the allowable soil pressure is low, or the building loads are heavy, the use of spread footings would cover more than one half the area and it may prove more economical to use mat or raft foundation.





b) DEEP FOUNDATIONS

Deep foundations may be of the following types:

- Deep strip, rectangular or square footings.
- Pile foundation.
- Pier foundation or drilled caisson foundation.
- Well foundation or caissons.

As stated earlier, the usual strip, rectangular or square footings come under the category of deep foundations, when the depth of the foundation is more than the width of the footing. Well foundations are generally adopted for bridge piers etc. and not for building foundations.

Shallow foundation-constructional details of : Spread foundations for walls, thumb rulesfor depth and width of foundation and thickness of concrete block

Spread foundations for walls

Fig. 2.3 (a) shows the spread footing for a wall, consisting of concrete base without any steps. Usually, masonry walls have stepped footing as shown in Fig. 2.3 (b), with a concrete base.

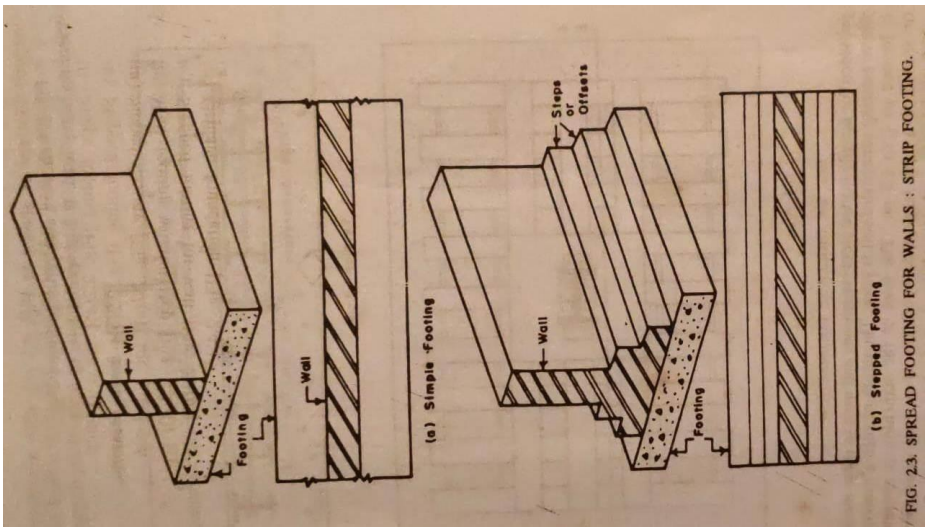
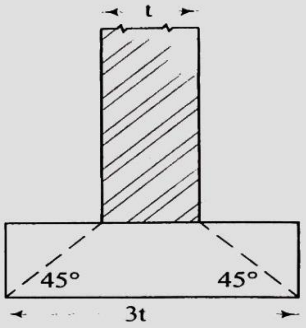


FIG. 2.3. SPREAD FOOTING FOR WALLS : STRIP FOOTING.

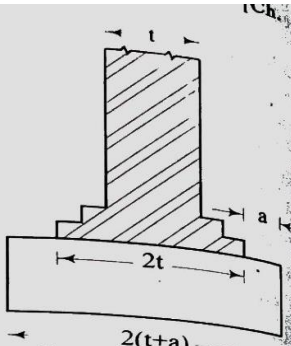
Thumb rules for depth and width of foundation and thickness of concrete block

(1) Width of foundations: The width of foundations is decided by adopting the following rules:

(i) If no footings are to be provided to the wall i.e. for simple footing, the width of foundations should be equal to three times the thickness of wall as shown in fig. 5-4.



Simple footing
FIG. 5-4



Stepped footing
FIG. 5-5

(ii) The total load including dead load, live load and wind load coming on the wall per metre length or in case of a pier, at the centre of the pier, is worked out. Then the width of foundation is obtained from the following relations:

$$\text{For walls,} \quad \text{width of foundations} = \frac{\text{total load per metre length}}{\text{allowable bearing capacity of the soil}}$$

$$\text{For piers,} \quad \text{width of foundations} = \left\{ \frac{\text{total load on the pier}}{\text{allowable bearing capacity of the soil}} \right\}^{\frac{1}{2}}$$

(iii) Usually the walls and piers are given footings such that the width at the base becomes equal to twice the width of wall at the plinth level as shown in fig. 5-5. By adding the width of offsets of concrete, the total width of foundations can be obtained for stepped footing.

If t = thickness of wall, and

a = offset of concrete.

width of foundation = $2\ (t + a)$.

Note: The greater result obtained from rules (ii) and (iii) should be adopted.

(2) Depth of foundations: The depth of foundations is obtained by keeping in view the following rules:

(i) As a general rule, all the shallow foundations should be taken to a minimum depth of 800 mm below natural ground level unless hard soil is available within 800 mm.

(ii) The total load to be transferred to the soil per square metre can be worked out and after the study of the results of the trial pits, the foundations should be taken to such a depth at which the soil has allowable bearing capacity greater than this value.

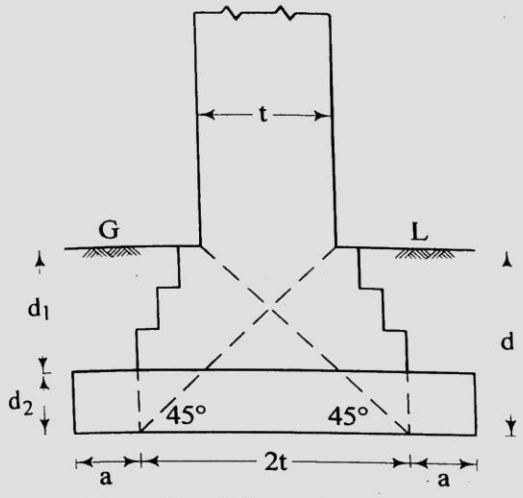
(iii) The depth of foundations can also be obtained by drawing the lines of angles 45° and 60° as shown in fig. 5-6 and fig. 5-7 respectively.

Let d_1 = Depth of footings

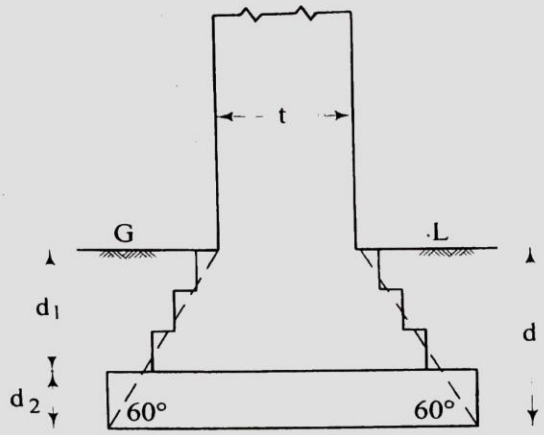
d_2 = Depth of concrete block.

d = Total depth of foundations

Then, $d = d_1 + d_2$.



Depth of foundations
FIG. 5-6



Depth of foundations
FIG. 5-7

(iv) For loose soil, Rankine's formula, as given below, can be used to find the minimum depth of foundations:

$$d = \frac{p}{w}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)^2$$

where d = minimum depth of foundation in metres

w = weight of soil in kg/m^3 or kN/m^3

ϕ = angle of repose

p = load on soil in kg/m^2 or kN/m^2.

The above formula is not to be used when the building to be constructed rests on hard soil. The values of angles of repose of some of the soils are given in table 5-1.

TABLE 5-1
ANGLES OF REPOSE OF VARIOUS SOILS

No.	Type of soil	Angle of repose in degrees
1.	Loose earth	30 to 45
2.	Compact earth	50
3.	Dry sand	25 to 35
4.	Moist sand	30 to 45
5.	Wet sand	15 to 30
6.	Clay	25 to 45
7.	Gravel	30 to 40
8.	Ashes	40
9.	Gravel and sand	25 to 40

(v) For finding out the depth of concrete block, the following formula can be used:

(a) Depth of concrete block in cm $= \frac{1}{58}\sqrt{\frac{pa^2}{m}}$

(b) Depth of concrete block in cm $= \frac{5}{6}\,t.$

Deep foundations: Pile foundations-their suitability, classification of piles based on materials, function and method of installation.

Pile foundations-their suitability

Deep foundations are those in which the depth of the foundation is very large in comparison to its width. Deep foundations are not constructed by ordinary methods of open pit excavations. Deep foundations are of the following types:

a. Pile foundation.
b. Pier foundation
c. Caisson or well foundation.

Out of these, pile foundation is more commonly used in building construction. Following are the situations in which a pile foundation is preferred:

- The load of the super structure is heavy and its distribution is uneven.
- The top soil has poor bearing capacity.
- The subsoil water level is high so that pumping of water from the open trenches for the shallow foundations is difficult and uneconomical.
- There is large fluctuations in subsoil water level.
- If deep strip foundation is attempted, timbering of sides is difficult to maintain or retain the soil of sides of the trench.
- The structure is situated on the sea shore or river bed, where there is danger of scouring action of water.
- Canal or deep drainage lines exist near the foundations.
- The top soil is of expansive nature.

Classification of piles based on materials

Timber piles:

Timber piles are made from tree trunks and are well seasoned, straight and free from all defects. In India, timber piles mostly made up of sal tree trunks. These piles are available in length between 4 to 6 m. Timber piles are used where good bearing stratum is available at a relatively shallow depth.

Concrete piles:

Concrete piles are either precast or cast in situ. Precast piles are cast and cured at the casting site and then transported to the site for installation. These piles are adequately reinforced to withstand handling stresses along with working stresses. Precast piles are normally suitable for short lengths. Cast-in-situ piles are constructed by drilling hole in the ground and then filling the hole by concrete after placing the reinforcement.

Steel piles:

Steel piles are usually of rolled H-sections or thick pipe sections. These piles are used to withstand large impact stresses and where less disturbance from driving is desired. Steel sheet piles and H-piles are generally used to support the open excavation and to provide seepage barrier.

Composite piles:

A pile which is made up of two materials like concrete and timber or concrete and steel is called composite pile. Composite piles are used in situations where a part of the pile is permanently under

water. The part of the pile which will be under water can be made of untreated timber and the other part can be of concrete.

Classification of piles based on function

Based on the function or the use, piles may be classified as:

1. End bearing pile
2. Friction pile
3. Compaction pile
4. Tension pile or uplift pile
5. Anchor pile
6. Fender pile and dolphins
7. Sheet pile.

End bearing piles are used to transfer load through water or soft-soil to a suitable bearing stratum [Fig 4.1(a)].

Friction piles are used to transfer loads to a depth of a friction load carrying material by means of skin friction along the length of piles (Fig. 4.1(b)).

Compaction piles are used to compact loose granular soils, thus increasing their bearing capacity. The compaction piles themselves do not carry any load. Hence they may be of weaker material-sometimes of sand only. The pile tube, driven to compact the soil, is gradually taken out and sand is filled in its place thus forming a 'sand pile [Fig. 4.1 (c)]

Tension or uplift piles anchor down the structures subjected to uplift due to hydrostatic pressure or due to over-turning moment [Fig. 4.1 (d)]

Anchor piles provide anchorage against horizontal pull from sheet piling or other pulling forces (Fig. 4.1 (e)].

Fender piles and dolphins are used to protect water from structures against impact from ships or other floating objects.

Sheet piles are commonly used as bulkheads, or as impervious cut off to reduce seepage and uplift under hydraulic structures. The batter piles are used to resist large horizontal or inclined forces [Fig. 4.1 (f)].

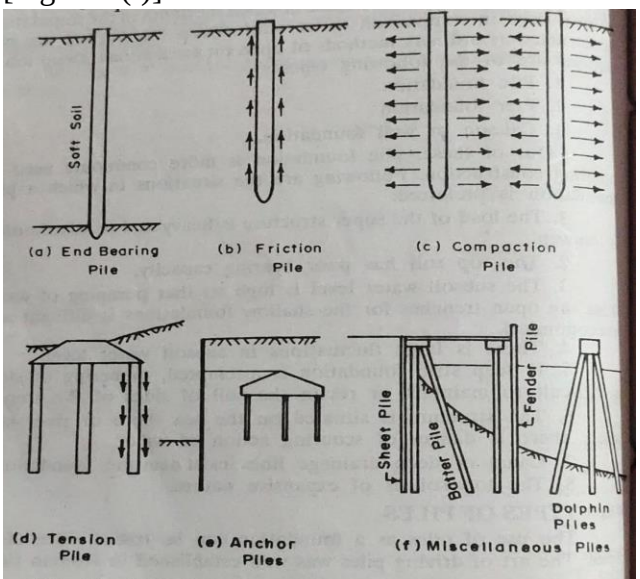


FIG. 4.1. CLASSIFICATION OF PILES BASED ON FUNCTION.

Classification of piles based on method of installation

According to the method of construction, there are three types of piles

1. DRIVEN PILES

   Piles may be of timber, steel or concrete. When the piles are of concrete, they are to be precast. They may be driven either vertically or at an angle to the vertical. Piles are driven using a pile hammer. When a pile is driven into granular soil, the soil so displaced, equal to the volume of the driven pile, compacts the soil around the sides since the displaced soil particles enter the soil spaces of the adjacent mass which leads to densification of the mass. The pile that compacts the soil adjacent to it is sometimes called a *compaction pile*. The compaction of the soil mass around a pile increases its bearing capacity.

2. CAST-IN-SITU PILES

   Cast-in-situ piles are concrete piles. These piles are distinguished from drilled piers as small diameter piles. They are constructed by making holes in the ground to the required depth and then filling the hole with concrete. Straight bored piles or piles with one or more bulbs at intervals maybe cast at the site. The latter type are called under-reamed piles. Reinforcement may be used as per the requirements.

3. DRIVEN AND CAST-IN-SITU PILES

   A steel shell is driven into the ground with the aid of a mandrel inserted into the shell. The mandrel is withdrawn and concrete is placed in the shell. The shell is made of corrugated and reinforced thin sheet steel (mono-tube piles) or pipes (Armco welded pipes or common seamless pipes). The piles of this type are called a shell type. The shell-less type is formed by withdrawing the shell while the concrete is being placed. In both the types of piles the bottom of the shell is closed with a conical tip which can be separated from the shell. By driving the concrete out of the shell an enlarged bulb may be formed in both the types of piles.

Walls & Masonry Works :

Purpose of walls

Classification of walls – load bearing, non-load bearing walls, retaining walls. 3.3.Classification of walls as per materials of construction: brick, stone, reinforced brick,

reinforced concrete, precast, hollow and solid concrete block and composite masonry walls (Concept Only).

3.4.Partition Walls : Suitability and uses of brick and wooden partition walls

3.5.Brick masonry : Definition of different terms.

Bond – meaning and necessity: English bond for 1and 1-1/2 Brick thick walls. T, X and rightangled corner junctions. Thickness for 1and 1-1/2 brick square pillars in English bond

Stone Masonry :

Glossary of terms –String course, corbel, cornice, block-in-course, grouting, mouldings, templates, throating, through stones, parapet, coping, pilaster and buttress

Purpose of walls

Wall is one of the most essential components of a building. The primary function of a wall is to enclose or divide space of the building to make it more functional and useful. Walls provide privacy, afford security and give protection against heat, cold, sun and rain. Walls provide support to floors and roofs. Walls should therefore be so designed as to have provision of adequate.

i. Strength and stability
ii. Weather resistance
iii. Durability
iv. Fire resistance
v. Thermal insulation and
vi. Sound insulation

Classification of walls – load bearing, non-load bearing walls, retaining walls.

Load Bearing Walls

- Load bearing wall is a structural element.
- It carries the weight of a house from the roof and upper floors, all the way to the foundation. It supports structural members like beams, slab and walls on above floors above. A wall directly above the beam is called load bearing wall.
- It is designed to carry the vertical load. In another way, if a wall doesn't have any walls, posts or other supports directly above it, it is more likely to be a load-bearing wall.
- Load bearing walls also carry their own weight. This wall is typically over one another on each floor.
- Load bearing walls can be used as interior or exterior wall. This kind of wall will often be perpendicular to floor joists or ridge.
- Concrete is an ideal material to support these loads. The beams go directly into the concrete foundation. Load bearing walls inside the house tend to run the same direction as the ridge

Non load bearing wall:-

- A wall which doesn't help the structure to stand up and holds up only itself is known as a non-load bearing wall.
- It doesn't support floor roof loads above. It is a framed structure. Most of the time, They are interior walls whose purpose is to divide the structure into rooms. They are built lighter.
- One can remove any non-load bearing walls without endangering the safety of the building. Non-load bearing walls can be identified by the joists and rafters.
- They are not responsible for gravitational support for the property. It is cost effective. This wall is referred to as "curtain wall".

Types of Non-Load Bearing Wall:

- Hollow Concrete Block
- Facade Bricks
- Hollow Bricks
- Brick Walls

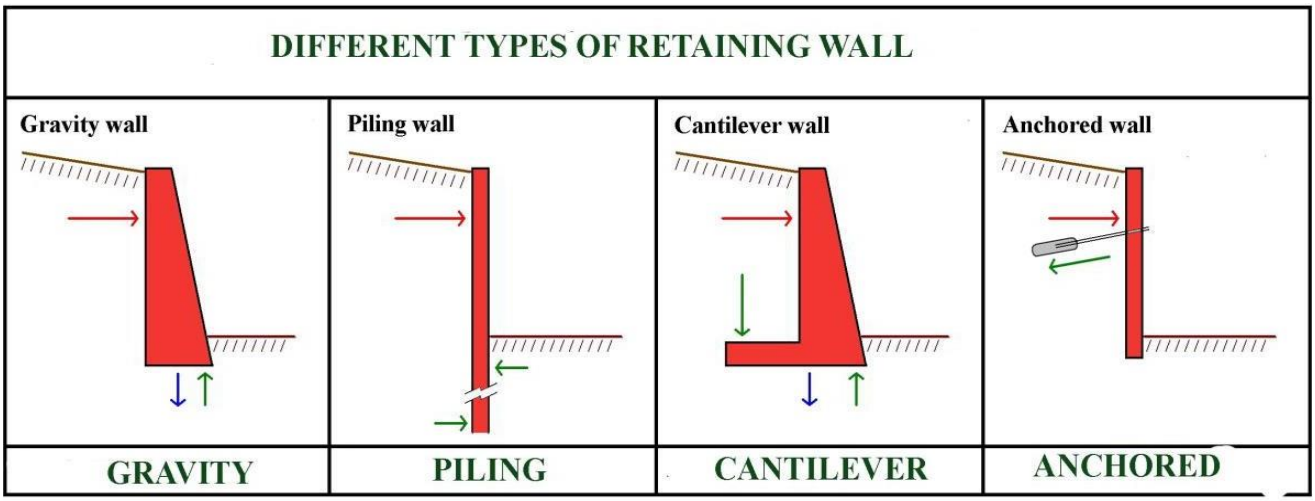
Retaining wall:-

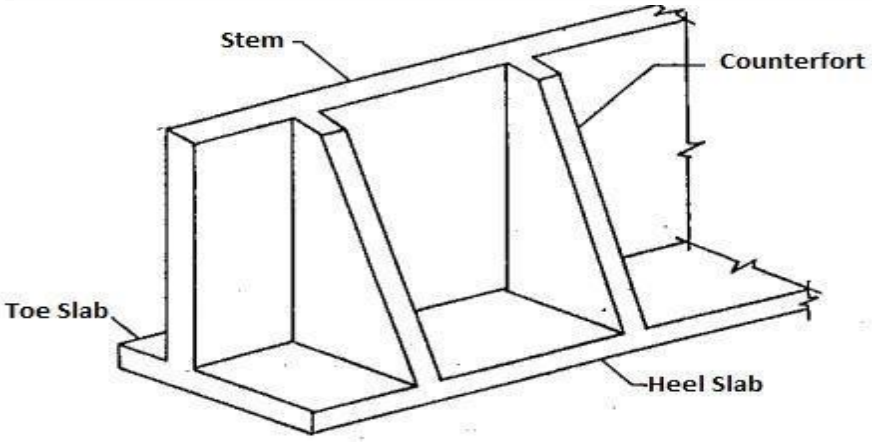
Retaining wall is a structure that retain (holds back) any material (usually earth) and prevents it from sliding or eroding away. It is designed so that to resist the material pressure of the material that it is holding back.

Types of Retaining Wall

An earth retaining structure can be considered to have the following types:

- Gravity Walls
- Reinforced Gravity Walls
- Concrete Cantilever retaining wall
- Counter-fort / Buttressed retaining wall
- Precast concrete Retaining wall
- Pre-stressed retaining wall





Classification of walls as per materials of construction: brick, stone, reinforced brick, reinforced concrete, precast, hollow and solid concrete block and composite masonry walls (Concept Only).

Brick wall

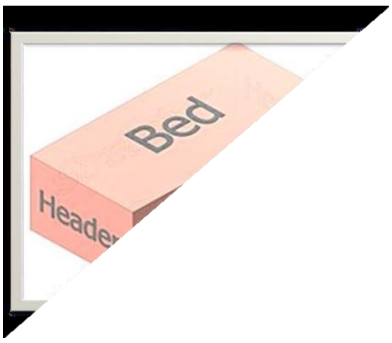
- Brickwork is masonry produced by a bricklayer, using bricks and mortar. Typically, rows of bricks called courses are laid on top of one another to build up a structure such as a brick wall. Bricks may be differentiated from blocks by size.

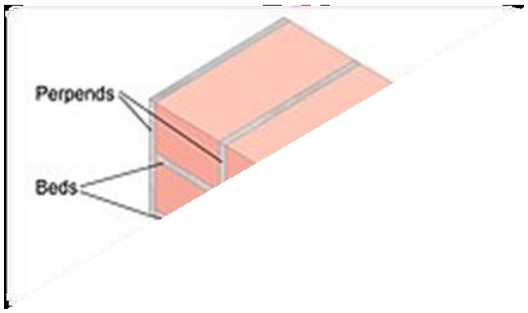
As the most common bricks are rectangular prisms, six surfaces are named as follows:

- Top and bottom surfaces are called Beds
- Ends or narrow surfaces are called Headers or header faces
- Sides or wider surfaces are called Stretchers or stretcher faces

Mortar terminology-showing perpends and bed.

- Mortar placed between bricks is also given separate names with respect to their position. Mortar placed horizontally below or top of a brick is called a bed, and mortar Placed vertically between bricks is called perpend.





Stone wall

Stone walls are a kind of masonry construction that has been used for thousands of years. The first stone walls were constructed by farmers and primitive people by piling loose field stones into a dry stone wall. Later, mortar and plaster were used, especially in the construction of city walls, castles, and other fortifications before and during the Middle Ages.

Reinforced brick wall

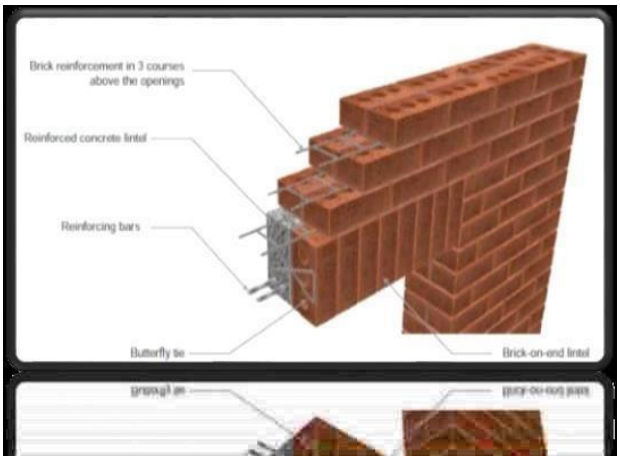
- Reinforced Brick Wall: Walls that are prone to tensile forces are needed to be reinforced, and both horizontal and vertical reinforcement is required.
- Horizontal reinforcement is provided at every third or fourth course, consists of steel mesh spread flat on the cement mortar and pressed uniformly.

Reinforced concrete wall

These wall are of 3 types

1. Solid concrete block
2. Hollow concrete Block
3. Precast concrete block

The residential and commercial construction industries make wide use of a form of concrete building material known officially as a concrete masonry unit (CMU). These hollow-core blocks can be made of standard concrete with traditional sand and gravel aggregate held together with Portland cement. Or, they can substitute lighter-weight industrial waste materials, such as fly ash or coal cinders, for the sand and gravel aggregate, in which case they are usually known as cinder block.

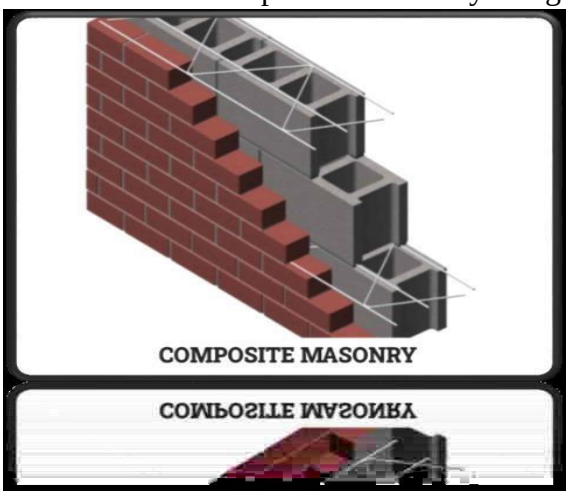


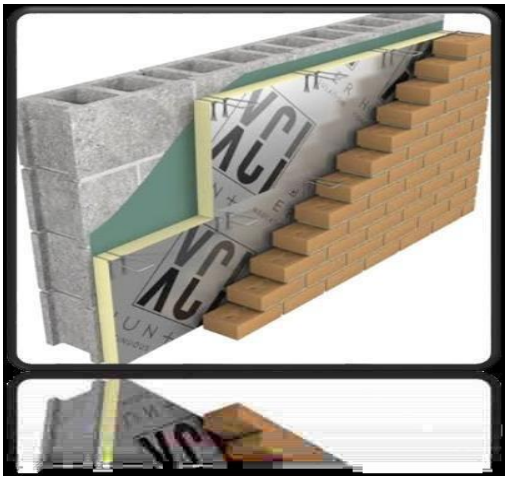
Composite Masonary Wall:-

Composite masonry is a masonry constructed with two or more types of construction units or different types of construction materials, i.e. bricks, stones, etc.

Composite masonry is adopted for the following reasons:

1. This masonry reduces the overall cost of construction.
2. It makes the structure more durable by providing better quality and good workmanship material in the faces.
3. Also, improves the appearance of the structure by concealing the inferior function.
4. It achieves optimum economy using locally available materials





Partition Walls : Suitability and uses of brick and wooden partition walls

- A partition wall is defined as a wall or division made up of bricks, studding, glass or other such material and provided for dividing from one room to another room.
- Partition Wall can be constructed as a load-bearing wall or non-load bearing wall. The load-bearing partition wall is known as the internal wall.
- These internal walls do not carry any weight. Sometimes partition wall may be folded, collapsible, or fixed type.
- Partition walls or walls can be constructed in various shapes like –thin, light, or thick, heavy, etc. Mainly, these walls' requirements are based on building materials and design.

BRICK PARTITIONS

Brick partitions are quite common since they are the cheapest.

Brick partitions are of three types:

1. Plain brick partitions
2. Reinforced brick partitions.
3. Brick nogging partitions.

Plain Brick Partitions

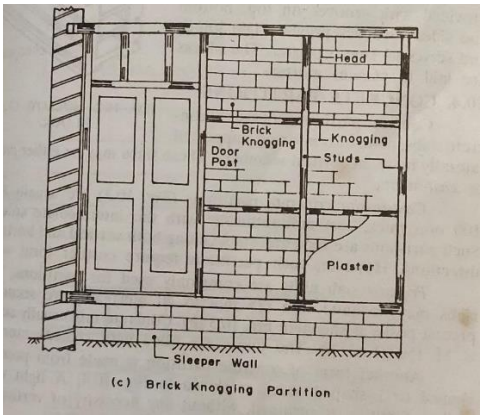
Plain brick partitions are usually half brick thick. The bricks are laid as stretchers , in cement mortar. Vertical joints are staggered alternate blocks. The wall is plastered on both the sides. The wall is considerably strong and fire resistant.

Reinforced Brick Partitions

These are stronger than the ordinary brick partitions, and is used when better longitudinal bond is required, and when the partition wall has to carry other super-imposed loads. The thickness of the wall is kept equal to half brick (10 cm). The reinforcement consists of steel meshed strips, called Exmet.

Brick Nogging Partitions

Brick nogging partition wall consists of brick work (half brick thickness) built up within the frame work of wooden members. The timber frame work consists of (i) sill, (ii) head, (iii) vertical members, called studs, and (iv) horizontal members called nogging pieces. The vertical members or studs are spaced at 4 to 6 times the brick length. The nogging pieces are housed into the studs at vertical interval of 60 to 90 cm. The framework provided stability to the partition against lateral loads and vibrations caused due to opening the adjoining door.



(c) Brick Knogging Partition

TIMBER PARTITIONS

Timber partitions consist of wooden frame work, properly supported on floor and fixed to the side walls. This frame work, made of horizontal and vertical members, can either be plastered or covered with boarding etc. from both the sides. Wooden partitions are light weight, but are costlier. It is likely to decay, or eaten away by termites. Also, it is not fire resistant. It's use is reducing day by day.

Two types of wooden partitions may be used:

i. Common or stud partition.
ii. Trussed or braced partition.

1. Common or stud partition

It consists of a frame work of vertical members (called studs), and short horizontal pieces, called noggings. Horizontal pieces impart rigidity to studs. A stud of short length, such as the one provided on an opening, is called, puncheon. The upper and lower horizontal members of the frame are known as head and sill respectively.

2. Trussed or braced partitions

Such partitions are provided where there is no means of supporting the partition except at their ends. The frame work is similar to the stud partition, but inclined members called braces, and steel straps and bolts are additionally used. Sometimes, such partitions carry floor load also, in addition to its own weight. For more rigidity and strength, an additional horizontal member, known as inter-tie is provided between head and sill.

Brick masonry : Definition of different terms.

Brick masonry is made of brick units bonded together with mortar. Two essential components of brick masonry are therefore:

i. Bricks
ii. Mortar

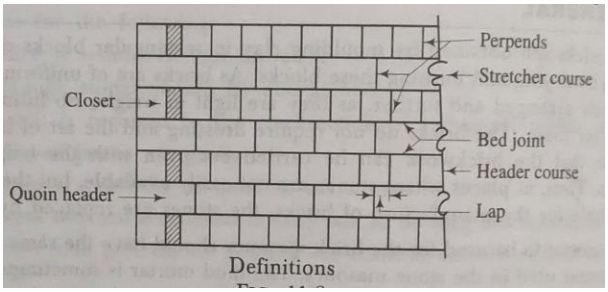
Mortar acts as a cementing material and unites the individual brick units together to act as a homogeneous mass. Following types of mortar may be used in brick masonry :

1. Cement mortar
2. Lime mortar
3. Cement-lime mortar
4. Lime-surkhi mortar and
5. Mud mortar

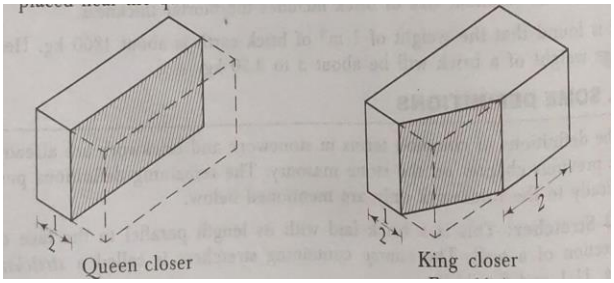
Mud mortar is used only for low-rise buildings which carry light loads. Cement mortars are used for high-rise buildings, where strength is of prime importance. Lime mortar and lime-surkhi mortars are used for all types of construction.

Bricks are manufactured by moulding clay in rectangular blocks of uniform predetermined size, drying them and then burning them in a kiln.

Definition of different terms



Definitions



Lap: The horizontal distance between the vertical joints in successive courses is termed as a lap and for a good bond, it should be one-fourth of the length of a brick.

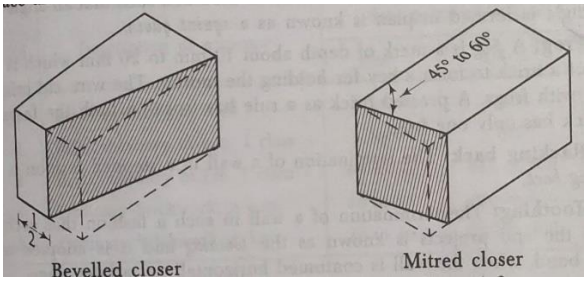
Closer: A piece of brick which is used to close up the bond at the end of brick courses is known as a closer and it helps in preventing the joints of successive courses to come in a vertical line. Generally the closer is not specially moulded. But it is prepared by the mason with the edge of the trowel. Following are the types of closers

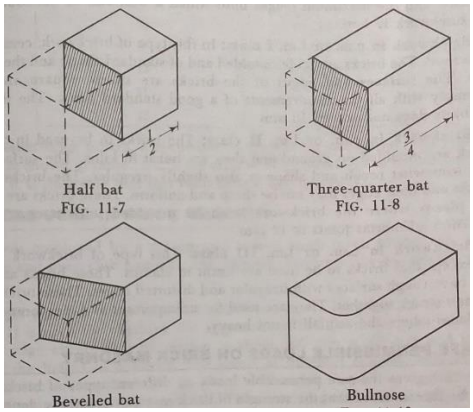
Queen closer: This is obtained by cutting the brick longitudinally in two equal parts.

King closer: This is obtained by cutting a triangular portion of the brick such that half a header and half a stretcher are obtained on the adjoining cut faces.

Bevelled closer: This is obtained by cutting a triangular portion of half the width but of full length.

Mitred closer: This is obtained by cutting a triangular portion of the brick through its width and making an angle of 45° to 60° with the length of the brick.
Bat: This is a piece of brick, usually considered in relation to the length of brick and accordingly known as half bat or three-quarter bat





Bullnose: A brick moulded with a rounded angle is termed as a bullnose and it is used for a rounded quoin. A connection which is formed when a wall takes a turn is known as a quoin.
Cownose: A brick moulded with a double bullnose on end is termed as a cownose.
Frog: A frog is a mark of depth about 10 mm to 20 mm which is placed on the face of a brick to form a key for holding the mortar. The wire cut bricks are not provided with frogs. A pressed brick as a rule has frogs on both the faces. A hand- made brick has only one frog.
Racking back: The termination of a wall in a stepped fashion is known as the racking back.
Toothing: The termination of a wall in such a fashion that each alternate course at the end projects is known as the toothing and it is adopted to provide adequate bond, when the wall is continued horizontally at a later stage.

Bond – meaning and necessity: English bond for 1and 1-1/2 Brick thick walls. T, X and right angled corner junctions. Thickness for 1and 1-1/2 brick square pillars in English bond

Bond- meaning and necessity
It is the method of arranging the bricks in courses so that individual units are tied together and the vertical joints of the successive courses do not lie in same vertical line. Bond of various types are distinguished by their elevation or face appearance. Bricks used in masonry are all of uniform size. If they are not arranged (or bonded) properly, continuous vertical joints will result. An unbonded wall, with its continuous vertical joints has little strength and stability. Bonds help in distributing the concentrated loads over a larger area. Since bricks are small units, having uniform dimensions, the process of bonding is easily performed.
Following are the types of bonds provided in brick work:

1. Stretcher bond.
2. Header bond.
3. English bond.
4. Flemish bond.
5. Facing bond.

6. English cross bond.
7. Brick on edge bond.
8. Dutch bond.
9. Raking bond.
10. Zigzag bond.
11. Garden wall bond.

English bond

This is the most commonly used bond, for all wall thicknesses. This bond is considered to be the strongest. The bond consists of alternate courses of headers and stretchers. In this bond, the vertical joints of the header courses come over each other, similarly, the vertical joints of the stretcher courses also come over each other. In order to break the vertical joints in the successive courses, it is essential to place queen closer after the first header (quoin header) in each heading coarse.
Following are the essential features of English bond.

1. Alternative courses will show either headers or stretchers in elevation.
2. Every alternate header comes centrally over the joint between two stretchers in course below.
3. In the stretcher course, the stretchers have a min. lap of ¼ th their length over headers.
4. There is no continuous vertical joint.
5. Walls of even multiple of half bricks (i.e. 1 brick thick wall, 2-bricks thick wall, 3-bricks thick wall) present the same appearance on both faces. Thus a course showing stretchers on the front face will also show stretchers on the back face.
6. Wall of odd multiple of half bricks (i.e. 1 ½ brick thick wall, 2 ½ brick thick wall etc.) will show stretchers on one face and headers on the other face.
7. Since the number of vertical joints in the header course are twice the number of vertical joints in the stretcher course, the joint in the header course are made thinner than the joints in the stretcher course.

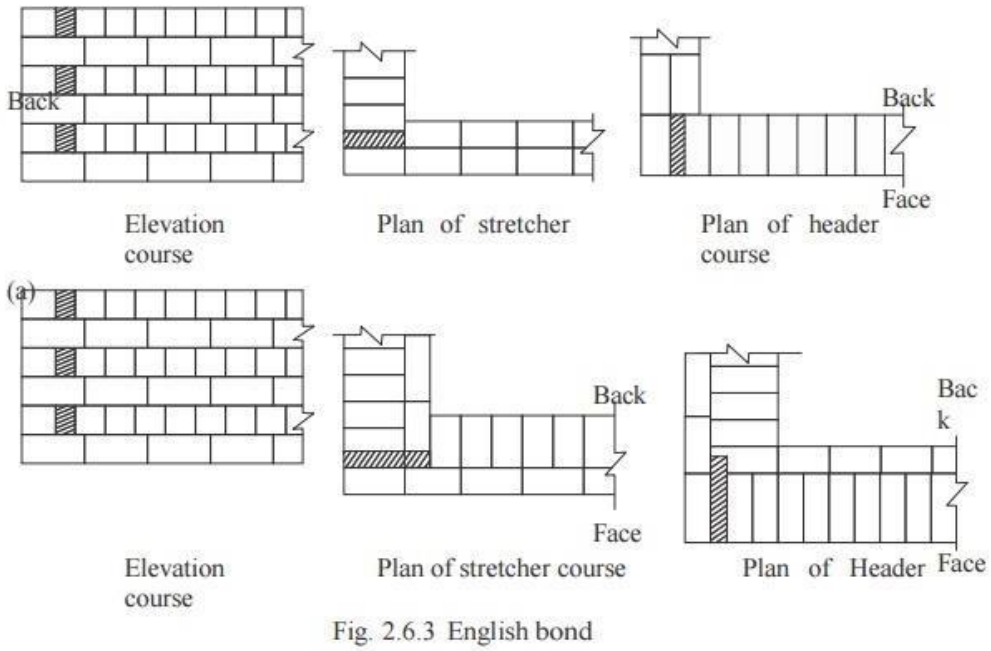


Fig. 2.6.3 English bond

T, X and right angled corner junctions

Junctions: A connection between a main wall and a partition wall is termed as a junction.

A junction is classified in two categories:

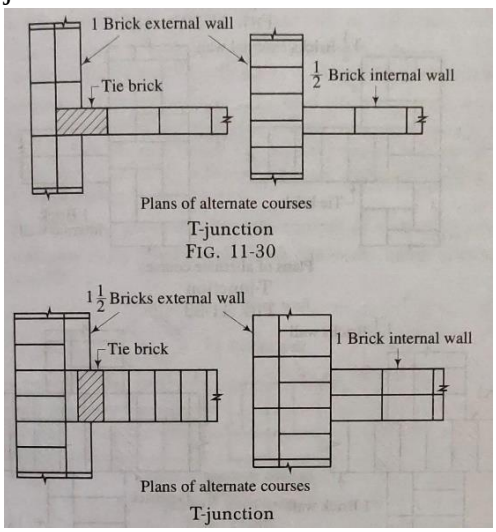
i. Right-angled junction
ii. Squint junction.

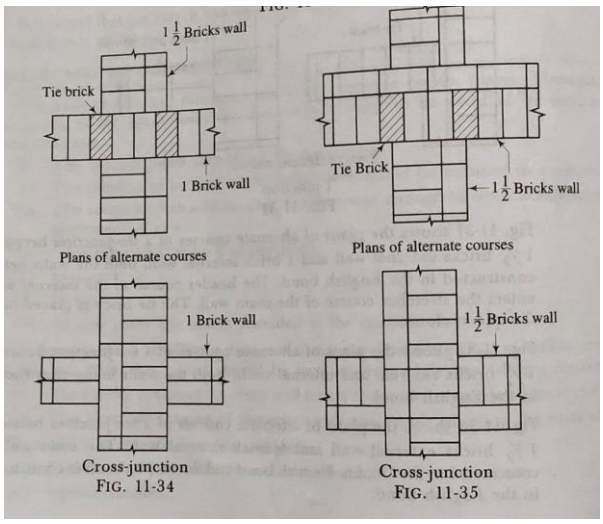
Right-angled junction: This type of junction has two forms:

a. Tee-junction
b. Cross-junction or intersection.

Tee-junction: This type of right-angled junction forms the shape of the English letter T in plan.

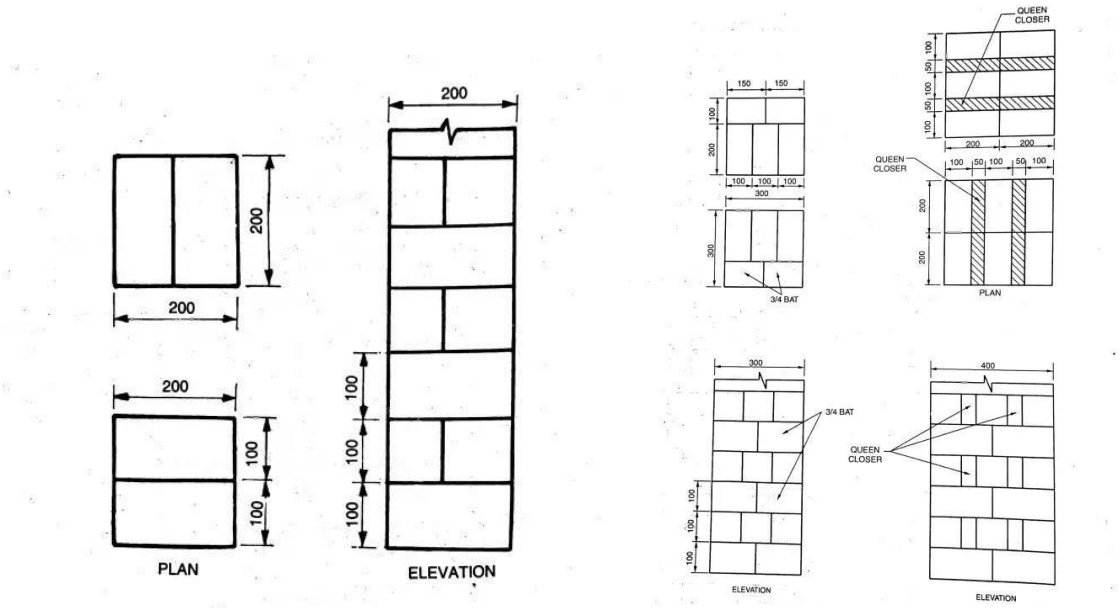
Cross-junction or intersection: When two continuous walls cross or intersect each other, a cross-junction or an intersection.





Thickness for 1and 1-1/2 brick square pillars in English bond

Fig. Shows alternate courses of pillar for various thickness along with their elevation in English Bond



Stone Masonry :

- Masonry may be defined as the construction of building units bonded together with mortar. The building units (commonly known as masonry units) may be stones, bricks or precast blocks of concrete. When stones are used as the building units or building blocks, we have stone masonry.
- Masonry work is one of the major building crafts and one of the oldest. It has built itself great reputation as one of the premier traditional materials of building. Even though new principles of construction and new materials become prominent in building construction practices, masonry has got the highest importance in building industry.
- Masonry is normally used for the construction of foundations, walls, columns and other similar structural components of buildings. The basic advantage of masonry construction lies in the fact that in load-bearing structures, it performs a variety of functions such as
  i. Supporting loads,
  ii. Subdividing space,
  iii. Providing thermal and acoustic insulation,
  iv. Affording fire and weather protection etc,

3.7.Glossary of terms

The meanings, attached to some technical terms used in the masonry, are given below for ready reference.

String course

It is the continuous horizontal course of masonry, projecting from the face of the wall for shedding rain water off the face. It is generally provided at every floor and sill level. A string course breaks the monotony of a plane surface, and thus imparts aesthetic appearance to the structure. The string course is suitably weathered and throated so as to throw off water clear of the wall surface.

Corbel

A corbel is a projecting stone which is usually provided to serve as support for joist, truss, weather shed etc. Corbels are generally moulded and given ornamental treatment. Corbels should extend at least two-thirds of their length into the wall, so that they do not overturn or come out of the wall.

Cornice

It is a projecting ornamental course near the top of a wall or at the junction of wall and the ceiling. It penetrates the full width of the wall. It is weathered and throated to dispose off rain water. In order to prevent overturning of cornice, extra weight in the form of parapet wall should be provided.

Block-in-course

It is another course of stone placed immediately above the cornice. Apart from improving the appearance of the wall, it adds to the stability of the cornice against overturning.

Grouting

The process of filling voids in masonary of stone is known as grouting.

Templates

It is defined as the block of stone or concrete provided under a beam or girder to distribute the concentrated load over a greater area of the bearing surface.

Throating

It is a groove provided on the underside of projecting elements such as sills cornices, copings etc., so that rain water can be discharged clear of the wall surface.

Through stones

A through stone is a stone header. Through stones are placed across the wall at regular interval. If the thickness of the wall is small, through stone may be of length equal to the full width of the wall. If, however the wall is considerably thick, two through stones with an overlap are provided, Through stones should be strong, and non-porous, and should be of sufficient thickness.

Parapet

It is the portion of low height wall constructed along the edge of the roof to protect the users.

Coping

It is a covering of stone, concrete, brick of terracota, placed on the exposed top of a wall, to prevent seepage of water. It may also be provided on the top of compound wall. A coping is suitably weathered and throated.

Pilaster

Pier is an isolated vertical mass of stone or brick masonry to support beams, lintels, arch etc, the width of which exceeds four times its thickness. If it is made monolithic with the wall and projecting a little beyond to support the ends of a beam or truss etc, then it is called a pilaster.

Buttress

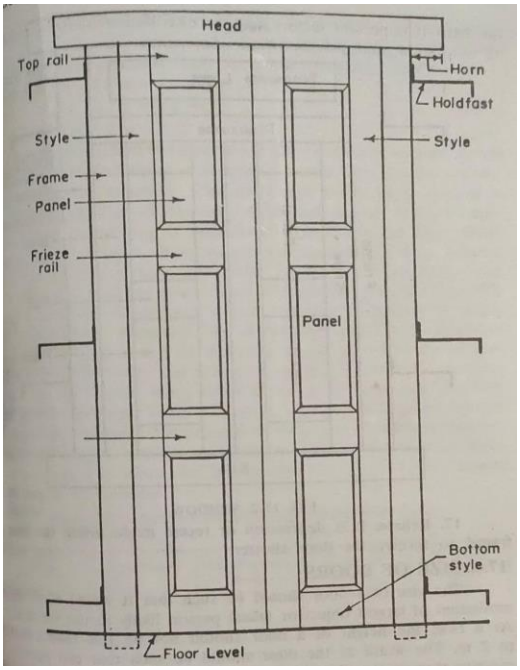
It is a sloping or stepped masonry projection from a tall wall intended to strengthen the wall against the thrust of a roof or arch.

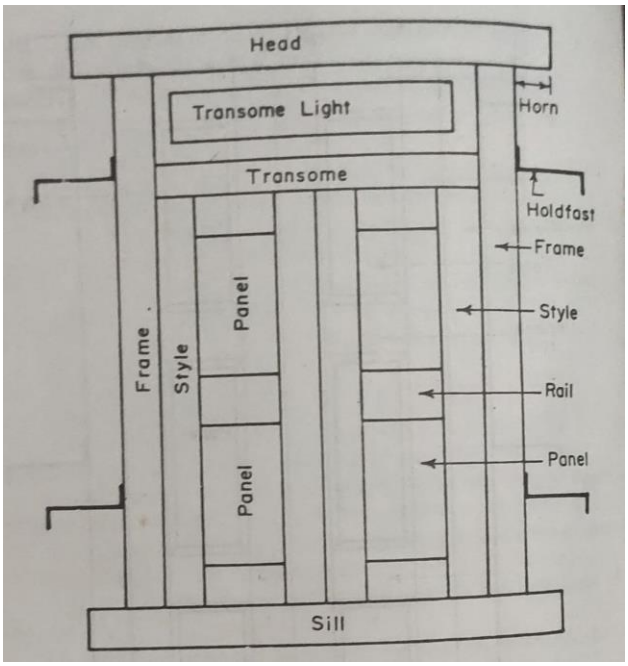
Doors, Windows And Lintels

4.1.Glossary of terms used in doors and windows
4.2.Doors – different types of doors
4.3.Windows – different types of windows
4.4.Purpose of use of arches and lintels

Glossary of terms used in doors and windows

Figs. show respectively a door and a window. The following are the technical terms applied to doors and windows:





1. **Frame**. It is an assembly of horizontal and vertical members, forming an enclosure, to which the shutters are fixed.
2. **Shutters**. These are the openable parts of a door or window. It is an assembly of styles, panels and rails.
3. **Head**. This is the top or uppermost horizontal part of a frame.
4. **Sill**. This is the lowermost or bottom horizontal part of a window frame. Sills are normally not provided in door frames.
5. **Horn**. These are the horizontal projections of the head and sill of a frame to facilitate the fixing of the frame on the wall opening. The length of horns is kept about 10 to 15 cm.
6. **Style**. Style is the vertical outside member of the shutter of a door or window.
7. **Top rail**. This is the top most horizontal member of a shutter.
8. **Lock rail**. This is the middle horizontal member of a door shutter, to which locking arrangement is fixed.
9. **Bottom rail**. This is the lowermost horizontal member of a shutter.
10. **Intermediate or cross-rails**. These are additional horizontal rails, fixed between the top and bottom rails of a shutter. A rail fixed between the top rail and lock rail is called frieze rail.
11. **Panel**. This is the area of shutter enclosed between the adjacent rails.
12. **Mullion**. This is a vertical member of a frame, which is employed to sub-divide a window or a door vertically.

13. **Transom**. This is a horizontal member of a frame, which is employed to sub-divide a window opening horizontally.
14. **Hold fasts**. These are mild steel flats (section 30 mm X 6 mm), generally bent into Z-shape, to fix or hold the frame to the opening. The horizontal length of hold fast is kept about 20 cm, and is embedded in the masonry.
15. **Jamb**. This is the vertical wall face of an opening which supports the frame.
16. **Reveal**. It is the external jamb of a door or window opening at right angles to the wall face.
17. **Rebate**. It is depression or recess made inside the door frame, to receive the door shutter.

Doors – different types of doors

A door may be defined as an openable barrier secured in a wall opening. A door is provided to give as access to the inside of a room of a building. It serves as a connecting link between the various internal portions of a building. Basically, a door consists of two parts:

i. Door frame, and
ii. Door shutter.

The door shutter is held in position by the door frame which in turn is fixed in the opening of the wall by means of hold-fasts etc.

Depending upon the type of materials, arrangement of different components, method of construction, etc., the common types of doors used in the buildings can be grouped as follows:

1. Ledged doors
2. Ledged and braced doors
3. Ledged and framed doors
4. Ledged, framed and braced doors
5. Framed and panelled doors
6. Glazed or sash doors
7. Flush doors
8. Louvered doors
9. Collapsible steel doors
10. Revolving doors
11. Rolling steel doors
12. Sliding doors
13. Swing doors.

Ledged doors

A ledged door is formed of the vertical boards, known as the battens, which are secured by horizontal supports, known as the ledges.

Ledged and braced doors

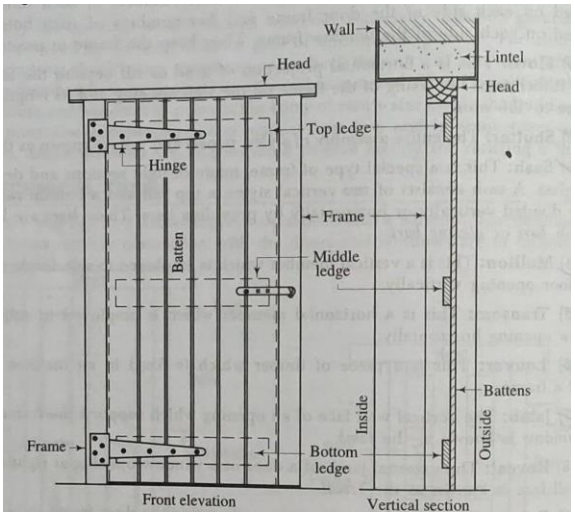
These are similar to the ledged doors except that the diagonal members, known as the braces, are provided as shown.

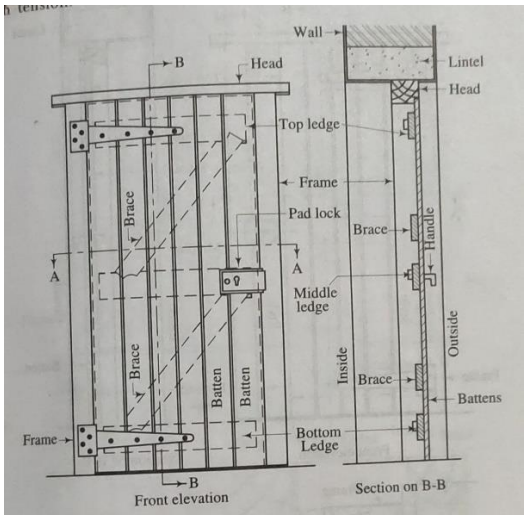
Ledged and framed doors

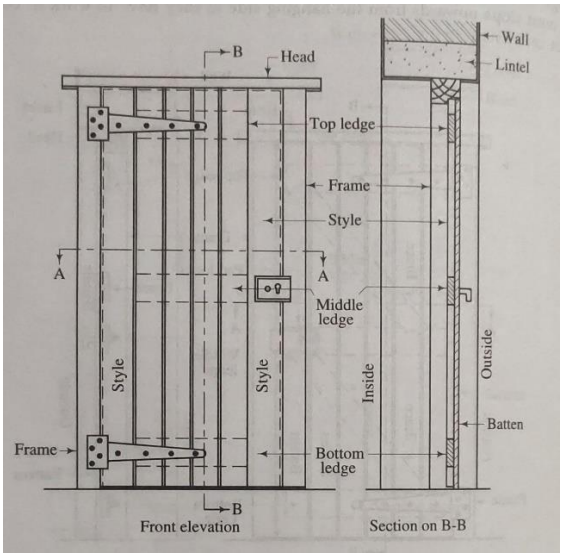
In this type of doors, a framework for shutters is provided to make the doors stronger and better in appearance as shown.

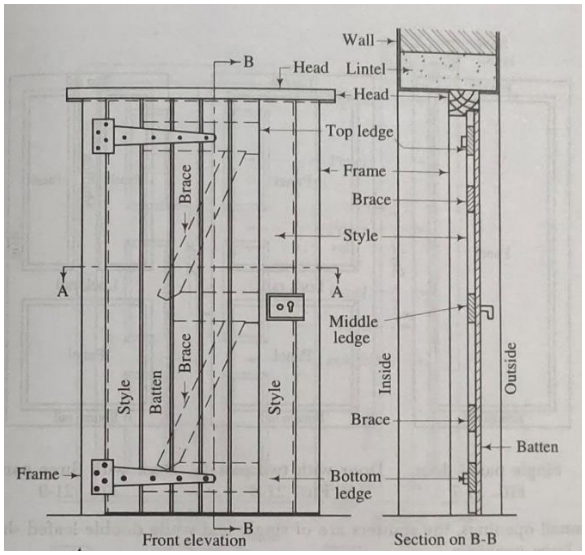
Ledged, framed and braced doors

This is just similar to the above type except that the braces are introduced as shown. This type of door is more durable and stronger and hence it can be adopted for external use.









Framed and panelled doors

This is the most usual variety of door and it consists of a framework in which panels are fitted. This type of door reduces the tendency of shrinkage and presents a decent appearance. The styles are continuous from top to bottom and rails are joined to the styles. The mullions, if any, are joined to rails. The panels are secured in position by grooves made inside the edges of the framework. The number and size of panels depend upon the architect's design or owner's desire. But the number varies from one to six and panels are moulded to add to the beauty of the door.

Glazed or sash doors

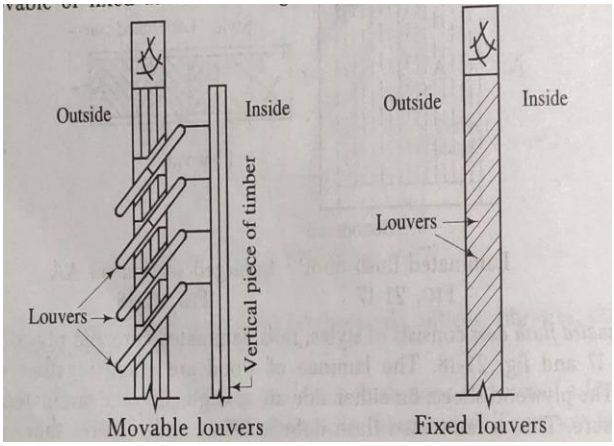
In order to admit more light, in addition to that coming from the windows, the fully glazed or partly panelled and partly glazed doors are used.

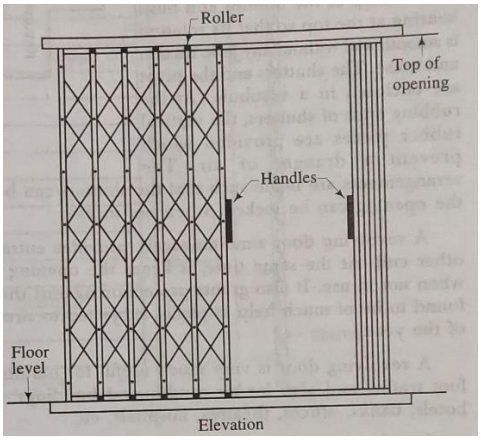
Flush doors

A flush door consists of a framework of rails and styles and it is covered with plywood or hard-board. There are two varieties of flush doors a framed flush door and a laminated flush door.

Louvered doors

In this type of doors, the shutters are provided with louvers, either fully or partly. The louvers are arranged at such an inclination that the horizontal vision is obstructed. The louvers may be movable or fixed as shown.





Collapsible steel doors

A collapsible steel door consists of a mild steel frame. The rollers, mounted on horizontal piece, are provided at top and bottom ends of the vertical pieces. The door can be opened or closed manually by a slight pull or push. The door is also provided with handles, locking arrangement, stoppers, etc.

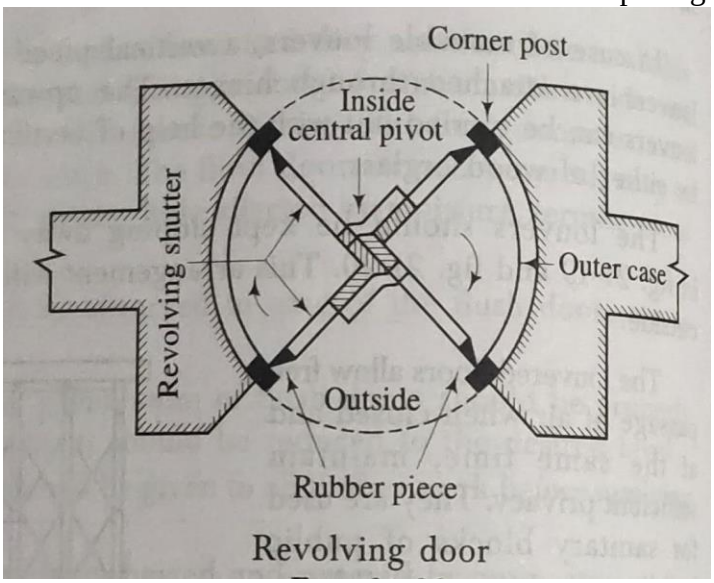
Revolving doors

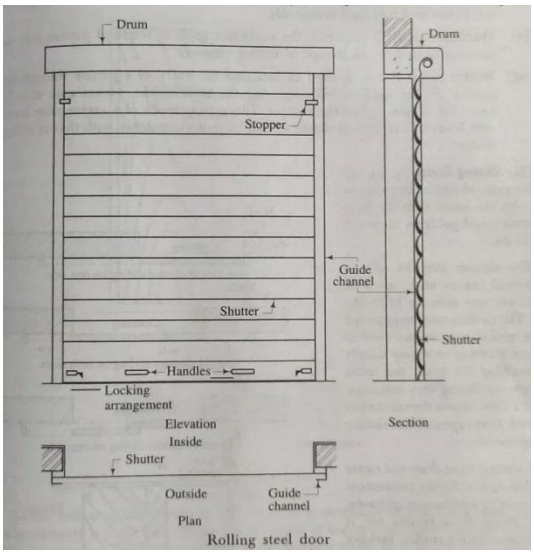
A revolving door essentially consists of a centrally placed mullion or pivot in a circular opening. The revolving shutters or leaves which are four in number are radially attached to the pivote as shown. A revolving door simultaneously provides entrance on one side and exit on the other end.

Rolling steel doors

A rolling steel door consists of a frame, a drum and a shutter of thin steel plates or iron sheets of thickness about 1 mm, as shown. The shutter usually rolls in turns. Thus a slight pull or push will close or open the shutter.

The rolling steel doors are sufficiently strong and as they can be easily rolled up at the top, they cause no obstruction to the floor as well as the opening.





Sliding doors

In this type of doors, the shutter slides on the sides with the help of runners and guide. The shutter may be of one or several leaves and can slide either on one side or both the sides. The cavities may be provided in the wall to receive the door in an open position or it may simply lie touching the wall.

Swing doors

A swing door is provided with special hinges known as the double action spring hinges and thus the shutters of the door are held in closed position, when the door is not in use. The shutter may be of one or several leaves. When the door is to be used, a slight push is made and then the action of spring brings the shutter in closed position.

Windows – different types of windows

A window is also a vented barrier secured in a wall opening. The function of the window is to admit light and air to the building and to give a view to the outside. Windows must also provide insulation against heat loss, and in some cases, against sound. Some windows are also required to give a measure of resistance to fire. A window also consists of two parts:

i. Window frame, secured to the wall opening with the help of hold fasts, and
ii. Window shutters held position by the window frame.

Depending upon the manner of fixing, materials used for construction, nature of operational movements of shutters, etc., the common varieties of windows used in the buildings can be grouped as follows:

1. Casement windows
2. Double-hung windows
3. Pivoted windows
4. Sliding windows
5. Louvered windows
6. Sash or glazed windows
7. Metal windows
8. Circular windows
9. Corner windows
10. Gable windows
11. Dormer windows
12. Bay windows
13. Clerestorey windows
14. Lanterns or lantern lights
15. Skylights.

Casement windows

These are the windows, the shutters of which open like doors. The construction of a casement window is similar to the door construction and it consists of a frame, styles, rails, vertical and horizontal sash bars and sometimes, it also includes mullions and transoms.

Double-hung windows

These windows consist of a pair of shutters which can slide within the grooves provided in the frame. A pair of metal weights connected by cord or chain over pulleys is provided for each sash. It is so arranged that the upper sash moves in the downward direction, thus opening at the top and the lower sash moves in the upward direction, thus opening at the bottom.

Pivoted windows

In this type of windows, the shutters are allowed to swing round the pivots. The frame of a pivoted window is just similar to casement window except that no rebates are provided. The windows may be vertically pivoted or horizontally pivoted. The pivoted windows are easy to clean and they admit more light than the side-hung windows.

Sliding windows

These windows are similar to the sliding doors and the shutters move on the roller bearings, either horizontally or vertically. Suitable openings are provided in the walls to receive the shutters when windows are opened out.

Louvered windows

In this type of windows, the louvers are provided as in case of louvered doors. They allow free passage of air when closed and at the same time, they maintain sufficient privacy.

Sash or glazed windows

These are fully glazed casement windows. The sashes are rebated to receive glass panels.

Metal windows

These are now-a-days widely used, especially for public buildings. The metal used in the construction may be mild steel, bronze or other alloys. The steel windows are manufactured in standard sizes and are widely used.

Circular windows

These are pivoted windows of circular shape. They are useful for factories, workshops, etc.

Corner windows

These windows are provided at the corner of a room. They are placed at the corner of a room and thus they have two faces in two perpendicular directions. Due to such situation, there is entry of light and air from two directions and in many cases, the elevation of the building is also improved.

Gable windows

These are the windows which are provided in the gable ends of a roof

Dormer windows

These are the windows provided on the sloping roofs. The main purpose of providing dormer windows is to admit light and air to rooms which are constructed within or below the roof slopes.

Bay windows

These windows project outside the external walls of a room. They may be square, splayed, circular, polygonal or of any shape. These windows admit more light, increase opening area, provide ventilation and improve the appearance of the building.

Clerestorey windows

These windows are provided near the top of main roof. The pivoted windows are used for this purpose. The clerestorey windows provide ventilation to the inside of the room as the front is blocked by the verandah. They also improve the appearance of building. The care should be taken to see that the upper part opens inside and the lower part opens outside. Otherwise the rain water will accumulate in the room.

Lanterns or lantern lights

These are the windows which are fixed on flat roofs to provide light to the inner portion of the building where light coming from the windows in the external walls is insufficient. They may be square or rectangular or curved. The glass panels are generally fixed. But if ventilation is also required in addition to light, the pivoted windows may be provided.

Skylights

These are the windows which are provided on the sloping surface of a pitched roof. As skylights are mainly meant for light, they are usually provided with the fixed glass panels. The opening of skylight is properly treated by lead flashing to make water proof the roof area surrounding the opening.

Purpose of use of arches and lintels

The openings for doors, windows, ventilators, cupboards, wardrobes, etc., are invariably required in a wall. These openings are bridged by the provision of either an arch or a lintel. Thus the arch as well as the lintel are structural members which are designed to support the loads of the portion of wall above the openings and to transmit such loads to the ends of walls or piers or jambs or columns over which they are supported.

A lintel is a horizontal member which is placed across an opening to support the portion of the structure above it. The function of a lintel is just the same as that of an arch or a beam. However the lintels are easy and simple in construction. The ends of lintels are built into the masonry and thus the load carried by lintels is transferred to the masonry in jambs. At present, the lintels of R.C.C. are widely used to span the openings for doors, windows, etc. in a structure. Arch and lintel are provided above doors, windows and passages. The function of an arch and a lintel is to carry the weight of the structure above the opening.

Floors, Roofs and Stairs

Floors: Glossary of terms ,Types of floor finishes – cast-in-situ, concrete flooring(monolithic,bonded), terrazzo tile flooring, cast in situ Terrazzo flooring, timber flooring (Concept only)

Roofs: Glossary of terms, Types of roofs, concept and function of flat, pitched, hipped andSloped roofs

Stairs: Glossary of terms; Stair case, winder, landing, stringer, newel, baluster, rise, tread,width of stair case, hand rail, nosing, head room, mumty room.

Various types of stair case – straight flight, dog legged, open well, quarter turn, half turn (neweland geometrical stairs), bifurcated stair, spiral stair, cantilever stair, tread riser stair.

Floors: Glossary of terms ,Types of floor finishes – cast-in-situ, concrete flooring(monolithic, bonded), terrazzo tile flooring, cast in situ Terrazzo flooring, timber flooring (Concept only)

The purpose of a floor is to provide a level surface capable of supporting the occupants of a building, furniture, equipment and sometimes, internal partitions. To perform this function, and in addition, others which may vary according to the situation of the floor in the building and the nature of the building itself, a floor must satisfy the following requirements:

i. Adequate strength and stability
ii. Adequate fire resistance.
iii. Sound insulation
iv. Damp resistance
v. Thermal insulation.

The floors resting directly on the ground surface are known as ground floors, while the other floors of each storey, situated above the ground level are known as upper floors.

Glossary of terms

A floor is composed of two essential components:

i. Sub-floor, base course or floor base
ii. Floor covering, or simply, flooring.

The floor base is a structural component, which supports the floor covering. For the ground floors, the object of floor base is to give proper support to the covering so that it does not settle, and to provide damp resistance and thermal insulation.

When you look at your floors, you may only think of the one layer that you see. Like the ground you stand on, there are several layers underneath the top one that ensures the stability and function of your floor.

Joists

At the very bottom of the layers that make up your floors is what we call joists. Think of these as the bones of your flooring. They are used to frame the layer that will sit upon your home's foundation. It's crucial that joists are installed correctly because if not, it will result in a great deal of noise, squeaking, and shifting when you walk on your floors.

Subfloor

After the joists, you'll find the subfloor. This is the solid base that your floor will stand on and will support the floor covering. The subfloor is most often constructed with concrete. It's crucial that the subfloor has room to breathe, as in expanding and contracting over the years to make your floor function properly.

Underlayment

On top of the subfloor is the underlayment. The underlayment is installed for noise control because it softens any noise that will result from someone walking on the floor. Not only is it installed for noise cancellation, but underlayment gives the floor covering a smooth surface to sit on. Most often made of foam or cork, underlayment is required for "floating" floors such as laminate or engineered wood. These floors are not attached to the subfloor so they "float" on top.

Floor Covering

The last part of your flooring is the top layer called the floor covering. Floor coverings come in a variety of materials and styles. They can be hardwood, tile, stone, carpet, vinyl, or laminate. This is what you think of when you think of floors.

Adhesive

Floor coverings are often secured to subfloor or underlayment depending on the material. With tile, thinset is used to adhere the tiles to the subfloor. Thinset will need to be expertly removed when looking to install new flooring where the tile was once installed.

Types of floor finishes

Concrete flooring

This is commonly used for residential, commercial and even industrial building, since it is moderately cheap, quite durable and easy to construct. The floor consists of two components:

- Base concrete, and
- Topping or wearing surface.

The two components of the floor can be constructed either monolithically (i.e. topping laid immediately after the base course is laid) or non-monolithically. When the floor is laid monolithically, good bond between the two components is obtained resulting in smaller over all thickness. However, such a construction has three disadvantages:

- The topping is damaged during subsequent operations,
- Hair cracks are developed because of the settlement of freshly laid base course which has not set, and
- Work progress is slow because the workman has to wait at least till the initial setting of the base course.

Hence in most of the cases, non-monolithic construction is preferred:

The base course may be 7.5 to 10 cm thick, either in lean cement concrete (1:3:6 to 1:5:10) or lime concrete containing 40% mortar of 1:2 lime-sand (or 1 lime : 1 surkhi: 1 sand) and 60% coarse aggregate of 40 mm nominal size. The base course is laid over well-compacted soil, compacted properly and levelled to rough surface. It is properly cured.

Terrazzo flooring

Terrazzo flooring is another type of floor finish that is laid in thin layer over concrete topping. It is very decorative and has good wearing properties. Due to this, it is widely used in residential buildings, hospitals, offices, schools and other public buildings. Terrazzo is a specially prepared concrete surface containing cement (white or grey) and marble chips (of different colours), in proportion to $1:1\frac{1}{4}$ to 1:2. When the surface has set, the chips are exposed by grinding operation. Marble chips may vary from 3 mm to 6 mm size. Colour can be mixed to white cement to set desired tint. The flooring is, however, more expensive.

Timber flooring

Timber flooring is used for carpentry halls, dancing halls, auditoriums, etc. They are not commonly used in residential buildings in India, because timber flooring is also quite costlier. However, in hilly areas, where timber is cheaply and readily available, and where temperature drop very low, timber flooring is quite common. One of the major problems in timber flooring is the damp prevention. This can be done by introducing D.P.C. layer below the flooring.

Roofs: Glossary of terms, Types of roofs, concept and function of flat, pitched, hipped and Sloped roofs

A roof may be defined as the uppermost part of the building, provided as a structural covering, to protect the building from weather (i.e. from rain, sun, wind, etc.). Structurally, a roof is constructed in the same way as an upper floor, though the shape of its upper surface may be different. Basically, a roof consists of structural elements which support roof coverings. The structural element may be trusses, portals, beams, slabs (with or without beams), shells or domes. The roof coverings may be A.C. sheets, G.I. sheets, wooden shingle, tiles, slates or slab itself.

Types of roofs

Roofs may be divided into three categories:

1. Pitched or sloping roofs,
2. Flat roofs or terraced roofs, and
3. Curved roofs.

The selection of the type of roof depends upon the shape or plan of the building, climatic conditions of the area and type of constructional materials available. Pitched roofs have sloping top surface. These are suitable in those areas where rainfall/snowfall is very heavy. Flat roofs are considered suitable for buildings in plains or in hot regions, where rainfall is moderate, and where snowfall is not there. Flat roofs are equally applicable to buildings of any shape and size. Curved roofs have their top surface curved. Such roofs are provided to give architectural effects. Such roofs are more suitable for public buildings like libraries, theatres, recreation centres etc.

Pitched or Sloped roofs

A roof with sloping surface is known as a pitched roof. Pitched roofs are basically of the following forms:

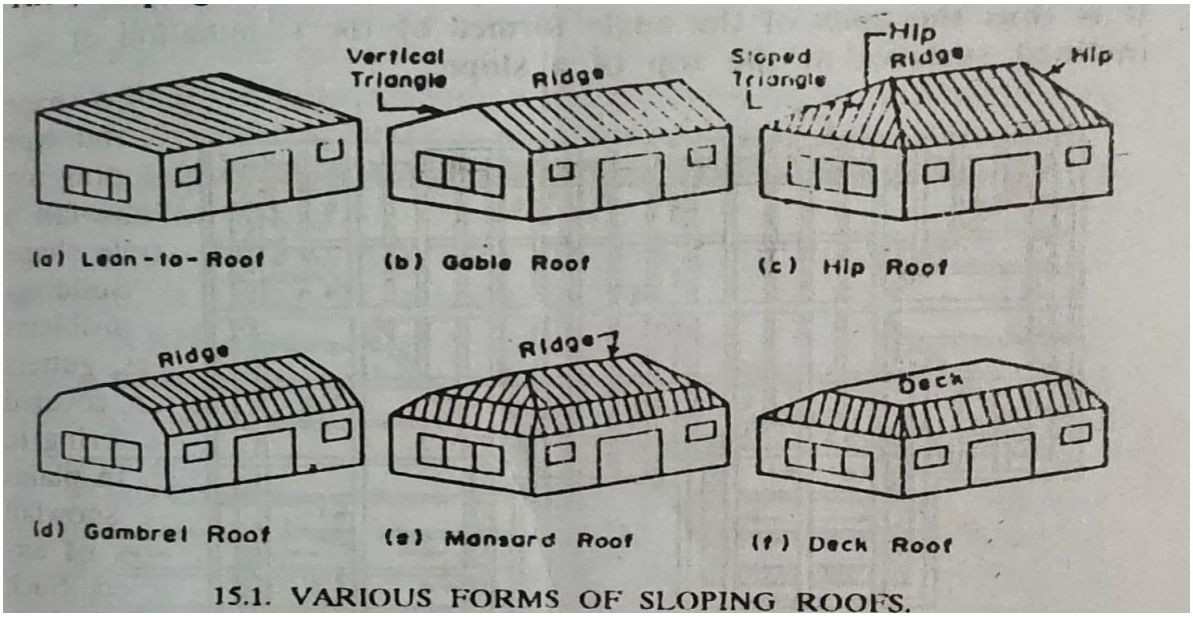
1. Lean-to-roof
2. Gable roof
3. Hip roof
4. Gambrel roof
5. Mansard or curb roof
6. Deck roof.

Lean-to-roof: This is the simplest type of sloping roof, provided either for a room of small span, or for the verandah. It has slope only one side (Fig. 15.1 a).

Gable roof: This is the common type of sloping roof which slopes in two directions. The two slopes meet at the ridge. At the end face, a vertical triangle if formed (Fig. 15.1 b).

Hip roof: This roof is formed by four sloping surfaces in four directions (Fig. 15.1c). At the end faces, sloped triangles are formed.

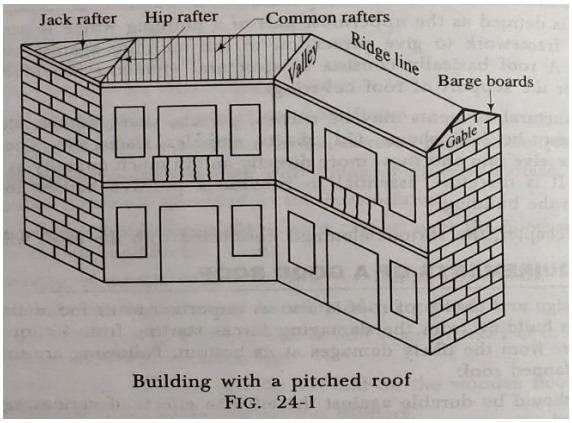
Gambrel roof: This roof, like gable roof, slopes in two directions, but there is a break in each slope, as shown in Fig. 15.1(d). At each end, vertical face is formed.
Mansard roof: Mansard roof, like a hip roof, slopes in the four directions, but each slope has a break, as shown in Fig. 15.1(e). Thus, sloping ends are obtained.
Deck roof: A deck roof has slopes in all the four directions, like a hip roof, but a deck or plane surface is formed at the top, as shown in Fig. 15.1 (f).



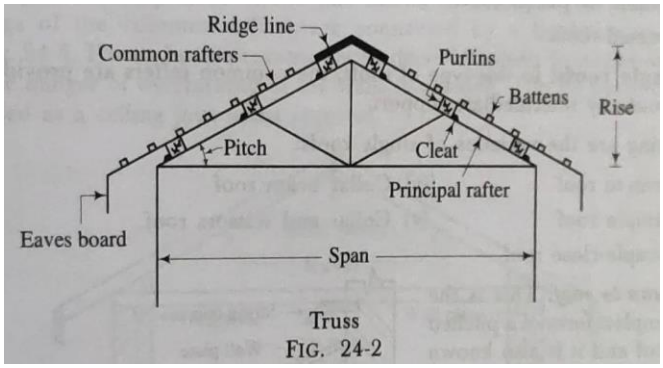
15.1. VARIOUS FORMS OF SLOPING ROOFS.

Glossary of terms

Barge boards: These are the wooden planks or boards which are fixed on the gable end of a roof as shown in fig. 24-1. They connect the ends of ridge, purlins and wall-plates.
Battens: These are the thin strips of wood which are fixed on the rafters or ceiling as shown in fig. 24-2. They support the roof ceiling.
Cleats: These are small blocks of wood which are fixed on the trusses to prevent the sliding of purlins as shown in fig. 24-2.
Dragon beam: The lower end of a hip rafter is generally supported on a diagonal piece of wood which is laid across the corner of the wall. This diagonal piece is known as a dragon beam or a dragon tie or an angle tie.
Eaves: The lower edges of a roof which are resting upon or projecting beyond the supporting walls are known as the eaves. Sometimes a thin board of wood is provided at the eaves to cover the ends of the common rafters. Such a board is known as an eaves board or facia board as shown in fig. 24-2. It is usually 25 mm thick and 250 mm wide.
Gable: The triangular upper part of a wall formed at the end of a pitched roof is known as a gable as shown in fig. 24-1.
Hip: The angle formed at the intersection of two roof slopes is known as the hip.
Pitch: The inclination of sides of a roof to the horizontal plane is known as the pitch and it can be expressed either in terms of degrees or as a ratio of rise to span.
Purlins: The wooden pieces which are placed horizontally on principal rafters to carry the common rafters are known as the purlins as shown in fig. 24-2.



Building with a pitched roof
FIG. 24-1



Truss
FIG. 24-2

Rafters: These are the pieces of timber which extend from the eaves to the ridge. Following are the various types of rafters:

i. **Common rafters**: These are the intermediate rafters which give support to the roof covering as shown in fig. 24-1 and fig. 24-2.
ii. **Hip rafters**: These are the rafters which are provided at the junction of two roof slopes as shown in fig. 24-1.
iii. **Jack rafters**: Any rafter which is shorter than a common rafter is known as a jack rafter as shown in fig. 24-1.
iv. **Principal rafters**: These are the inclined members of a truss as shown in fig. 24-2.

Ridge: A wooden piece provided at the ridge line of a sloping roof is known as the ridge or ridge board or ridge piece.

The common rafters are attached to the ridge. A ridge line indicates the apex line of a sloping roof as shown in fig. 24-1 and fig. 24-2.

Span: The horizontal distance between the internal faces of walls or supports is known as a span or a clear span as shown in fig. 24-2. The effective span indicates the horizontal distance between the centres of wall or supports.

Template: A bedding block is generally provided at the end of a truss. This block is known as a template and it helps in spreading the load over a large area. A template may be of wood or stone or R.C.C.

Truss: The framework, usually of triangles and designed to support the roof covering or ceiling over rooms, is known as a roof truss.

Verge: The edge of a gable, running between the eaves and ridge, is known as a verge.

Valley: When two roof surfaces meet together and form an internal angle, a valley is formed as shown in fig. 24-1.

Wall-plates: These are the long wooden members which are embedded on top of walls to receive the common rafters. They actually connect the walls to the roof.

Flat terraced roofs

Flat roof is the one which is either horizontal, or practically horizontal with slope less than 10°. Even a perfectly horizontal roof has to have some slope at top so that rain water can be drained off easily and rapidly. Similar to the upper floor, the flat roofs can be constructed of flag stones, R.S.J.

and flag stones, reinforced cement concrete, reinforced brick work, jack arch roof or precast cement concrete units. However, the flat roof differ from the upper floor only from the point of view of top finish, commonly called terracing, to protect it from adverse effects of rain, snow, heat etc.

Stairs: Glossary of terms; Stair case, winder, landing, stringer, newel, baluster, rise, tread, width of stair case, hand rail, nosing, head room, mumty room.

Stair case

A stair is a set of steps leading from one floor to the other. It is provided to afford the means of ascent and descent between various floors of a building. The room or enclosure of the building, in which the stair is located is known as stair-case.

Winder

This is a tapering step and it is used to change the direction of a flight. The winders radiate from a common centre.

Landing

The horizontal platform between two flights of a stair is known as the landing. A landing facilitates change of direction and provides an opportunity for taking rest during the use of a stair.

Stringer

These are the sloping members which support the steps in a stair. They run along the slop of the stair.

Newel post

This is the vertical member which is placed at the ends of flights to connect the ends of strings and handrails

Baluster

This is the vertical member which is fixed between string and handrail to give support to the handrail

Rise

This is the vertical distance between two successive treads.

Tread

The horizontal upper portion of a step is known as a tread.

Width of stair case

Staircase width refers to the side-to-side distance if you were walking up or down the stairs.

Hand rail

The inclined rail over the string is known as a handrail. The handrail serves as a guard rail and it should be provided at a convenient height so as to give grasp to the hand during ascent and descent.

Nosing

The projecting part of the tread beyond the face of riser is known as a nosing. It is usually rounded off from architectural considerations.

Head room

The vertical distance between the nosing of one flight and the bottom of flight immediately above is known as the headroom and it should be of sufficient value so as not to cause any difficulty to the persons using the stair.

Mumty room

A small room with only one purpose; to provide a roof over staircase.

Various types of stair case – straight flight, dog legged, open well, quarter turn, half turn (newel and geometrical stairs), bifurcated stair, spiral stair, cantilever stair, tread riser stair.

Stairs can be classified in two broad heads:

1. Straight stairs
2. Turning stairs
    i. Quarter turn stairs
    ii. Half turn stairs (dog-legged and open well stairs
    iii. Three-quarter turn stairs
    iv. Bifurcated stairs.

Each of the turning stairs are of three types:

a. Newel stairs
b. Well or open-newel stairs, and
c. Geometrical stairs

1. Straight stairs

In this type, the stair runs straight between the two floors. It is used for small houses where there are restrictions in available width. The stair may consist of either one single flight or more than one flight (usually two) with a landing,

2. Quarter turn stairs

A quarter turn stair is the one which changes its direction either to the left or to the right, the turn being affected either by introducing a quarter space landing or by providing winders.

Quarter turn stairs are of two types:

a. Newel quarter turn stairs
b. Geometrical quarter turn stairs.

Newel quarter turn stairs

These stairs have the conspicuous newel posts at the beginning and end of each flight. At the quarter turn, there may either be quarter space landing or there may be winders.

Geometrical quarter turn stairs

In geometrical stairs, the stringer as well as the hand rail is continuous, with no newel post at the landing.

3. Half turn stairs

Half turn stair is the one which has its direction reversed, or changed for 180°. Such stairs are quite common. These may be of three types :

a. Dog-legged or newel half turn stairs.
b. Open newel half turn stairs.
c. Geometrical half turn stairs.

Dog-legged stairs

This name is given because of its appearance in sectional elevation. The flights run in opposite directions and there is no space between them in plan. These stairs are useful where total width of space available for the staircase is equal to twice the width of steps.

Open newel half turn stair

Open well or open newel half turn stair has a space or well between the outer strings. This is the only aspect in which it differs from the dog-legged stair. The additional width is required between

the two flights; the space between the two strings may vary from 15 cm (min) to 100 cm. When the space left is more, a small flight containing two to four steps may be introduced at the turn, between the two quarter space landings. Otherwise, for small width well, a half space landing may be provided.

Geometrical half turn stairs

The essential features of such stairs are that the stringers and the hand rails are continuous, without any intervening newel post. These may be either with half-space landing or without landing.

4. Bifurcated stairs

This type of stair is commonly used in public building at their entrance hall. The stair has a wider flight at the bottom, which bifurcates into two narrower flights, one turning to the left and the other to the right, at the landing. It may be either of newel type with a newel or of geometrical type, with continuous stringer and rails.

5. Spiral stair

Such a stair is usually made either of R.C.C. or metal, and is employed at a location where there are limitations. These are also used as emergency stairs, and are provided at the back side of a building. All the steps are winders. The stair is, therefore, not comfortable.

6. Cantilever tread slab steps

In this type, the steps are formed by treads only, made of thick stone slabs, without any riser. The tread slab is fixed at one end into the wall, and acts as cantilever. The steps may either be rectangular or triangularly shaped.

7. Slab tread and riser steps

In this type, flag stone slabs are used as tread and risers. The stone slab risers and treads may be connected through dowels. The thickness of the stone slabs may vary from 5 cm to 8 cm.

Protective, Decorative Finishes, Damp and Termite Proofing

Plastering – purpose – Types of plastering, Types of plaster finishes – Grit finish, rough cast,smooth cast, sand faced, pebble dash, acoustic plastering and plain plaster etc. Proportion of mortars used for different plasters, preparation of mortars, techniques of plastering and curing.

Pointing – purpose –Types of pointing.

Painting – objectives – method of painting new and old wall surfaces, wood surface and metalsurfaces – powder coating and spray painting on metal surfaces.

White washing – Colour washing – Distempering – internal and external walls.

6.6.Damp and Termite proofing – Materials and Methods.

Plastering – purpose – Types of plastering, Types of plaster finishes – Grit finish, rough cast, smooth cast, sand faced, pebble dash, acoustic plastering and plain plaster etc.

Plastering

Plastering is the process of covering rough surfaces of walls, columns, ceilings and other building components with thin coat of plastic mortars to form a smooth durable surface. The coating of plastic material (i.e. mortar) is termed as plaster. Plastering on external exposed surfaces is known as rendering,

Purpose

Plastering is done to achieve the following objects:

1. To protect the external surfaces against penetration of rain water and other atmospheric agencies.
2. To give smooth surface in which dust and dirt cannot lodge.
3. To give decorative effect.
4. To conceal inferior materials or defective workmanship.

Types of plastering

There are different types of plasters available such as

- Lime mortar
- Cement mortar
- Lime cement mortar

Lime mortar

The lime mortar consists of equal volumes of lime and sand, and these two materials are carefully ground in a mortar mill. The fat lime is recommended for plaster work because the fat lime contains 75% of CaO and it combines with CO_2 of atmosphere and gives $CaCO_3$ quickly. Thus, the lime sets quickly, but it imparts low strength and hence, it can be used only for plaster work. The hydraulic lime contains particles which slake slowly as they come in contact with atmosphere. This results into blisters on the surface known as the blowing. The sand to be used for preparing lime mortar for plastering work should be clean, coarse and free from any organic impurities.

Cement mortar

The cement mortar consists of one part of cement to four parts of clean, coarse and angular river sand by volume. The materials are thoroughly mixed in dry condition before water is added to them. The mixing of materials is done on a watertight platform and mortar of one cement bag only is prepared at a time and this quantity of mortar is consumed within 30 minutes after adding water.

Lime cement mortar

Lime-cement mortar contains properties of both the lime mortar as well as cement mortar. Cement mortar as such does not possess sufficient plasticity. Addition of lime to it imparts plasticity, resulting in smooth plastered surface. Mix proportions generally used are 1:1:6 (cement : lime : sand), 1:1:8 or 1: 2: 8. Generally, fat lime is used.

Types of plaster finishes

Plastered surface may be finished in the following varieties:

1. **Smooth cast finish**. In this finish, smooth, levelled surface is obtained. The mortar for the finish may be made of cement and fine sand in the ratio of 1: 3. Mortar is applied with the help of wooden float. Steel floats are not recommended for external renderings since they give a very smooth finish which is liable to cracking and crazing under exposure to atmospheric conditions.
2. **Sand faced finish**. This is obtained by plastering in two coats. The first coat is applied in 1: 4 cement sand mortar for 12 mm thickness. It is provided with zig-zag lines. After curing it for 7 days, the second coat is applied in the thickness of 8 mm. The mortar for the second coat is prepared from cement sand mix ratio 1 : 1. The sand for this is perfectly screened so that uniform size is obtained. Sponge is used in the second coat when it is still wet. The surface of final coat is finished by rubbing clean and washed sand of uniform size by means of wooden float. This results in the surface having sand grains of equal and uniform density.
3. **Rough cast finish or spatter dash finish**. In this, the mortar for the final coat contains fine sand as well as coarse aggregate in the ratio of 1:1.5: 3 (cement : sand : aggregate). The coarse aggregate may vary from 3 mm to 12 mm in size. The mortar is dashed against the prepared plastered surface by means of large trowel. The surface is then roughly finished using a wooden float. Such a finish is water proof, durable and resistant to racking and crazing, and may be used for external renderings.
4. **Pebble dash or dry dash finish**. In this the final coat, having cement : sand mix proportion of 1 : 3 is applied in 12 mm thickness. Clean pebbles of size varying from 10 to 20 mm size are then dashed against the surface, so that they are held in position. The pebbles may be lightly pressed into the mortar, with the help of wooden float.
5. **Depeter finish**. This is similar to pebble dash finish in which the 12 mm coat is applied and while it is still wet, the pieces of gravel or flint are pressed with hand on the surface. Flints of different colours may be used to obtain beautiful patterns.
6. **Scrapped finish**. In this, the final coat of 6 to 12 mm thick is applied and after it has stiffened for few hours, the surface is scrapped in patterns for a depth of 3 mm. For scrapping, steel straight edge, old saw blade or such other tool may be used. Such scrapped surface is less liable to cracks.
7. **Textured finish**. This is used with stucco plastering. Ornamental patterns or textured surfaces are made on the final coat of stucco plastering, by working with suitable tools.
8. **Acoustic plaster**. This contains gypsum mixtures applied as final coat in finishing the plastered surface. Such a coat undergoes chemical reaction resulting in production of gas bubbles and consequent formation of tiny openings in the coat. These honey-combed minute openings absorb sound. Such plaster is useful in the interior walls of halls, auditoriums etc. The plaster is applied in two coats each of 6 mm thickness, using wooden float.

Proportion of mortars used for different plasters, preparation of mortars, techniques of plastering and curing.

Proportion of mortars used for different plasters

Mix Ratio of Mortar	General Usage Recommended
1:3	Very rich mortar mix. Not recommended for general usage at sites. Can act as a repair mortar with a waterproofing/bonding agent
1:4	For External Plaster and for ceiling plaster
1:5	Brickwork Mortar and for Internal plaster (If sand is not fine Fineness Modulas > 3)
1:6	For Internal Plaster (if fine sand is available)

Preparation of mortars

Mortar is mixed on-site in a mechanical mixer but can be mixed in smaller amounts by hand, using a hoe and a mixing tub or wheelbarrow.

1. Use a dry bucket to measure out the materials.
2. Pre-wet mortar containers before filling them with fresh mortar.
3. Prepare a container with a flat, solid surface base and tall sides for mixing mortar, if mixing by hand.
4. Add the masonry cement, lime, and sand in the appropriate amounts to your mixing container, then add water on top of the dry ingredients.
5. Fold the mortar mix from the bottom into the water, when mixing by hand. Keep mixing until the water is mixed in. Then, add more water and keep mixing. Keep adding water until the mortar attains a smooth consistency.
6. Stop mixing when the mortar is wet enough to slip easily off the shovel but holds its shape if you make a hollow in the mix. Mortar has attained the correct viscosity when you can make a few ledges in the mortar mix and ledges stand up.

Techniques of plastering and curing

Preparation of Surface for Plastering

1. Keep all the mortar joints of wall rough, so as to give a good bonding to hold plaster.
2. Roughen the entire wall to be plastered.
3. Clean all the joints and surfaces of the wall with a wire brush, there should be no oil or grease etc. left on wall surface.
4. If there exist any cavities or holes on the surface, then fill it in advance with appropriate material.
5. If the surface is smooth or the wall to be plastered is old one, then rake out the mortar joint to a depth of at least 12 mm to give a better bonding to the plaster.
6. Wash the mortar joints and entire wall to be plastered, and keep it wet for at least 6 hours before applying cement plaster.
7. If the projection on the wall surface is more than 12 mm, then knock it off, so as to obtain a uniform surface of wall. This will reduce the consumption of plaster.

Groundwork for Plaster

1. In order to get uniform thickness of plastering throughout the wall surface, first fix *dots* on the wall. A dot means patch of plaster of size 15 mm * 15 mm and having thickness of about 10 mm.

2. Dots are fixed on the wall first horizontally and then vertically at a distance of about 2 meters covering the entire wall surface.
3. Check the verticality of dots, one over the other, by means of plumb-bob.
4. After fixing dots, the vertical strips of plaster, known as *screeds*, are formed in between the dots. These screeds serve as the gauges for maintaining even thickness of plastering being applied.

Applying Under Coat or Base Coat

1. In case of brick masonry the thickness of first coat plaster is in general 12 mm and in case of concrete masonry this thickness varies from 9 to 15 mm.
2. The ratio of cement and sand for first coat plaster varies from 1:3 to 1:6.
3. Apply the first coat of plaster between the spaces formed by the screeds on the wall surface. This is done by means of trowel.
4. Level the surface by means of flat wooden floats and wooden straight edges.
5. After leveling, left the first coat to set but not to dry and then roughen it with a scratching tool to form a key to the second coat of plaster.

Curing of Plastering works

1. After completion of the plastering work, it is kept wet by sprinkling water for at least 7 days in order to develop strength and hardness.
2. Use of gunny bags or other materials is used to keep the plastering works wet in external works.
3. Improper curing may lead to cracks formation or efflorescence in plaster work.

Pointing – purpose –Types of pointing.

The term pointing is used to denote the finishing of mortar joints of either stone masonry or brick masonry. The joints are raked out to a depth of about 20 mm and then, these spaces are filled up by suitable mortar in the desired shape.

Following are the main objects for providing pointing and plastering to the exposed surfaces:

- To improve the appearance of the structure as a whole and to give smooth surface.
- To protect the exposed surfaces from the effects of atmospheric actions.
- To rectify the defective workmanship or to conceal inferior materials.

Types of pointing

Pointing is carried out in the following common shapes:

1. Flush pointing (Fig. 19.3 a)

This type of pointing is formed by pressing mortar in the raked joint and by finishing off flush with the edge of masonry units. The edges are neatly trimmed with trowel and straight edge. It does not give good appearance. However, the pointing is more durable since it does not provide any space for the accumulation of dust, water, etc. Due to this reason, flush pointing is extensively used.

2. Recessed pointing (Fig. 19.3 b)

The pointing is done by pressing the mortar back from the edges by 5 mm or more. The face of the pointing is kept vertical, by a suitable tool. The pointing gives very good appearance.

3. Rubbed, keyed or grooved pointing (Fig. 19.3 c)

This pointing is a modification of flush pointing by forming a groove at its mid height, by a pointing tool. It gives better appearance.

4. Beaded pointing (Fig. 19.3 d)

This is the special type of pointing formed by a steel or ironed with a concave edge. It gives good appearance, but is liable to damage easily.

5. Struck pointing (Fig. 19.3 e)

This is a modification of flush pointing in which the face of the pointing is kept inclined, with its upper edge pressed inside the face by 10 mm. This pointing drains water easily.

6. Tuck pointing (Fig. 19.3 S).

The pointing is formed by first pressing the mortar in the racked joint and finishing flush with the face. While the pressed mortar is green, groove or narrow channel, having 5 mm width and 3 mm depth is cut in the centre of the groove. This groove is then filled in or tucked in with white cement putty, kept projecting beyond the face of the joint by 3 mm. If projection is done in mortar, it is called Bastard pointing or half tuck pointing.

7. V-pointing (Fig. 19.3 g)

This pointing is formed by forming V-groove in the flush finishing face.

8. Weathered pointing (Fig. 19.3 h)

This pointing is made by making a projection in the form of V- shape.

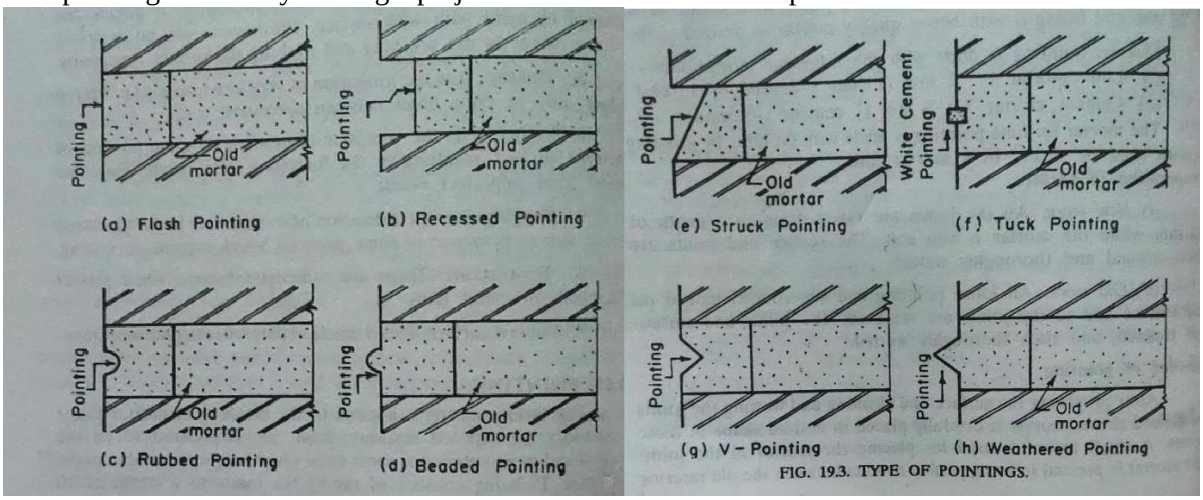


FIG. 19.3. TYPE OF POINTINGS.

Painting – objectives – method of painting new and old wall surfaces, wood surface and metal surfaces – powder coating and spray painting on metal surfaces.

Paints are liquid compositions of pigments and binders which when applied to the surface in thin coats dry to form a solid film to impart the surface a decorative finish, apart from giving protection to the base material (i.e., concrete, masonry and plaster surfaces) from weathering, corrosion and other chemical and biological attacks.

Following are the objects of painting a surface:

- It protects the surface from weathering effects of the atmosphere and actions by other liquids, fumes and gases.
- It prevents decay of wood and corrosion in metal.
- It is used to give good appearance to the surface. The decorative effects may be created by painting and the surface becomes hygienically good, clean, colourful and attractive.
- It provides a smooth surface for easy cleaning.

Method of painting

The process of painting depends on the nature of the surface to be painted. A brief description of painting on each of the various surfaces is given below:

New woodwork: Normally four coats of paint are required for new woodwork. The process of painting is carried out as follows:

- The surface of woodwork is prepared to receive the paint. For satisfactory working, it is necessary that the woodwork is sufficiently seasoned and it does not contain more than 15 per cent moisture at the time of painting. The surface of woodwork is thoroughly cleaned and the heads of nails are punched to a depth of 3 mm below the surface.
- The surface of the woodwork is then knotted.
- The priming coat is then applied on the surface of new woodwork. Generally, the priming coat is applied before the woodwork is placed in position.
- The process of stopping is then carried out.
- The subsequent coats of paint, namely, under coats and finishing coats, are then applied on the surface. The extreme care should be taken to see that the finishing coat presents smooth and even surface and that no brush marks are seen on the finished work.

Repainting old woodwork: If the paint on the old woodwork has cracked or has developed blisters, it is to be removed. If the surface has become greasy, it should be cleaned by rubbing down sand-paper or fine pumice stone. The old paint can also be removed by applying any one of the following three paint solvents:

- A solution containing 2 N of caustic soda to a litre of water is prepared and used to wash the surface. The paint dissolves and the surface becomes clean.
- A mixture consisting of one part of soft soap and two parts of potash is prepared and one part of quicklime is then added afterwards. This mixture is applied on the surface in a hot state and allowed to stay for about 24 hours. The surface is then washed with hot water.
- A mixture consisting of equal parts of washing soda and quicklime is brought to a paste form by adding required quantity of water. It is applied on the surface and kept for about an hour. The surface is then washed with water.

After removing old paint from the surface, the woodwork is painted as in case of painting on new woodwork.

Metals: The surface of the metal to be painted should be clean and free from dirt, grease, etc. It should be such as to provide key for the paint. Depending upon the nature of metal, suitable paint is selected. For instance, the priming coat for aluminium surface should be of zinc chromate and that for zinc surface, it should be of zinc oxide.

Plastered surfaces: For successful application of paint on cement plastered surfaces, the following factors should be carefully considered:

- There is considerable amount of moisture in newly constructed wall with cement plaster. It is necessary to allow the moisture content to escape. The actual time required for drying of wall surface will depend on atmospheric conditions and ventilation. But it usually requires about 3 to 6 months for the surface to be ready to receive paint. The artificial heating may in some cases be employed in combination with ventilation.
- The cement plaster is alkaline in nature because lime is liberated during the hydration of cement. Hence the oil based paints and distempers are liable for alkali attack, especially in presence of moisture. Hence the coats of alkali resistant primer paints should be applied or paints not containing any oil should be used.
- The defects in cement plastered surfaces develop due to various reasons. Such defects should be removed and the surface should be prepared to receive the paint.

- The spots showing efflorescence should be brushed off and the surface should be watched for few days. If spots of efflorescence develop again, the process of brushing off should be repeated. It is desirable to postpone painting till such spots cease to appear.
- If there are chances for the discolouration of the painted surface, it is necessary to clean the surface and to sterilise it with an antiseptic wash. The organisms responsible for discolouration are algae, mould, lichen, etc. They may be black, green purple or red in colour. The conditions favourable for their easy and free growth are dampness and high humidity.

With the above remarks in view, the usual paints recommended for cement plastered surfaces are cement paints, emulsion paints, oil paints, silicate paints, etc.

Powder coating and spray painting on metal surfaces

Powder coating uses an oven-cured dry powder to provide a thick, highly-durable coating for metal and other high-temperature resistant objects such as a fence, appliance face or deck furniture. The parts require disassembly and shipping because it is done offsite at a specialty shop.

Spray Painting uses a wet finish sprayed or electrostatically applied in a thin coat for both corrosion protection and appearance. Usually done on-site, it requires masking for overspray and curing times of up to 24 hours.

White washing – Colour washing – Distempering – internal and external walls.

Whitewashing

The fresh lime is slaked at site of work and mixed thoroughly with sufficient quantity of water in a tub. It is then screened through a clean cloth. The clean gum dissolved in hot water is then added at the rate of 20 N per m3 of lime. The rice may be used in place of gum.

The surface to be whitewashed should be cleaned before the work is started. For whitewashing walls which are whitewashed before, the old loose whitewash is to be first removed and repairing to the plaster is carried out, if necessary.

The whitewash is applied with jute brush and the brush is so worked that a surface with uniform colour is obtained. The three coats are generally applied, each after the previous coat has completely dried.

The lime is toxic for germs. It reflects light and thus it increases the brightness of the surface. The whitewashing therefore is extensively used for interior wall surfaces and ceilings of houses.

The process of whitewashing is sometimes used for exterior wall surfaces also. A satisfactory work gives an opaque smooth surface with uniform white colour and does not readily come off on the hand, when rubbed.

Colourwashing

This is prepared by adding the colouring pigment to the screened whitewash. It should be seen that the colouring pigment is not affected by the presence of lime. Ordinarily, the yellow earth is popular for colourwashing. Generally, the walls are colourwashed and ceilings are whitewashed. The mixture is to be kept constantly stirred during use.

The colourwash is applied in the same fashion as the whitewash. A satisfactory work does not give out powder when the finished surface is rubbed with the fingers.

The process of colourwashing imparts cleanliness and pleasant appearance of the surfaces which are treated.

Distempering- internal and external walls

The main object of applying distemper to the plastered surfaces is to create a smooth surface. The distempers are available in the market under different trade names. They are cheaper than paints and varnishes and they present a neat appearance. They are available in a variety of colours.

The application of distemper is carried out in the following way:

Preparation of surface: The surface to receive the distemper is thoroughly rubbed and cleaned. The important facts to be kept in mind are:

- The new plastered surfaces should be kept exposed for a period of two months or so to dry out before distemper is applied on them. The presence of dampness on the surface results in failure of distemper coating.
- The surface to receive distemper should be free from any efflorescence patches. These are to be wiped out by clean cloth.
- The irregularities such as cracks, holes, etc. of the surface are to be filled by lime putty or gypsum and allowed to become hard before distemper is applied on the surface.
- If distemper is to be applied on the existing distempered surfaces, the old distemper should be removed by profuse watering

Priming coat: After preparing the surface to receive the coats of distemper, a priming coat is applied and it is allowed to become dry. For ready made distempers, the priming coat should be composed of materials as recommended by the makers of distempers. For local made distempers, the milk is used for priming coat. One litre of milk will cover about 10 m^2 of the surface.

Coats of distemper: The first coat of distemper is then applied on the surface. It should be of a light tint and applied with great care. The second coat of distemper is applied after the first coat has dried and become hard. Following facts are to be remembered:

- The distempering should be done in dry weather to achieve better results.
- The oil-bound distemper or washable distemper adheres well to oil painted walls, wood, corrugated iron, etc. But a priming coat of pure milk should be applied before distempering is done on such surfaces.
- The application of distemper by a spraying pistol is superior to that by brushes. The spraying affords smooth and durable film of distemper.

Damp and Termite proofing – Materials and Methods.

Damp proofing

One of the basic requirement of a building is that it should remain dry or free from moisture travelling through walls, roofs or floors. Dampness is the presence of hygroscopic or gravitational moisture. Dampness gives rise to unhygienic conditions, apart from reduction in strength of structural components of the building. Damp prevention is therefore one of the important items of building design. Every building should be damp proof. Provision of damp proof courses prevent the entry of moisture in the building.

Materials

Following are the materials which are commonly used for the damp-proofing:

1. Hot bitumen
2. Mastic asphalt
3. Bituminous felts

4. Metal sheets
5. Combination of sheets and felts
6. Stones
7. Bricks
8. Mortar
9. Cement concrete
10. Plastic sheets

Methods

Following methods are adopted to make a building damp proof :

1. Use of damp proofing course (D.P.C.) : membrane damp proofing
2. Integral damp proofing.
3. Surface treatment.
4. Cavity wall construction.
5. Guniting.
6. Pressure grouting.

Termite proofing

Termites, popularly known as white ants cause considerable damage to wood work, furnishings etc. of buildings. In some countries the loss caused due to termites is estimated to be as high as 10% of the capital outlay of the buildings. Anti-termite treatment is, therefore, necessary so that damages are either reduced or stopped all together.

Methods

The methods of termite proofing can broadly be classified into the following two groups:

1. Soil treatment with chemicals
2. Structural barriers.

Soil treatment with chemicals: The soil insecticides are thoroughly mixed and well spread in soil so as to provide an effective chemical barrier for termites. The various patented chemical insecticides such as DDT, BHC, PCP, etc. are available.

Structural barriers

To prevent the entry of termites through walls, the impenetrable physical structural barriers may be provided continuously at plinth level. Such structural barrier may be in the form of a cement concrete layer or metal layer at plinth level.

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ESTIMATION & COST EVALUTION-I

SPINTRONIC TECHNOLOGY & ADVANCE RESEARCH (STAR)

DEPARTMENT OF CIVIL ENGINEERING

3RD SEMESTER

DIPLOMA

LECTURE NOTES ON

ESTIMATION & COST EVALUTION-I

BY

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CHAPTER - 1

INTRODUCTION

Estimate :-

An estimate is the probable cost of work and usually preparced before the construction is taken up.

Data required for Estimate :-

The following 3 types of data are required for estimate.

1. Drawing (plan, elevation, section)
2. Specification (it includes nature, quality, and type of work, material etc)
3. Rate (The reate per unit various items of work, labour etc.)

Types of Estimate :-

The different types of estimates are:

1. Prelimnary estimate / Aproximate estimate
2. Plinth area estimate
3. Cube reate estimate
4. Approximate quantity menthod estimate
5. Detailed estimate.
6. Revised estimate
7. Supplimentary ~~and revised estimate~~.
8. Supplimentary and revised estimate
9. Annual or Repair / Maintenance estimate.

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1. Prelimnarcy estimate / Aprcoximate estimate.

- It is reqdireed for prelimnarcy studies of varicus item of work or project to decide the financial posltion and policy for administrative section by the competent authority.

- In case of commercial projects such as erricgation project or similarc projects which earn revenue income, then it may be seen wheatherc investment is justified or not.

2. Plinth arcea estimate :-

- This is preparced on the basis of plinth arcea of building. The rate being deduced from the cost of similarc building having similarc specification, height and construction in the locality.

- Plinth arcea ~~and~~ estimate is calculated by taking by finding the plinth arcea of the building and multiplying by the plinth arcea rate.

- The plinth arcea should be calculated for the covered arcea by taking external dimensions of the building at the floor level. courtyards and otherc open arcecs should not be included in the plinth arcea estimate.

3. Cube rate Estimate :-

- Cube reate estimate is a preliminarcy estimate or an approximate estimate and is prepared on the basis of cubical content of a similarc building having similarc specification and construction in the locality.

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- This is calculated by finding the cubical content of the building and multiplying it with the cube rate. The length and breath should be taken as the external dimension of the building at the floor level and the height should be taken from the floor level of one storey to the top of next heigher floor.

- The foundation, plinth and the prepare above roof are not taken into consideration in finding the cubical content.

4. Approximate quantity method estimate:

- In this method approximated total length of wall is found in running meter and this total length multiplied by the rate per running meter wall gives fairly accurate cost.

- For this method the structure is devided two parts.
  1. Foundation encluding plinth.
  2. super structure.

- The reunning meter cost for foundation and superstructure should be calculated first and these reunning meter should be multiplied by the total length of wall.

- Similarly for superstructure the prices or rate per running meter is determined from the approximate quantities of bricks, wood works, roof, floor, finishing etc.

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5. Detailed estimate :-

- Detailed estimate is an accurcate estimate and consicts of working out the quantities of each item of works and working the cost.
- The dimensions such as length, breath and height of each item aree calculated and abstreating and billing aree done.
- The detailed estimate is preparced in two steages.

(1) Details OF measurment and calculation of quantities.

- The details of measurment of each item of work are taken out correctly from the quantites under each items was computed ena tabulare form.

(2) Abstract of Estimate cost.

- The totel ~~of~~ cost of a item es calculated from the analysis of reates ~~of~~ ore shedule of reate chart.

6. Revised Estimate :-

Revised estimate is the detailed estimate and it required to be prepared under any one of the following circumstances.

*. when the original sanction estimate es executed ore likely to execeted by more than 5%.

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* when the expenditure on a work exceeded or likely to exceed the amount of administrative sanction by more than 10%.

* when there are material evacuation from the original material even through the cost may be meet the original sanction amount.

7. Supplementary Estimate :-

* Supplementary is the detailed estimate and it prepared when additional work are required to supplement the original work or when further development is required during the progress of work.

* The abstract should show the amount of original estimat and the amount including the supplementary amount for which sanction is required.

8. Supplementary and Revised Estimate :-

If at any time either before or during the execution of original works it is found that original estimate is excess in then the divicional officers may saction a revised estimate of reduced amount

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* while giving such sanction the accountant general and othere heighere authority arce in fream,

* It is preeparced when the work is parctically abandon and the estimated cost of reemaining work is less than is less than 95% of oreiginal work.

Annual Repair or Maintenance Estimate:-

* Annual reepaire ore maintenance estimate is a detailed estimate and its preeparced to maintain the structure in preeparced ordere and save condition.

* Fore building this encludes white washing, colour washing, painting, minore reepairs etc.

* Furethere there may be speeical estimate such as moonsoon damage reepaire estimate etc.

Estimate OF Building:

A building is estimated by two stages.

1. Meesurement of quantity
2. Calculation of cost.

1. Measurement of Quantity:

- The whole work is devided into different items of work such as earth work, concreet brick work etc.

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- The items are classified and groups under different sub heads.
- Details of measurment of each item of work are taken out and quantities under each item are computed in a prescript form known as details of measurment form.

Details of Measurment :

Item no	Particulars of item	No	Length (L)	Breath (B)	Height (H) in metre	Quantity in m^3 (Q)	Explanary Note.

2. Calculation of cost :-

- The cost under each item of work is calculated from the quantities already computed at workable reate and the total cost is work down in a prescript from known as abstract of estimate form.
- A percentage of 3 to 5 % is added for contingency to allow petty expenditerey and unforeowning expenditere.
- The ground total

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Abstreact OF Estimate Form.:

Item no	Particulars of item	Quantity	unit	Rate	Amount

Type off Areea :-

There are 3 types of areea in a building

1. Plinth areea
2. Floore areea
3. carepet areea

Plinth areea :- It is the built up coverced areea of a building measured at floore of any storey.

- It is calculated by taking external diamensions of building at the floore level excluding plinth.

- The following shall be encluded in plinth areea.
  1. All floore areea of walls at the floore level.
  2. Interenal shaffts for sanitarey installation whose areea don't exceed 2 m^2.
  3. The areea of bare Sati and areea of munty roof
  4. Areea of pureches other than cantely cource.

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— The following shall not be included in plinth Area:

① Area of loft

② Internal shaft for sanitary installation whose exceed ~~1000~~ 2m2.

③ on covered balconys.

Floor Area: It is the area of building that is the area of floor in between walls and consists up floor of all room, varandah, passage, staircase room, interance, bathroom, ketecheh, storee room etc.

*. In short floor area = Plinth area - Area of walls

* The floor of each storey is and different types of floors are measured and taken separately

* The floor area of basements, baresaties, mumties and purches etc. should be measured separatly.

Carpet Area: Carepet area of a building is the useful area to livable area.

* This the total floor area - Area varandah, corredoor, passage, staircase and entrance hall.

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*. For officeal building carpet is the rentable areea or useful areea. For reeidensical building carpet areea is the livable areea and should be used for living purepose.

*. The carepet areea of an office building is up to 15%. of the plinth areea and for recidensical building it is 50 to 65 % of the plinth areea.

Units of Measurement for various items of work :-

The units of different works depends upon theire size, nature and sape. In generally the units of different items of work are based on the following preinciple.

(1) Mass, voluminous and thick work shall be taken in qubic unit or volum.

(2) Shallow, thin and surface work shall be taken in squeare unite per in areea.

(3) Piece work job work shall be teken in number.

(4) Long and thin work shall be taken in meter per running unit.

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SI NO	Particulars OF Items	Units
1	Eearth work excavation	M3
2.	Rock excavation	M3
3.	Eeath work in filling	M3
4.	Sand filling	M3
5.	Surface dressing	Sqm
6.	Cutting of trees	nos
7.	Quarrying of stone or boulder	M3
8.	Lime concrete in roof terracing thickness specified	Sqm
9.	Lime concrete in foundation	M3
10.	cement concrete	M3
11.	Reinforced cement concreete	M3
12.	Jail work	Sqm
13.	Damp proof course	Sqm
14.	Brick work in foundation in plinth and superstructure	M3
15.	sun dried brick work	Cu.m
16.	Honey comb brick work	Sqm
17.	Half - brick work	Sqm
18.	Thin partition wall	Sqm

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19.	— Reinforced brick work	Cum
20.	string course, weather course, coping (projection specified)	meter
21.	Brick edging	Meter
22.	— stone masonry, rubble masonry	Cum
23.	stone work in wall facing	Sqm
24.	wood work in door and window framing	Cum
25.	Door and window shutters	Sqm
26.	Timbering of trenches	Sqm
27.	wood work in partition	Sqm
28.	steel reinforcement bard etc in R.C.C. R.B, work	Quintal
29.	Bending and binding of steel reinforcement	Quintal
30.	Barbed wire fencing	Meter
31.	Iron railing	Meter
32.	Iron hold fast	Quintal
33.	steel door and windows	Sqm
34.	Asbestos cement sheet	Sqm
35.	R.C.C / R.B roof slab	

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36.	Centereing and shuttering form work	3qm	
37.	~ Plastering	Sqm	
38.	Pointing	Sqm	
39.	white washing	Sqm	
40.	Distempering	Sqm	
41.	Painting, distempering	Sqm	
42.	Polishing of wood work	3qm	
43.	Cement concrete floor	Sqm	
44.	Bituminous road sufacing	Sqm	
45.	Dismantling of brick masonry	Cum	
46.	Supply of cement	Bag.	

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Quantity Estimate of Building

2.1 Estimate Methods :-

There are 2 types of estimate methods.

1. Long wall - short wall method.
2. Center line method.

Long Wall short Wall Method :-

- In this method the centers to center length of long wall and short wall is calculated separately
- The long wall is always calculated from out to out.
- The short wall is always calculated from in to in.
- Length of long wall = centers to center length + $\left(2 \times \frac{\text{width of item}}{2}\right)$
- Length of short wall = Centers to center length - $\left(2 \times \frac{\text{width of item}}{2}\right)$

CENTER LINE METHOD :-

- In this method the centers to centers length of long wall and short wall is calculated separately.
- This method is used to calculated the quantities rapidly.
- + special deduction is applied at junction on this method.
- The point at which more than 2 walls meet is called as a joint.

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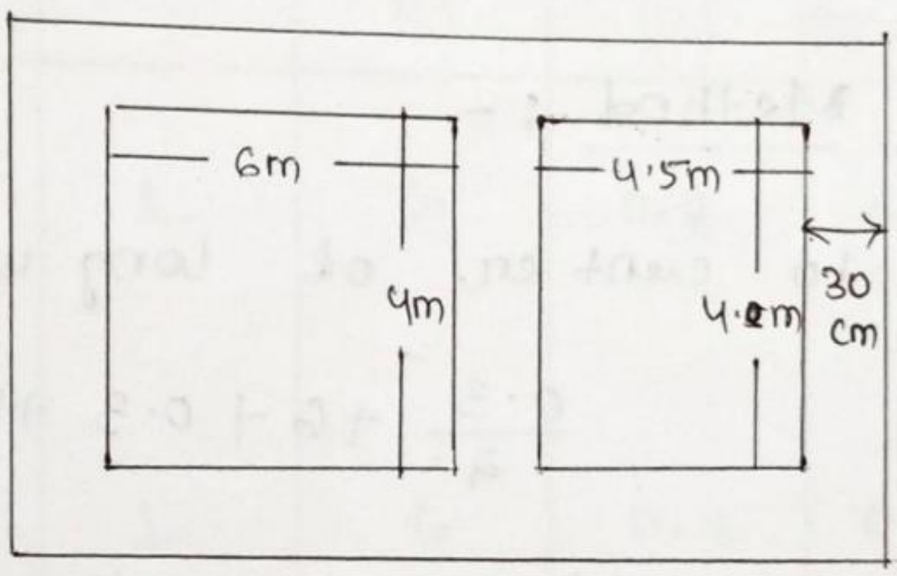
At joints Total length of walls = sum of centre to centre length of all walls – (no of joints × $\frac{\text{width of item}}{2}$)

1. calculate the length of long wall, short wall centere to centere length of all walls of the figure given below?



30cm = 0.3m

* long wall short wall Method

there no of long wall = 2

no of short wall = 3

centere to centere of long wall = $\frac{0.3}{2} + 6 + 0.3 + 4.5 + \frac{0.3}{2}$

= 11.1

Centere to centere of short wall =

$\frac{0.3}{2} + 4 + \frac{0.3}{2}$

= 4.3

Length of long wall = center to center length + $\frac{2 \times \text{width of item}}{2}$

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$$\rightarrow \quad 11.1 + 2 \times \frac{0.3}{2}$$

$$\rightarrow \quad 11.4\,m$$

Length of short wall = center to center length − $2 \times \frac{\text{width of item}}{2}$

$$\Rightarrow \quad 4.3 - 2 \times \frac{0.3}{2}$$

$$\Rightarrow \quad 4m$$

<u>center line Method</u> :-

Center to center of long wall =

$$\frac{0.3}{2} + 6 + 0.3 + 4.5 + \frac{0.3}{2} = 11.1$$

Center to center of short wall = $\frac{0.3}{2} + 4 + \frac{0.3}{2}$

$$= 4.3$$

Here no of function or joints = 2

total length of center line =

Sum of center to center length of all walls − no of joint $\times \frac{\text{width of item}}{2}$

$$\rightarrow \quad 2(11.1) + 3(4.3) - 2 \times \frac{0.3}{2}$$

$$\Rightarrow \quad 34.8$$

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Ex-1

ESTIMATING OF WALL

Plan and Section

Fig. 2-1

WALL WITH STANDARD MODULAR BRICKS.

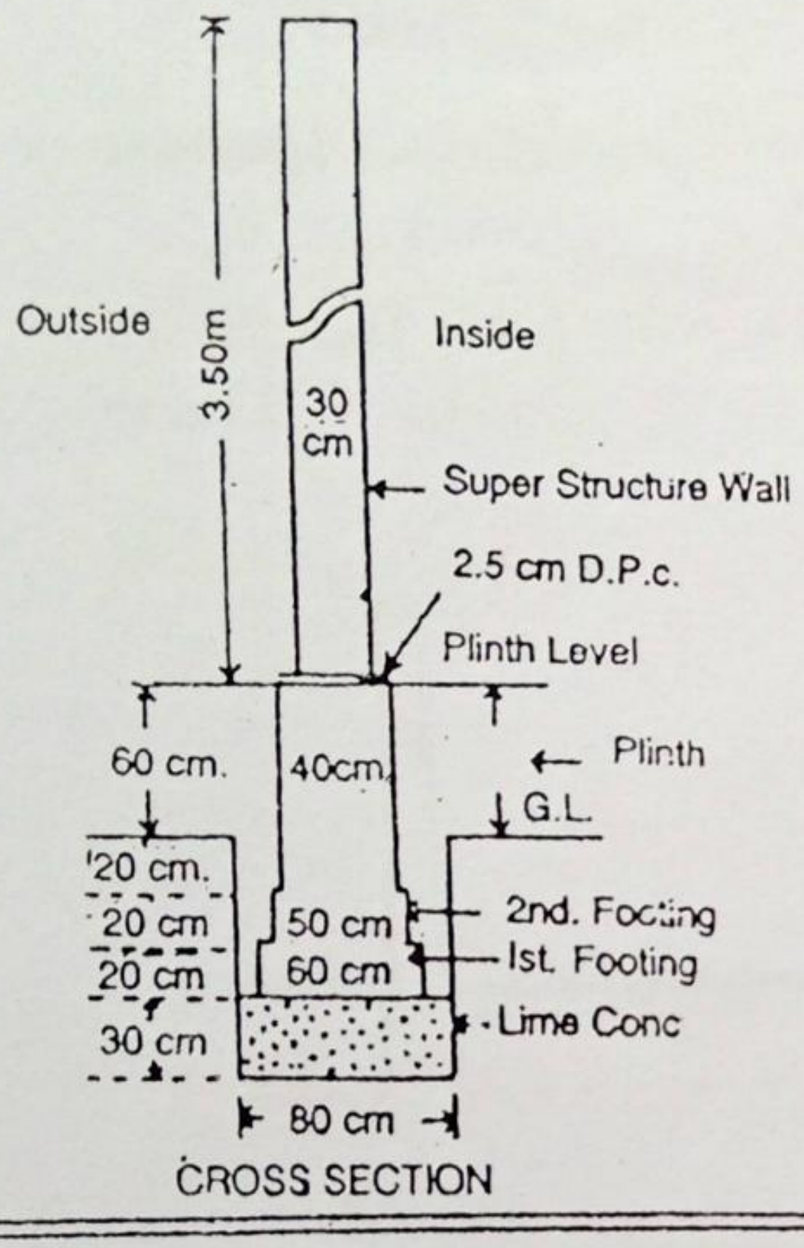
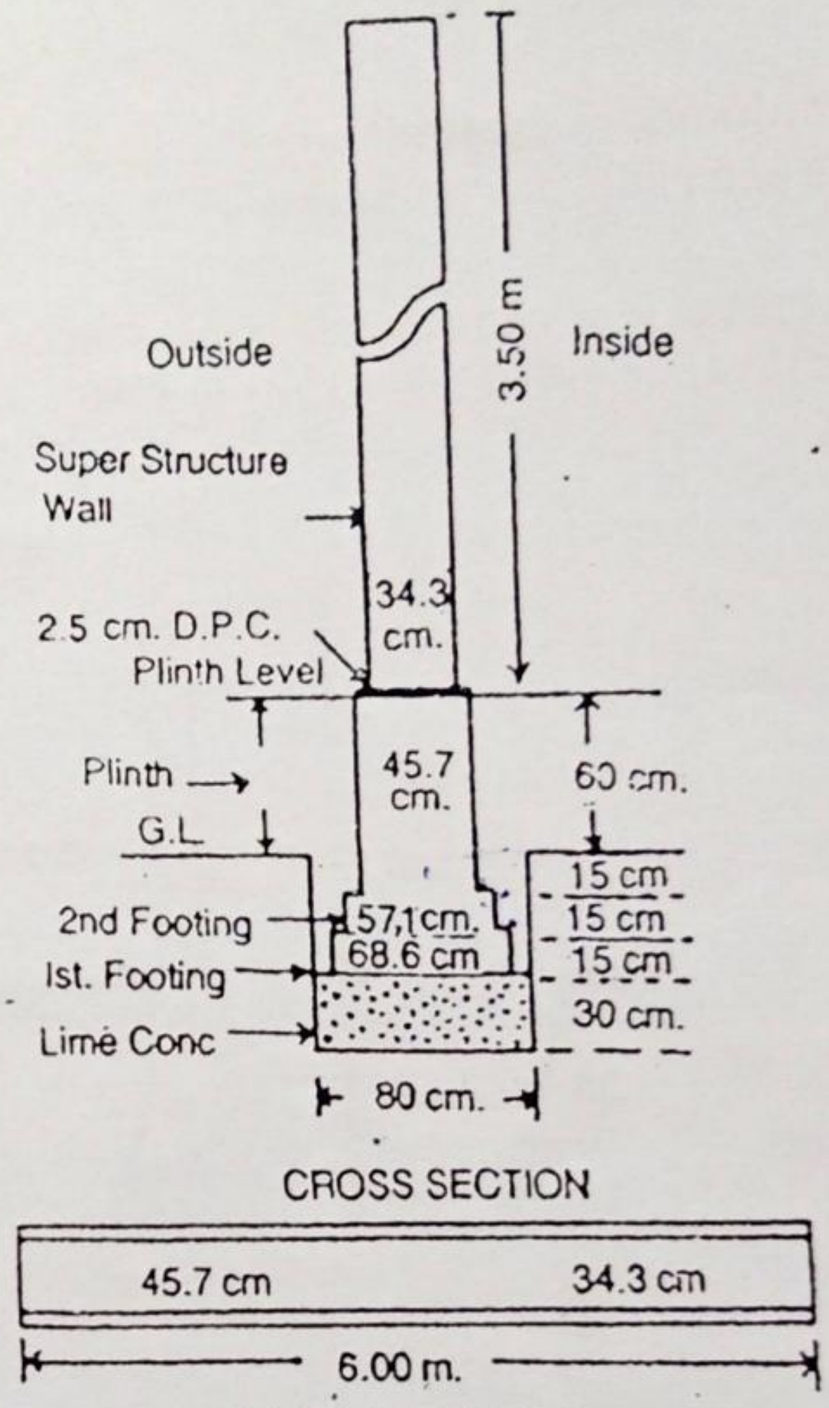


Fig. 2-2

WALL WITH TRADITIONAL BRICKS.



With Standard Bricks, Fig. 2-1 —

DETAILS OF MEASUREMENT AND CALCULATION OF QUANTITIES (Ex. 2)

Item No.	Description of items of work	No.	Dimensions			Quantities or Contents	Total quantities
			Length	Breadth	Ht. or Depth		
1.	Earthwork in excavation in foundation ...	1	6.00 m	.80 m	90 m	4.32	4.32 cu m
2.	Lime concrete in foundation ..,	1	6.00 m	.80 m	.30 m	1.44	1.44 cu m
3.	1st class brickwork in lime mortar in foundation and plinth —						
	1st footing ...	1	6.00 m	.60 m	.20 m	.72	
	2nd footing ...	1	6.00 m	.50 m	.20 m	.60	
	Plinth wall up to G.L. ...	1	6.00 m	.40 m	.20 m	.48	3.24 cu m
	Plinth wall above G.L. ...	1	6.00 m	.40 m	.60 m	1.44	

(Contd.)

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2. calculate the quantity of following items of the walls given in figure 1 and 2.

i) Eearth work estavation
ii) Cement concreet in foundation or lime concreet foundation.
iii) Brick work in foundation in super structure.

Figure - 2.1.

Item NO	Particulars of Item	NO	Length (L)	Breath (B)	Height (H) in meter	Quantity in m^3 (Q)	Explanatory Note.
						LxbxhxNO	
1.	Eearth work and Estavation	1	6	0.8	0.9	4.32 m^3	H = 0.3 + 0.2 + 0.2 + 0.2
2.	Lime concreet on foundation	1	6	0.8	0.3	1.44 m^3	
3.	Brick ~~and~~ work and Foundation in super structure						
	a. 1st footing	1	6	0.6	0.2	0.72	
	b. 2nd footing	1	6	0.5	0.2	0.6	
	c. 3rd footing	1	6	0.4	0.8	1.92	
4.	Brick work in super structure.	1	6	0.3	3.5	6.3	

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Figurce - 2.2

Item NO	Paerticeulars of Item	NO	Length (L)	Breeath (b)	Height (h) in (m)	Quantity in m^3 (Q)	Explanatorey Note.
1.	Earth work and Estavation	1	6	0.8	0.75	3.6 m^3	H = 0.15 + 0.15 + 0.15 + 0.3 = 0.75
2.	Lime concreet of foundation	1	6	0.8	0.3	1.44 m^3	
3.	Breick work and foundation in superc structure.						
a. 1st Footing		1	6	0.686	0.15	0.6174	
b. 2nd Footing		1	6	0.571	0.15	0.5139	
c. 3rd Footing		1	6	0.457	0.75	2.0565	
d. 4th Footing		1	6	0.343	3.5	7.203	

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The following examples (Exs. 3a, 4a, and 5a) illustrate this method :—

Example 3(a). — Fig. 2-3, the plan represents the plan of superstructure wall of a single room building of 5 m × 4 m, and Sections represent the cross-sections of the walls with foundation. Estimate the quantities of —

(1) Earthwork in excavation in foundation, (2) Concrete in foundation, (3) Brickwork in foundation and plinth and (4) Brickwork in superstructure.

The length of long wall centre to centre = 5.00 + ½ × .30 + ½ × .30 = 5.30 m. The length of short wall centre to centre = 4.00 + ½ × .30 + ½ × .30 = 4.30 m.

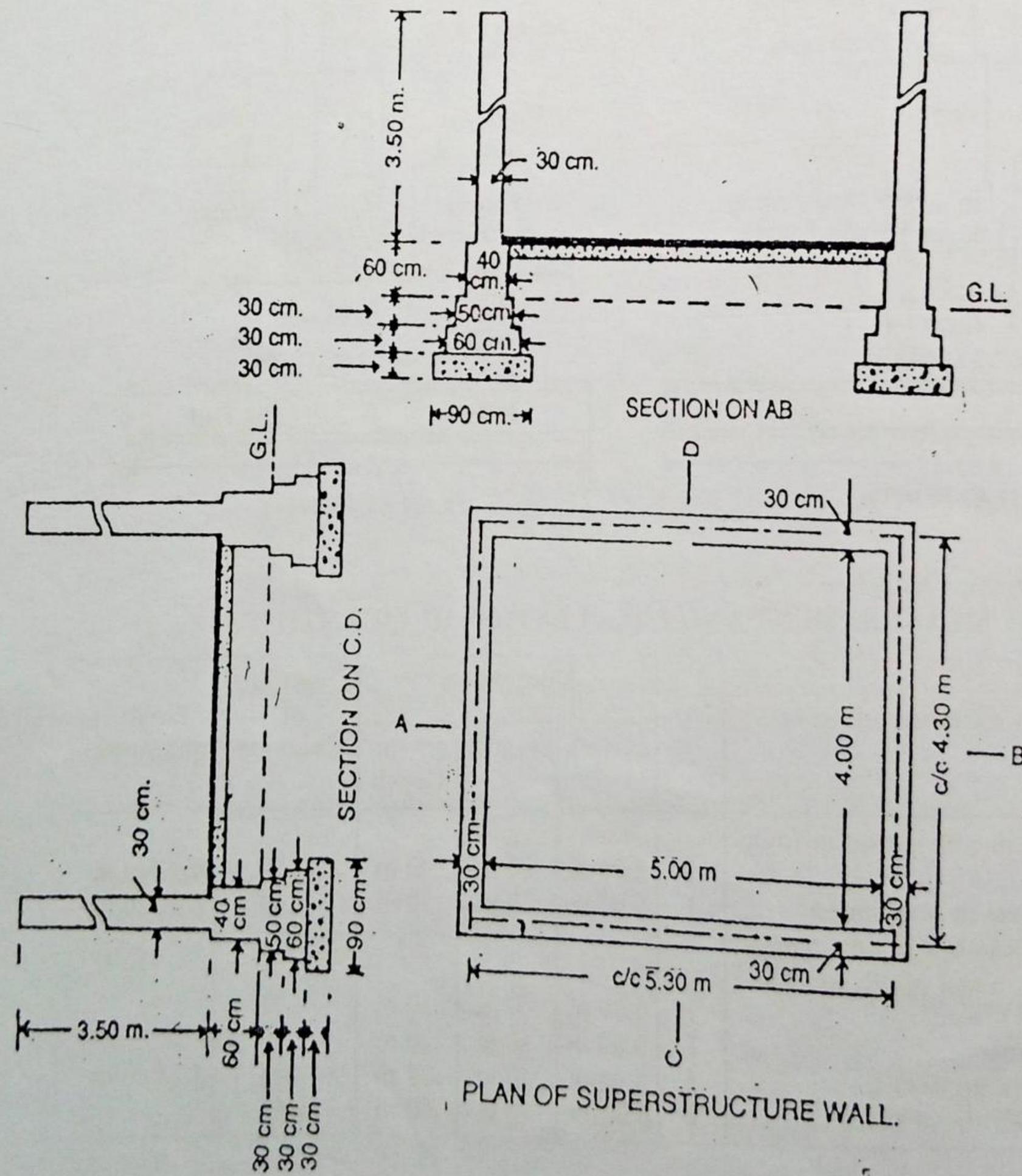

Fig. 2-3

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Ex-2(a)

calculate the quantity of following items from the given drawing.

1. Earth work in Estavation.
2. Cement concreet or lime concreet in foundation.
3. Brick work foundation in plinth
4. Brick work in super structure.

Ans

Centere to centere length of long wall ⇒

$$\frac{0.3}{2} + 5 + \frac{0.3}{2} = 5.3$$

Centere to center length of short wall ⇒

$$\frac{0.3}{2} + 4 + \frac{0.3}{2} = 4.3$$

No of long wall = 2

No of short wall = 2

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Item No	Particulars of item	No	Length (l)	Breadth (b)	Height (h) in (m)	Quantity in m3 (q)	Explanatory.
1.	Earth work estavation						
	a. Long wall	2	6.2	0.9	0.9	10.04	$L_L = 5.3 + \frac{0.9}{2} + \frac{0.9}{2} = 6.2$
	b. short wall	2	3.4	0.9	0.9	+ 5.50	$L_S = 4.3 - \frac{0.9}{2} - \frac{0.9}{2} = 3.4$
						15.54	
2.	Lime concret cement in Foundation						
	a. Long wall	2	6.2	0.9	0.3	3.34	$L_i = 5.3 + \frac{0.9}{2} + \frac{0.9}{2} = 6.2$
	b. Short wall	2	3.4	0.9	0.3	+ 1.83	$L_S = 4.3 - \frac{0.9}{2} - \frac{0.9}{2} = 3.4$
						5.17	
3.	Brick work Foundation and Plinth.						
	① First Footing						
	ⓐ long wall	2	5.9	0.6	0.3	2.12	$L = 5.3 + \frac{0.6}{2} + \frac{0.6}{2} = 5.9$
	ⓑ Short wall	2	3.7	0.6	0.3	+ 1.33	$L_S = 4.3 - \frac{0.6}{2} - \frac{0.6}{2} = 3.7$
						3.45	

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	② Second footing						
	ⓐ Long wall	2	5.8	0.5	0.3	1.74	$L = 5.3 + \frac{0.5}{2} + \frac{0.5}{2} = 5.8$
	ⓑ Short wall	2	3.8	0.5	0.3	1.14	$L_s = 4.3 - \frac{0.5}{2} - \frac{0.5}{2} = 3.8$
						2.88	
	③ Third footing						
	ⓐ Long wall	2	5.7	0.4	0.6	2.73	$L = 5.3 + \frac{0.4}{2} + \frac{0.4}{2} = 5.7$
	ⓑ Short wall	2	3.9	0.4	0.6	1.87	$L_s = 4.3 - \frac{0.4}{2} - \frac{0.4}{2} = 3.9$
						4.6	
4.	Brick work in Super structure						
	① Long wall	2	5.6	0.3	3.5	11.76	$L = 5.3 + \frac{0.3}{2} + \frac{0.3}{2} = 5.6$
	② Short wall	2	4	0.3	3.5	8.4	$L_s = 4.3 - \frac{0.3}{2} - \frac{0.3}{2} = 4$
						20.16	

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CENTERLINE METHOD :- Ex-(3a)

Item No	Particulars of Item	No	Length (l)	Breath (b)	Height (h)	Quantity in m^3	Explanatory
1	Earth work Excavation	1	19.2	0.9	0.9	15.552 m^3	L = 5.3 x 2 + 4.3 x 2 = 19.2
2.	Lime concrete in foundation	1	19.2	0.9	0.3	5.184 m^3	L = 5.3 x 2 + 4.3 x 2 = 19.2
3.	Brick work in foundation						
	(a) 1st Footing	1	19.2	0.6	0.3	3.456	L = 5.3 x 2 + 4.3 x 2 = 19.2
	(b) 2nd footing	1	19.2	0.5	0.3	2.88	
	(c) 3rd footing	1	19.2	0.4	0.6	4.608	
						10.944 m^3	
4.	Brick work in Super structure	1	19.2	0.3	3.5	20.16	L = 5.3 x 2 + 4.3 x 2 = 19.2

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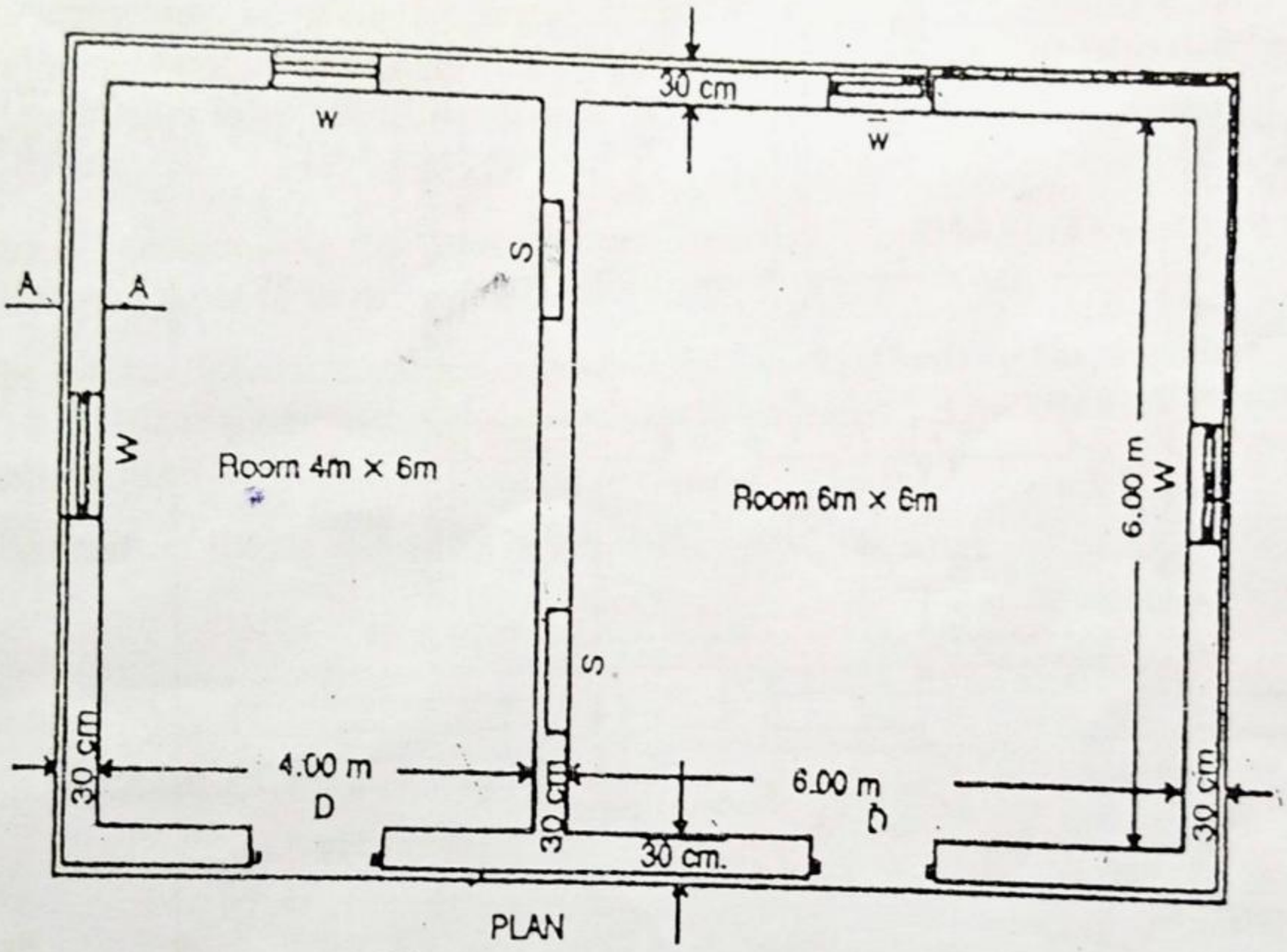
Example 4(a). — Estimate the quantities of the following items of a two roomed building from the given plan and section (Fig. 2-6) :—

(1) Earthwork in excavation in foundation, (2) Lime concrete in foundation, (3) 1st class brickwork in cement mortar 1 : 6 in foundation and plinth, (4) 2.5 cm c.c. damp proof course, and (5) 1st class brickwork in lime mortar in superstructure.

TWO ROOMED BUILDING



CROSS SECTION OF WALL ON AA

All Walls are of same section
Lintels over Doors, Windows and Shelves are 15 cm thick R.B.

Doors D-1.20 m × 2.10 m
Windows W-1.00 × 1.50 m
Shelves S-1.00 m × 1.50 m

Fig. 2-6

Note : — *No beam has been shown in the plan as the object of this example is to explain the method of estimating the walls only.*

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Ex - 4 (a)

calculate the quantity of following items of work from the given drawing.

① Eeareth work in Estavation
② Lime concret in foundation
③ Breick work in foundation in plinth
④ Damp proof course work
⑤ Brick work in super structure

Ans

Here no of long wall = 2
no of short wall = 3

centere to centere length of long wall =

$$\frac{0.3}{2} + 4 + 0.3 + 6 + \frac{0.3}{2} = 10.6$$

centere to centere short wall =

$$\frac{0.3}{2} + 6 + \frac{0.3}{2} = 6.3$$

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Item no	Particulars OF Item	NO	Length (l)	Breath (b)	Height (h)	Quantity	Explanatory Note in(M)
1.	Earth work in estavation.						H = 0.3+ 0.2+0.1+0.1+0.1 +0.2 = 1
	a. long wall	2	11.7	1.1	1	24.74	LL = $10.6 + 2 \times \frac{1.1}{2} = 11.7$
	b. short wall	3	5.2	1.1	1	17.16	LS = $6.3 - 2 \times \frac{1.1}{2} = 5.2$
						+ 41.9	
2.	Lime concrete in Foundation						
	a. long wall	2	11.7	1.1	0.3	7.722	L = $10.6 + 2 \times \frac{1.1}{2} = 11.7$
	b. short wall	3	5.2	1.1	0.3	5.148	LS = $6.3 - 2 \times \frac{1.1}{2} = 5.2$
						+ 12.87	
3.	Brick work foundation in plinth						
	a. first Footing						
	(i) Long wall	2	11.4	0.8	0.2	3.648	LL = $10.6 + 2 \times \frac{0.8}{2} = 11.4$
	(ii) short wall	3	5.5	0.8	0.2	2.64	LS = $6.3 - 2 \times \frac{0.8}{2} = 5.5$
						+ 6.288	
	b. second footing						
	(i) Long wall	2	11.3	0.7	0.1	1.582	LL = $10.6 + 2 \times \frac{0.7}{2} = 11.3$
	(ii) short wall	3	5.6	0.7	0.1	1.176	LS = $6.3 - 2 \times \frac{0.7}{2} = 5.6$
						+ 2.758	

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						2.758	
c. 3rd Footing							
(i) long wall	2	11.2	0.6	0.1	1.344	$L_L = 10.6 + 2 \times \frac{0.6}{2} = 11.2$	
(ii) Short wall	3	5.7	0.6	0.1	1.026	$L_S = 6.3 - 2 \times \frac{0.6}{2} = 5.7$	
					2.37		
d. 4th Footing							
(i) long wall	2	11.1	0.5	0.1	1.11	$L_L = 10.6 + 2 \times \frac{0.5}{2} = 11.1$	
(ii) Short wall	3	5.8	0.5	0.1	0.87	$L_S = 6.3 - 2 \times \frac{0.5}{2} = 5.8$	
					1.98		
e. 5th footing							
(i) long wall	2	11	0.4	0.8	7.04	$L_L = 10.6 + 2 \times \frac{0.4}{2} = 11$	
(ii) short wall	3	5.9	0.4	0.8	5.664	$L_S = 6.3 - 2 \times \frac{0.4}{2} = 5.9$	
					12.704		
4. 2 cm D.P.C work							
(a) long wall	2	11	0.4		8.8	$L_L = 10.6 + 2 \times \frac{0.4}{2} = 11$	
(b) short wall	3	5.9	0.4		+ 7.08	$L_S = 6.3 - 2 \times \frac{0.4}{2} = 5.9$	
					15.88		
(c) Deduction for door	2	1.2	0.4		0.96		
					14.92		

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5.	Brick work in super structure.							
	(a) long wall	2	10.9	0.3	4.2	27.468	$L_L = 10.6 + 2 \times \frac{0.3}{2} = 10.9$	
	(b) short wall	3	6	0.3	4.2	22.68	$L_S = 6.3 - 2 \times \frac{0.3}{2} = 6$	
						+		
						50.148		
	(c) Deduction for							
	(i) Door	2	1.5	0.3	2.1	0.135	$L = 1.2 + 2 \times 0.15$	
	(ii) window	4	1.3	0.3	1.5	0.234	$L = 1 + 2 \times 0.15$	
	(iii) shelves	2	1.3	0.2	1.5	0.117	$L = 1 + 2 \times 0.15$	
						+		
						0.486		
						50.148		
						- 4.398		
						45.75		

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CENTER LINE METHOD

Item No	Particulars of item	No	length	breadth	height	Quantity	Explanatory Note
1.	Earth work in estavation.	1	39	1.1	1	42.9	$(2\times 10.6)+(3\times 6.3)-(2\times\frac{1.1}{2})$ $= 39$
2.	Lime concreete in foundation	1	39	1.1	0.3	12.87	$(2\times 10.6)+(3\times 6.3)-(2\times\frac{1.1}{2})$ $= 39$
3.	Breick work in foundation						
	① first footing	1	39.3	0.8	0.2	6.288	$(2\times10.6)+(3\times6.3)-(2\times\frac{0.8}{2})=$ $= 39.3$
	② 2nd footing	1	39.4	0.7	0.1	2.758	$(2\times10.6)+(3\times6.3)-(2\times\frac{0.7}{2})=39.4$
	③ 3rd footing	1	39.5	0.6	0.1	2.37	$(2\times10.6)+(3\times6.3)-(2\times\frac{0.6}{2})=39.5$
	④ 4th footing	1	39.6	0.5	0.1	1.98	$(2\times10.6)+(3\times6.3)-(2\times\frac{0.5}{2})=39.6$
	⑤ 5th footing	1	39.7	0.4	0.8	12.704	$(2\times10.6)+(3\times6.3)-(2\times\frac{0.4}{2})=39.7$
						26.1	
4.	2 cm D.P.C work	1	39.7	0.4		15.88	
	@ Deduction for Door	2	1.2	0.4		-0.96	
						14.92	

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5.	Brick work in super structure	1	39.8	0.3	4.2	50.148	$(2\times10.6)+(3\times6.3)-(2\times\frac{0.3}{2})=39.8$
	(a) Deduction for						
	(i) Door	2	1.2	0.3	2.1	1.512	
	(ii) window	4	1	0.3	1.5	1.8	
	(iii) shelves	2	1	0.2	1.5	0.6	
						3.912	
	(b) Deduction for lintel over						
	(i) Door	2	1.5	0.3	0.15	0.135	
	(ii) window	4	1.3	0.3	0.15	0.234	
	(iii) shelves	2	1.3	0.3	0.15	0.117	
						0.486	
						45.75	

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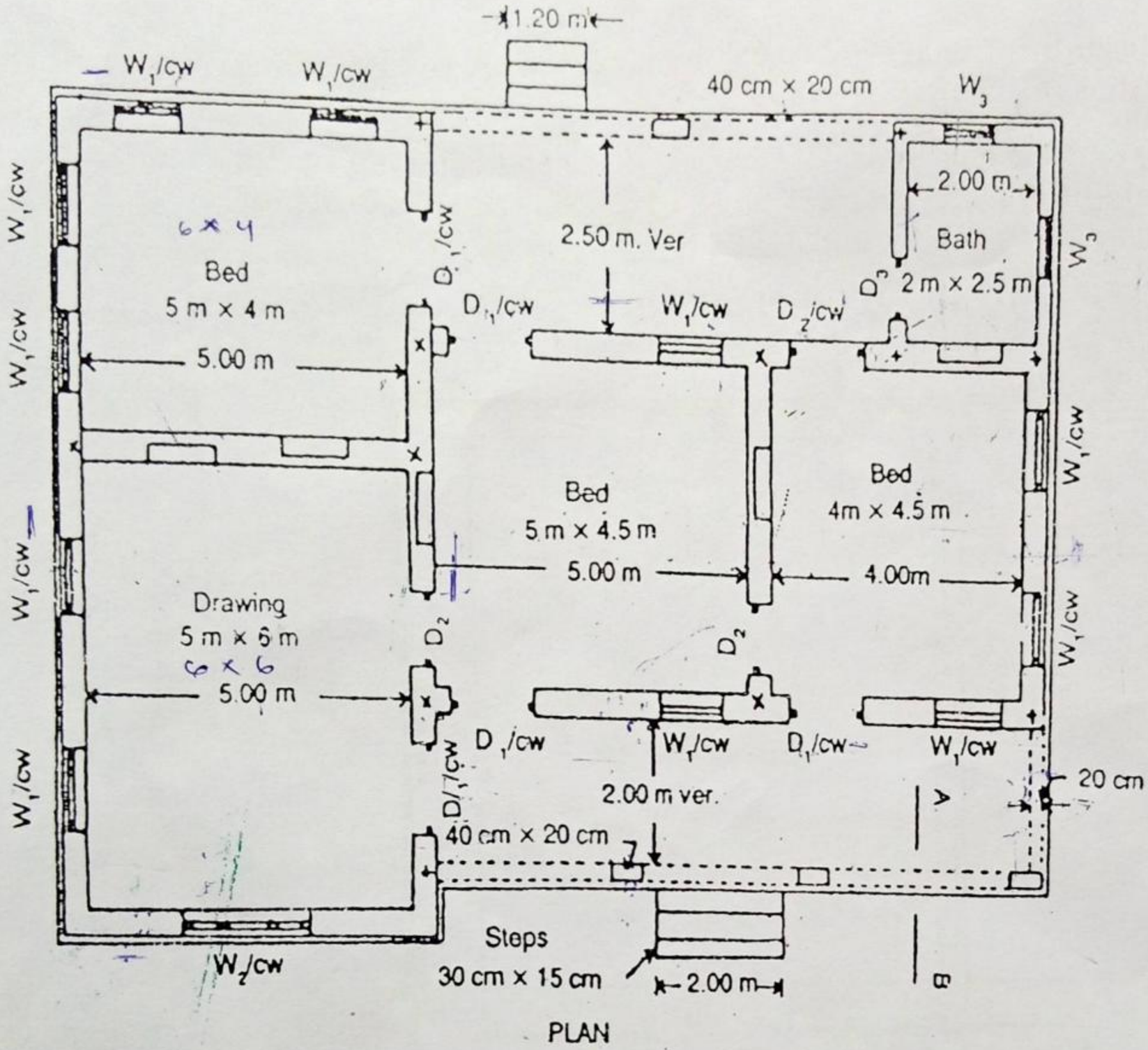
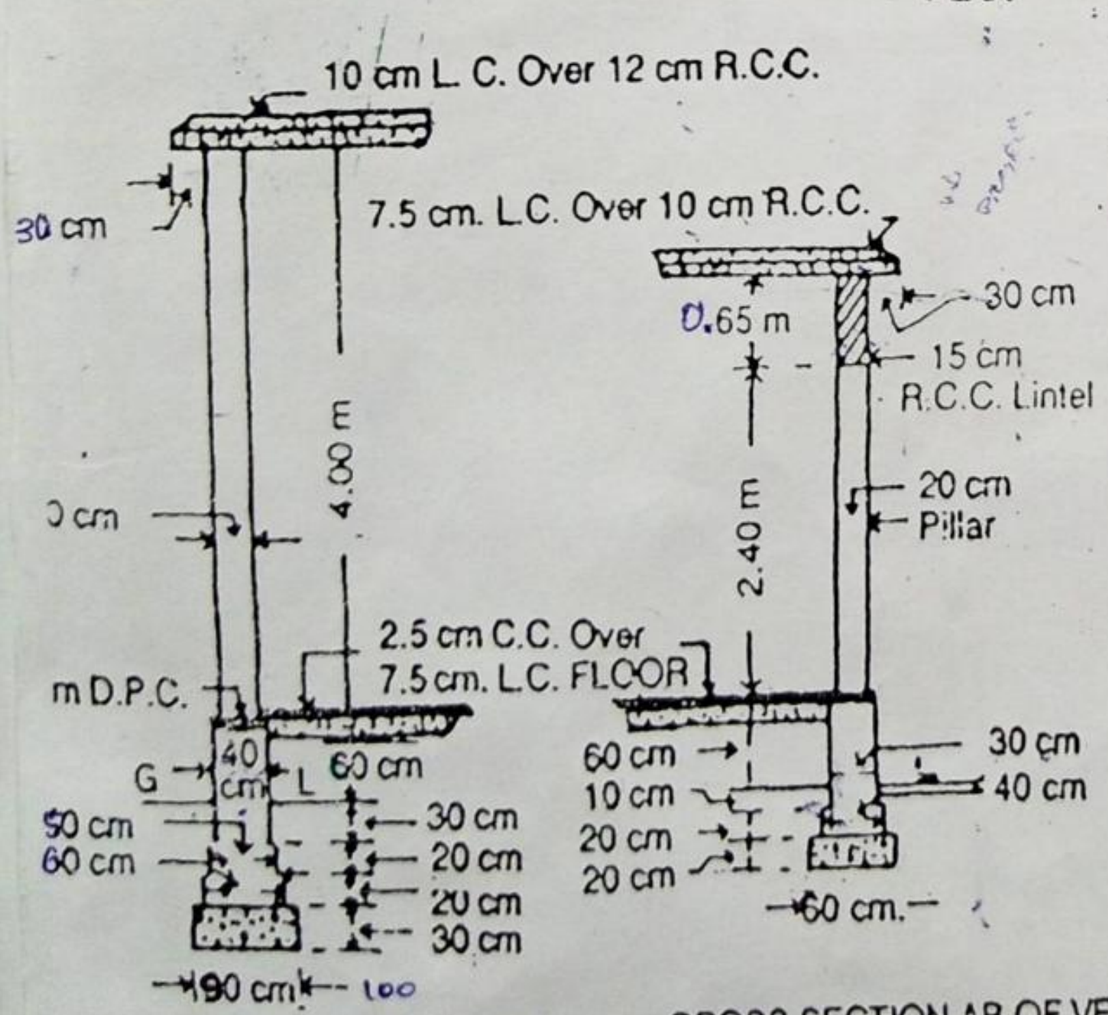
RESIDENTIAL BUILDING





CROSS SECTION OF MAIN WALLS

CROSS SECTION AB OF VER.WALL

Doors:-

D_1 – 120 cm × 210 cm (1.20 m × 2.10 m)

D_2 – 100 cm × 200 cm (1.00 m × 2.00 m)

D_3 – 75 cm × 180 cm (.75 m × 1.80 m).

Windows:-

W_1 – 100 cm × 150 cm (1.00 m × 1.50 m)

W_2 – 200 cm × 150 cm (2.00 m × 1.50 m)

W_3 – 75 cm × 120 cm (.75 m × 1.20 m)

C.W. – 75 cm × 60 cm (.75 m × .60 m).

Shelves:-

S – 100 cm × 150 cm (1.00 m × 1.50 m)

Lintel Over Doors, Windows Etc.

15 cm R.B.

All walls of Drawing Rooms and Bed Rooms have same section

Bath Room walls have similar section.

Note—No beam has been shown in the plan.

Fig. 2-7

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Drawing :- 4

Calculation of center to center leagth.

(a) Left side Bed and drawing combined center to center length of long wall =>

$$\Rightarrow \frac{0.3}{2} + 4 + 0.3 + 6 + \frac{0.3}{2} = 10.6$$

center to center length of short wall

$$\Rightarrow \frac{0.3}{2} + 5 + \frac{0.3}{2} = 5.3$$

No of long wall = 2

No of short wall = 3

(b) Right side double bedroom comined.

centere to center length of long wall

$$\Rightarrow \frac{0.3}{2} + 5 + 0.3 + 4 + \frac{0.3}{2} = 9.6$$

Center to center length of short wall

$$\Rightarrow \frac{0.3}{2} + 4.5 + \frac{0.3}{2} = 4.8$$

No of long wall = 2

No of short wall = 2

(c) Front varcandah

center to center length of long wall

$$\Rightarrow \frac{0.3}{2} + 5 + 0.3 + 4 + 01 + \frac{0.2}{2} = 9.65$$

center to center length of short wall

$$\Rightarrow \frac{0.3}{2} + 2 + \frac{0.2}{2} = 2.25$$

No of long wall 1

No of short wall 1

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(d) Back varandah including bathroom.

center to center length of long wall

$$= \frac{0.3}{2} + 5 + 0.3 + 4 + 0.1 + \frac{0.2}{2} = 9.65$$

center to center length of short wall

$$= \frac{0.2}{2} + 2.50 + \frac{0.3}{2} = 2.75$$

No of long wall - 1

No of short wall = 2

(e) Bathroom

center to center length of long wall

$$\Rightarrow \frac{0.3}{2} + 2.5 + \frac{0.2}{2} = 2.9$$

center to center length of short wall

$$\Rightarrow \frac{0.2}{2} + 2 + \frac{0.2}{2} = 2.2$$

No long wall = 2

No of short wall = 1

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Item NO	Particulars of Item	NO	Length (l)	breadth (b)	Height (h)	Quantity	Explanatory Note
1.	Earth work in Excavation						H = 0.3+0.2+0.2+0.3 = 1
	(a) Left side bed and drawing combined						
	① Long wall	2	11.5	0.9	1	20.7	LL = $10.6 + 2 \times \frac{0.9}{2} = 11.5$
	② short wall	3	4.4	0.9	1	11.88	Ls = $5.3 - 2 \times \frac{0.9}{2} = 4.4$
	(b) Right side double bedroom combined						
	① Long wall	2	9.6	0.9	1	17.28	$L_L = 9.6 + \frac{0.9}{2} - \frac{0.9}{2} = 9.6$
	② short wall	2	3.9	0.9	1	7.02	$L_S = 4.8 - \frac{0.9}{2} - \frac{0.9}{2} = 3.9$
	(c) front varandah						H = 0.2 + 0.2 + 0.1 = 0.5
	① Long wall	1	9.5	0.6	0.5		$L = 9.65 - \frac{0.9}{2} + \frac{0.6}{2} = 9.5$
	② short wall	1	1.5	0.6	0.5		$L_S = 2.25 - \frac{0.6}{2} - \frac{0.9}{2} = 1.5$

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	ⓓ Back varandah						
	ⓘ Long wall	1	9.5	0.6	0.5	2.85	$L = 9.65 - \frac{0.9}{2} + \frac{0.6}{2} = 9.5$
	ⓘⓘ Short wall	2	2	0.6	0.5	1.2	$L = 2.75 - \frac{0.9}{2} - \frac{0.6}{2} = 2$
	ⓔ step						
	① Front step	1	2.2	1	0.1	0.22	$L = 2 + 2 \times 0.1 = 2.2$
	② Back step	1	1.4	1	0.1	0.14	$b = 0.3 + 0.3 + 0.3 + 0.1 = 1$
2.	Lime concrete in foundation.						
	ⓐ Left side bed and drawing comined						
	ⓘ Long wall	2	11.5	0.9	0.3	6.21	$L = 10.6 + \frac{0.9}{2} + \frac{0.9}{2} = 11.5$
	ⓘⓘ Short wall	3	4.4	0.9	0.3	3.564	$L = 5.3 - \frac{0.9}{2} - \frac{0.9}{2} = 4.4$
	ⓑ Right side double bedroom comined						
	① Long wall	2	9.6	0.9	0.3	5.184	$L = 9.6 - \frac{0.9}{2} + \frac{0.9}{2} = 9.6$
	② short wall	2	3.9	0.9	0.3	2.106	$L = 4.8 - \frac{0.9}{2} - \frac{0.9}{2} = 3.9$

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ⓒ Front varandah						
① Long wall	1	9.7	0.6	0.2	1.164	$L = 9.65 - \frac{0.5}{2} + \frac{0.6}{2} = 9.7$
② short wall	1	1.7	0.6	0.2	0.204	$L = 2.25 - \frac{0.5}{2} - \frac{0.6}{2} = 1.7$
ⓓ Back varandah including bath-room						
① Long wall	1	9.7	0.6	0.2	1.164	$L = 9.65 - \frac{0.5}{2} + \frac{0.6}{2} = 9.7$
② short wall	2	2.2	0.6	0.2	0.204	$L = 2.75 - \frac{0.5}{2} - \frac{0.6}{2} = 1.7$
② steps						
① Front steps	1	2.2	1	0.1	0.22	
ii) Back steps	1	1.4	1	0.1	1.14	
③ Brick work in foundation and plinth.						
ⓐ Left side bed and drawing combined						
① First footing						
- long wall	2	11.2	0.6	0.2	1.344	$L = 10.6 + 2 \times \frac{0.6}{2} = 11.2$
- short wall	3	4.7	0.6	0.2	1.692	$L = 5.3 - 2 \times \frac{0.6}{2} = 4.7$

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(ii) second footing						
- long wall	2	4.1	0.5	0.2	2.22	$L_L = 10.6 + 2 \times \frac{0.5}{2} = 11.1$
- short wall	3	4.8	0.5	0.2	1.44	$L_S = 5.3 - 2 \times \frac{0.5}{2} = 4.8$
(iii) 3rd footing						
- long wall	2	11	0.4	0.9	7.92	$L_L = 10.6 + 2 \times \frac{0.4}{2} = 11$
- short wall	3	4.9	0.4	0.9	5.292	$L_S = 5.3 - 2 \times \frac{0.4}{2} = 4.9$
~~(iv) 4th footing~~						
~~- long wall~~						
~~- short wall~~						
(b) Right side double bedroom combined						
(i) First Footing						
- long wall	2	9.6	0.6	0.2	2.304	$L_L = 9.6 + \frac{0.6}{2} - \frac{0.6}{2} = 9.6$
- short wall	2	4.2	0.6	0.2	1.008	$L_S = 4.8 - \frac{0.6}{2} - \frac{0.6}{2} = 4.2$
(ii) Second Footing						
- long wall	2	9.6	0.5	0.2	1.92	$L_L = 9.6 + \frac{0.5}{2} - \frac{0.5}{2} = 9.6$
- short wall	2	4.3	0.5	0.2	0.82	$L_S = 4.8 - \frac{0.5}{2} - \frac{0.5}{2} = 4.3$
(iii) 3rd Footing						
- long wall	2	9.6	0.4	0.9	6.912	$L_L = 9.6 + \frac{0.4}{2} - \frac{0.4}{2} = 9.6$
- short wall	2	4.4	0.4	0.9	3.168	$L_S = 4.8 - \frac{0.4}{2} - \frac{0.4}{2} = 4.4$

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ⓒ Front varandah						
ⓘ First Footing						
+ Long wall	1	9.65	0.4	0.2	0.772	$L = 9.65 - \frac{0.4}{2} + \frac{0.4}{2} = 9.65$
short wall	1	1.85	0.4	0.2	0.148	$Ls = 2.25 - \frac{0.4}{2} - \frac{0.4}{2} = 1.85$
ⓘⓘ 2nd footing						
- long wall	1	9.6	0.3	0.7	2.016	$L = 9.65 - \frac{0.4}{2} + \frac{0.3}{2} = 9.6$
- short wall	1	1.9	0.3	0.7	0.399	$Ls = 2.25 - \frac{0.4}{2} - \frac{0.3}{2} = 1.9$
ⓓ Back varandah including bathroom						
ⓘ First footing						
Long wall	1	9.65	0.4	0.2	0.772	$L = 9.65 - \frac{0.4}{2} + \frac{0.4}{2} = 9.65$
Short wall	2	2.35	0.4	0.2	0.376	$Ls = 2.75 - \frac{0.4}{2} - \frac{0.4}{2} = 2.35$
ⓘⓘ Second footing						
long wall	1	9.6	0.3	0.7	2.016	$L = 9.65 - \frac{0.4}{2} + \frac{0.3}{2} = 9.6$
short wall	2	2.4	0.3	0.7	1.008	$Ls = 2.75 - \frac{0.4}{2} - \frac{0.3}{2} = 2.4$
ⓔ Steps						
ⓐ - Front step						
- First step	1	2	0.9	0.15	0.27	
- 2nd step	1	2	0.6	0.15	0.18	
- 3rd step	1	2	0.3	0.15	0.09	

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	ⓑ back step						
	first step	1	1.2	0.9	0.15	0.162	
	2nd step	1	1.2	0.6	0.15	0.108	
	3rd step	1	1.2	0.3	0.15	0.054	
4.	2.5 cm D.P.C work.						
	ⓐ left side bed and drawing combined						
	(i) Long wall	2	11	0.4	8.8	$L = 10.6 + \frac{0.4}{2} + \frac{0.4}{2} = 11$	
	(ii) short wall	3	4.9	0.4	5.88	$L = 5.3 - \frac{0.4}{2} - \frac{0.4}{2} = 4.9$	
	ⓑ Right side bed room combined						
	(i) Long wall	2	9.6	0.4	7.68	$L = 9.6 + \frac{0.4}{2} - \frac{0.4}{2} = 9.6$	
	(ii) short wall	2	4.4	0.4	3.52	$L = 4.8 - \frac{0.4}{2} - \frac{0.4}{2} = 4.4$	
	ⓒ Bath room						
	(i) Long wall	2	2.7	0.3	1.62	$L = 2.75 - \frac{0.4}{2} + \frac{0.3}{2} = 2.7$	
	(ii) short wall	1	1.9	0.3	0.57	$L = 2.2 - \frac{0.3}{2} - \frac{0.3}{2} = 1.9$	

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④	Pillars		0.4	0.2		7.68	
⑤	Deduction for Doors						
	(i) D_1	5	1.2	0.4		2.4	
	(ii) D_2	3	1	0.4		1.2	
	(iii) D_3	1	0.75	0.3		0.225	
5.	Brick work in Super structure.						
ⓐ	Left side bed drawing combined						
	① Long wall	2	10.9	0.3	4	26.16	$L_1 = 10.6 + \frac{0.3}{2} + \frac{0.3}{2} = 10.9$
	② short wall	3	5	0.3	4	18	$L_2 = 5.3 - \frac{0.3}{2} - \frac{0.3}{2} = 5$
⑤	Right side bed room combined						
	① Long wall	2	9.6	0.3	4	23.04	$L_1 = 9.6 + \frac{0.3}{2} - \frac{0.3}{2} = 9.6$
	② short wall	2	4.5	0.3	4	10.8	$L_2 = 4.8 - \frac{0.3}{2} - \frac{0.3}{2} = 4.5$

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ⓒ Front varandah (varandah wall above lintel)						$H = 0.65 - 0.15 = 0.5$
① Long wall	1	9.6	0.2	0.5	0.96	$L = 9.65 - \frac{0.3}{2} + \frac{0.2}{2} = 9.6$
② short wall	1	2	0.2	0.5	0.2	$Ls = 2.25 - \frac{0.3}{2} - \frac{0.2}{2} = 2$
ⓓ Back varandah (varandah wall above lintel)						
① Long wall	1	7.2	0.2	0.5	0.72	$L = 9.65 - \frac{0.3}{2} - \frac{0.2}{2} - 2 - 0.2 = 7.2$
ⓔ Bath room						
① Long wall	2	2.7	0.2	3.05	3.294	$H = 2.4 + 0.65 = 3.05$
② short wall	1	2	0.2	3.05	1.22	$L = 2.75 - \frac{0.3}{2} + \frac{0.2}{2} = 2.7$
ⓕ Pillars	4	0.4	0.2	2.4	0.768	$Ls = 2.2 - \frac{0.2}{2} - \frac{0.2}{2} = 2$
ⓖ Deduction for						
① Door, window and CW						
① Door						
- D_1	5	1.2	0.3	2.1	3.78	
- D_2	3	1	0.3	2	1.8	
- D_3	1	0.75	0.2	1.8	0.27	

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② window						
w_1	11	1	0.3	1.5	4.95	
w_2	1	2	0.3	1.5	0.9	
w_3	2	0.75	0.2	1.2	0.36	
③ CW	18	0.75	0.3	0.60	2.43	
④ shelves	5	1	0.2	1.5	1.5	
⑤ Deduction for lintel over door, window, CW, shelves.						
① Door						
D_1	5	1.5	0.3	0.15	0.3375	$L = 1.2 + 2 \times 0.15 = 1.5$
D_2	3	1.3	0.3	0.15	0.1755	$L = 1 + 2 \times 0.15 = 1.3$
D_3	1	1.05	0.2	0.15	0.0315	$L = 0.75 + 2 \times 0.15 = 1.05$
② window						
w_1	11	1.3	0.3	0.15	0.6435	$L = 1 + 2 \times 0.15 = 1.3$
w_2	1	2.3	0.3	0.15	0.1035	$L = 2 + 2 \times 0.15 = 2.3$
w_3	2	1.05	0.2	0.15	0.063	$L = 0.75 + 2 \times 0.15 = 1.05$
③ CW	18	1.05	0.3	0.15	0.3505	$L = 0.75 + 2 \times 0.15 = 1.05$
④ shelves	5	1.3	0.2	0.15	0.195	$L = 1 + 2 \times 0.15 = 1.3$

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5.	**Plastering**					
	out side					
	ⓐ Plinth height, Plinth projection and 10cm below ground level	1	52.6	0.75		H = 0.5 + 0.6 + 0.1 = 0.75 L = 2 × (11 + 15.3) = 52.6
	ⓑ out side of left side bed and drawing combined	1	33	4		L = 2 × [(0.3 + 5 + 0.3) + (0.3 + 6 + 0.3 + 4 + 0.3)] = 33
	ⓒ out side of right side double bed room combined	1	24.3			L = (5 + 0.3 + 4 + 0.3) + (5 + 4 + 0.3 + 0.3) + (4.5 + 0.3 + 0.3) = 24.3
	ⓓ out side of bathroom.	1	7.8	3.05		L = (2 + 0.2 + 0.2) + 2 × (2.5 + 0.2) = 7.8
	ⓔ Pillar	4	1.2	2.4		L = 2 × 0.4 + 0.2 =
	ⓕ out side of Front varandah	1	11.8	0.65		L = 9.65 − 0.3/2 + 0.2/2 + 2 + 0.2 =
	ⓖ out side of back varandah	1	7.2			

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	In side						
(h)	In side of left side bed room	1	18		4		L = 5+4+5+4 = 18
(i)	In side of drawing room.	1	22		4		L = 5+6+5+6 = 22
(j)	In side of right side bedroom 1	1	14.5		4		L = 5+4.5+5 = 14.5
(k)	In side of right side bedroo 2	1	17		4		L = 4+4.5+4+4.5 = 17
(l)	Inside of bath room.	1	7		3.05		l = 2+2.5+2.5 = 7 H = 2.40+0.65 = 3.05
(m)	In side of front varandah	1	11.4		0.65		L = 9.65 - 0.3/2 - 0.2/2 + 2
(n)	In side of back varandah.	1	7.2				
(o)	(i) Step						
	(ii) Front step						
	- 1st step	1	2.3	0.3			l = 0.15+2+0.15 = 2.3
	- 2nd step	1	2.6	0.3			L = 0.3+2+0.3 = 2.6
	- 3rd step	1	2.9	0.3			L = 0.45+2+0.45 = 2.9
	(ii) Back step						

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(ii) Back step							
1st step	1	1.2	0.3				$l = 0.15 + 1.2 + 0.15 = 1.2$
2nd step	1	1.8	0.3				$l = 0.3 + 1.2 + 0.3 = 1.8$
3rd step	1	2.1	0.3				$l = 0.45 + 1.2 + 0.45 = 2.1$
P. Deduction for							
• Door							
→ D_1	5 x 1	1.2		2.1			
D_2	3 x 1	1		2			
D_3	1 x 1	0.75		1.8			
window							
w_1	11 x 1	1		1.5			
w_2	1 x 1	2		1.5			
w_3	2 x 1	0.75		1.2			
CW	18 x 0	1		1.5			
7. R.B work							
(a) 15 cm R.B lintel over door							
(i) D_1	5	1.5	0.3	0.15	0.3375		$l = 1.2 + 2 \times 0.15 = 1.5$
(ii) D_2	3	1.3	0.3	0.15	0.1755		$l = 1 + 2 \times 0.15 = 1.3$
(iii) D_3	1	1.05	0.2	0.15	0.0315		$l = 0.75 + 2 \times 0.15 = 1.05$

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(iv) w_1 (v) w_2 (vi) w_3 (vii) CW (viii) shelves						
R.C.C work (a) Roof of left side bed and drawing combined	1	11.5	6.2	0.12		$l = 0.3+0.3+6+0.3+4+0.3+0.3 = 11.5$ $b = 0.3+0.3+5+0.3+0.3 = 6.2$
(b) Roof of Right side double bed room combined	1	9.6	5.7	0.12		$l = 3+0.3+5+0.3+4+0.3-0.3 = 9.6$ $b = 0.3+4.5+0.3+0.3+0.3 = 5.7$
(c) Roof of Front varandah	1	10	2.6	0.1		$l = 5+4+0.3+0.3+0.3+0.1 = 10$ $b = 2+0.3+0.2+0.1 = 2.6$
(d) Roof of back varandah including bath room	1	10	3.1	0.1		~~Loss~~ $l = 5+4+0.3+0.3+0.3+0.1 = 10$ $b = 0.3+0.2+2.5+0.1 = 3.1$

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Brick work

(e) 15 cm R.C.C lintel in Front varandah						
(i) Long wall	1		9.6	0.2	0.15	
(ii) short wall	1		2	0.2	0.15	
(f) 15 cm R.C.C. lintel in back varandah						
(i) Long wall	1		7.2	0.2	0.15	

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Rules for plastering and pointing

- No deduction are made for ends of beam, post, rafter etc.
- For small opening upto $0.5\ m^2$ no deduction is made and at the same time no additions are made for soffits, jambs and sill etc.
- For opening exceeding $0.5\ m^2$ but not exceeding $3\ m^2$ deduction is made for one face only. and the other face is allow for jambs, soffits and sill.
- For opening above $3\ m^2$ deduction is made for both faces and the jambs, soffits and sills are taken en to account and added. seperately.

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SINGLE ROOM BUILDING WITH FRONT VERANDAH

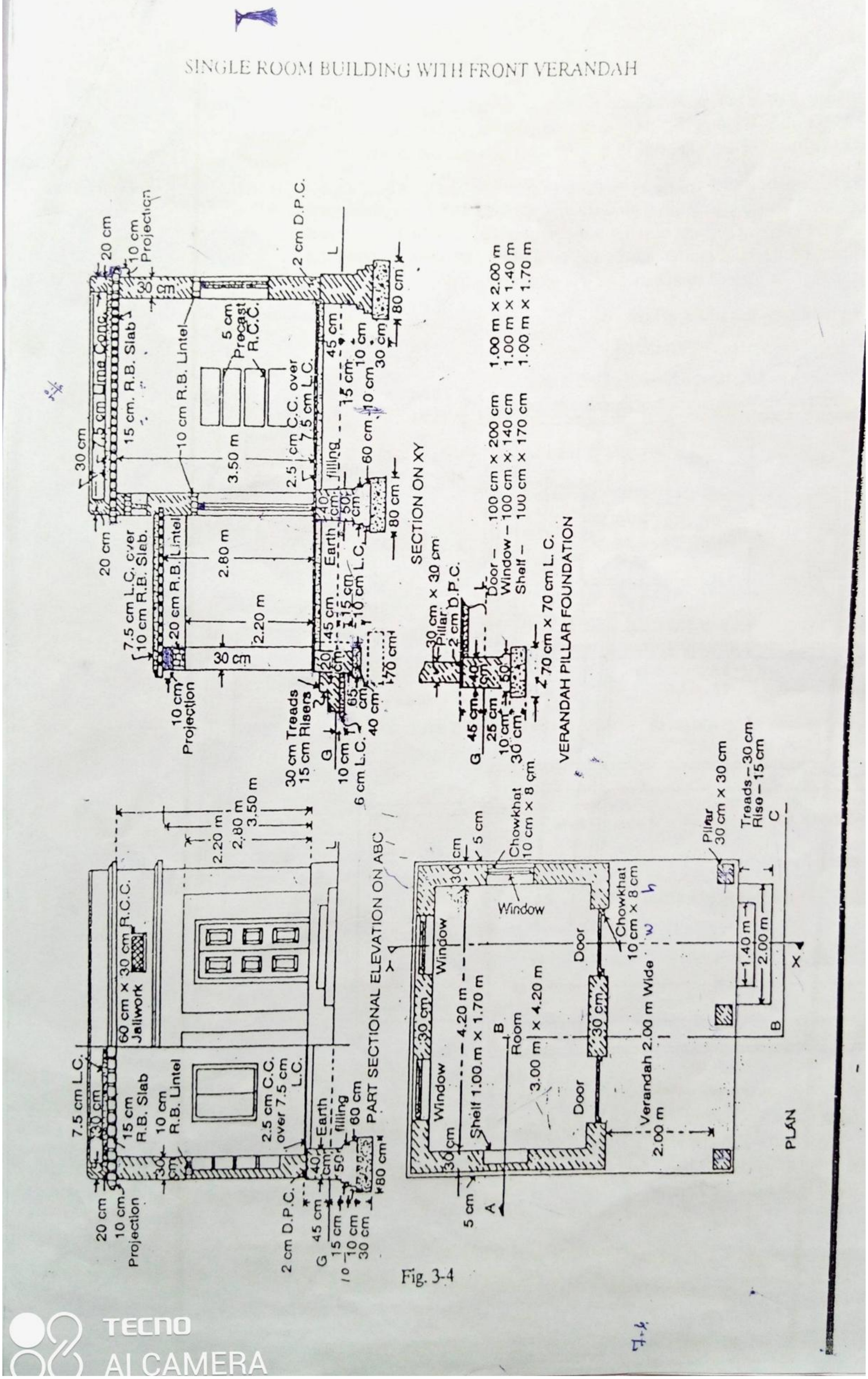

Fig. 3-4

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(A) Room :-

(1) Calculation of the center to center length of

long wall = $\frac{0.3}{2} + 4.2 + \frac{0.3}{2} = 4.5$

(2) calculation of the center to center of short

wall = $\frac{0.3}{2} + 3 + \frac{0.3}{2} = 3.3$

B. Varandah :-

(1) center to center length of long wall (center to center of pillar) = $\frac{0.3}{2} + 4.2 + \frac{0.3}{2} = 4.5$

(2) center to center length of short wall (center of main wall ~~center~~ to the center of pillar) = $\frac{0.3}{2} + 2 + \frac{0.3}{2}$

= 2.3

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No	Particulars of Item	No	Length	breadth	Height	Quantity	Explanatory Notes.
1.	Earth work in Excavation						
	(a) Room						H = 0.3+0.1+0.1+0.15 = 0.65
	(i) Long wall	2	5.3	0.8	0.65		L = $\frac{0.8}{2}$ + 4.5 + $\frac{0.8}{2}$ = 5.3
	(ii) short wall	2	2.2	0.8	0.65		Ls = 3.3 - $\frac{0.8}{2}$ - $\frac{0.8}{2}$ = 2.2
	(b) Pillar	3	0.7	0.7	0.65		
	(c) Varandah						H = 0.25 + 0.1 + 0.3 = 0.65
	(i) Long wall	1	5.2	0.7	0.65		L = $\frac{0.7}{2}$ + 4.5 + $\frac{0.7}{2}$ = 5.2
	(ii) short wall	2	1.6	0.7	0.65		Ls = 2.3 - $\frac{0.7}{2}$ - $\frac{0.7}{2}$ = 1.6
	(c) varandah						H = 0.1 + 0.15 = 0.25
	(i) Long wall	1	3.1	0.4	0.25		L = 4.5 - $\frac{0.7}{2}$ - $\frac{0.7}{2}$ - 0.7 = 3.1
	(ii) short wall	2	1.55	0.4	0.25		Ls = 2.3 - $\frac{0.8}{2}$ - $\frac{0.7}{2}$ = 1.55
	(d) step	1	2.1	0.65	0.1		L = 2 + (2 × 0.05) = 2.1
							b = 0.3 + 0.3 + 0.05 = 0.65

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2.	Lime concrete in Foundation					
	ⓐ Room					$L_L = \frac{0.8}{2} + 4.5 + \frac{0.8}{2} = 5.3$
	ⓘ long wall	2	5.3	0.8	0.3	$L_S = 3.3 - \frac{0.8}{2} - \frac{0.8}{2} = 2.5$
	ⓘⓘ short wall	2	2.5	0.8	0.3	
	ⓑ Pillar	3	0.7	0.7	0.3	
	ⓒ Varandah					$L = 4.5 - \frac{0.4}{2} - 0.4 - \frac{0.4}{2} = 4.1$
	ⓘ long wall	1	4.1	0.4	0.1	$L_S = 2.3 - \frac{0.4}{2} - \frac{0.5}{2} = 1.85$
	ⓘⓘ short wall	2	1.85	0.4	0.1	
	ⓓ Step	1	2.1	0.65	0.06	
3.	Brick work and Plinth					
	ⓐ Room					
	ⓘ First footing					
	ⓘⓘ long wall	2	5.1	0.6	0.1	$L = 4.572 \times \frac{0.6}{2} = 5.1$
	= short wall	2	2.7	0.6	0.1	$L_S = 3.3 - 2 \times \frac{0.6}{2} = 2.7$

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(ii) 2nd Footing			0.5	0.1		$L = 4.5 + 2 \times \frac{0.5}{2} = 5$
+ long. wall	2	5	0.5	0.1		
short wall	2	2.8				$B = 3.3 - 2 \times \frac{0.5}{2} = 2.8$
(iii) 3rd Footing						
- long wall	2	4.9	0.4	0.5		$L = 4.5 + 2 \times \frac{0.4}{2} = 4.9$
- short wall	2	2.9	0.4	0.5		$B = 3.3 - 2 \times \frac{0.4}{2}$ 2.9
(b) Pellar						
(i) First footing	3	0.5	0.5	0.1		
(ii) 2nd Footing	3	0.5	0.4	0.7		$H = 0.45 + 0.25 = 0.7$
(c) Verandah wall						
i) Long wall	1	3.7	0.2	0.6		$H = 0.45 + 0.15 = 0.6$
(ii) short wall	2	1.9	0.2	0.6		$L = 4.5 - \frac{0.4}{2} - \frac{0.4}{2} - 0.4$
						$= 3.7$
						$B = 2.3 - \frac{0.4}{2} - \frac{0.4}{2}$
						$= 1.9$
(d) Steps						
1st step	1	0.2	0.6	0.19		$H = 0.15 + 0.4 = 0.19$
2nd step	1	0.14	0.3	0.15		

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4.	2cm D.P.C work						
	a. Room						$L_L = 4.5 + 2 \times \frac{0.4}{2} = 4.9$
	i - long wall	2	4.9	0.4			
	ii - short wall	2	2.9	0.4			$L_S = 3.3 - 2 \times \frac{0.4}{2} = 2.9$
	b. Pillar	3	0.4	0.4			
	c. Deduction for Door	2	0.1	0.4			
5.	Brick work in Super Structure						
	a. Room						
	i long wall	2	4.8	0.3	3.5		$L = 4.5 + 2 \times \frac{0.3}{2} = 4.8$
	ii short wall	2	3	0.3	3.5		$L_S = 3.3 - 2 \times \frac{0.3}{2} = 3$
	b. Pillar	3	0.3	0.3	2.2		
	c. Verandah wall above lintel						$H = 0.6 - 0.2 = 0.4$
	i. long wall	1	4.8	0.3	0.4		$L = 4.5 + 2 \times \frac{0.3}{2} = 4.8$
	ii. short wall	2	2	0.3	0.4		$L_S = 3.3 - 2 \times \frac{0.3}{2} = 3$

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Parapate above roof			0.2			$H = \frac{30+7.5}{100}$
i- Long wall	2	4.8	0.2	0.37		$L = 4.5 + 2\times\frac{0.3}{2} = 4.8$
ii- short wall	2	2.4	0.2	0.37		$L_s = 2.3 + 2\times\frac{0.1}{2} = 2.4$
⑤ Deduction for						
i- Door	2	1	0.3	2		
ii- window	3	1	0.3	1.4		
iii- shelve	1	1	0.2	1.7		
iv- Jali work	1	0.6	0.3	0.3		
⑦ Deduction for						
Lintel over						
i- Door	2	1.2	0.3	0.2		
ii- window	3	1.2	0.3	0.2		
iii- shelve	1	1.2	0.3	0.2		
iv- Jali work	2	0.8	0.3	0.3		
⑥ ⓐ Plastering and Pointing wall out side						
ⓐ Plinth height and Plinth Projection 10 cm below ground level	1	21.8		0.6		$H = 0.05 + 0.45 + 0.1 = 0.6$ $L = 2\times4.9 + 2\times6 = 21.8$

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below ground level

⑤ out side of room	1	16.8		3.5		L = 2 x (4.8 + 3.6) = 16.8
⑥ Pillar	4	1.2		2.2		L = 0.3 x 4 = 1.2
⑦ out side of verandah wall above pillar	1	9.4		0.6		H = 2.8 - 2.2 = 0.6 L = 4.8 + 2.8 2.3 = 9.4
⑧ out side of parapate.	1	16.8		0.375		L = 2 (4.8 + 3.6) = 16.8 H = 30 + $\frac{7.5}{100}$ = 0.375
⑨ Steps						L = 2 + 0.15 + 0.15 = 2.3
1- First step	1	2.3	0.3			
2- Second step	1	2.6	0.3			L = 1.4 + 4 x 0.15 + 2 x 0.3 = 2.6
Inside						
⑩ inside of room	1	14.4		3.5		L = 4.2 + 4.2 + 3 + 3 = 14.4
⑪ verandah wall above pillar	1	8.2		0.6		L = 4.2 + 2 x 2 = 8.2 H = 2.8 - 2.2 = 0.6
⑫ parcerpate	1	15.2		0.3		L = 2 x 4.4 + 2 x 3.2 = 15.2

4.4
3.2

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	Deduction for						
	i- Door	2	1		2		
	ii- window	3	1		1.4		
	iii- Jali work	2	0.6		0.3		No deduction shall be made.
7.	wood working frame						
	a. Door	2	5.08	0.1	0.08	0.081	L = 1+ ~~0.08~~(2x2)+ (0.04x2) = 5.08
	b. window	3	4.8	0.1	0.08	0.1152	L = 2x1 + 2x1.4 = 4.8
8.	wood working shutter						
	(a) Door	2	0.87		1.935		L = (100 - 2x8 + 2x1.5)/100 (Repate=1.5) = 0.87
	(b) window	3	0.87				H = (200 - 8 + 1.5)/100 = 1.935
							L = (100 - (2x8) + (2x1.5))/100 = 0.87

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	~~Lime concrete~~						$H = 100 - (2\times8) - (2\times1.5)$
9.	Lime concrete terasing						
	a. 7.5 L.C terasing over room	1	4.4	3.2			$L = 4.2 + (2\times0.1) = 4.4$ $b = 3 + (2\times0.1) = 3.2$
	b. 7.5 cm L.C terasing	1	5	2.4			$L = 4.8 + (2\times0.1) = 5$ $b = 2 + 0.3 + 0.1 = 2.4$
10.	R.B work						
	a. 10 cm R.B lintel over						
	i- Door	2	1.2	0.3	0.2		
	ii- window	3	1.2	0.3	0.2		
	iii- Shelves	1	1.2	0.3	0.2		
	iv- Jali work	2	0.8	0.3	0.3		
	b. 20 cm RB Lintel over pillar in verandah						
	i- long wall	1	4.8	0.3	0.2		$L = 0.3 + 4.2 + 0.3 = 4.8$
	ii- short wall	2	2	0.3	0.2		

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c.	15 cm R.B Slab over. Roof	1	5	3.8	0.15		L = 0.3+4.2+0.3+0.1 +0.1 = 5 b = 0.3+3+0.3+0.1+0 = 3.8
d.	10 cm R.B Slab over. Verandah	1	5	2.5	0.1		L = 0.3+4.2+0.3+0.1 +0.1 = 5 b = 0.1+2+0.3+0.1 = 2.5
11.	R.C.C work						
a.	5cm precast R.C.C slab on shelve	3	1.1	0.2	0.05		l = 1+(2×0.05)

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TWO-ROOM BUILDING WITH FRONT VERANDAH

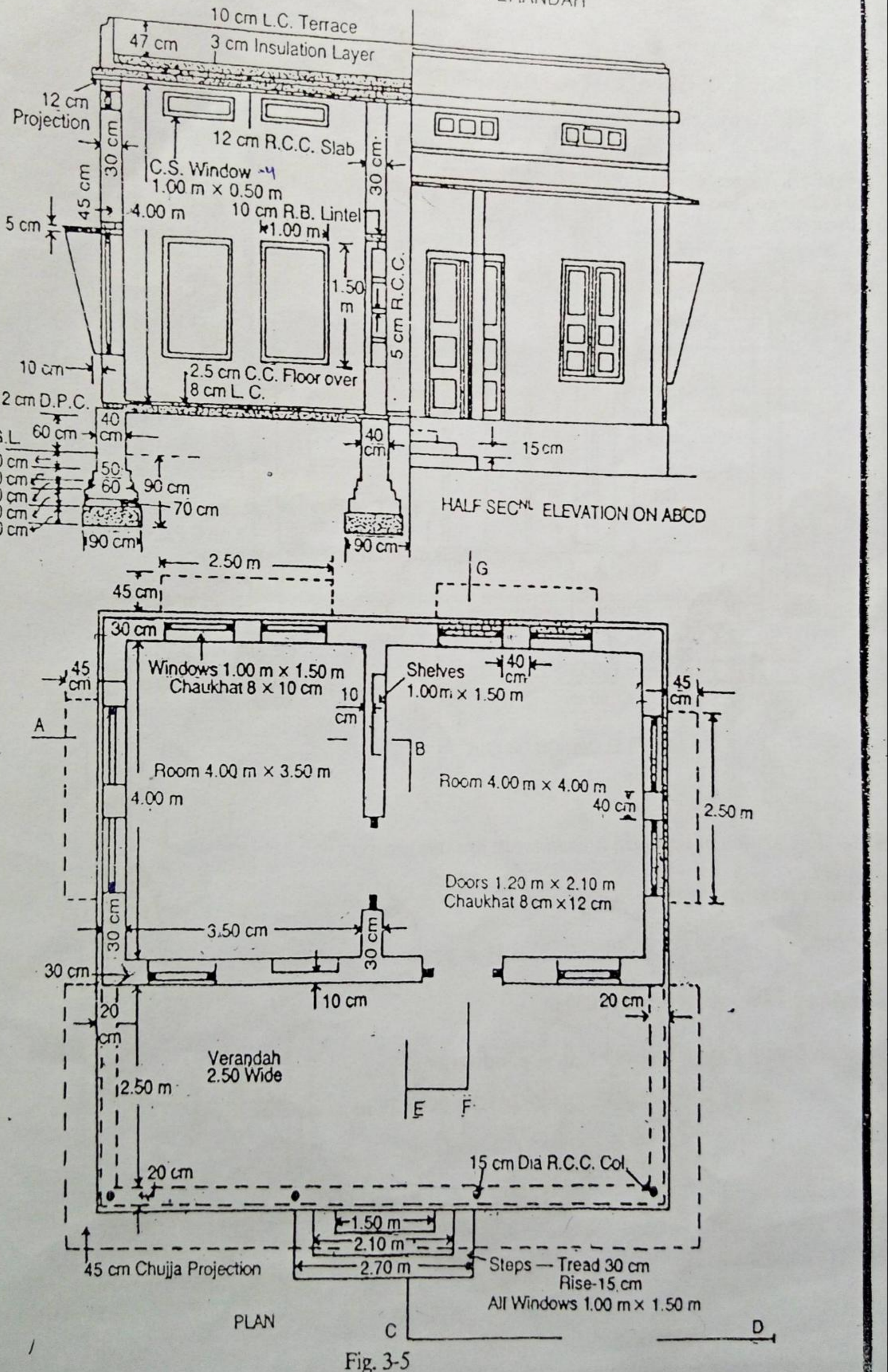

Fig. 3-5

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Room

center to center length of long wall

$$= \frac{0.3}{2} + 3.5 + 0.3 + 4 + \frac{0.3}{2} = 8.1$$

center to center length of short wall

$$= \frac{0.3}{2} + 4 + \frac{0.3}{2} = 4.3$$

Front Verandah

center to center length OF long wall

$$= \frac{0.2}{2} + 0.1 + 3.5 + 0.3 + 4 + 0.1 + \frac{0.2}{2} = 8.2$$

Center to center Length OF short wall

$$= \frac{0.3}{2} + 2.5 + \frac{0.2}{2} = 2.75$$

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Item no	Particulars of item	NO	Length	breadth	height	Quantity	Explanatory note.
1.	Earth work on excavation						$H = 0.3+0.2+0.1+0.1+0.2$
	a. Room						
	- long wall	2	9	0.9	0.9		$L_L = 8.1+2\times\frac{0.9}{2} = 9$
	- short wall	3	3.4	0.9	0.9		$L_S = 4.3-2\times\frac{0.9}{2} = 3.4$
	b. verandah						
	- long wall	1	8.9	0.7	0.9		$H = 0.3+0.2+0.1+0.3 = 0.9$
	- short wall	2	1.95	0.7	0.9		$L_L = 8.2+2\times\frac{0.7}{2} = 8.9$
	c. step	1	2.9	1	0.15		$L_L = 2.75-\frac{0.9}{2}-\frac{0.7}{2} = 1.95$
							$L = 2.7+2\times0.1$
2.	Lime concrete in foundation						
	a. Room						
	- long wall	2	9	0.9	0.3		
	- short wall	3	3.4	0.9	0.3		
	b Verandah						LL
	- long wall	1	8.9	0.7	0.3		
	- short wall	2	1.95	0.7	0.3		

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c. step	1	2.9	1	0.15		
3 Brick work in foundation and plinth						
a. Room						
i First footing						
- long wall	2	8.8	0.7	0.2		$L_L = 8.1 + 2\times\frac{0.7}{2} = 8.8$
- short wall	3	3.6	0.7	0.2		$L_S = 4.3 - 2\times\frac{0.7}{2} = 3.6$
ii - 2nd footing						
- long wall	2	8.7	0.6	0.1		$L = 8.1 + 2\times\frac{0.6}{2} = 8.7$
- short wall	3	3.7	0.6	0.1		$L_S = 4.3 - 2\times\frac{0.6}{2} = 3.7$
iii 3rd footing						
- long wall	2	8.6	0.5	0.1		$L = 8.1 + 2\times\frac{0.5}{2} = 8.6$
- short wall	3	3.8	0.5	0.1		$L_S = 4.3 - 2\times\frac{0.5}{2} = 3.8$
iv - 4th footing						
- long wall	2	8.5	0.4	0.8		$L = 8.1 + 2\times\frac{0.4}{2} = 8.5$
- short wall	3	3.9	0.4	0.8		$L_S = 4.3 - 2\times\frac{0.4}{2} = 3.9$

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b. Verandah					
a First footing					$L_L = 8.2 + 2 \times \frac{0.5}{2} = 8.7$
- long wall	1		0.5	0.2	
- short wall	2		0.5	0.2	$L_S = 2.75 - \frac{0.5}{2} - \frac{0.7}{2} = 2.15$
b. 2nd footing					
- long wall	1		0.4	0.1	$L = 8.2 + 2 \times \frac{0.4}{2} =$
- short wall	2		0.4	0.1	$L_S = 2.75 - \frac{0.4}{2} - \frac{0.6}{2} =$
c. 3rd footing					
i - long wall	$\frac{1}{2}$		0.3	0.9	$L = 8.2 + 2 \times \frac{0.3}{2} =$
ii - short wall			0.3	0.9	
= 10 cm height of 3rd footing from bottom	2		0.3	0.1	$L_S = 2.75 - \frac{0.3}{2} - \frac{0.5}{2}$
= 80 cm height above 10 cm height	2		0.3	0.8	$L_S = 2.75 - \frac{0.3}{2} - \frac{0.4}{2}$
c. Step					
- 1st step	1	2.7	0.9	0.15	
- 2nd step	1	2.1	0.6	0.15	
- 3rd step	1	1.5	0.3	0.15	

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4.	2 cm D.P.C work						
	a. Room						
	- long wall	2	8.5	0.4			$L_L = 8.1 + 2 \times \frac{0.4}{2} = 8.5$
	- short wall	3	3.9	0.4			$L_S = 4.3 - 2 \times \frac{0.4}{2} = 3.9$
	Deduction for Door	2	1.2	0.4			
5.	Brick work in Superstructure						
	a Room						
	- long wall	2	8.4	0.3	4		$L = 8.1 + 2 \times \frac{0.3}{2} = 8.4$
	- short wall	3	4	0.3	4		$L_S = 4.3 - 2 \times \frac{0.3}{2} = 4$
	b verandah wall above lintel.						
	- long wall	1	8.4	0.2	0.3		$L = 8.1 + 2 \times \frac{0.2}{2} = 8.4$
	- short wall	2	2.5	0.2	0.3		$L_S = 4.3 - 2 \times \frac{0.2}{2} = 2.5$

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Parapate over room						
- long wall	2	8.4	0.2	0.6		H = 0.47 + 0.1 + 0.3 = 0.6
- short wall	2	4.2	0.2	0.6		L = 0.3 + 3.5 + 0.3 + 4 + 0.3 = 8.4
Parapate over varandah						
- long wall	1	8.4	0.2	0.4		
- short wall	2	2.5	0.2	0.4		
Deduction						
- DOOR	2	1.2	0.3	2.1		
- window	10	1	0.3	1.5		
- C.S window	11	1	0.3	0.5		
- shelves	2	1	0.2	1.5		
Deduction for lintel over.						
- DOOR	2	1.4	0.3	0.1		D = 1.2 + 0.1 + 0.1 = 1.4
- window	10	1.2	0.3	0.1		w = 1 + 0.1 + 0.1 = 1.2
- shelves	2	1.2	0.3	0.1		S = 1 + 2 × 0.1 = 1.2
- C.S window.	11	1.2	0.3	0.1		C.S = 1 + 2 × 0.1 = 0.2

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4.	Plastering work out side					
	a. Plinth, heigh, plinth projection, including 10cm below ground leble.	1	31.8		0.75	L = 2x8.5 + 2x7.4 = 31.8 H = 0.05 + 0.6 + 0.1 = 0.75
	b. out side of double room combined.	1	26		4	L = 2x(8.4 + 4.6) — only wall
	c. out side of varandah wall	1	13.8		0.6	L = 8.4 + 2x2.7 = 13.8 H = 3 − 2.4 = 0.6
	d. outer side of parcapate					
	① over room	1	26		0.6	L = (2x8.4) + (2x4.6)
	② owere varandah	1	13.8		0.4	L = 2x(2.5 + 0.2) + 8.4

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e. Steps					
- 1st step	1	3	0.3	0.15	L = 2.7 + (2 × 0.15) = 3
- 2nd step	1	3.3	0.3	0.15	L = 2.7 + (4 × 0.15) = 3.3
- 3rd step	1	3.6	0.3	0.15	L = 2.7 + (6 × 0.15) = 3.6
Inside					
f. Inside of left side room 1	1	15		4	L = 2 x (3.5 + 4) = 15
g. Right side room.	1	16		4	L = 2x (4 + 4) = 16
h. Inside of varandah wall	1	13		0.6	L = 8 + (2 x 2.5) = 13 H = 3 - 2.4 = 0.6
i. Inside of parapet.					
i- over room	1	24.4		0.47	L = (2 x 8) + (2 x 4.2) = 24.4
ii- over varandah	1	13		0.27	L = 8 + (2 x 2.5)
j. Plastering of Chajja (window)					
(i) Top chajja	4x2	2.5	0.45		

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	(Top, bottom)						
(ii)	Bottom chajja (Top, bottom)	4×2	2.5	0.1			
(iii)	Side chajja (~~Top~~ inner, outer, ~~bottom~~)	4×4		0.275	1.5		$b = \frac{0.45+0.1}{2} =$
K.	Chajja (Verandah)						
	i- Front chajja (Top and bottom)	1×2	9.3	0.45			$L = 8.4+(2\times 0.45)$
	ii- Side chajja	2×2	2.7	0.45			
L.	Deduction For						
	i- Door	2×1	1.2		2.1		Deduction is made for 1 face only.
	ii- window	10×1	1		1.5		"
	iii- C.S window	11×0	1		0.5		no deduction is made.
M.	wood working						
	1- Door	2	5.4	0.12	0.8		$L = 1.2+2.1+2.1+0.04+0.04$
	2- window	10	5	0.1	0.8		$L = 2\times1+2\times1.5$

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wood working shutter					
- DOOR	2	1.07	2.035		$L = 1.2 - 2(0.08) + 2\left(\frac{1.5}{100}\right)$ $= 1.07$
+ window	10	0.87	1.37		$H = 2.1 - \left(0.08 + \frac{1.5}{100}\right)$ $= 2.035$
					$L = 1 - 2(0.08) + 2\left(\frac{1.5}{100}\right)$ $= 0.87$
					$H = 1.5 - 2 \times 0.08 + 2 \times \left(\frac{1.5}{100}\right)$ $= 1.37$

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Superstructure. — Wall shall be of first class brickwork in lime mortar. Roof shall be of lime concrete terracing over R.C.C slab. Window chowkhat shall be of sal wood, and shutters shall be 4 cm thick battenes of shisham wood. Gate will be of 20 B.W.G. sheet iron in angle iron frame.

Finishing. — Floor of garage room shall be 2.5 cm c.c. over 7.5 cm L.C. and floor of approach ramp shall be of 4 cm c.c. over 7.5 cm L.C. Walls shall be 12 mm plastered with 1 : 6 cement local sand mortar. Ceiling and the exposed surfaces of outer projections of slab shall be 6 mm cement plastered with 1 : 3 cement coarse sand mortar. Inside and outside shall be white washed three coats. Windows and gate shall be painted two coats over one coat of priming.

Calculate also the plinth area rate of the garage.

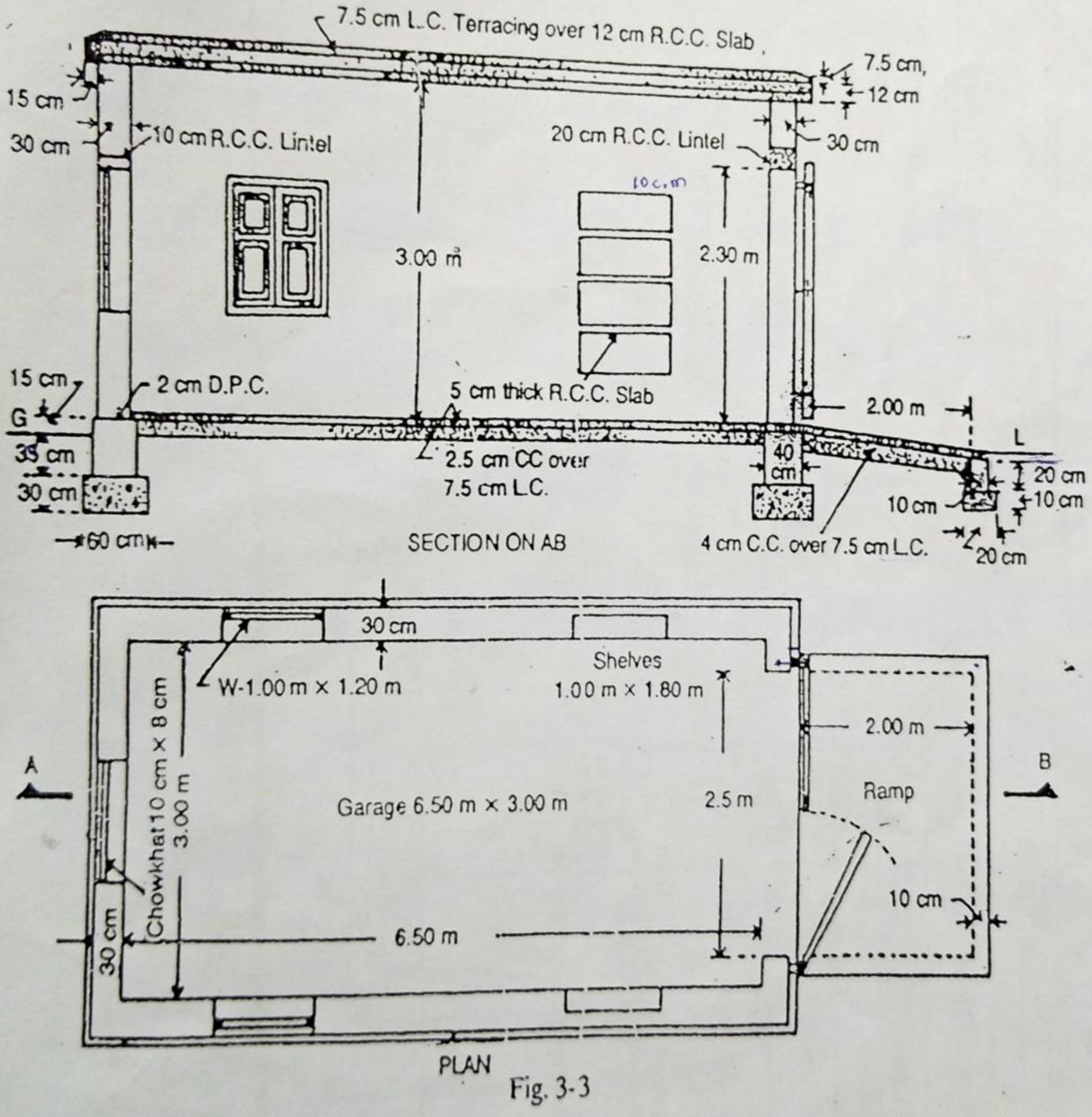

Fig. 3-3

Motor Garage -- Long wall c. to c. length = 6.50 + 0.30 = 6.80 m, and short wall c. to c. length = 3.00 + 0.30 = 3.30 m.

Approach Ramp — Front wall c. to c. length = 2.50 + 0.10 = 2.60 m, and side walls c. to c. length = 2.00 + .20 + .05 = 2.25 m.

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calculation of center to center length :-

a - Garage

center to center length of longwall =

$$\frac{0.3}{2} + 6.5 + \frac{0.3}{2} = 6.8$$

center to center length of short wall =

$$\frac{0.3}{2} + 3 + \frac{0.3}{2} = 3.3$$

b. Ramp

Center to center length of long wall =

$$\frac{0.1}{2} + 2.5 + \frac{0.1}{2} = 2.6$$

center to center length of short wall =

(Plinth Projection)

$$\frac{0.3}{2} + 0.05 + 2 + \frac{0.1}{2} = 2.25$$

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Item NO	Particulars of Item	NO	Length	Breadth	Height	Quantity	Explanatory Note.
1.	Earth work in excavation						H = 0.3 + 0.35 = 0.65
	a. Garage						
	i - long wall	2	7.4	0.6	0.65		L = $\frac{0.6}{2}$ + 6.8 + $\frac{0.6}{2}$ = 7.4
	ii - short wall	2	2.7	0.6	0.65		S = $\frac{0.6}{2}$ - 3.3 - $\frac{0.6}{2}$ = 2.7
	b. Ramp						H = 0.1 + 0.2 + 0.04 = 0.34
	i - long wall	1	2.8	0.2	0.34		L = 2.6 + 2 × $\frac{0.2}{2}$
	ii - short wall	2	1.85	0.2	0.34		= 2.8
							S = 2.25 - $\frac{0.6}{2}$ - $\frac{0.2}{2}$
							= 1.85
2.	Cement concrete in foundation						
	a. Garage						
	i - long wall	2	7.4	0.6	0.3		L = 6.8 + 2 × $\frac{0.6}{2}$ = 7.4
	ii - short wall	2	2.4	0.6	0.3		S = 3.3 - 2 × $\frac{0.6}{2}$ = 2.7
	b Ramp						
	i - long wall	1	2.8	0.2	0.1		L = 2.6 + 2 × $\frac{0.2}{2}$ = 2.8
	ii - short wall	2	1.95	0.2	0.1		S = 2.25 - $\frac{0.6}{2}$ - $\frac{0.2}{2}$
							= 1.95

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3.	Brick work in foundation in plinth					$H = 0.35 + 0.45 = 0.5$
	a. Garage					
	- long wall	2	7.2	0.4	0.5	$L = 6.8 + 2 \times \frac{0.4}{2} = 7.2$
	- short wall	2	2.9	0.4	0.5	$L = 3.3 - 2 \times \frac{0.4}{2} = 2.9$
	b. Ramp					
	- long wall	1	2.7	0.1	0.2	$L = 2.6 + 2 \times \frac{0.1}{2} = 2.7$
	- short wall	2	2	0.1	0.2	$L = 2.25 - \frac{0.4}{2} - \frac{0.1}{2} = 2$
4.	2cm D.P.C					
	a. Garage					
	- long wall	2	7.2	0.4		$L = \frac{0.4}{2} \times 2 + 6.8 = 7.2$
	- short wall	2	2.9	0.4		$L = 3.3 - 2 \times \frac{0.4}{2} = 2.9$
5.	b. Deduction for gate	1	2.5	0.4		

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5. Brick work in super structure

a. Garage					
i. Long wall	2	7.1	0.3	3	$L_1 = 6.8 + 2 \times \frac{0.3}{2} = 7.1$
ii - Short wall	2	3	0.3	3	$L_3 = 3.3 - 2 \times \frac{0.3}{2} = 3$
b. Deduction for					
i - Gate	1	2.5	0.3	2.3	
ii - window	3	1	0.3	1.2	
iii - shelve	2	1	0.2	1.8	
C. Deduction for lintel over					
i - Gate	1	2.9	0.3	0.2	$L = 2.5 + 0.2 + 0.2 = 2.9$
ii - window	3	1.2	0.3	0.1	$L = 1 + 0.1 + 0.1 = 1.2$
iii - shelves	2	1.2	0.3	0.1	

6. Plastering

Out side

① Plinth height including plinth projection and

$H_D = 0.15 + 0.05 + 0.1 = 0.3$

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10 cm below ground level	1	19.1		0.3		L = 2(3.7+7.2) = 21.8 21.8 − 2.7 = 19.1
(ii) Out side of garage	1	21.4		3		L = 2(7.1+3.6) = 21.4
Inside						
i Inside of garage	1	19		3		L = 2x(6.5+3) = 19
(iii) Deduction for						
- gate	1 x 2	2.5		2.3		Deduction is made for 2 faces only
- window	3 x 1	1		1.2		Deduction is made for 1 face only.
7. R.C.C work						
a. 12 cm R.C.C slab over garage	1	7.4	3.9	0.12		L = 6.5+0.3+0.3+0.15+0.15 b = 3+0.3+0.3+0.15+0.15
b. 10 cm R.C.C lintel over						
a- window	3	1.2	0.3	0.1		L = 1+0.1+0.1
b. shelve	2	1.2	0.3	0.1		
c. 20 cm R.C.C lintel over gate	1	2.9	0.3	0.2		L = 2.5+0.2+0.2

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D.	5cm R.C.C ~~lintel~~ slab in shelve	2 x 3	1	0.2	0.05		L = 1 + 0.05 + 0.05
8.	wood working in frame - window	3	4.4	0.1	0.08		L = 2 x (1 + 1.2)
9.	wood working in shutter. - window	3	0.87				L = 1 - 2(0.08) + 2(0.015) b = 1.2 - (2 x 0.08) + (2 x 0.015)

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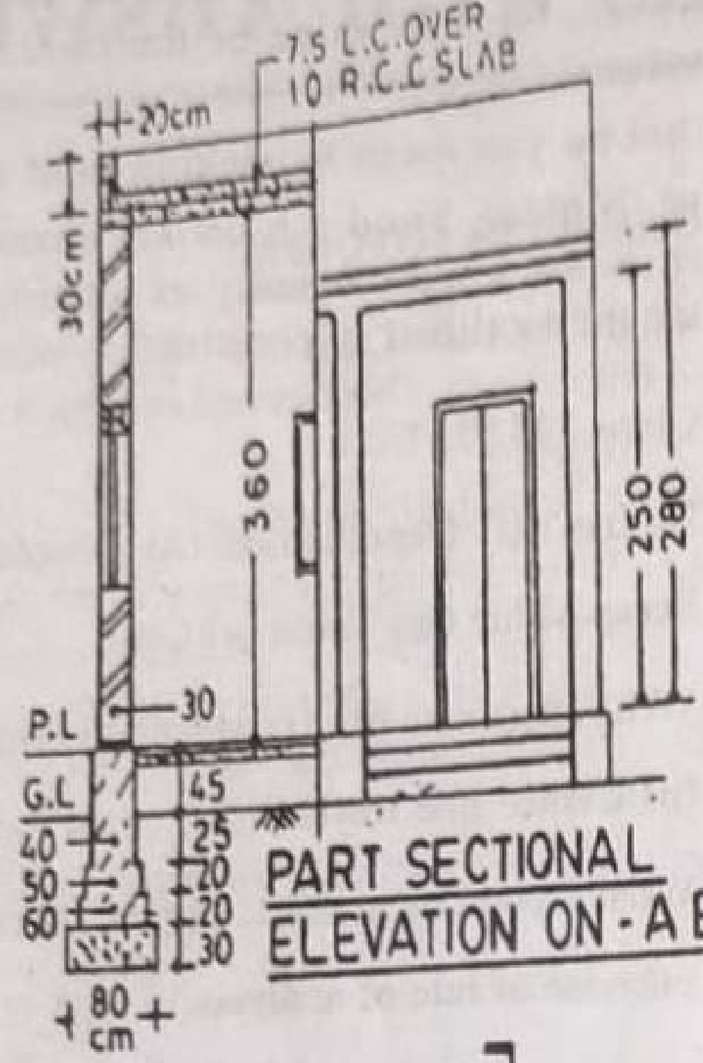
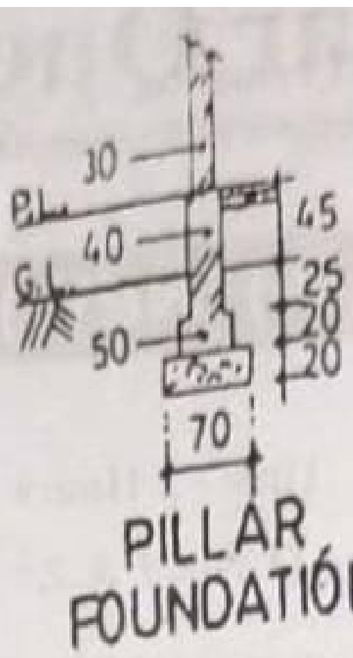


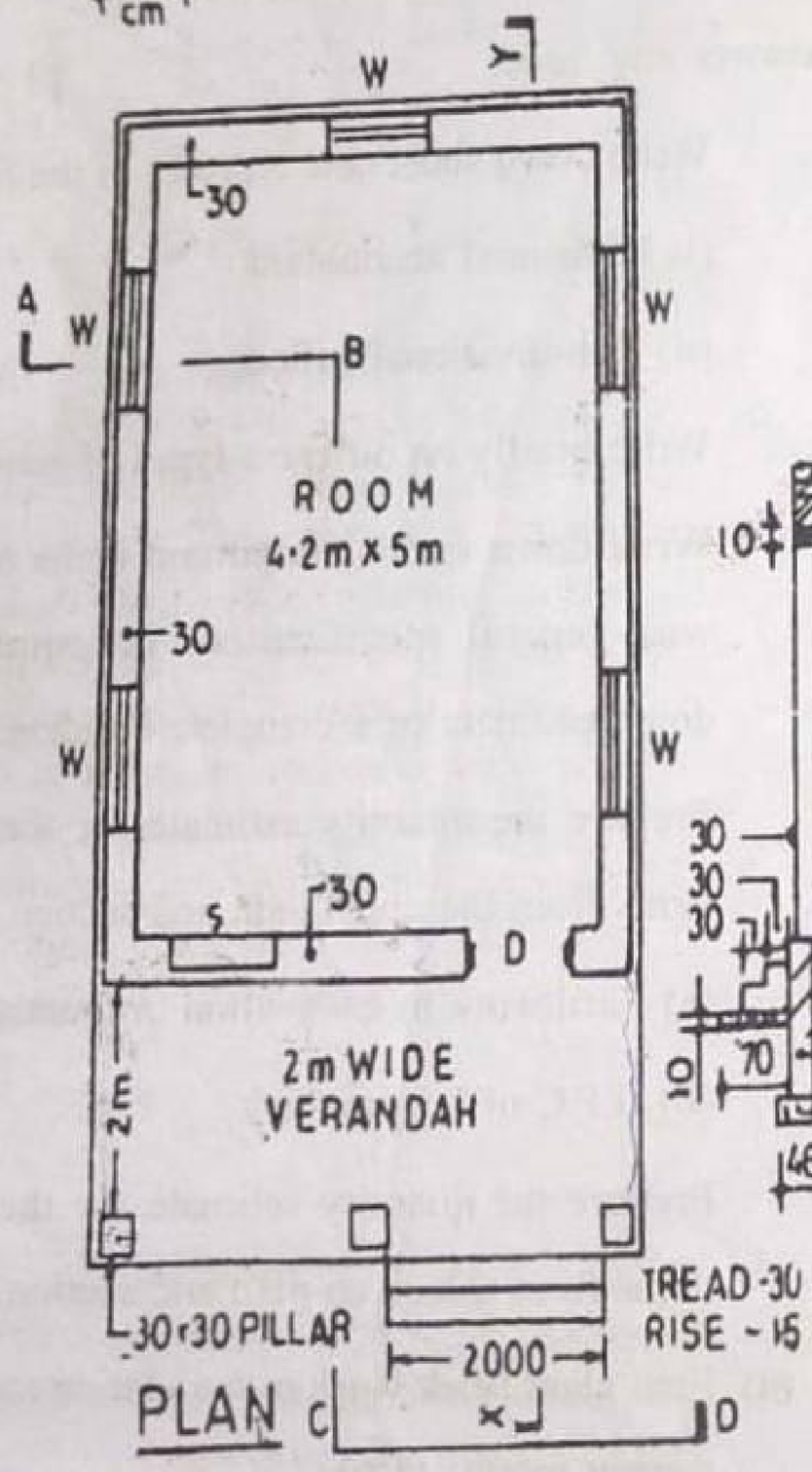
PART SECTIONAL
ELEVATION ON - ABCD

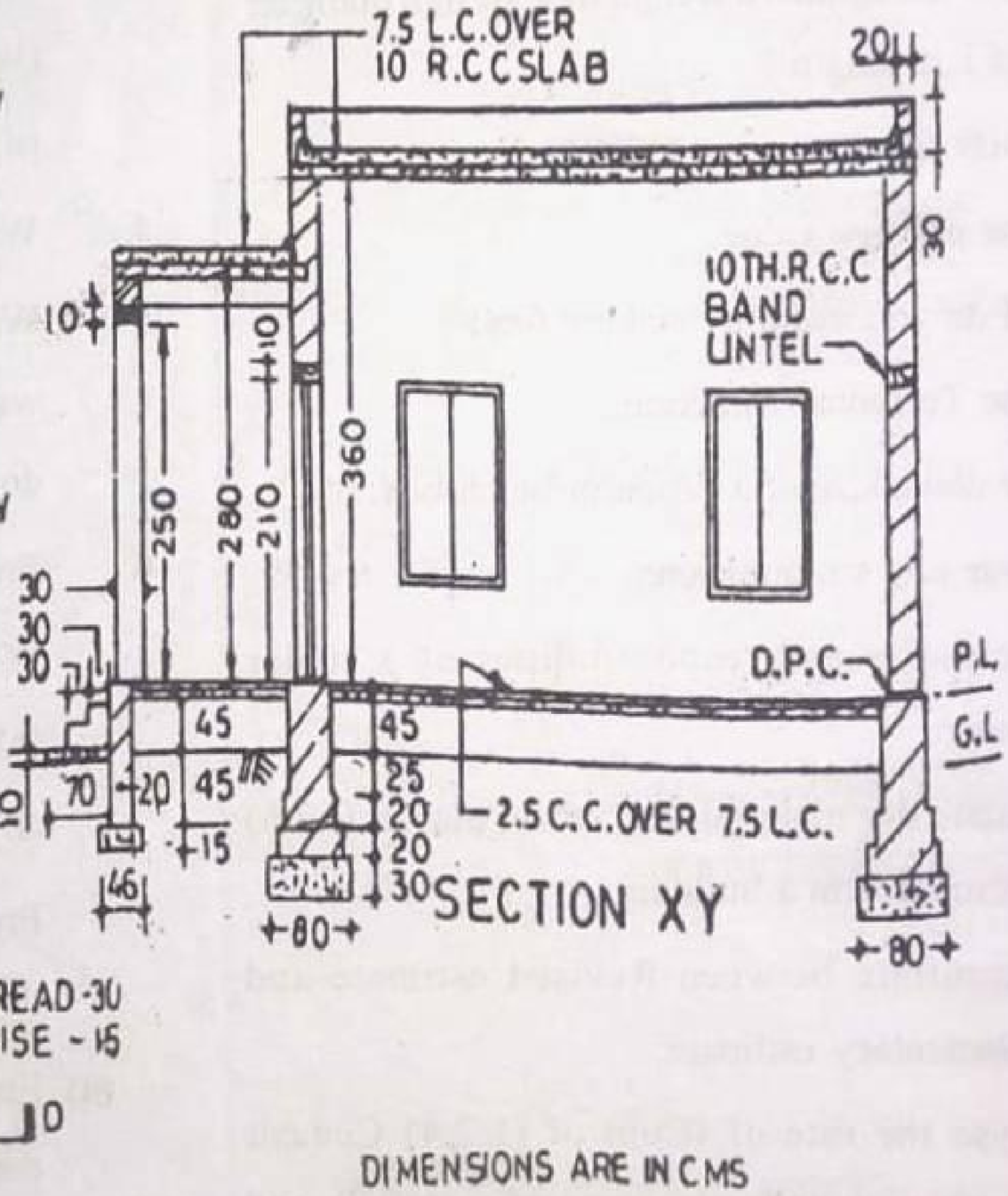


PILLAR
FOUNDATION

SCHEDULE

DOOR D = 110×210
WINDOW W = 90×190
SHELF S = 90×150





SECTION XY

DIMENSIONS ARE IN CMS

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Previous Year Q. 2019

Calculation of center to center length

a Room

Center to center length of long wall

$$= \frac{0.3}{2} + 5 + \frac{0.3}{2} = 5.3$$

Center to center length of short wall

$$= \frac{0.3}{2} + 4.2 + \frac{0.3}{2} = 4.5$$

b. Verandah (Pillar)

center to center length of long wall (Pillars)

$$= \frac{0.3}{2} + 4.2 + \frac{0.3}{2} = 4.5$$

center to center length of short wall (Pillar to wall)

$$= \frac{0.3}{2} + 2 + \frac{0.3}{2} = 2.3$$

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Item No	Particulars of Item	No	length	breadth	height	Quantity	Explanatory Note
1.	Earth work in excavation						
	a. Room						$H = 0.3+0.2+0.2+0.25 = 0.95$
	- long wall	2	6.1	0.8	0.95		$L = 5.3 + \frac{0.8}{2} + \frac{0.8}{2} = 6.1$
	- short wall	2	3.1	0.8	0.95		$L = 4.5 - 2 \times \frac{0.8}{2} = 3.7$
	b. Pillar	3	0.7	0.7	0.65		$H = 0.4 + 0.25 = 0.65$
	c. Verandah						
	- long wall	1	3.1	0.4	0.6		$H = 0.45 + 0.15 = 0.6$
	- short wall	2	1.55	0.4	0.6		$L = 4.5 - \frac{0.7}{2} - 0.7 - \frac{0.7}{2} = 3.1$
	d. step	1	2.2	0.7	0.1		$L = 2.3 - \frac{0.8}{2} - \frac{0.7}{2} = 1.55$
							$l = 2 + 0.1 + 0.1$
							$b = 0.3 + 0.3 + 0.1$
2.	Lime concrete in foundation						
	a. Room						
	- long wall	2	6.1	0.8	0.3		$L = 5.3 + 2 \times \frac{0.8}{2} = 6.1$
	- short wall	2	3.7	0.8	0.3		$L = 4.5 - 2 \times \frac{0.8}{2} = 3.7$

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	b. Pillar	3	0.7	0.7	0.2		
	c. vereandah						
	- long wall	1		0.4	0.15		$L_L = 4.5 - 2\times\frac{0.7}{2} - 0.7 = 3.1$
	- short wall	2		0.4	0.15		$L_S = 2.3 - \frac{0.7}{2} - \frac{0.6}{2} = 1.65$
	d. step	1	2.2	0.7	0.1		
3	Brick work in foundation and plinth						
	a - Room						
	i - first footing						
	- long wall	2	5.9	0.6	0.2		$L_L = 5.3 + 2\times\frac{0.6}{2} = 5.9$
	- short wall	2	3.9	0.6	0.2		$L_S = 4.5 - 2\times\frac{0.6}{2} = 3.9$
	ii 2nd footing						
	- long wall	2	5.8	0.5	0.2		$L_L = 5.3 + 2\times\frac{0.5}{2} =$
	- short wall	2		0.5	0.2		$L_S = 4.5 - 2\times\frac{0.5}{2} =$
	iii - 3rd footing						$H = 0.25 + 0.45 = 0.7$
	- long wall	2		0.4	0.7		$L_L = 5.3 + 2\times\frac{0.4}{2} =$
	- short wall	2		0.4	0.7		$L_S = 4.5 - 2\times\frac{0.4}{2} =$

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5. Pillar					H = 0.45 + 0.25 = 0.7
- first footing	3	0.5	0.5	0.2	
- 2nd footing	3	0.4	0.4	0.7	H = 0.45 + 0.45 = 0.9
c verandah	1		0.2	0.9	
1- 1st 20cm height from bottom					
- long wall	1	3.5	0.2	0.2	L = 4.5 - $\frac{0.5}{2}$ x 2 - 0.5 = 3.5
- short wall	2	1.8	0.2	0.2	L = 2.3 - $\frac{0.5}{2}$ - $\frac{0.5}{2}$ = 1.8
ii 2nd 70 cm height bellow plinth level					
- long wall	1	3.7	0.2	0.7	L = 4.5 - $\frac{0.4}{2}$ x 2 - 0.4 = 3.7
- short wall	2	1.9	0.2	0.7	L = 2.3 - $\frac{0.4}{2}$ - $\frac{0.4}{2}$ = 1.9
d. step					
- First step	1	2	0.6	0.15	
- 2nd step	1	2	0.3	0.15	

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	e. D.P.C work						
	a. Room						
	- long wall	2	5.7	0.4			$L = 5.3 + 2 \times \frac{0.4}{2} = 5.7$
	- Short wall	2	4.1	0.4			$L = 4.5 - 2 \times \frac{0.4}{2} = 4.1$
	b. Pillar	3	0.4	0.4			
	c. Deduction for door.	1	1.1	0.4			
4.	Brick work in Super structure						
	a Room						
	- long wall	2	5.6	0.3	3.6		$L = 5.3 + 2 \times \frac{0.3}{2} = 5.6$
	- short wall	2	4.2	0.3	3.6		$L = 4.5 - 2 \times \frac{0.3}{2} = 4.2$
	b. Pillar	3	0.3	0.3	2.5		
	C. Verandah wall above lintel.						$H = 2.8 - 2.5 - 0.1 = 0.2$
	- long wall	1	4.8	0.3	0.2		$L = 4.5 + 2 \times \frac{0.3}{2} = 4.8$
	- short wall	2	2	0.3	0.2		$L = 2.3 - 2 \times \frac{0.3}{2} = 2$

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4- Parapet around the room						
- long wall	2	5.6	0.2	0.3		$L_l = 5 + 0.3 + 0.3 = 5.6$
- short wall	2	4.4	0.2	0.3		$L_s = 4.2 + 2 \times 0.1 = 4.4$

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Imporetant Note :-

- The drey volume of $1m^3$ of cement concreete = $1.5\ m^3$
- Brick worek -(both superi. structuree and sub structuree)

Standard size of 1 breick = 190 cm x 9 cm x 9 cm

Nominal size of 1 brick = 20 cm x 10 cm x 10 cm

In brick worek volume of mortar = Total volume of brick work - volume(~~quantity~~ of bricks

No of bricks present in $1\ m^3$ of brick work =

= 500

- wet volume of mortar is ~~after~~ increasing 15% for freog filling, brick bonding coarse, westage etc.

Drey volume of mortear = wet volume + 1/3 of wet volume

3. Plastering :- volume of plastering = Areea of plastering x thickness of plastering

- wet volume of mortar is increase 20%. for westage and joint filling etc.
- Drey volume of mortear = wet volume + 1/3rd of wet volume

4. D.P.C -: a. Moretear D.P.C = cement + sand + ~~Dam~~ damp proofing materical

TECNO

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- wet volume of D.P.C = Area of D.P.C x thickness of D.P.C

- Dry volume of D.P.C = wet volume + 1/3rd of wet volume.

- D.P.C material (sealing material) = 2% or 1% of weight of cement (given in question)

5. Concreete D.P.C :- wet volume of D.P.C = Area of D.P.C x thickness of D.P.C

- Dry volume of D.P.C = 1.54x wet of volume.

1 bag of cement = 50kg = $0.034 m^3$

1 m3 of cement = $\frac{1440}{50}$ kg = 29 bag

6. volume of reain forcement for horizontal streeteere like beam, slab, chajja = 2% of total volume of streeteere

7. volume of reainforceement for verticeal streeteere like column = 1% of toteel volume of streeteere.

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1. Calculate the quantity of drey matericel required for 5m3 of cement concrete (1:2:4).

t. Data given

volume of ~~drey~~ wet concrete = $5m^3$

Preperetion of concrete = 1:2:4

we knew drey volume of 1m3 of concrete

= 1.54 m3

So drey volume of concrete = 5 x 1.54 = 7.7 m3

Let volume of cement = x

" " sand = 2x

" " aggregate = 4x

$\Rightarrow x + 2x + 4x = 7.7$

$\Rightarrow 7x = 7.7$

$\Rightarrow x = \frac{7.7}{7} = 1.1\ m^3$

So volume of cement = x = $1.1\ m^3 = 1.1 \times 1440\ kg$

~~sand = 2x = 2x 1.1~~ = 1584kg

$= \frac{1584}{50}$ bag

= 32 bag

sand = 2x = 2 x 1.1 = 2.2 m3

aggregeate = 4x = 4 x 1.1 = 4.4 m3

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:. Calculate the quantity of dry material required for 6m3 of brick work (1:2) (c.s)

Data given

Quantity of brick work = 6m3

Proportion of mortar = 1:2

we know No of bricks present in 1m3 of brick work = 500

so here total no of bricks = 6 × 500 = 3000 No

volume of 1 brick = 0.19 × 0.09 × 0.09

= 1.539 × 10^-3 m3

Total volume of bricks = 3000 × 1.539 × 10^-3

= 4.617 m3

wet volume of mortar = 6 - 4.617 = 1.383 m3

Increasing 15% for fraog filling, wastage and brick bonding coarse, wet volume of mortar = 1.383 + 15/100 × 1.383

= 1.59045

Dry volume of mortar = 1.59045 + 1/3 × 1.59045

= 2.1206

Let volume of cement = x

sand = 2x

⇒ x + 2x = 2.1206

⇒ 3x = 2.1206

⇒ x = 2.1206/3 = 0.706

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So volume of cement = x 0.706

Sand = $2x$ = 2×0.706

= 1.4137 m^3

Cement = x = 0.706×1440 kg

= $\frac{1016.64}{50}$ bag

= 21 bag

③ Calculate the quantity of dry materials required for 200m^2 of 12mm thick cement plastered. (1:4)

Soln

Data given

Area of plastering = 200m^2

Thickness of plastering = 12mm = 0.12m

Proportion of mortar = 1:4

Volume of plastering = Area of plastering × thickness of plastering

= 200×0.12

= 2.4 m^3

Increase 20% for wastage and joint filling

= $2.4 + \frac{20}{100} \times 2.4$

= 2.88 m^3

Dry volume mortar = $2.88 + \frac{1}{3} \times 2.88$

= 3.84 m^3

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Let the volume of cement = x

sand = 4x

$\Rightarrow x + 4x = 3.84$

$\Rightarrow 5x = 3.84$

$\Rightarrow x = \frac{3.84}{5} = 0.768\ m^3$

volume of cement $x = 0.768\ m^3$

$= 0.768 \times 1440$ kg

$= \frac{1105.92}{50}$ bags

$= 23$ bags

volume of Sand $4x = 4 \times 0.768 = 3.072\ m^3$

(4) Calculate the quantity of dry material required for $150m^2$ of 2 cm D.P.C (1:3)

Sol^n

Data given

Area of D.P.C $= 150m^2$

thickness of D.P.C $= 2m = 0.02mm$

volume of D.P.C $= 150 \times 0.02$

$= 3\ m^3$

Dry volume mortar $= 3 + \frac{1}{3} \times 3$

$= 4\ m^3$

Let the volume of cement $= x$

Sand $= 3x$

$\Rightarrow x + 3x = 4\ m^3$

$\Rightarrow 4x = 4$

$\Rightarrow x = 1\ m^3$

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volume of cement $x = 1$

$= 1 \times 1440$ kg

$= \frac{1440}{50}$ bag

$= 29$ bag

volume of sand $= 3x = 3 \times 1 = 3\ m^3$

— Quantity of sealing material @ at the rate 1% of

weight of cement $= \frac{1}{100} \times 1440 = 14.4$ kg

(5) Calculate the quantity of ~~500m²~~ dry materials for 500m² 2.5 cm D.P.C (1:3:6)

A. Given data

Area of D.P.C = 500 m²

thickness of D.P.C = 2.5 cm = 0.025 mm

Volume of D.P.C = 12.5 m3

Dry volume of ~~concrete~~ concrete = 12.5×1.54

$= 19.25\ m^3$

let the volume of cement $= x$

sand $= 3x$

aggregate $= 6x$

$\Rightarrow x + 3x + 6x = 19.25$

$\Rightarrow 10x = 19.25$

$\Rightarrow x = \frac{19.25}{10} = 1.925$

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volume of cement $x = 1.925$

$= 1.925 \times 1440$ kg

$= \frac{2772}{50}$ bag

$= 56$ bag

sand $3x = 3 \times 1.925 = 5.775\ m^3$

aggreegate $6x = 6 \times 1.925 = 11.55\ m^3$

Quantity of sealing material at the rate 1% of

weight of cement $= \frac{1.925}{100} \times 1440$

$= 27.72$ kg.

6. Calculate the quantity of drey materials required for $5\ m^3$ of R.C.C slab (1:2:4)

Data given

~~Area of~~ volume of R.C.C $= 5\ m^3$

Drey volume of concrete $= 5 \times 1.54$

$= 7.7\ m^3$

let volume of cement $= x$

sand $= 2x$

aggregate $= 4x$

$\Rightarrow x + 2x + 4x = 7.7$

$\Rightarrow 7x = 7.7$

$\Rightarrow x = 1.1$

volume of cement $= 1.1$

$= 1.1 \times 1440$ kg

$= 1584$

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$= \frac{1584}{50}$ bag $= 32$ bag

sand $= 2x = 2 \times 1.1 = 2.2\ m^3$

aggregate $= 4x = 4 \times 1 = 1 = 4.4\ m^3$

Quantity of reinforcement at the reate @2% o

total volume $= \frac{2}{100} \times 5 = 0.1\ m^3$

we know density of steel $= 7850\ kg/m^3$

so total wt. of reinforcement $= 0.1\ m^3 \times$

$7850 = 785$

$= 7.85$

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ANALYSIS OF RATES, RATES OF MATERIALS AND LABOUR CURRENT IN LUCKNOW DURING 2016

Particulars	Rate at Site
Materials :-	
① Brick II class	Rs-8000/- %oo Nos
② Brick II class	Rs-7000/- %oo Nos
③ Brick Ballast (morra) 40mm gauge	Rs-1000/- Per cu.m
④ Brick ballast 25mm	Rs:-1100/- Per cu.m
⑤ stone ballast 40mm	Rs-2400/- Per cu.m
⑥ stone ballast 20mm	Rs-2400/- Per cu.m
⑦ stone ballast 12 mm	Rs-1900/- Per cu.m
⑧ stone ballast 8 mm	Rs-1500/- Per cu.m
⑨ cement	Rs-330/- Per bag
⑩ steel	Rs-4200/- Per q
⑪ white orc stone lime (unslaked)	Rs-1000/- Per q cu.m
⑫ white orc stone lime (slaked)	Rs-1000/- Per cu.m
⑬ surkhi	Rs-800/- Per cu.m
⑭ sand fine (local)	Rs-1500/- Per cu.m
⑮ sand coarse (maircang)	Rs-1800/- Per cu.m
⑯ Teak wood	Rs-50000/- Per cu.m
⑰ Shisham wood	Rs-35000/- Per cu.m
⑱ [illegible]	Rs-50000/- Per cu.m

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Particulars	Rates
LABOUR	
① Head mason (mistry)	RS. 425/- Per day
② Mason	Rs. 400/- Per day
③ Maz door (Belder)	Rs. 250/- Per day
④ Bay or woman coolie	Rs. 230/- per day
⑤ Bishti	Rs. 230/- Per day
⑥ carpenter	Rs. 400/- Per day
⑦ Blacksmith	Rs. 375/- Per day
⑧ Painter	Rs. 375/- Per day

NO OF LABOUR

1. Lime concrete in foundation with 40mm gauge brick ballast unit 1 cu.m. (Quantity - $10m^3$)

Labourr :-

Mistri (Head mason)	–	1/2 Nos
Mason	–	1 Nos
Maz door (Belder)	–	12 Nos
Bay or woman coolie	–	12 Nos
Bhisti (waterr-man)	–	2 Nos
Sundries T. and P. etc (misc petty thwings)	–	Lump sum.

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2. Cement concrete 1:2:4 - unit 1 cu m. Take 10 cu.m

Labour :- (quantity - 10m3)

Mestri (Head mason) — 1/3 no

Mason — 2 nos

Mazdoor (Beldar) — 12 nos

Bay or women coolie — 20 nos

Bhishti (including curings) - 6 nos

Forems etc. (according to requirement) - Lump-sum.

sundries T. and P. etc. — Lump sum.

3. I class Brick work in foundation and plinth with 20 x 10 x 10 cm (nominal size) Bricks with cement sand mortar 1:6 - unit 1 cu.m Take - 10 cu.m.

Labour :- (Quantity = 10m3)

Mistri (Head mason) — 1/2 no

Mason — 7 nos

Mazdoor (Beldar) - 7nos

Bay or women coolie - 7 nos

Bhishti (waterc man) - 2nos

Sundries .T. and p etc

misc petty things — Lump sum.

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4. I class Brick work en superstructure with 20x10x10 Brick with 1:6 cement sand moretar - unit 1 cu.m

Take - 10 cu.m.

Labour :- Quantity ($10 m^3$)

Mistrei (Head mason) - 1/2 no

Mason - 10 nos

Mazdoor (Belder) - 7 nos

Boy or woman coolie - 10 nos

Bhishti - 2 nos

Scaffolding - lump sum

Sundries T. and P etc - Lump sum

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5. 12mm Plastering 1:6 - unit 1 sqm - Take 100 sqm.

Labour :-

Mistri (Head mason) - 1/3 no

Mason - 10 nos

Mazdoor (Belder) including

Making of joints - 15 nos

Bhishti including (curing) - 3/4 nos

Scaffolding sundries T. and P. etc - Lumpsum

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(7). Calculate the quantity of dry materials required for 200m² of 7.5cm lime terracing (1:1.5:3)
(c.s.a)

Soln

Data given

Area of lime terracing = 200m²

thickness of " = 7.5cm = 0.075

Volume of lime concrete = 200 × 7.5 = 1500 m³
= 15 m³

Dry volume of lime concret
= 15 × 1.54
= 23.1 m³

Let the volume of ~~cement~~ lime = x

Sand = 1.5x

aggregate = 3x

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$= x + 1.5x + 3x = 23.1$

$= 5.5x = 23.1$

$= x = \frac{23.1}{5.5} = 4.2\ m^3$

sand $= 1.5x = 1.5 \times 4.2 = 6.3\ m^3$

aggregate $= 3x = 3 \times 4.2 = 4.5\ m^3$

Rate Analysis

The calculation of cost both material and labour is known as rate analysis. ~~steeedule~~

- It is done according to odicha P.W.D schedule of rate.
- The total cost is calculated by multiplying the quantity with unit rate

Purpose of Rate Analysis :-

- To determine the current rate per unit of an item at the locality.
- To determine the variability of rates offered by the contractor.
- To fix up the labour contract rate.
- To know the cost of structure and the amount to be sanctioned.
- To calculate the strength of labour and material required for the project.

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27.12.21

(1) Calculate the cost of construction of $15m^3$ of cement concrete in foundation (1:2:4) :

A. Data given

Quantity of cement concrete = $15m^3$

Properetion = 1:2:4

Calculation of drey material ($10m^3$)

Drey volume $10m^3$ of concrete = $1.54 \times 10 = 15.4 m^3$

Let the volume of cement = x

sand = $2x$

aggregeate = $4x$

$\Rightarrow \quad x + 2x + 4x = 15.4$

$\Rightarrow \quad 7x = 15.4$

$\Rightarrow \quad x = \frac{15.4}{7} = 2.2$

cement = x = 2.2

sand = $2x$ = $2 \times 2.2 = 4.4$

aggregate = $4x$ = $4 \times 2.2 = 8.8$

cement = 2.2×1440

= $\frac{3168}{50}$ bag

= 64 bag

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Item No	Types of material or Labour	Quantity	Rate	unit	Cost
1	Material				
	Cement	64	330	bag	21120
	Sand	4.4	1500	m^3	6600
	Aggregate	8.8	2400	m^3	21120
2.	Labour				
	Head Mason	$\frac{1}{3}$	425	No	141.66
	Mason	2	400	No	800
	Mazdoor	12	250	No	3000
	Boy or women collies	20	230	No	4600
	Bhesti	6	230	No	1380
	Sundries T & P etc	—	—	—	200
	Forms etc.				150
				Total	59111.67
	adding 1½ % of water charge			Total 65909.506	$59111.67 \times \frac{1.5}{100}$ = 886.67
	adding 10 % as contractor profit				$5914.67 \times \frac{10}{100}$ = 5911.16

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cost for $10 m^3$ of cement concrete = ₹5909.50

cost for $1 m^3$ of cement concrete = $\frac{₹5909.50}{10}$

= 590.95

cost FO $15 m^3$ of cement concrete = 590.95 × 15

= 8864.25 Rs

2. Calculate the cost of construction of $200 m^2$ of 12mm cement plaster (1:6)

A

Data given

Quantity of cement plaster = $200 m^2$

Preperation = 1:6

Calculation of drey material = $100 m^2$

• volume of concrete = 1.200 × 0.012

= 1.2m3

Increase 20% = $1.2 + \frac{20}{100} \times 1.2 = 1.44 m^3$

Drey volume mortare = $1.44 + \frac{1}{3} \times 1.44$

= $1.92 m^3$

Let the volume of = x

Sand = 6x

$\Rightarrow x + 6x = 1.92$

$\Rightarrow 7x = 1.92$

$\Rightarrow x = \frac{1.92}{7} = 0.27$

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Cement $x = 0.27$

$= 0.27 \times 1440$

$= \frac{388.8}{50}$ bag

$= 8$ bag

Sand $= 6x = 6 \times 0.27 = 1.62\ m^3$

Item NO	Types of material or labour	Quantity	Rate	unit	cost
1.	Material				
	Cement	8	330	bag	2640
	Sand	1.62	1500	m^3	2430
2.	Labour				
	Head mason	1/3	425	Nos	141.66
	Mason	10	400	Nos	4000
	Mazdoor	15	280	No	3750
	~~Joints~~				
	Bhishti including	3/4	230	No	172.5
	scaffolding				200
	sundries				
	T and P etc			Total	13334.16
Adding 1 1/2 % as ~~water~~ charge		as water charge =			$13334.16 \times \frac{1.5}{100}$
					$= 200.0124$
Adding 10%		as - - -		=	$13334.16 \times \frac{10}{100}$
					~~$= 20.00224$~~
					$= 1333.41$
				Total	~~13554.17~~
					14867.58

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cost for 100m³ of cement concrete = 14867.58

cost for 1 m³ of " " = $\frac{14867.58}{100}$

= 148.67

cost for 200m³ of " " =

= 148.67 × 200

= 29735.16 /—

29.12.21

1. Calculate the cost of construction of 12m3 of brickwork (1:4) in super structure.

A

Data given

Quantity = 12 m³

Proportion = 1 : 4

Calculation For Dry material (10m³)

~~volume of the cement =~~

we know the no of bricks present in 1m³

= 500

So here no of bricks = 10 × 500 = 5000 Nos

volume of 1 brick = 0.19 × 0.09 × 0.09

= 1.539×10^{-3} m3

Total volume of brick = $5000 \times 1.539 \times 10^{-3}$

= 7.695 m³

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wet volume of mortar = 10 - 7.695 = 2.305 m^3

Increase 15% = $2.305 + \frac{15}{100} \times 2.305$

= 2.65075

Dry volume of mortal = $2.\frac{650}{305} + \frac{1}{3} \times 2.\frac{650}{305}$

= 3.53

Let the volume of cement = x

sand = 4x

$\Rightarrow x + 4x = 3.53$

$\Rightarrow 5x = 3.53$

$\Rightarrow x = \frac{3.53}{5} = 0.706$

$= 0.706 \times 1440$

$= \frac{1017.6}{50}$ bag

= 21 bag.

Sand 4x = $4 \times 0.706 = 2.824\ m^3$

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= 27145.92

1.	Material				
	cement	21	330		6930
	sand	2.824	1500		4236
	brick	5000	8		40000
2.	Mistri	1/2	425		212.5
	Mason	10	400		4000
	Mazdoor	7	250		1750
	Boy or women	10	230		2300
	Bhishti	2	230		460
	Scaffolding				200
	sundries				200
					60288.5

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Adding $1\frac{1}{2}$ as water = $60288.5 \times \frac{1.5}{100} = 904.32$

Adding 10% as contractor's profit = $60288.5 \times \frac{10}{100}$ =

= 6028.85

cost for 10 m^3 of cement concrete = 67221.67

" " 1 m^3 " " " = $\frac{67221.67}{10}$

= 6722.167

" " 12 m3 " " " = 6722.167 x 12

= 80666.

② Calculate the cost of construction of 8 m^3 of R.C.C beam (1:2:4)

A Give data

Quantity = 8 m^3

Proportion = (1:2:4)

Dry volume of concret = 10 x 1.54

= 15.4 m^3

let the volume of cement = x

sand = 2x

aggregate = 4x

$\Rightarrow$ x + 2x + 4x = 15.4 m3

$\Rightarrow$ 7x = 15.4

$\Rightarrow$ x = $\frac{15.4}{7}$ = 2.2

= 2.2 x 1440

= $\frac{3168}{50}$ = 64 bag

= 64 bag

Scanned by TapScanner

Sand $2x = 2 \times 1.76 = 3.52$

aggregate $4x = 4 \times 1.76 = 7.04$

Quantity of reinforcement @ 2% of total

$$\text{volume} = \frac{2}{100} \times 10 = 0.2\ m^3$$

We know density of scabe $= 7850\ kg/m^3$

so total wt. of reinforcement $= 0.2 \times 7850$

$= 1570\ kg$

$= 1570/100\ q$

$= 15.7\ q$

Item No	Types of material or labour	Quantity	Rate	Unit	cost
1.	Material				
	Cement	64	330	bag	21120
	Sand	4.4	1500	m^3	6600
	Aggregate	8.8	2400	m^3	21120
	Reinforcement	15.7	4200	q	65940
2.	Labour				
	Mistri	1/3	425		141.66
	Masoon	3	400		1200
	Mazdoor	12	250		3000
	Boy or woman Kully	20	230		4600
	Bhichti	6	230		1380
	Sundries P and T etc	lumpsum	200		200
	Bending Cranking and binding of				

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steel bas in position				
Blacksmith	8	375-		3000
Mazdoor	8	250		2000
Lumpsum/ Sundries	—	200		200
centering and shuttering				
Timber planks	—			1000
carpainter	10	400		4000
Mazdoor	10	250		2500
Nail	—	100		100
Tand P		200		200
			Total	138301.66

Adding $1\frac{1}{2}$ % as water charge = $138301.66 \times \frac{1.5}{100}$

= 2074.52

Adding 10 % as contractors charge = $138301.66 \times \frac{10}{100}$

= 13830.166

Total cost = 138301.66 + 2074.52 + 13830.166

= 154206.346 m³

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Cost of $1 m^3$ of cement plaster = 154206.346 m^2

" " $1 m^3$ " " " = $\frac{154206.346}{10}$

= 15420.6346

cost of $8 m^3$ " " " = 15420.6346 × 8

= 123365.0768.

Valuation

Introduction -: Valuation is the process of estimating or determining the faire price or value of a property such building, other engineering structure etc. The value also depends on supply of demand and purpose for which valuation is required.

- The cost means original cost of constructions. But value means present value.

Different values of a structure :-

The different values of a structure of a structure are as follows.

1. Scrap value
2. Salvage value
3. Market value
4. Book value
5. Assessed value.

Purpose of valuation

- Buying or selling property.
- Taxation
- Rent fixation
- Security of Loans
- Compulsory acquisition

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Scrap Value :-

scrap value is the value of dismanted material. For a building when the life is over at the end of its utility period, the dismantled material such as steel, bricks timber will fetch a certain amount which is the scarp value of the building.

The scareap value of a building is about 10 percent of its total cost of construction.

- The cost of dismantling and removal of the rubbish material is deducted from the total receipt from the sale of the useable materials to get the scarp value.

Salvage Value :-

- It is the value at the end of the utility period without being dismantled. A machine after the completion of its usual span of life or when it become uneconomical, may be sold, and one may purchase the same for use for some other purpose. The sale value of a machine is its salvage value. It does not include the cost of removal.
- The ~~scrap~~ salvage value of a RCC structure will be negative as the removal and dismantling is costly.

Asset Value :-

- For the purpose of taxation, a property is assessed for its monetary worth. This ascertained price is known as assessed value.
- In general the assessed value tends to be less than the fair or actual market price of the property.
- This is normally done by govt departments.

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Asset Value -:

- For the purpose of taxation, a property is assessed for its monetary worth. This ascertained price is known as assessed value.
- In general the assessed value tends to be less than the fair or actual market price of the property.
- This is normally done by govt. department.

Depreciation :-

- Depreciation is the gradual decrease of usefulness of a property. This may be defined as the decrease or loss in the value of a property due to structural deterioration use, life wear and tear, decay and obsolescence.
- Usually a percentage on depreciation per annum is allowed. The general annual decrease in the value of property is known as annual depreciation.
- Usually the percentage rate of depreciation is less at beginning and gradually increase during later year.

Obsolescence :-

- Obsolescence is the cause due to which depreciation occurs.
- Obsolescence may be due to the reason such as progress in arts, changes in fashion, changes in planning ideal new, inventions, improvement in design techniques.

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Method of Valuation :-

The different method used for valuation of a structure are :

- Rental method of valuation
- Direct comparision with the capital value
- valuation based on profit
- valuation based on cost
- Development method of valuation
- Depreciation method of valuation.

ADMINISTRATIVE SET UP OF ENGINEERING ORGANISATION

Duties And Liabilities of Owner :-

- To appoint an engineer and give him power to work on his behalf.
- To intimate the engineer required for the project including his desired time of completion.
- To obtain necessary sanction for his construction from competent authority.
- To give possession of the site to the contractor.
- To make payment to the contractor on production of certified bill from the engineer.
- To take over possession of the completed projects timely from the contractor.
- In case of conflict (problem) with the contractor, he will appoint lawer for defending his side.

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DUTIES OF CONTRACTOR -:

- To inspect the site and study soil conditions before tendering. He should investigate the availability, accessibility of electric power, water supply etc.
- he should collect the local rates of material, labour and accordingly prepare the analysis of rates of all the items.
- It is duty and liability of the contractor to follow the labour act truly.
- The contractor is liable to safeguard his own men and material issued to him.
- The contractor should submit his claims for extra works and submit bills for payment in due time.
- It is the responsibility of contractor to hand over the completed work in sound condition.

Duties of Engineer :-

- To prepare the necessary drawing, specifications and estimate in accordance with the requirement of the owner.
- To check up the soil condition.
- To supervise the work and ensure that the drawing and specifications are faithfully followed.
- To check up the progress of work with passage of time and submit progress report to the owner.
- To check the quality of work, measurment of works done, quantities, rates and pass the bill for payment.
- To notify the owner about the progress of work, if the work progress very slowly.
- To ensure that no damage is being made on any part of the completed work at the time of handing over the same to the owner.

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Duties of ~~Executive~~ Engineer :-

- To prepare the necessary drawing, specifications and estimate in accordance with the requirement of the owner.
- To check up the soil ~~owner~~. condition.
- To supervise the work and ensure that the drawing and specifications are faithfully followed.
- To check up the progress of work with passage of time and submit progress report to the owner.
- To check the quality of work, measurement of works done, quantities, rates and pass the bill for payment.
- To notify the owner about the progress of work, if the work progress very slowly.
- To ensure that no damage is being made on any part of the completed work at the time of handling over the same to the owner.

Duties OF Executive Engineer :-

- To organize and supervise the execution of works and to see that they are economically and suitably carried out with specified quality of material.
- To prepare estimate of proposed works through his sub-ordinates and submit the same to the super-intending engineer.
- To invite tenders for works valued under his power.
- To inform the probability of excess of actual over estimated cost of work to his superintending engineer before execution of work.
- To check that the accounts are posted from day to day and that the accountant carries his duty regularly and punctually.
- To prevent ~~any~~ encroachment on govt. lands under his

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- To inspect works and checks measurement of at least 10% of works according to the standing rule.

Duties of Assistant Engineer or Sub-divisional Officer:-

- Efficient management and execution of works within his jurisdiction.
- To maintain the initial account records of cash and stores under his charge.
- To ensure all correct account returns and submit it punctually to the divisional officer.
- To check a certain percentage (at least 25%) of measurements recorded in the measurement book.
- To keep a control over the expenditure against the sanctioned estimate and to report monthly progress work.
- to check stores at least twice a year and tools and plants once in a year.
- To modify the tools, plants and labors whenever required.

Duties of sectional officer / sub-assistant Engineer:-

- To supervise the day-to-day progress of works under his control. To check up whether the materials, proportion of mix, details of item, workmanship etc. are provided as per specification.
- To take detailed measurement of works during progress and enter the same in the measurement book and prepare timely bill for payment.
- To maintain accounts of materials, tools and plants issued for the work and to make timely recovery of the same from the bill of contractors.

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To draw the attention of his assistant engineer in charge for any irregularities of contract, specifications, shotage of supply of departmental material or any other difficulties.

To maintain accounts of material, tools and plants, labour etc.

Duties of Divisional Accountant:-

To assist divicional officers in the discharge of their responsibilities.

- compile the accounts of the division with the prescribed rules and from the date furnished to him.
- To apply certain preliminarey checks to the internal accounts, voucher etc.
- Advice the divisional officer in all matters relating to account and budget estimates.
- To see the comperative statements correctly and incorporate the totals are checked on individual tenders.

Lead and Lift:-

Normally earth work is estimated for 30 m horizontal distance and 1.5m vertical distance or height. The horizontal distance of 30m is known as lead and the vertical distance of earth work is for 30m lead and 1.5m lift. For greater horizontal distance (30m) and greater vertical distance (1.5m) the rate of earth work is different.

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Sundries And Overhead Charges -:

SUNDRIES -: It is the item of work can't be measured but it is required in the work at site. A lumpsum amount is kept as provision to meet sundry expenditure.

OVERHEAD CHARGES -: It includes general office expenses, rent, tax, purchase of stationary, printing of papers, telephone bill, electric bill and postage etc. This is an indirect expence.

Diffrent Govt. And Public Sectors Employing Civil Diploma Holdem -:

- Govt organizations employing civil diploma holders in odisha are - (i) Opsc, Ssc
  (ii) Housing Board Organization
  (iii) PWD, Irrigation department, NHPC & B department, RWSS etc.
- Public sectors oreganizations employing civil diploma holders en odisha are -
  (i) Indian oil
  (ii) JINDAL, TATA, ONGC, OPTCL, NALCO, SAIL, HAL etc.

Explain the terem A.R ESTIMATE -:

Annual repair estimate is a detailed estimate and is prepared to maintain the streucture or work in proper order and safe condition. For building, thes includes white washing colour washing, painting and minor repaires etc. For road it includes patch reparing, renewals repaire of bridge and culvert.

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UNIT RATE ESTIMATE -:

In this estimate the unit reate of all materials and labors aree noted. Then this unit reate is multiplied with quantity of item orr labor to calculate cost of all items.

→ List out the Enclosurees / Attachments to be submitted with an ESTIMATE -:

- Detailed measurement
- Rate analysis forr material and labor
- Labor reate chart
- Proposed dreawing including plan

→ Mention the different types of inclined roofs available -:

- Lean to roof
- Gable roof
- Hip roof
- Gambreel roof
- Mansard orr curb roof
- Deck roof

→ Specify size of Nominal and Traditeonal Bricks.

Types of brick	Actual size	Nominal size
Standard brick	19cm x 9cm x 9cm	20cm x 10cm x 10cm
Traditional brick	22.9cm x 11.2cm x 7cm	22.9cm x 11.4 cm x 7.6cm

→ Administrative aprovall orr sanction -:

- Forr any work orr project requirred by a departtment, an approval orr sanction of the competent authority of the department with reespect to the cost orr work is necessary at the firest instance.
- It denotes the foremal acceptance by the departtment concerned of the preposal and after administrative approval is given the engineering departtment teake up the work and prepares details, designs, plans, estimates

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and then executes the work.

TECHNICAL SANCTION:

- Technical sanction means the sanction of detailed estimate design calculations, quantities of work by the competent authority of the engineering department
- After the technical sanction of the estimate is given, then only the work is teaken up for construction.
- In case of original work, the counter signature of the local head of the department should be obtained in the plan and estimate, before technical sanction is accorded by the engineering department.
- The power of technical sanction differs from state to state.

HEIRACHY OF ENGINEERING DEPARTMENT

Heirachy of Engineering

P.W.D. (Ex-OPWD)

(Adminstrative head) → Chief Engineer (odisha circle)

- Regional cheif Engg.
- Additional chief Engg.
- Deputy cheif Engg.

(East / west circles) = superintending Engineer (S.E) / surveyor of work

Division = Executive Engineer (EE) / Divisional Enggineer

Sub-division = Assistant Engineer (A.E) / sub-divisional Engineer (S.D.O)

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↓

Panchayat / block / section = over seer / sectional officer /
Junior Engg.
↳ work supervisor.

→ write the size of cement batch box.

- 400mm x 300mm x 300mm.

03.01.22

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REFERNCES BOOKS:

Sl. NO.	Name of Authors	Titles of Book	Name of Publisher
1.	M.Chakraborty B.N.Dutta Estimating &Costing	Estimating, Costing, specification &Valuation in Civil Engineering	Published by author
2.	B.N.Dutta	Estimating &Costing	UBSPD
3.	A. Panigrahi	Accounts & contracts	Vikas Publication
4.	Govt. of Odisha	Latest Orissa PWD Schedule of Rates & Analysis of rates	Govt. of Odisha

ENVIROMENTAL STUDIES

SPINTRONIC TECHNOLOGY & ADVANCE RESEARCH (STAR)

DEPARTMENT OF CIVIL ENGINEERING

3RD SEMESTER

DIPLOMA

LECTURE NOTES ON

ENVIROMENTAL STUDIES

BY

SASMITA SAHOO

ENVIRONMENTAL STUDIES

(Common to all Branches of Engg.)

BST-501

Period/Week: 05
Total Periods: 75

Total Marks: 100
Theory End Exams: 70; CT (20) +IA (10)

Rationale:

Due to various aspects of human developments including the demand of different kinds of technological innovations, most people have been forgetting that, the Environment in which they are living is to be maintained under various living standards for the preservation of better health. The degradation of environment due to industrial growth is very much alarming due to environmental pollution beyond permissible limits in respect of air, water industrial waste, noise etc. Therefore, the subject of Environmental Studies to be learnt by every Engineering student in order to take care of the environmental aspect in each and every activity in the best possible manner.

OBJECTIVES:

After completion of study of environmental studies, the student will be able to:

1. Gather adequate knowledge of different pollutants, their sources and shall be aware of solid waste management systems and hazardous waste and their effects.
2. Develop awareness towards preservation of environment.

Unit 1: The Multidisciplinary nature of environmental studies (04 periods)

Definition, scope and importance, Need for public awareness.

Unit 2: Natural Resources (12 periods)

Renewable and non renewable resources:

a) Natural resources and associated problems.
   - Forest resources: Use and over-exploitation, deforestation, case studies, Timber extraction mining, dams and their effects on forests and tribal people.
   - Water resources: Use and over-utilization of surface and ground water, floods, drought, conflicts over water, dam's benefits and problems.
   - Mineral Resources: Use and exploitation, environmental effects of extracting and using mineral resources.
   - Food Resources: World food problems, changes caused by agriculture and over grazing, effects of modern agriculture, fertilizers- pesticides problems, water logging, salinity, .
   - Energy Resources: Growing energy need, renewable and non-renewable energy sources, use of alternate energy sources, case studies.
   - Land Resources: Land as a resource, land degradation, man induces land slides, soil erosion, and desertification.

b) Role of individual in conservation of natural resources.

c) Equitable use of resources for sustainable life styles.

Unit 3: Systems (12 periods)

- Concept of an eco system.
- Structure and function of an eco system.
- Producers, consumers, decomposers.
- Energy flow in the eco systems.
- Ecological succession.
- Food chains, food webs and ecological pyramids.
- Introduction, types, characteristic features, structure and function of the following eco system:
- Forest ecosystem:
- Aquatic eco systems (ponds, streams, lakes, rivers, oceans, estuaries).

Unit 4: Biodiversity and it's Conservation **(08 periods)**

- Introduction-Definition: genetics, species and ecosystem diversity.
- Biogeographically classification of India.
- Value of biodiversity: consumptive use, productive use, social ethical, aesthetic and optin values.
- Biodiversity at global, national and local level.
- Threats to biodiversity: Habitats loss, poaching of wild life, man wildlife conflicts.

Unit 5: Environmental Pollution. **(18 periods)**

Definition Causes, effects and control measures of:

a) Air pollution.
b) Water pollution.
c) Soil pollution
d) Marine pollution
e) Noise pollution.
f) Thermal pollution
g) Nuclear hazards.

Solid waste Management: Causes, effects and control measures of urban and industrial wastes.
Role of an individual in prevention of pollution.
Disaster management: Floods, earth quake, cyclone and land slides.

Unit 6: Social issues and the Environment **(12 periods)**

- Form unsustainable to sustainable development.
- Urban problems related to energy.
- Water conservation, rain water harvesting, water shed management.
- Resettlement and rehabilitation of people; its problems nd concern.
- Environmental ethics: issue and possible solutions.
- Climate change, global warming, acid rain, ozone layer depletion, nuclear accidents and holocaust, case studies.
- Air (prevention and control of pollution) Act.
- Water (prevention and control of pollution) Act.
- Public awareness.

Unit 7: Human population and the environment **(09 periods)**

- Population growth and variation among nations.
- Population explosion- family welfare program.

- Environment and human health.
- Human rights.
- Value education
- Role of information technology in environment and human health.

Recommended Books:

1. Textbook of Environmental studies, Erach Bharucha, #UGC
2. Fundamental concepts in Environmental Studies, D.D. Mishra, S.Chand & Co-Ltd,
3. Text book of Environmental Studies by K.Raghavan Nambiar, SCITECH Publication Pvt. Ltd.
4. Environmental Engineering by V.M.Domkundwar- Dhanpat Rai & Co.
5. Environmental Engineering & Safety by B.K.Mohapatra.

UNIT – 1

The Multidisciplinary nature of environment studies

* 1.1 *
DEFINITION

The word environment is derived from the French word "environner" which means encircle or surrounding.

It is a complex world where living organisms were lived or a group of organisms living with non-living things which are inter-related to each other.

OR

Environment is the natural world or surroundings or conditions in which a person, animal or plant lives or operates.

→ * Biologist "Jacob Van Uexkal" introduced the term "Environment" *

Environment consists of two main components :-

Biotic components :- (living things)
animals, plants, fungi, bacteria etc.

Abiotic component :- (Non-living things)
Temperature, air, minerals, water, atmosphere, soil light etc.

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{1.2}

Scope and Importance

Scope means the extent of area that something deals with or to which it is relevant.

The scope of environmental studies deals with environmental science, environmental engineering and environmental management.

→ **Environmental science** :- It deals with the scientific study of environmental system (air, water, soil) i.e changes on living organisms and environmental damages done by various human activity.

→ **Environmental Engineering** :- It deals with the study of technical processes involved in the protection of environment from the dangerous effects of human activity and improving the environmental quality for the good health of humans.

→ **Environmental management** :- It consists of the adiministrative functions i.e development, implementation and monitoring of environmental policy of an organization. It promotes a planned investment for controlling and prevention of various environmental pollution.

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Importances

(1) → To clarify modern environmental concept

(2) → To know more sustainable way of living.

(3) → To use natural resource more efficiently

(4) → To know the behaviour of organism under natural conditions

(5) → To aware and educate people regarding environmental issues at local, national and international level

(6) → To know the inter-relation b/w people and non-living things

1.3 Need for public Awareness

Now-a-days, it is essential to make the public aware of the dangerous consequences of the environmental degradation.

If reformative measure not undertaken for these deleterious effect our life and properties will rapidly extincted.

Various environmental challenges are

- Growing population
- poverty
- Need for Ground water
- Degradation of land

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— Reduction of biodiversity.

— Air and water pollution.

— Rapid industrialisation etc.

It is not only the duty of govt but also the people to take active role for protecting environment.

The public awareness may be promoted by newspaper, radio, television, advertisement, awareness programes etc.

There are various institutions and organisation which plays a vital role towards the environment to make people aware of various consequences and challenges.

* Note Various organisations

BSI - Botanical survey of india, 1890

ZSI - Zoological survey of india, 1916

WII - Wildlife Institute of india, 1982

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UNIT-3

NATURAL RESOURCES

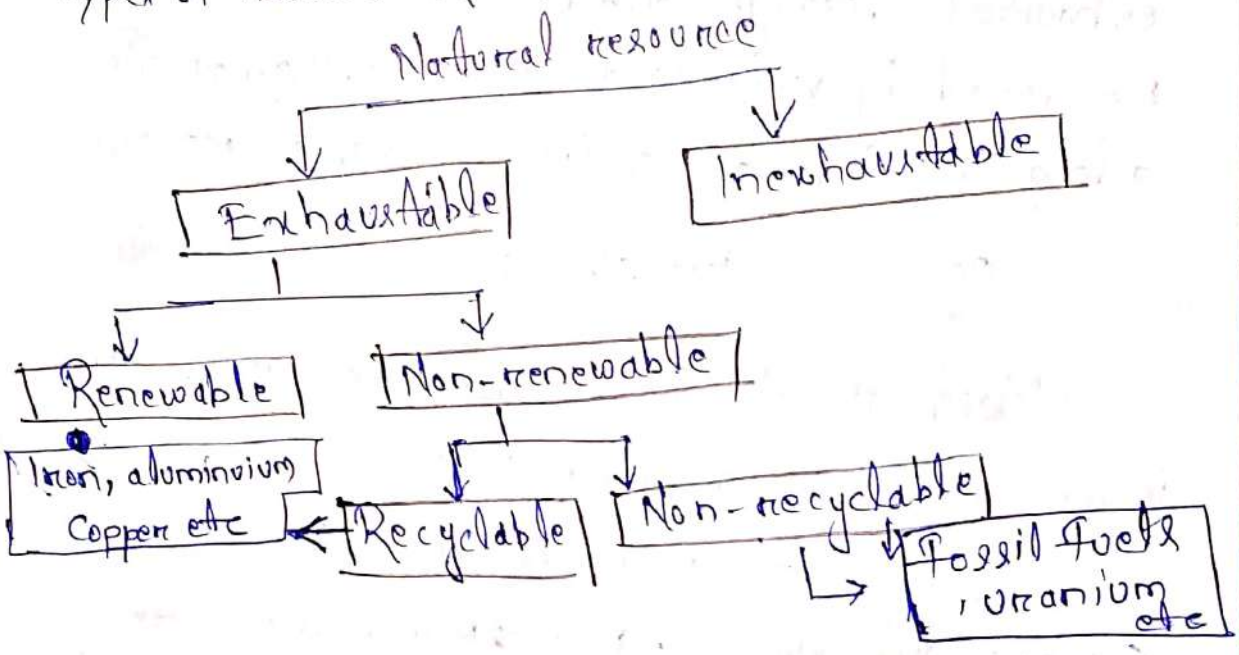
INTRODUCTION :-

SOURCE :- A place, person or thing where something comes from and ~~starts~~ from or from where something obtained.

RESOURCE :- A supply of something, a peice of equipment etc i.e available for somebody to use.

Natural Resource :- Things or matterials of the nature, that can be put to some use by human beings for their growth, development, comfort and other necessities are called as natural resources.

Types of natural Resources



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Exhaustable natural resources

These are the resources which are consumed or exhausted through continous use or mis-use.

eg:- soil, forest, petroleum, minerals, natural gas etc.

It is of two types.

→ Renewable
→ Non-renewable

In-exhaustable natural resources

These are the resources which can not be exhausted through continous use or misuse.

e.g → air, sunlight etc.

Renewable natural Resources

The natural resources which are consumed exhausted through continous use and can be recovered by very hard efforts taken of for a long periods are called renewable resources

eg: soil, forest, ground water etc.

Most of the removable resources are interdependent to each others

eg. forest maintained the environmental condition and climate. plants and trees helps to prevent soil erosion and soil helps the plants for their growth.

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Non-renewable natural resources

There are not replenishable or we cannot get back these resources at any cost if once it get emptied.

It means, we can't get back our coal and petroleum petroleum reserves in our life time if once they are consumed or exhausted completely.

Maxm no of metals are non-renewable resources. i.e iron, copper, zinc, coal, oil deposits, minerals etc.

Note * Minerals also called - stock Resources.
* Coal, petroleum and natural gas - Fossil fuels

2.1 FOREST RESOURCE

Introduction

India is one of the 12 mega diversity country of the world, commanding 7% of world's diversity & with 16% of the major forest types varrying from alpine pastures in Himalayas to temperate sub-tropical, tropical forest and mangroves in coastal areas.

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Country data

- Total land area - 297,319 thousand hectare
- Total forest area - 65,005 hectre (21% of total land area)
  - Natural forest - 50,385 hectre.
- Total protected area - 14.8 million hectre
  - National parks - 80 Nos
  - Wild life sanctuaries - 441 nos.

2.1.1 Use of forests

① Commercial use

It provide a large no of commercially used materials like timber, fire wood, pulp wood, food items, gum, Rubber, fibres., fooder, medicines and many more.

Many forest lands are used for mining, agriculture, grazing, recreation and fore-development of dams and other major works.

② Ecological use

→ production of oxygen : Trees are rightly called "the lungs of world".

→ Trees produce oxygen by photosynthesis, which is essential for the life on the earth.

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③ Reducing global warming

The main green house gas "CO_2" is absorbed by forests as a raw material for photo-synthesis.
So it is reducing global warming.

④ wild life habitat

forest are the home of million of wild animals and plants.

⑤ Regulation of hydrological cycle.

- Forests, watersheds act like gaint spongers absorbing rainfall, slowing down the runoff.
- 50-80% of moisture above forest comes from transpiration which helps in bringing rain.

⑥ Soil conservation

Forests bind the soil particle thightly in their root and prevent soil erosion.

⑦ pollution moderation:

Forest absorbs many toxic gases and keep the air pure and clean.
It also absorbs noise and prevents noise pollution.

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Over. exploitation of forest

- Since time immemorial (not constant) human depends heavily on forest, food, medicine, sheeter, wood fuel etc.
- With growing population demand of forest raw materials are increasing by mining, road construction by clearing the forest.
- Excessive use of fuel wood, expansion of urban agricultural and industrial areas and over-grazing togetherly leads to rapid degradation of forest resource.

2.1.2 Deforestation

It is defined as loss or continual degradation of forest area due to natural or human related causes.

OR

The conversion of forested area into non-forested area is known as deforestation.

OR

It is defined as the reduction of forest quality, Density.

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Causes of deforestation

→ **shifting cultivation** :- There are an estimated 300 million people living as shifting cultivators who practice slash and burn agriculture and supported to clear the total forest.

In india we have this practice in north-east region. and some in andhra-pradesh, Bihar and madhya pradesh.

→ **Fuel requirment** :- Increasing demand for fuel wood by the growing population is a major cause of deforestation.

→ **Raw material for industrial use**

Wood for making boxes, furnitures, rail-way sleepers, plywood, pulp of papers have applied a tremendous pressure on forest areas.

→ **Developement projects**

Massive destruction of forests occur for various development projects like hydro-electric projects, big dams, road construction, and mining etc.

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→ Growing food needs

To meet the demand of rapid growing population agricultural lands and settlements are created permanently by clearing forests.

→ Over grazing:

- Lots of tree cover and the cleared lands are turned into the grazing lands.
- Over grazing by cattle leads to further degradation of these lands.

Effect of deforestation:

→ It threatens the existance of many wild life species due to destruction of their natural habitat.

→ Bio-diversity along with genetic diversity get erroded by deforestation.

→ Hydrological cycle get affected thereby influencing rainfall.

→ problems of soil errosion and loss of soil fertility is increasing rapidly.

→ In hilly areas, deforestation also leads to landslides.

→ Global warming also increasing with loss of forest

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CASE STUDIES:

→ **Timber Extraction** :- Timber is the major resource of forest. The pre and post independent periods of india observed recorded use of timber for various applications like -

furnitures, doors, windows, roofing, flooring etc.

Till 1990, wood is used as fuel energy in rural area, to an extent of 50% of consumption is reduced from 1990 due to imposition of ban on cutting of trees (valuable trees)

paper board and news-print also consume much of our wood. With increasing population growth our per capita consumption of paper is likely to increase from 2 Kg to 4.5 Kg per year.

There are several interconnected effect of timber extraction and fuel wood cutting on forest and tribal people, such as :-

→ Thining of forest
→ Loss of biodiversity (Especially on tree breeding birds)
→ Soil erosion and loss of fertility.
→ loss of tribal culture
~~→ Loss of tribal cult~~

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Dams and their effect on forest and tribal people

* Dams are projected as "temples of modern india". However these dams are also responsible for the destruction of vast areas of forest.

* Big dams have been in sharp focus of various environmental groups all over the world which is mainly because of several ecological problems. (i.e deforestation, socio-economic problem related to tribal people)

WATER RESOURCE

Water is an indispensable natural resources on our earth on which all life depends. about 71% of the earth's surface is covered by water from which 96.5% is salty ocean water and 3.5% is fresh water in the form of rivers, lakes, frozen water locked up in glaciers and poles.

(2% - in glacial form
1.5% - usable drinking water)

* Most of the animal and plants have 62-65% water in their body.

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Use of surface and ground water

Consumptive use:

In such uses, water is completely utilized and can-not be re-used.

Ex: Domestic, industrial and irrigational use.

Non-consumptive use:

In such uses, water is not completely utilized and can be re-used again. Ex: hydropower-plant.

Other uses

→ Water is used for domestic purposes like: washing, bathing, cooking, drinking etc.

→ Water is used in commercial establishment like hotels, theatre, educational institutions, offices etc.

→ Almost 60-70% of fresh water is used for irrigation.

→ Water plays a key role in moderating climate change and diluting pollutants.

* Over utilization of surface and ground water

The rapid increase in population and industrial growth leads to severe demands on water resource. After using all available surface water resource to max^m point, human beings began using ground water to meet their needs.

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Effects of over-utilization of water

* The increase in extraction of ground water in excess leads to decreasing ground water level.
* Building construction activities seal permiabe soil zone and reduce the area for percolation of rain water thereby increasing surface run-off.
* Over-utilization of ground water in arid and semi-arid regions for agriculture disturbs equillibrium of reservoir in that region causing problems like lowering of water table.
* Over-utilization coastal area leads to rapid intrusion of salty water from the sea which causes un-suitability of drinking water.
* Over-utilization causes dry of borewells and dug well.

FLOOD

Flood is an overflow of an expanse of water that submerges land.

In the sense of flowing water the word "flood" may also be applied to the inflow of the tide.

Causes of flood

Heavy rainfall, melting of ice, sudden release of water from dams often causes flood in the low laying coastal areas.

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→ pro-longed heavy rainfall can also cause the over flowing of rivers and lakes resulting in floods.

→ Reduction in carrying capacity of the river channels due to abstraction built on flood ways

→ Deforestation, over-grazing, mining increases the run-off from rains causing flood.

Effect of flood

→ Water spreads in the surrounding areas and sub-merges them causing various problems to the mankind.

→ plain surfaces get eroded and silted with mud and sand thereby affecting cultivated land areas.

→ Extinction of civilization in some coastal area also occurs.

FLOOD MANAGEMENT

→ Floods can be controled by constructing dams and reservoirs.

→ Channel management and embankment also control floods.

→ Encrouchment of flood ways should be banned.

→ Flood hazard may be reduce by forecasting or flood warning.

→ Flood hazard may be reduced by reduction of run-off and this can be achieved by increasing infilteration through appropriate afforestation in the catchment area.

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DROUGHT

Drought is mainly happened at the place of scarce of water. Drought is nothing, but scarcity of water.

drought causes due to :- (causes of drought)

→ When annual rainfall is below normal and less than evaporation, drought occurs.

→ High population also leads to drought. population growth leads to poor land use and worsens the situation.

→ ~~Deforestation~~ Deforestation leads to desertification and drought. Deforestation leads to top soil exposed to erosion by heavy rains, wind and the sun.

→ Over-exploitation of surface water resources by digging wells or bore well ~~for~~ for high productivity also causes drought.

Effects of drought

* Drought causes hunger, malnutrition and scarcity of drinking water. It also degrades the quality of drinking water.

* Drought causes wide-spread crop failure leading to accute shortage of food.

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→ Drought indicates the initiation of desertification.

→ Drought accelarates degradation of natural resources.

Drought Management

→ Indigenious Knowledge in control of droughts and desertification is very useful for dealing with drought problems.

→ Construction of large capacity reservoirs is essential in drought-prone areas.

→ Rainwater harvesting programme is very useful technique to conserve water.

→ Afforestation activities improve the potential of water in drought-prone areas.

→ Mixed cropping is an essential method to minimize drought.

Benefits of dams

* Water for drinking and industrial use

Due to large variation in hydrological cycle, Dams and reservoirs are should be constructed to store water during the periods of surplus water availability.

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Water stored in reservoirs is also used for various industrial use.

② Irrigation

Dams are constructed to store surplus water during wet periods, which can be used for irrigating arid lands. One of the major benefit of dams is that waters flows can be regulated as per agricultural requirements throughout the year.

③ Flood Control

Flood in the rivers have been playing many time and an important role in our life and property of the people. Dams can be effectively used to control floods by regulating river's waterflow downstream of the dam.

④ Hydropower Generation

Energy (electric) plays a key role for socio-economic development of a country, Hydro-power provides a cheap, clean and renewable source of energy.

Hydropower is the most advanced and economically viable resources of renewable energy.

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Inland navigation

Advanced inland navigation is a result of Comprehensive basic planning and development, of ~~utilizing~~ dams. Dams plays a vital role in realizing large economic benefits.

Problem of dams

- Soil errosion :-

One of the vital problem with the dams is , errosion of land of local areas and forest.

A major example of soil errosion problem is near "Aswan Dam."

Species extinction

Fisheries become an increasingly important sources of food supply. More attention to be paid to the harmful effects of dams on many fish and other aquatic animals.

There is no bypass systems for these animals are not including in many of dams. So the fishermen are interfering with their life cycle.

Spreading of deseases

Dams in tropical areas, due to their slow movement are literally breeding grounds for mosquitoes, snails and flies. These are the vectors that carry mallaria and various diseases.

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Siltation

It is the outcome of silt being deposited at the bottom of the reservoirs which affects, by utility of dams.

Salinization of water

1st, irrigation increases the salinity of ~~soil~~ surface water when evapouration removes water but leaves salt behind. This occurs when irrigation water contains some salt and irrigation rates are not high enough to flush the salt away. Saline water ~~in Afric~~ may ~~pollute surfac~~ cause soil pollution.

CONFLICTS OVER WATER

The term "water conflict" is used to describe tensions or disputes between states, countries or people groups for the utilization, consumption or control of water resource.

Only 0.4% of the world's drinkable water is accessable to humans. With a growing population of 7 billion people, disputes over portable water sources comes specially in the scarce region of water.

Whenever a water source such as a lake, a river or an underground aquifer crosses national borders, rightful ownership is often contested.

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MINERAL RESOURCE

Mineral - (homogenious) metal - (Not - homogenious.)

Mineral :- A mineral is a pure inorganic substance that occurs naturally in the earth's crust.

* More than 2000 minerals are identified till date

Among all the minerals, the following 08 elements contains the most. i.e. O, Si, Al, Fe, Ca Na, K, Mg.

Na, K → Non-metal.

Minerals are valuable natural resources being finite and non-renewable.

study of minerals → mineralogy

Mineral Resource
- Metallic → ferrous / Non-ferrous
- Non-metallic → Eg - Graphite, Kaoline, felspar, mica limestone, sandstone.
- mineral fuel → Natural gas, petroleum, fossil fuels.

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ferrous :- Iron, chromite, cobalt, mild steel, cast iron
non-ferrous - copper, zinc, lead, aluminium, Gold.

Use and exploitation of mineral

→ For development of industrial plants and machinary, we vastly use various types of minerals.
→ Generation of energy (coal, uranium etc)
→ Construction, housing settlement
→ form different equipments, weapons etc
→ for transportation means.
→ for communication i.e telephone wires, cables, electronic devices.
→ Medicinal system (particularly in Ayurvedic system)
→ formation of Alloys for various purposes.
→ Jewellary and ornaments that is : Gold, silver, platinum and diamond.
→ for agricultural mean as fertilizers, sheed-dressings etc.

Environmental Effects of extracting and using mineral resource.

→ Land degradation due to lowering of surface levels. The loss of valuable soil cover.
→ Deforestation in the mining areas
→ Erosion of soil is enhanced.

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→ The agricultural lands are affected by silt.
→ Materials mined but not recovered.
→ The disturbance caused to the floral and faunal population is increased.
→ Mine drainage has polluted streams, rivers, lakes and even seas.

FOOD RESOURCE

— Food is defined as any substance consume to provided nutritional support for our organisms.
— food is an essential requirment for our life.

- Component of food
  - → Carbohydrate
  - → fat
  - → protien
  - → Vitamins
  - → minerals.

→ Food comes from three sources mainly

Crop lands	Range lands	Fisheries
76% world's food	(private firm) 17%	(7%)

WORLD FOOD PROBLEM

Natural disaster

Climate change is having an increasing impact on food production as drought and flooding becomes more frequent and severe

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poverty :- The main reason why people are unable to feed themselves is not that food is not available, but they can not afford it.

Global food price :- Rising G.F.P affects people's ability to buy enough to feed their families, especially urban poors.

Uncontrolled population :- The balance of production and consumption of food is another reason of world food problem.

Under-nourishment :- It is the lack of sufficient nutrition or calloricies of food. According to F.A.O a man need 2500 calloricies food per day. People who receive less than this value are under-nourished

MAL-NOURISHMENT and OVER-nourishment are also affects the world food problem vastly.

Effects of/changes by agriculture & over grazing

Soil erosion

When grasses are removed by cattle, top soil become loose and get eroded by wind and water.

Land degradation

The humus content of soil decreased & fertility of soil affected.

Soil looses infilteration capacity by trambling of cattle.

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Flooding :- Soil erosion by over grazing also results flood.

pollution of river :- chemicals and fertilizers flown by wind and irrigation water and at last polluting our water resource.

Desertification

The poor agricultural practice like shift cultivation and slash & burn cultivation results desertification.

Over-grazing also results reduction of plant diversity.

Effects of modern agriculture.

→ It leads to soil erosion
→ Altercation in fertility of soil.
→ leads to soil pollution
→ HYV (High yielding varieties) encourage monoculture in highly populated areas.
→ Increases diseases of crop and attack of pathogen.
→ Water logging condition may occur by modern agricultural practice though root zone of the crops cannot get enough water.
→ Salinization of crops.

Fertilizer and pesticide problem

1) Micro-nutrient imbalance :-

Most of the chemical fertilizer used in modern agriculture are consists of nytrogen, potassium and phosphorous which are very essential for the crops.

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→ Excessive use of fertilizers causes micro-nutrient imbalance in our body.

(ii) Nitrate pollution: Nitrogenious fertilizers applied in the field often leach into the soil and contaminate the ground water.

** If excessive nitrogen of 25 mg per litre is appeared in waters causes "BLUE BABY SYNDROME"

(iii) Eutroficaftion

A large properation of N and P used in crop field is washed out by run-off water to various water bodies, causing over-nourishment of lakes ~~etc~~ and ponds etc. ~~which causes~~ The process is known as Eutrofication.

Due to this various algal blooms population increases and toxic products of fertilizer affects the life in water.

(iv) ~~PESTICIDES~~ Creating Resistance in ~~pesticides~~ pests

Various ~~about~~ ~~20~~ species of pastes are now known which can resist various types of pesticides and pesticides become immune to them. These type of pastes are known as supers pastes.

(v) Biological Magnification

Many of pesticides are non-biodegradable and keep on accumility food chain & the process is known as bio-logical magnification and the magnified pesticides are very harmful to the life system.

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(vi) Death of non-target organism.

Many insecticides which are very useful for the crops are also killed by various pesticides along with the dangerous pests.

Water-logging :-

Defn :- If water stands on land most of the year, it is called water logging.

Causes → Excessive water supply
→ Excessive rain.
→ poor drainage system.

Effects In water logging conditions pore voids in the soil get filled by water and though roots of crops can't balance their respiration and can't get enough air to survive.

Prevention
→ Avoid excessive irrigation.
→ Sub-surface drainage technology.

Salinity When water i.e absorbed by soil and evaporated leaving behind a thin layer of dissolved salt on top soil is called salinity of soil.

Salinity effects low yield of crops and affects the soil fertility.

Various salts i.e (NaCl, $CaCl_2$, $MgCl_2$, Na_2SO_4, Na_2CO_3 etc)

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ENERGY RESOURCE

Energy may be defined as the property can be converted into work.

OR

The capacity of doing work is known as Energy

Energy is available in earth in various forms and both energy production and utilization indicates countries progress.

Growing energy needs

- Approx 80% of world's energy is produced by fossil fuels.
- With the growth of population and their demand the needs of energy resource are also increasing
- In india coal, oil, water are the main source of energy.
- Commercial consumption of energy from coal is 56% and petroleum is 36% and other from natural gas and water.
- Industrial sectors consuming about 50% of total commercial energy.
- (Ex - fertilizer industry, cement, paper, textile, aluminium, steel etc)

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The growing needs purely shown in the following lines.

Energy (per house-hold)	In 1970-71	In 1991-95
Electricity	7 Kwh	53 Kwh
Kerosene	6.6 Kg	9.9 Kg
cooking gas	0.33 Kg	3.8 Kg
Firm sector energy consumption	3.9% (951)	32.5% (1997)

Renewable energy Resource

These resources can be generated continously and inexhaustable.

Ex:- Solar energy, wind energy, Hydro-power, Geo-thermal energy etc

Advantage

→ It is eco-friendly
→ It is a in-exhaustable source of energy
→ Leads to job creation
→ Less maintanance facilities

Non-renewable energy resource

These resource can't be regenerated once they are exhausted.

Ex:- coal, petroleum, natural gas and nuclear fuel (uranium)

Advantages

- They are abundant and affordable.
- They cannot be exhausted in short period.
- The products are very easy to use.

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- These energy has a great role to stabilize global energy price.

Disadvantage

- Electricity generation capacity still not enough per demand.
- It may be unreliable
- Requires a huge financial outlay (money for starting).

Disadvantage

- They cannot be replace if they exhausted once
- The mining of these resources causes environmental pollution.
- fossil fuels burning generates CO_2 which causes global warming

Use of alternative Energy Resources

Alternative source of energy is generally refers to any group of non-traditional fuel sources that doesn't burn fossil fuels or use of any type of natural resources which will damage / harm to the environment.

It provides many benefits over fossil fuels as less harmful and cheaper fuel price.

Solar Energy

- It is the biggest domestic use alternative energy resource till now.
- The radiation of sun light is converted to electrical energy by photo-voltaic cell or PV panel etc.

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→ The heat energy of sun can also be use in solar pool heating system in swimming pool cases.

Wind energy

— Wind energy is derived from the movement of the air via wind mills and turbines etc.

— Wind energy is all about converting the kinetic energy of wind into mechanical energy for driving pumps and machines & then into electrical energy to power our homes.

Geothermal energy

— This type of energy derived from the extreme heat of inner portion of the earth.

— Water heated by geothermal heat below the earth surface are pumped out to generate electricity or to heat buildings.

Ex:- Hot spring (Geothermal heat)

Hydrogen energy

— Hydrogen is a completely clean burning fuel. Once produced, Hydrogen gas cell emit only water vapour and warm air after use.

— The major issue about this energy i.e mostly derived from use of natural gas and fossil fuel, so it is not as popular as other alternative energy resource.

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Bio-mass energy

- It is derived from wood industry, agricultural crop, raw material from forest, household waste.
- Burning wood has been used as a source of fuel
- Bio-mass energy is used in the form of bio-gas.

LAND RESOURCE

Land as a resource

Land is a very valuable resource. It provides food, wood, fibre, medicines and many more.

* Soil is a ~~mixtu~~ compound of organic (dead materials and plant materials) & in-organic materials (Rocks, minerals)

* (The top layer of the land is called soil)

Uses

- land provides food, wood, minerals etc...
- land acts a dustbin for the waste of the modern society.
- Land nurtures (protection, care) when something growing plants and animals ~~as~~ that provides us food and shelter.
- Land is used for constructing road, buildings, industries, statues etc.

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Land degradation

It is defined as the process of decay in the quality of the soil or loss of fertility.

causes

→ Increasing population and due to their increasing food need.

→ Urbanization - It causes land degradation as agricultural land and forests get cleared.

— fertilizer and pesticides use in excess also causes land degradation.

— Human activities like deforestation, over-grazing mining, pollution from industry also causes land degradation.

Effects

→ soil texture (inner shape) & structure are collapsed.

→ Loss of fertility

→ water logging

→ Salinity of soil.

→ Loss of social, economic and biodiversity level.

Land slides

It is defined as the sudden collapse or downward movement of rocks, earth from a slopy ground.

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man induces landslide (Role of human in landslide)

→ Under ground mining.
→ Forming roads in unstable slopes.
→ Over-exploitation of ground water.
→ Movement of heavy vehicles in weak, slopy road causes landslide.

Soil erosion

It is a process of removal of super-ficial layer of soil. The most valuable part of the soil washed out in this process.

Types of soil erosion

Normal soil erosion	Accelarated soil erosion
- The soil erosion caused by gradual removal of top soil by natural process.	- Erosion caused by man made activities.
- The rate of soil erosion is very slow in this process	- The process is very rapid than the formation of soil.

Causes of soil erosion

- Water :- causes soil erosion in the form of rain, runoff and flood.
- Wind :- It carries away the fine particles of soil and contributes in soil erosion.
- Over grazing, Mining are also causing soil erosion.
- Deforestation leads to soil erosion

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- land slides is also a part of soil erosion.
- Construction of dams, buildings, roads etc remove the protective vegetal cover causes soil erosion.

Effects

→ Loss of soil fertility
→ Loss of soil ability to hold water and sediments.
→ Leads to complete destruction of plant growth.
→ Sediment (waste) run-off can pollute water.

prevention

→ Contour farming
→ Terrace farming
→ Agro-forestry
→ Wind-breaks
→ No-till farming method.

DESERTIFICATION

It is the process by which productivity potential arid or semi arid land falls.

The decrease in productivity is varies from 10-50%.

Causes

→ Deforestation.
→ Over grazing (increase in cattle population)
→ Mining and quarying activities.
→ Salinization.

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Effects of desertification

- Conversion of irrigated crop land to desert.
- Depletion of ground water.
- Soil erosion.
- Loss of vegetal cover.

prevention of desertification

→ Afforestation in arid areas.
→ Eco-friendly mining.
→ Use of organic fertilizers and gypsum.

Role of individual in conservation of natural resource

→ We should follow 3R rule (Reduce, Reuse, Recycle)
→ We should save water which is so vital for survival.
→ We should save electricity as we need electricity in our daily life.
→ To save fuel is very essential for our future.
→ Use of plastic should be in limit as it takes thousands of year to decompose.
→ Trees are rightly called lungs of earth. So we should plant trees.
→ We should use renewable source of energy for power generation.
→ We should use natural resources judiciously.
→ public awareness is very essential for conservation of natural resource.

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Equitable use of resource for sustainable life style.

- The equal distribution of natural resources should be for all irrespective of rich or poor
- There are must be balance b/w the need and consumption, particularly for water, fuel (coal, petroleum), soil, food etc.
- The developed countries are utilizing maxm amount of natural resources compared to developing countries. For ex: Japan, USA, Canada, Australia have 22% of world's population are using 86% of natural resources.

  In this process rich are becoming richer and poor to poorer.

  This process should be strictly avoided.

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unit-3

ECO-SYSTEM

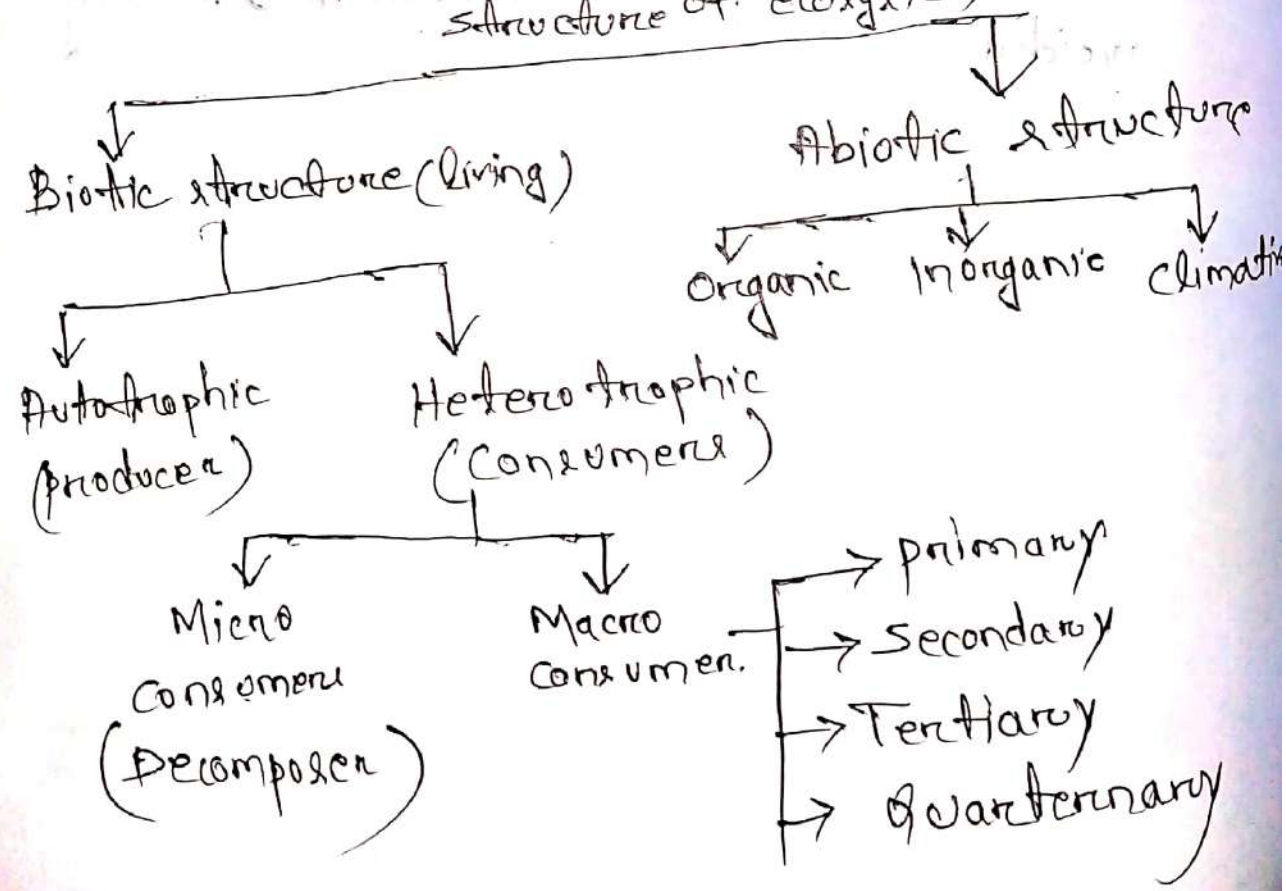
Ecosystem is defined as a community of both living and non-living components in a particular area.

A system involving the interaction b/w a community of living organism with its non-living environment in a particular area.

Ecology :- The study of inter-relationship b/w living and non-living components in an ecosystem

* **A.G. Tansley** coined the word **Ecosystem**

Structure of ecosystem :/ Components of eco-system.



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Biotic structure

Producer :- They are autotrophic in nature (self nourishing)
→ All green plants are producers.

→ They utilize sunlight as there energy source and with in-organic nutrients from soil, water & air, they made their own food in a process called photo-synthesis.

Consumers :- They are heterotrophic in nature.
→ They depend upon the producers to derive their ~~own~~ food directly or indirectly.

→ They digest their food into simple substance to metabolize their body & released the wastes to the environment.

primary consumers

— They are also called herbivores.
— They directly depend upon producers ~~to~~ derive their food and survival.
Ex :- deer, cow, grasshoppers, Goat etc.

Secondary Consumers :-

→ They are also known as carnivores.
→ They are meat eaters in nature and comparatively small in size.
→ Ex :- Birds, Cats, frogs.

Tertiary consumers

→ These are the large carnivores which fed on secondary consumers.
Ex :- Snakes, Jackels, wolves etc.

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Quarternary consumers

The large carnivores to depend upon meat for their survival are quarternary consumers.

e.g. Tiger, lion, crocodile, shark, whale etc.

* **Omnivores** :- A animal/person eating both plants and meat is called omnivores.

Ex:- Man, Dog, Bear etc.

Decomposer :- They are living components which break down the complex compound of dead plants and animals into simple organic compound and ultimately into inorganic nutrients.

Ex:- Bacteria, fungi (Mushroom)

Abiotic structure

- The organic matter like carbohydrates, lipids, protein, amino acid etc are some important factor of abiotic structure

- The inorganic compounds such as water, minerals, atmospheric gases etc are some important factor of abiotic structure and have an important role in eco.system.

→ It also include climative factors like moisture, wind, rainfall, solar radiation etc.

presence of Mineral in any time in ecosystem is known as **Standing state**

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Functions of eco-system

producing bio-mass

→ The trees are producing bio-mass and those trees are utilized by carnivores.

Energy flow

→ The living components get energy by producers.

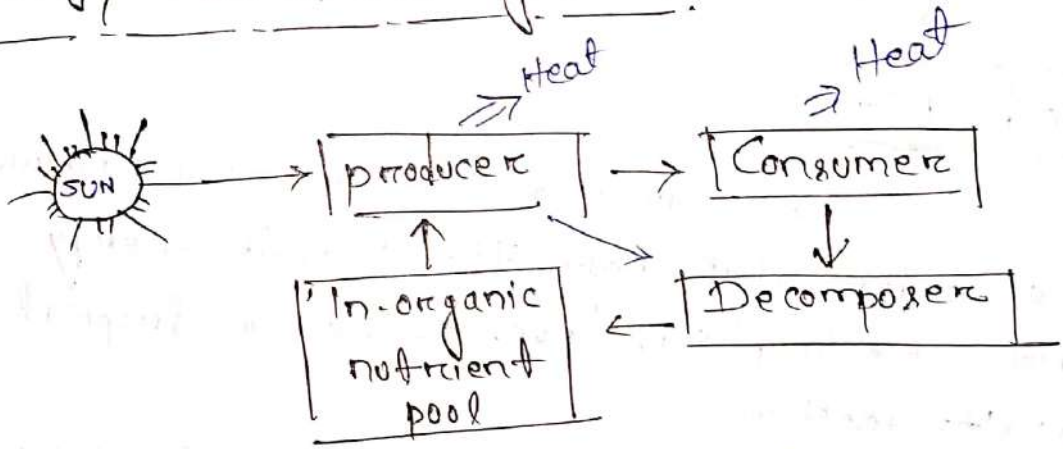
Decomposition

As the decomposers decomposes all the waste products which helps to clean the environment.

Nutrient cycle

The decomposed items gradually/ultimately gives us nutrition.

Energy flow in Eco-system:-



→ Energy is essential for every biological activities.

→ Sun is the parent source of energy in this earth

→ Solar energy is transferred into chemical energy by a process called "photo-synthesis".

→ This energy stored in plant tissue and then transformed into mechanical and heat energy during metabollic activities.

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→ Here, energy flows from sun to the producer and then to consumer.

→ This total process is based on the 2 laws of thermodynamics.

1stly Energy can never be created nor be destroyed, it only transfer from one trophic level to another level.

2ndly When energy is passed in an ecosystem from one trophic level to the next, only 10% of the energy will be passed on.

* Trophic level is the position of an organism in a food chain or energy pyramid

Food chain

A food chain is a chronological pathway or an order that shows the flow of energy from one trophical level to another tropical level.

- In a community which has producers, consumers and decomposers, the energy flows in a pathway.
- The food chain shows a single pathway of eaten and eaten.

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Ex:- Grass → Deer → Tiger

Grass → Grasshopper → Frog → snakes.

Foodchain is generally two types.

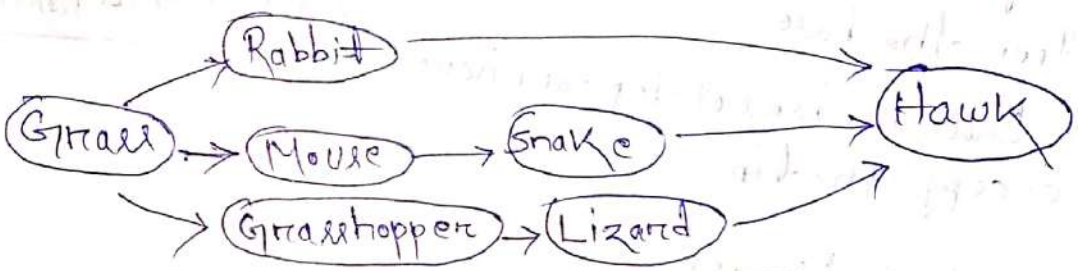
→ Grazing food chain

→ Detritus food chain

Food web

→ Many interconnected food chains make up a food web.

→ A network of food chains where different types of organism are connected at trophic level, so that there are a number of options of eating and being eaten at each trophic level.



ECOLOGICAL PYRAMID

— Ecological pyramids are graphical representation of food chains.

— In this pyramids the height of every trophic levels remains same only width is variable.

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→ pyramids represents the sequential order of eaten and eaten consumers, producers etc.

There are three types of ecological pyramids

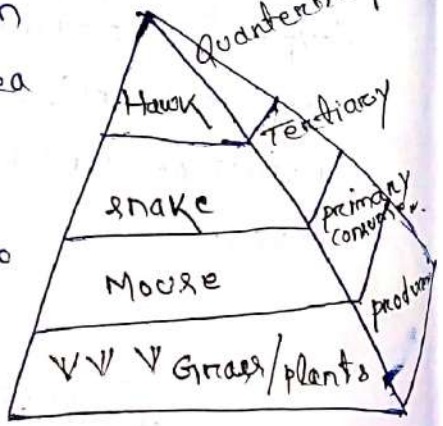
→ pyramids of numbers

→ pyramids of biomass.

→ pyramids of energy.

pyramid of numbers

→ It is the graphical representation of number of individuals per unit area of various trophic levels.



→ Large number of producers are tend to form the base.

→ Lower numbers of top carnivores occupy the tip

pyramid of biomass

It is the graphical representation of biomass present per unit area of different trophic levels with producers at the base and top carnivores at the tip.

- Biomass is calculated by the mass of each individual x number of individuals at each trophic level ($g.m^{-2}$ or gm^{-3}).

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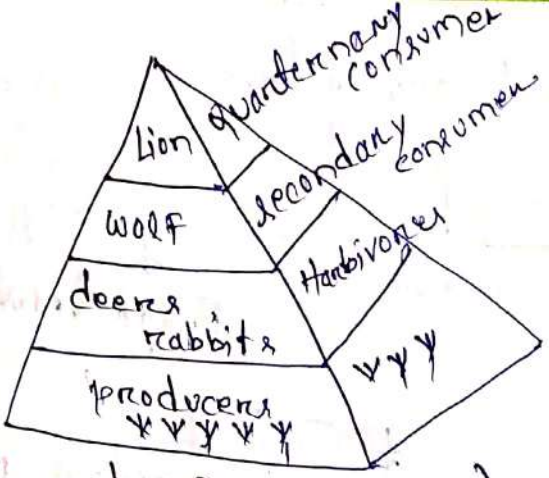
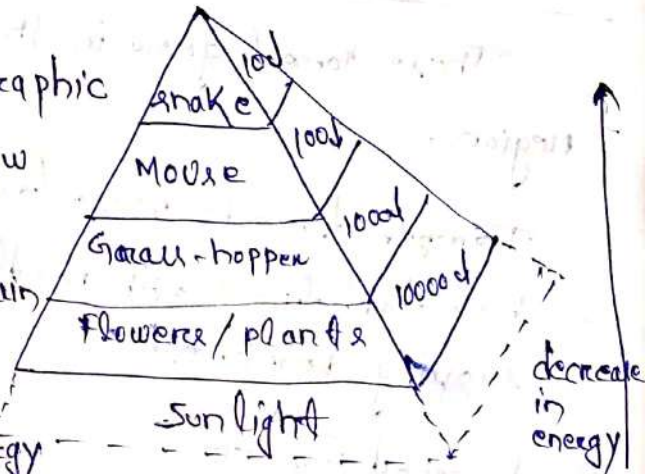


(pyramid of biomass)

pyramids of energy

- pyramids of energy is a graphic representation of the flow of energy through each trophic level of a food chain over a fixed time period.
- The input of solar energy may be indicated as an extra layer at the base.



Forest eco-system

- A forest eco-system is a community of organisms that lives within a forest.
- A forest is usually defined as a large group of trees.
- In india, one tenth of total forest of the world is available.
- Roughly 35% of the land is occupied by forest.

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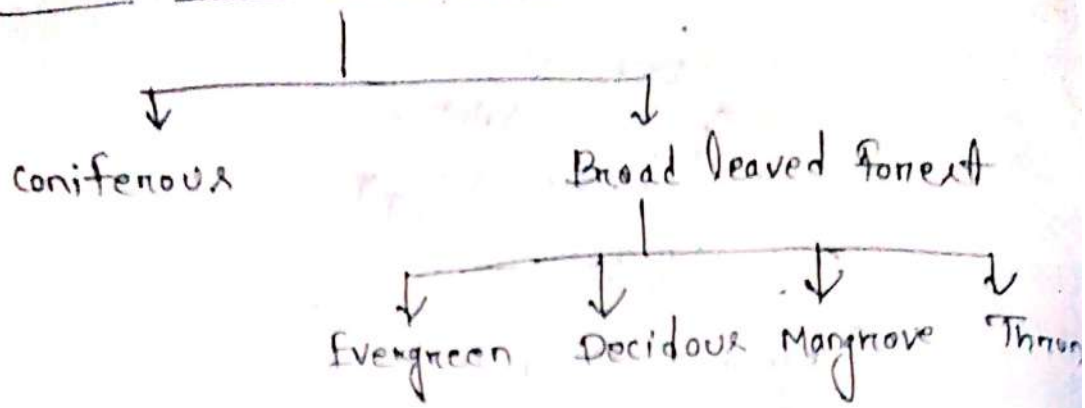
Types of forest eco-system



Coniferous forest

- These forest grow in the himalayan mountain region.
- Temperature is very low in these areas.
- Have tall trees with needle like leaves and downwar sloping branches.

Broad leaved forest

Ever. green forest :- Grow in high rainfall area of western ghat, north eastern region of india and andman.

- No leafless phase
- Only shade loving trees can grow in ground. layers of these type of forest. (because of canopy overla~~p~~)

Decidous forest

These are found in a region with a balanced amount of seasonal rainfall.

Sunlight can pentrate easily onto the forest floors.

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Mangrove forest

— Grows along the coast especially in river deltas.
— Are able to grow in both fresh and saline water, in muddy area.
— These have breathing roots

Throne forest

— These are found in semi-arid region.
— Have a long and fibrous roots to reach water at a great depth.
— Thrones of these plants protect itself from herbivores.

Characteristics features of forest eco-system.

→ A distinct community of organism is available.
→ A sizeable group of trees are found.
→ Generation of number of lakes and ponds.
→ protection to bio-diversity.
→ Forest soil is rich in organic matter and nutrient.
→ Since penetration of sunlight is low, the conversion of organic matter into nutrient is very fast.

Structure and function of forest ecosystem.

• biotic component/structure

All living organisms present in the food chain or web are biotic components.

Three functional group of a forest ecosystem are:-

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Producers

These are of different kinds depending upon the type of forest developed in that climate.

The main species of trees in major type of forest resource are.

- Tectona Grandis - (सागवान)
- Acer (maple) — pine (→ coniferous:
- Betula (बेतुला) — Cedrus)
- picea (coniferous)

Consumer

primary consumer :- These are herbivores which feed directly on producers.

Ex :- Ants, beetles, spiders, grasshoppers, etc. larger animal such as elephants, deer, giraffe, etc.

Secondary consumer :- These are carnivores and feed on primary consumers : Eg :- Birds, Lizards, frogs, snakes and foxes.

Tertiary consumers :- These are secondary carnivores and feed on secondary consumers.

Ex :- Lion, tiger etc.

decomposer :- These include a wide variety.

1 - Bacteria, fungi, streptomyces.

Rate of decomposition in tropical & subtropical forest is more rapid than temperate forests.

Abiotic structure

These include basic organic and inorganic compounds present in the soil and atmosphere.

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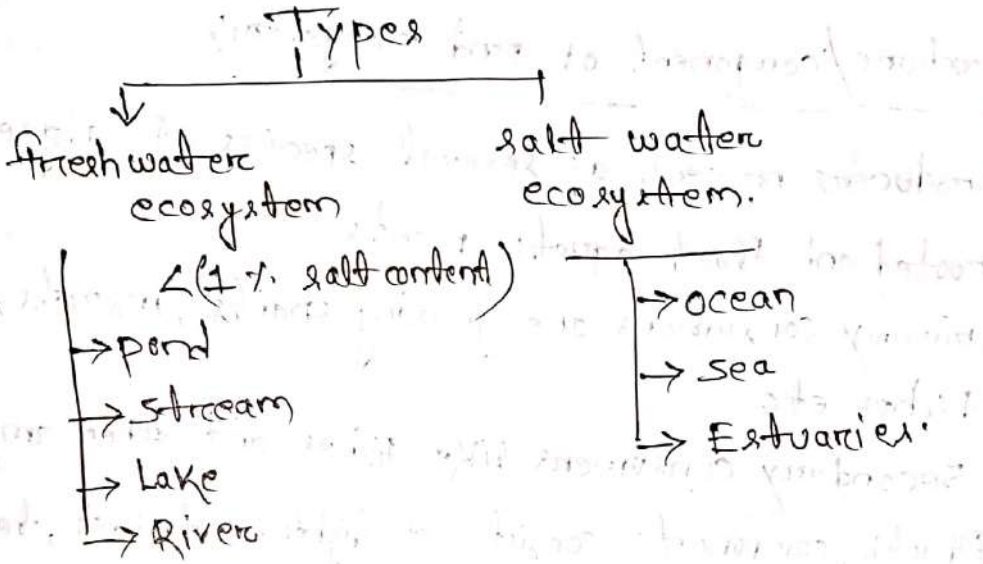
Aquatic Ecosystem

→ An aquatic ecosystem is an ecosystem in a body of water.

→ Communities of organisms that are dependent upon each other and on their environment live in aquatic eco-system.

Types of aquatic ecosystem.



Characteristics feature of aquatic eco-system:

— Special abiotic features includes physical aspects such as quality of water
  — clarity — salinity — oxygen content
  — Rate of flow

— Biotic features like.

* Wetland plants may produce in dense canopies off water
* Growing number of herbivores like snails, geese and mammals which are balancing the ecosystem.
* A huge source of food production and can stabilize the economy of a country.

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Pond ecosystem

- An eclosed body of water that houses different living and non-living components.
- It may be artificially enclosed or naturally enclosed

Characteristics

→ Still water
→ Surrounded by bank
→ Different trophical level.

Structure/component of pond ecosystem:-

→ producers present as several species of algae, rooted and float aquatic plants.
→ primary consumers are grazing snails, insects, small fishes etc.
→ Secondary consumers like fishes and other amphibions
→ Abiotic components consists of light, nutrient, temperature etc.

River or stream ecosystem

A large natural flow of water that crosses an area of land and goes into an ocean or sea etc.

Usually rivers are well oxygenated as it absorbs oxygen from air.

Characteristics of stream/river ecosystem

- It is a fresh water and free flowing water system
- Due to mixing of water dissolved oxygen content is more.
- River deposits a large number of nutrients.

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Structure of river/ stream ecosystem.

→ Temperature, light, nutrient and dissolved oxygen are the major abiotic component.
→ producers i.e phytoplanktons, algae etc.
→ primary consumers like snails, insects etc.
→ Secondary consumers like fishes, mammals etc.
→ Various Bacterias are the major decomposers.

Lake ecosystem.

This is the ecosystem whose large area surrounded by the land except a little space for the flow of water.

Lake eco-systems are may be of salty water or fresh water.

Types of lakes

↳ Tectonic lake → Glacial lake
→ Fluvial lake → Volcanic lake
→ Landslide lake

characteristics

→ still water.
→ surrounded by banks
→ Different trophical level.

Component of lake ecosystem.

- producers present in the form of algae, phytoplankton
- Snails, insects etc are the primary consumers.
- Bluegill, small fishes, carp fish and frogs are the secondary consumers.
- Loons, grieber, herons and otters etc are of tertiary consumers.

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— Nutrient, temperature, soil bed, light are the major abiotic component in lake ecosystem.

Sea/ocean Ecosystem.

— Oceans are the vast body of salty water that covers almost 3/4th of the earth surface.

— It consists of high concentration of salt and minerals.

— It also provides us iron, magnesium, phosphorous and natural gas.

Characteristics of salt water ecosystem

— It occupies the large area of saline water.

— It helps human in commercial activities.

— It moderates (balance) the temperature of earth.

Structure/component of sea/ocean Ecosystem.

Biotic component

— producers are coral reefs, are mainly algae and phytoplankton, microscopic photosynthetic organism.

— Consumers (primary) are hooplankton, small fishes and crustaceans.

— Secondary consumers are fish, coral penguins, whales etc.

— Tertiary consumers are tuna, barracuda and groupers, shark, whales, jelly fish, dolphins etc.

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Abiotic components

→ light - salts
- temperature - Nutrients.

Estuaries ecosystem.

- An estuary is a partially enclosed coastal area at the mouth of river.
- They are highly abundant in nutrients.

They are very useful for the human beings due to high potential of food.

Characteristics

- The zone that affect the tide of sea.
- Water characteristics are periodically changed.
- The living organism in it have wide tolerance level.
- Salinity remains high during summer and low during winter.

Structure/components:

Biotic - producers are mainly phytoplanktons in this ecosystem.
- primary consumers are fish, shell fish, filter feeders etc.
- Secondary consumers are crabs, birds and small fishes.

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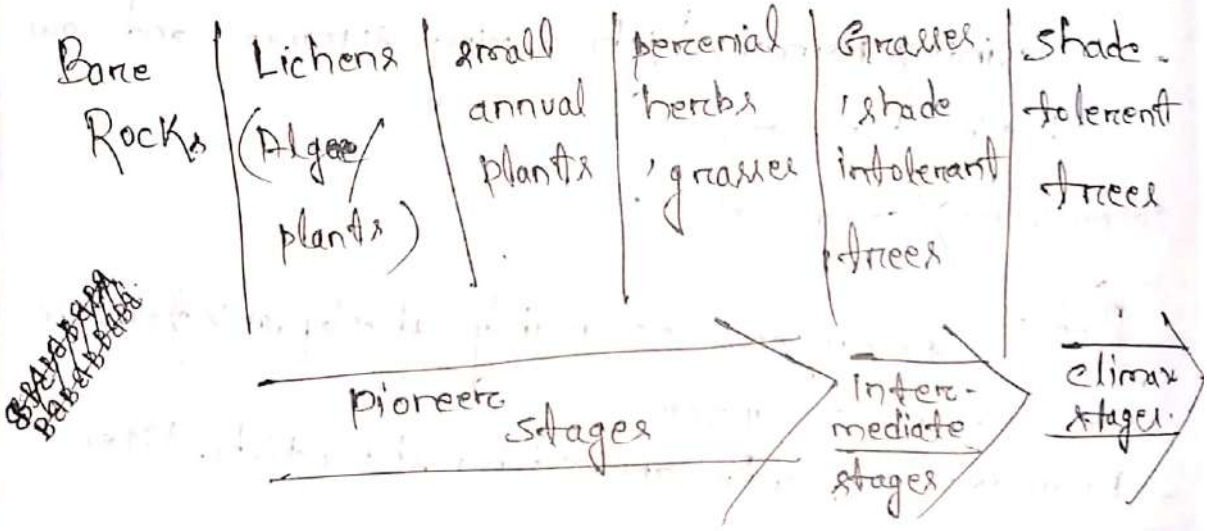
→ Tertiary consumers are fishes and birds, apex predators & humans, dolphins and sharks etc.

Abiotic components

Sunlight, water in estuaries, salt, temperature, dissolved oxygen and minerals.

Ecological succession

- It is a gradual process of change and replacement of some or all the species in a community.
- It can take 100 or 1000 of years.
- Each new community makes it harder for the previous one to survive.



Annual plant - completes its life cycle in one season and then dies germination the production of seeds

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Unit-4

BIO-DIVERSITY

Bio-diversity is the shorter form of the world.

* Biodiversity which means diversity in the biological world

* Thus one can define bio-diversity as the degree of variety in nature which regards to biological species.

→ WALTER G. ROSEN coined the word BIODIVERSITY.

Types of bio-diversity

→ Genetic diversity
→ Species diversity
→ Eco-system diversity.

Genetic diversity

— It induced the difference in DNA composition along individuals within a spacy.

— This level of biodiversity refers to the total number of genetic characteristics in the genetic make up of a species.

Ex:- Types of frogs, Various dog breeds, colourful roses.

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species biodiversity

It is the effective number of different species that are represented in a collection of individuals.

Ex:- Trees, tiger, various animals in forest.

Ecosystem biodiversity

It refers to the diversity present in a place at the level of ecosystem. Depending largely upon the availability of abiotic resources and conditions.

Ex:- Grassland, lakes, rivers, pond etc.

Biogeographically classification of india

India is one of the 12 mega biodiversity countries in the world.

The wide variety of physical feature and climatic conditions have results in a diversity of ecological habitat like forest grass land, deserts etc.

It has 47,000 species of plants and 1000 species of animals approximately and great marine diversity due to 7000 Km long coastline.

India mainly have 13 biogeographical regions.

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→ Himalaya
→ Thar desert
→ Gangetic plain
→ western ghat
→ Coromondal
Mahandian
→ Marine coast

→ Decan peninsula
→ Malabar
→ Laccadive island
→ Bangalian Forest.
→ Andaman islands
→ Nicobar islands.
→ Maldive island

Value of diversity:

Biodiversity is a valuable natural resource for the survival of mankind. Many plants and animals including wild life and very important for human beings.

They can be used directly or indirectly to have consumptive, productive, social, ethical, aesthetic, option value and lie in terms of money.

Consumptive use

Direct utilisation of timber, food, fuel wood, and fodder by local communities provides daily needs ie food, building material, medicines, herbs etc.

productive use

Bamboos, grasses, essential oils, gums, drugs, poisons, insecticides, rudrakshа, seeds, mahua etc have their high commercial values. Various animal body parts are sold in commercial market at both national and international level.

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Social value

Biodiversity has been preserved by traditional societies. These societies valued it as resource and believed that its depletion would be great loss to their society.

Ethical value

It is based on the principle of live and let others live. It is also related to biodiversity conservation and the importance of protecting all forms. All forms of life have the right to exist on earth. All species have an inherent right to exist.

Aesthetic values

It means (appearence.) It involves appreciation of the presence of biodiversity for its beauty as well as for the contribution. people from far and wide spend a lot of time and money to visit the beautiful areas where they can enjoy the aesthetic value of biodiversity.

Option values.

These values include the potentials of bio-diversity that are presently unknown and need to be explored. The option value of biodiversity suggests that any species may prove to be a miracle species some day.

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Biodiversity at local level

- The biodiversity at local levels can be well understand by locating points, places and zones that rich in biodiversity.
- From the following aspects we can study the biodiversity at local level

→ Richness of species at a given place
→ physical characteristics of habitat and vegetation in particular area.
→ change in species composition across different habitat.
→ Rate of change in gradient and conditions.

Threats of biodiversity

The following are the major causes and issues related to threats of biodiversity.

→ poaching of wild life.
→ Man wildlife conflicts.

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Habitat loss

Habitat loss due to human activities and other disturbances are well known factors.

- Due to pollution and the presence of toxic and hazardious pollutants, our fresh water resources have suffered and many species of aquatic birds, fish and mammals have been threatened.
- The natural forest and grasslands which were the natural homes of thousand species are going clear day by day for conversion into agricultural land, development projects, buildings etc
- Rain forest, tropical dry forest, wet land are threatend by desertification.
- problems of acid rain and global climate change are also well known for habitat loss.

poaching of wild life :-

(poaching :- illegal exercise)

- poaching is another threat of wild life.
- From ancient (old) periods, hunters, collecters and smuglers are the major threat to a number of species including endanger species.
- They collected furs, horns, herbal products and smugled for millions of dollars.
- The alarming point in this case is that for one animal they killed more than one. It is an illegal trade and internationally banned.

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→ It is advised that don't poach the parts and products made from wild animals specially of endangered species.

Man wild life conflict

"Struggle for existance."

This is applicable for both man and wild animal

→ Due to habitat loss animals come out of the forest and destroy the crops and livelihood & later on they become danger to human beings.

At last affected people kill them.

→ There are so many cases of conflict b/w man and wild life. In this conflict the group of animal are under in a vast stress.

← They suffer in pain & turn violent and destroy everything they see.

→ There are number of cases, when man eating tiger was reported several people because they like human flesh rather than animal flesh.

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01 JANUARY Tuesday

Tuesday	1	8	15	22	
Wednesday	2	9	16	23	
Thursday	3	10	17	24	
Friday	4	11	18	25	
Saturday	5	12	19	26	
Sunday	6	13	20	27	

1/364 01.week

Environmental pollution

Air pollution

Air pollution may defined as the presence ~~in the outdoor atmosphere~~ of one or more contaminants of some specific quantity for a specific duration in the outdoor atmosphere which is injurious to human, plant or animal life.

Those substances which leads to air pollution are known as Air pollutants.

Air-pollutants → primary pollutants
→ Secondary pollutants.

primary pollutants

↳ These are directly emitted into the atmosphere and are found directly.

E.g. CO, NO_2, SO_2 etc.

Secondary pollutants

↳ These are derived from the primary pollutants due to chemical or photo-chemical reactions in the atmosphere.

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Saturday
Sunday 3 10 17 24
01.week

e.g. Ozone, PAN (peroxy-acyl Nitrate), photo chemical smog.

Causes of air pollution

There are various causes of air pollution: such as:-

→ The burning of fossil fuel.

* Sulpher di-oxide emitted from the combustion of fossil fuels like coal, petroleum and other factory combustibles are the major cause of air pollution.

* Carbon monoxide emitted from vehicles (cars, bikes, jeeps, trucks) by incomplete combustion of fuel causes air pollution

* Nitrogen oxide which obtained from both natural and artificial process also causes air pollution.

→ Agricultural activities

* Ammonia is a very common by-product from agricultural-related activity and is one of the most hazardous gases in the atmosphere.

Experience may acquire the cleverest man, but the least wide-awake of children recognizes it. – Leo Tolstoy

FEB MAR APR MAY JUN

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3/362 ~~atmosphere~~.

* Use of insecticides, pesticides and fertilizers in agricultural activity emits harmful chemicals which pollutes ~~not chemicals~~ air as well as water.

→ **Exhaust from factories and industries**

* Manfacturing industries release a large amount of "CO", hydrocarbons, organic compound and chemicals into the air which causes serious air pollution.

* petroleum refineries releases hydro-carbon which pollutes air with land too.

→ **Mining operation**

During the mining process dust and chemicals are released in the air causing massive air pollution.

This is also responsible for various diseases to the workers and local residents.

→ **Indoor air pollution**

* House hold cleaning product, painting

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supplies & emit toxic chemicals in the air and cause air pollution.

* Toxic gases from Air conditioners and refrigerators causes severe air pollution

Effects of air pollution

→ Respiratory and heart problems

* The effect of air pollution is alarming.
* Air pollution creates dangerous diseases like cancer to ~~heart~~ respiratory and heart conditions along with other threats.
* Childrens in the polluted area suffers from "pneumonia" and asthma.

→ Global warming

* Another direct effect of air pollution is global warming
* With increased temperatures worldwide, increase in sea level and melting of ice takes place. Due to which displacement and loss of habitat will occur.

→ Acid rain

Harmful gases like nitrogen ovides and SO_2 released to the atmosphere during

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5/360 the burning of fossil fuel.

* When it rains, the water droplets combine with these air pollutants, becomes acidic and then falls on the ground in the form of acid rain.
* Acid rain can cause great damage to human, animals and crop.

→ **Effect on wildlife:**

* Toxic chemicals present in the air can force wildlife species to move to a new place and change their habitat.
* The toxic pollutants deposit over the surface of the water and can also affect sea animals.

→ **Ozone layer Depletion.**

* Ozone exists in the earth's stratosphere and is responsible for protecting humans from harmful ultra-violet rays of sun.
* It's getting depleted by CFC and hydro-CFC gases.
* So the layer is getting thin which can cause eye and skin related problems.
* UV rays can affects the crops.

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02/2019

Monday	–	4	11	18	25
Tuesday	–	5	12	19	26
Wednesday	–	6	13	20	27
Thursday	–	7	14	21	28
Friday	1	8	15	22	–
Saturday	2	9	16	23	–
Sunday	3	10	17	24	–

JANUARY
Sunday
06

01 week

Control measures/solution of air pollution 6/359

I. Use public mode of transportation.

* Encourage people to use more and more public modes of transportation to reduce pollution.
* Try to make "Carpooling"
* If you and your colleagues come from the same locality and have same timing, you can explore this option to save money and energy.

II. Conserve Energy

* Switch off all electrical appliances which like ACs, Refrigerators etc when they have no use.
* A large amount of fossil fuel are burnt to produce electricity. We should use less fossil fuel and replaces it by alternative source of energy.

III. Understand the concept Reduce, Reuse and Recycle.

* Do not throw away items that are no use. In-fact reuse them for some other purpose.

IV. Use of energy efficient devices

* CFL lights consumes less electricity compared to others.

Take chances. When rowing forward the boat may rock. – Chinese proverb

FEB MAR APR MAY JUN

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07 JANUARY Monday

Monday	-	7	14	21
Tuesday	1	8	15	22
Wednesday	2	9	16	23
Thursday	3	10	17	24
Friday	4	11	18	25
Saturday	5	12	19	26
Sunday	6	13	20	27

01/20

→ They live longer, consume less electricity and lower electricity bills and also helps to reduce pollution by consuming less energy.

V. Focus on clean energy resource

* clean energy technology like solar, wind and geothermal should have greater use than fossil fuels.
* Govt of various countries also grants solar panels for the private homes.

VI. Control the air pollution at source

VII. " " " by vegetation

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02/2019

Monday	–	4	11	18	25
Tuesday	–	5	12	19	26
Wednesday	–	6	13	20	27
Thursday	–	7	14	21	28
Friday	1	8	15	22	–
Saturday	2	9	16	23	–
Sunday	3	10	17	24	–

02.week

JANUARY
Tuesday
08

8/357

Water pollution

Water pollution is the contamination of water bodies (like oceans, seas, lakes, rivers, groundwater) usually caused by due to human activity.

Water pollution is any change in the physical, chemical or biological properties of water that will have some adverse effect for the living organisms.

Types of water pollution

(i) Groundwater pollution
(ii) Surface water pollution

Causes of water pollution

* Sewage :- Emptying the drains and sewers in fresh water bodies causes water pollution. The problem is severe in cities.

* Industrial waste :- Industrial wastes containing toxic chemicals, acids, alkalies, metallic salts etc are source of water pollution. They also cause thermal pollution of water.

* Synthetic Detergents :- Synthetic detergents used in washing produce foam and pollute water.

He who praises you for what you lack, wishes to take from you what you have. – Don Juan Manuel

FEB MAR APR MAY JUN

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09 JANUARY Wednesday

Monday	–	7	14	21	
Tuesday	1	8	15	22	
Wednesday	2	9	16	23	
Thursday	3	10	17	24	
Friday	4	11	18	25	
Saturday	5	12	19	26	
Sunday	6	13	20	27	

01/201

02.wee

9/356

* Agrochemicals: Agrochemicals like fertilizer (containing Nitrate and phosphate) and pesticides washed by rain water and pollute surface water.

* Oil: Oil spillage into sea water during drilling and shipment pollute it.

Effects of water pollution

* Diseases: In humans, consuming polluted water in any way has many disastrous effects on our health. It causes typhoid, cholera, hepatitis etc.

* Destruction of Ecosystem:

Total ecosystem is dependent upon water directly or indirectly. Water pollution can cause an entire ecosystem to collapse.

* Eutrophication: Chemical in water bodies, encourage the growth of algae. These algae form a layer on top of the pond and lake. Bacteria feed on this algae and this decreases the amount of oxygen which causes severe damage to aquatic life.

Reason can answer questions, but imagination has to ask them. – Ralph Gerard

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* Affects the food chain:
When toxins and pollutants in the water are consumed by aquatic animals which are then consumed by human indirectly affects food chain.

prevention/control measures/solution of water pollution;

→ possible re-use/recycle of industrial waste should be encouraged.

→ Minimum, appropriate quantity of agro-chemical should be used.

→ Treatment plant should be constructed and Govt should also help by funding.

→ To conduct seminar and training course, who are directly or indirectly engaged in water pollution control.

→ Various scientific processes like electro dialysis, ion exchange, reverse osmosis to be used for removal of water pollutants.

→ public awareness must be done through TV, radio, Newspaper to control water pollution.

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11 JANUARY Friday

					01/201
Monday	–	7	14	21	
Tuesday	1	8	15	22	
Wednesday	2	9	16	23	
Thursday	3	10	17	24	
Friday	4	11	18	25	
Saturday	5	12	19	26	
Sunday	6	13	20	27	

11/354 02.wee

SOIL POLLUTION

Soil pollution refers to anything that causes contamination of soil and degrades the soil quality.

Soil contamination or soil pollution can occur either because of human activities or because of natural processes.

Causes of soil pollution

Pesticides

There are synthetic toxic chemicals that definitely kill different types of pests and insects causing damage to agriculture but it has many ecological disadvantage like soil pollution.

They are also insoluble in water and non-biodegradable.

Industrial waste

Industries are the worst polluters of the soil with all chemicals they release into the environment in solid and liquid form.

Garbage pollution

Garbage that can not be recycled is disposed of carelessly to the land and pollute them effectively.

A man can stand anything except a succession of ordinary days. – Johann Wolfgang Von Goethe

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* Solid waste

Disposal of plastics, cans etc are also causes soil pollution. Disposal of electrical goods such as batteries causes an adverse effect on the soil. (e.i lithium present in batteries can cause leaching of soil).

* Urban activites like regular construction. with improper drainage polluted the soil. leaking of sewerage system can also affect soil quality.

Effects of soil pollution

* poor crop quality:- Regular use of chemical fertilizers, inorganic fertilizers decrease the fertility of soil and decreases the quality of soil.

* Harmful Effect on human Health:
Those people affected who are living nearby the degraded land. Living, working and playing in polluted soil can cause respiratory diseases, skin diseases etc.

* Water pollution :- polluted soil contaminated in water sources by runoff of water and causing water pollution.

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13 JANUARY Sunday

Monday	–	7	14	21
Tuesday	1	8	15	22
Wednesday	2	9	16	23
Thursday	3	10	17	24
Friday	4	11	18	25
Saturday	5	12	19	26
Sunday	6	13	20	27

01/26

02. We

13/352 * Negative impact on Ecosystem and biodiversity.

Soil is an important habitat and is the house of various species. So, due to pollution of soil they are lossing their life which affected ecosystem.

prevention/solution/control measure of soil pollution

→ Use of correct farming techniques.

→ Recycling of waste before disposal

→ Use of organic fertilizers instead of chemical fertilizers and pesticides.

→ Community education and awareness.

→ proper maintainance of sewage system.

→ proper disposal method of household and industrial waste.

*

Truth like surgery may hurt, but it cures. – Han Suyin

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Marine pollution
(Sea/ocean)

Sources / cause of marine pollution

→ Rivers are the main source of marine pollution. They carry wastes in their drainage and joins the sea/ocean. It carries industrial waste, detergents, plastics which pollute sea/ocean water.

→ Bigger countries like india, situated near or along the coast line. Every large amount of wastes from hotels, wastes from human activities mixed in sea water and pollute it.

→ ships which carry toxic substances, lubricating oil, paints, fuels, some times by accident or by leakages pollute the marine water.

→ Testing of atomic weapon, missiles are also causing marine pollution.

→ Harmful substances from nuclear power station fertilizers, pesticide industries also causing marine water pollution.

→ Marine pollution also caused by oil drilling in seas.

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15 JANUARY Tuesday

Monday	–	7	14	21
Tuesday	1	8	15	22
Wednesday	2	9	16	23
Thursday	3	10	17	24
Friday	4	11	18	25
Saturday	5	12	19	26
Sunday	6	13	20	27

01/20

03.We

15/350 Effects

→ When oil mixed with water, it becomes very dangerous for marine life. Thousands of birds killed every year because once the oiled

→ Oil of sea also affects sensitive flora and fauna.

→ When dumped plastic material consumed by marine animal, they get affected by "Ulcer". It reduces hunger.

→ Serious diseases like "cancer" are caused when affected animal are taken by man from sea/ocean.

→ By marine pollution, many countries are losing their economy.

Control measures/prevention/solution of marine pollution

→ Various research organizations, institutions are working in the prevention of marine pollution.

→ port authorities are alert and introduced antipollutant measures.

Music is moonlight in the gloomy night of life. – Jean Paul

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02/2019

Monday	-	4	11	18	25
Tuesday	-	5	12	19	26
Wednesday	-	6	13	20	27
Thursday	-	7	14	21	28
Friday	1	8	15	22	-
Saturday	2	9	16	23	-
Sunday	3	10	17	24	-

JANUARY
Wednesday
16

16/349

03.week

→ Dumping of oil, toxic substances etc should be banned.

→ Developmental activities on coastal areas should be minimized.

→ Nuclear explosion and other nuclear activities in sea should be minimized.

→ Awareness programmes to be done to reduce the amount of waste.

→ Drilling should be banned in coastal area.

→ Wastes should be recycled in waste management plants.

FEB
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MAY
JUN

It is impossible to walk rapidly and be unhappy – Howard Murphy

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17 JANUARY Thursday

unwanted sound

NOISE POLLUTION

Monday		7	14	21
Tuesday	1	8	15	22
Wednesday	2	9	16	23
Thursday	3	10	17	24
Friday	4	11	18	25
Saturday	5	12	19	26
Sunday	6	13	20	27

01/20

17/348

03.wee

The unwanted, unpleasant sound which causes discomfort and disturbance to all living beings is known as noise pollution.

Sound frequency → Unit → Hertz (Hz)

loudness → decibel (dB)

Humans can listen 20 Hz - 20000 Hz frequency of sound.

sounds < 20 Hz → infrasonic sound

sounds > 20000 Hz → ultrasonic sound.

→ Human speech is of 200 - 3000 Hz

Causes of noise pollution

→ Big machines which produces large amount of noise

→ Big machines which are used in industries are capable of producing noise.

→ Various equipment like generator, compressors, exhaust fan, grinding mills are the source of noise pollution.

→ Social events like marriage, parties, place of worship are the source of noise pollution as people play song on full volume for a long time.

After silence, that which comes nearest to expressing the inexpressible, is music. – Aldous Huxley

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→ A large number of vehicles on road, airplanes, underground trains produce heavy noise which leads to noise pollution

→ The construction equipments are also a leading cause/source of noise pollution.

→ Household products like TV, vaccum cleaner AC also contributes in noise pollution.

Effects

→ Sounds above 150 dB may cause permanent hearing loss of humans.

→ Contracts of blood vessels may occur

→ Heart attack, neurological problem, birth defects may occur.

→ The optical system of human beings is also affected by noise pollution.

→ Loud continous sound may reduce working efficiency as well as mental capability.

→ Loud noise may decrease output of blood from heart and thus heart beat rate affected

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19 JANUARY Saturday

Monday		7	
Tuesday	1	8	14
Wednesday	2	9	15
Thursday	3	10	16
Friday	4	11	17
Saturday	5	12	18
Sunday	6	13	19

19/346 prevention/control measures of noise pollution 03

⇒ "Reduction of noise at the source of its origin." It can be achieved by replacement of noisy rattling devices.

⇒ Application of sound proofing techniques to slowed down loud noises in industries and manufacturing plants.

⇒ Keeping residential localities free of noisy industries, busy highways and aerodromes.

⇒ Oiling of various noise producing machines and equipments can also control noise pollution.

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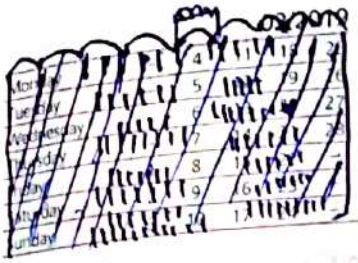

JANUARY
Sunday
20

03.week

20/345

Thermal pollution...

An increase in optimum water temperature by industrial process may be called as thermal pollution.

Causes

→ Coal-fired power plants :- Some thermal power plants use coal as fuel. Coal-fired power plants is a major source of thermal pollution. (To cool the coils they use water)

→ Industrial Effluents :- Industries generating electricity requires large amount of water for heat removal.

→ other industries like textile, paper, and pulp and sugar industries also release heat in less quantity.

→ Nuclear power plant :- It emits a large amount of unutilized heat and traces of toxic materials to nearby streams which causes thermal pollution.

→ Hydro-electric power :- The generation of hydroelectric power also causes thermal pollution.

FEB MAR APR MAY JUN

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21 JANUARY Monday

					01/20
Monday	-	7	14	21	
Tuesday	1	8	15	22	
Wednesday	2	9	16	23	
Thursday	3	10	17	24	
Friday	4	11	18	25	
Saturday	5	12	19	26	
Sunday	6	13	20	27	

21/344 Effects of thermal pollution 04.wee

→ Reduction of dissolved oxygen in water bodies.

i.e D.O. content

In 32°F → 14.6 ppm

In 64°F → 6.6 ppm.

→ The marine and water ecosystem affected largely by thermal pollution.

→ Change in water properties like decrease in density, solubility of gases and increases settling speed of suspended particles.

→ Increasing toxicity of water (i.e potassium cyanide doubled its toxic effect with extra heat)

→ Increasing various dangerous diseases in aquatic life.

→ The thermal pollution also affects the reproduction rate of aquatic animals.

→ Undesirable changes in algae production.

No bird soars too high, if he soars with his own wings. – William Blake

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02/2019

Monday	–	4	11	18	25
Tuesday	–	5	12	19	26
Wednesday	–	6	13	20	27
Thursday	–	7	14	21	28
Friday	1	8	15	22	–
Saturday	2	9	16	23	–
Sunday	3	10	17	24	–

JANUARY
Tuesday

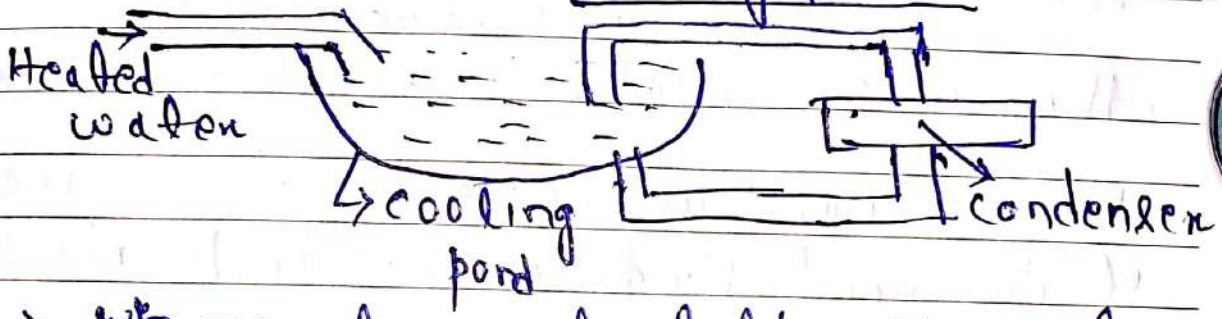
22

04.week

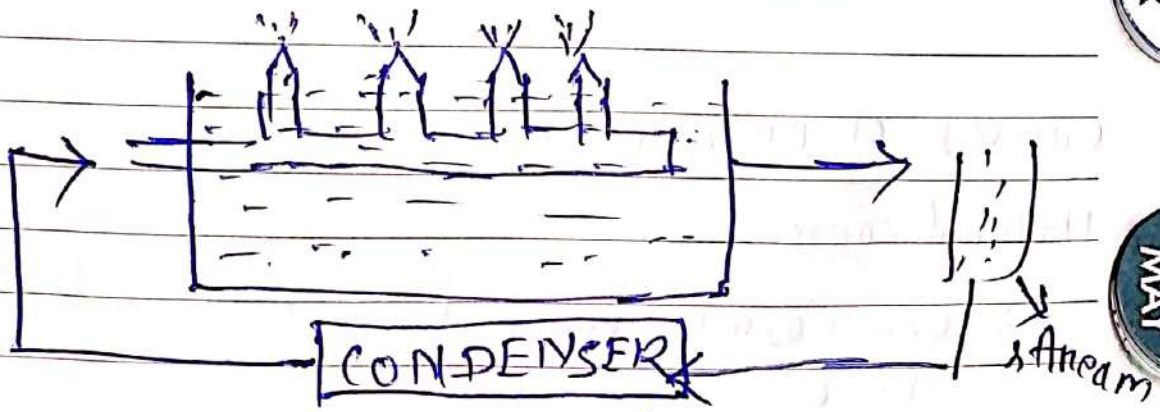
control measure of thermal pollution

→ Heated water from industries can treated before discharging directly to the water bodies.

→ Heated water can be treated by installation of "cooling ponds".



→ We can also control the thermal pollution by "spray pond"



→ Industrial treated water can be recycled for domestic use or industrial heating.

Happiness is not a matter of intensity, but of balance and order and rhythm and harmony. – Thomas Merton

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23 JANUARY Wednesday

Monday		7	14
Tuesday	1	8	15
Wednesday	2	9	16
Thursday	3	10	17
Friday	4	11	18
Saturday	5	12	19
Sunday	6	13	20

NUCLEAR HAZARD

23/342 Hazard: dangerous to human being

04

Nuclear hazards are the threats caused by the invisible and odourless contamination of environment by the presence of radio-active materials.

Radio-active elements are :-
Uranium-235, uranium 283, thorium 232, radium 226, carbon 14 etc.

Nuclear radiation arises from hundred of different kinds of unstable atom.

The various radiation are.
→ ~~From~~ Emission of alpha particle (~~return~~)
" " Beta "
" " Gama "

Causes of nuclear hazard

Natural cause

→ ~~Com~~ Cosmic radiation received from space

→ Radon gas emmited from by volcanic rocks and uranium ore.

→ Due to radio-active element's free presence in lithosphere

Man becomes man only by his intelligence, but he is man only by his heart. – Henri Frederic Amiel

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02/2019

Monday	-	4	11	18	25
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Wednesday	-	6	13	20	27
Thursday	-	7	14	21	28
Friday	1	8	15	22	-
Saturday	2	9	16	23	-
Sunday	3	10	17	24	-

JANUARY
Thursday
24

04.week

24/341

man made causes

→ Nuclear waste handling and disposal.
(Nuclear waste can not be treated chemically or biologically, it can stored in a tight container)
It may contaminate air.

→ Use of radioactive materials in defence weapons production.

→ Nuclear explosions and experiment of nuclear weapons.

→ Mining process of radio-active ores.

→ Use of radio-active isotopes in medical, industrial and research application like X'ray.

Effects of nuclear hazard / pollution

→ It causes damages to cell membranes, mitochondria, cell division.

→ people suffer from blood cancer and bone cancer.

→ Radiation cause "mutation" in DNA, which affect future generation

→ Loss of hair, vomiting also happens.

FEB MAR APR MAY JUN

To sit in the shade on a fine day, and look upon verdure is the most perfect refreshment. – Jane Austen

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25 JANUARY Friday

Monday				
Tuesday		7	14	
Wednesday	1	8	15	
Thursday	2	9	16	
Friday	3	10	17	
Saturday	4	11	18	
Sunday	5	12	19	
	6	13	20	

25/340 → Higher dose of nuclear radiation damages the tissues of brain, heart etc!

Control measures / prevention

→ Leakage from nuclear reaction, careless handling, transport and use of radio-active fuels, fission products should be stopped totally.

→ safety measures should be done against nuclear accidents.

→ Waste disposal must be careful, efficient, effective.

→

There is not a lot we can say to the

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02/2019

Monday	–	4	11	18	25
Tuesday	–	5	12	19	26
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Thursday	–	7	14	21	28
Friday	1	8	15	22	–
Saturday	2	9	16	23	–
Sunday	3	10	17	24	–

SOLID WASTE MANAGEMENT

JANUARY Saturday **26**

04.week

26/339

Intro

Any material that is thrown away as useless and unwanted by human or from animal activities is considered as solid waste.

Various sources of solid waste are
→ Residental, commercial, industrial buildings.
→ Municipal area, treatment plants
→ Agricultural products, construction site
→ Manufacturing plants.

Solid waste causes various diseases like cholera, diarrhoea, dehydration etc.
→ Asbestos is caused due to Asbestos.
→ Mina Matta disease " " " Mercury

Solid waste management :-

The management of solid waste from the point of generation to final disposal have been grouped into "six" Functional Elements which is as follows :-

FEB MAR APR MAY JUN

You only live twice. Once when you are born and once when you look death in the face. – Ian Fleming

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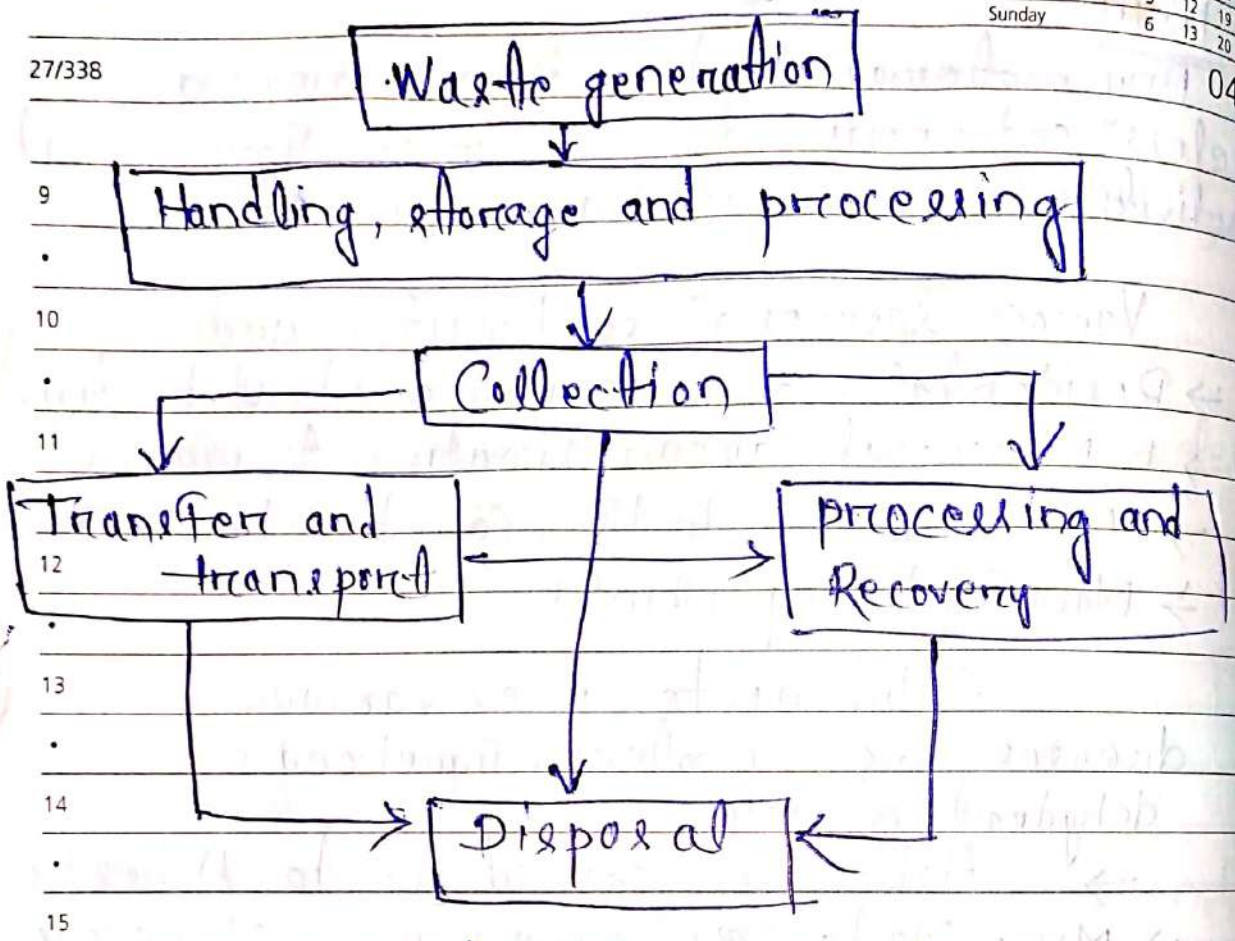
27 JANUARY Sunday

27/338



So, Solid waste management is defined as collection, transport, processing or disposal of waste material so as to reduce their effects on local environment and community.

Various methods are.

① physical removal :- It is done by manual activities like, collection of wastes by sorting out re-usable, decomposable and non-decomposable wastes.

A good plan implemented today is better than a perfect plan implemented tomorrow. – George Patton

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, then disposal becomes easy.

Dumping :- Transfer of solid waste from place of collection to the site of disposal is known as dumping. Municipal bodies collect the wastes and dump them in a suitable place far from human habitation.

3R or Reduce, Re-use and Recycle of solid waste

We should reduce household waste by using maximum part of goods. Before throwing outside, we should select the parts for Reuse and Recycle.

Composting :- The process of making usuable fertilizer by decomposable waste with the help of micro-organisms is known as composting.

Thermal process

Burning of solid waste should be done in a controlled condition both in presence and absence of air.

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29 JANUARY Tuesday

29/336

Role of an individual in prevention of pollution

→ We should go through the field of environmental awareness to protect the pollution.

→ We should go place to place to teach the lesson of awareness and prepare volunteers.

→ Give the message to save environment through paper, magazine, T.V. & Radio.

→ To promote for plantation and conservation of forest.

→ To organize seminar, on the subject related to forest.

→ population growth should be reduced.

→ We should not use CFC containing refrigerators.

→ We should discourage the use of more fertilizer, pesticides.

→ We should encourage the use of bio-fertilizer

Great things are not done by impulse, but by a series of small things brought together. – Vincent Van Gogh

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02/2019

Monday	–	4	11	18	25
Tuesday	–	5	12	19	26
Wednesday	–	6	13	20	27
Thursday	–	7	14	21	28
Friday	1	8	15	22	–
Saturday	2	9	16	23	–
Sunday	3	10	17	24	–

05.week

JANUARY
Wednesday
30

30/335

Disaster management

Flood management

→ Construction of suitable dams to store sudden flow of water due to heavy rain

→ Artificial reservoir should be done at the upper catchment area

→ proper network should be created in the inter-connected rivers which can decrease flood.

→ plantation of trees can save flow of water to human habitat.

→ Proper warning of heavy rainfall and regular update about flood can help people to stay in safe area.

FEB MAR APR MAY JUN

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31 JANUARY Thursday

Monday		7	14	01
Tuesday	1	8	15	
Wednesday	2	9	16	
Thursday	3	10	17	
Friday	4	11	18	
Saturday	5	12	19	
Sunday	6	13	20	

05.

31/334 Cyclone management

Cyclone are the atmospheric storm. it is caused due to rise in temperature in the sea surface for a long time.

→ Cyclone causes large damage to crops, animal, and human lives

→ It uproots trees and destroys house and buildings.

→ Electric and telephone towers are destroyed.

→ Communication is cut off due to cyclones.

Management

→ Cyclones are tracked by satellite pictures and accordingly preventions are made.

→ people are informed before the cyclone and should shift themselves along with their valuable goods.

Try not to become a man of success, but rather try to become a man of value. – Albert Einstein

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01 FEBRUARY Friday

32/333

→ Cyclone homes should built at different areas.

→ Tents, funds should help people the cyclone.

→ Tree plantation along the coastal area should be done. (mangrove)

→ Good scientific information can help the direction and speed of cyclone.

Earthquake management

Earthquakes are the effect of events happening inside the earth's interior.

Earthquakes are seismic waves coming from the interior of the earth.

The magnitude of the earthquakes (intensity of energy coming from the quake) is measured by a scale called "Richter scale".

Range of richter scale → 0 – 9

Earthquake measuring over "6 point" on richter scale considered to be very damaging

The reward of one duty done is the power to fulfill another. – George Eliot

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Monday	-	4	11	18	25
Tuesday	-	5	12	19	26
Wednesday	-	6	13	20	27
Thursday	-	7	14	21	28
Friday	1	8	15	22	29
Saturday	2	9	16	23	30
Sunday	3	10	17	24	31

FEBRUARY
Saturday
02

* The exact point where earthquake originates called — [focus]

* The point lying vertically above the focus is defined as [Epicenter]

Management

→ The house building procedure should be "earthquake resistant"

→ Bamboo material or very lightweight material should be used in earthquake prone areas.

→ Much scientific research should be done to know about the earthquake earlier.

→ All big structures i.e dams, bridges, buildings must be built by earthquake resistant.

→ people in earthquake prone area should be taught the ways to save themselves during earthquake.

→ The "Geological survey" Engencies of the countries should aware the people about the earthquake prone areas.

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03 FEBRUARY
Sunday

Monday	-	4	11
Tuesday	-	5	12
Wednesday	-	6	13
Thursday	-	7	14
Friday	1	8	15
Saturday	2	9	16
Sunday	3	10	17

34/331

LANDSLIDES MANAGEMENT

05

A landslide is the rapid mass movement of soil, mud or rocks downhill or downward due to the pull of gravity.

Deforestation, rock ~~mining~~ mining, bad agriculture practice may cause landslide.

Management

Before a landslide

→ The query of the ground property where house to be built should properly done.

→ Find out whether the area in which you live is prone to landslide or not.

→ If you live in a high risk area an evacuation (transfer) plans should be prepared.

→ Afforestation should be done highly

During a landslide

→ If you are inside of a building, stay inside; don't leave your home.

Silent gratitude isn't very much use to anyone. — Gertrude Stein

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FEBRUARY
Monday
04

→ If you are at the river, try to leave the area as soon as possible.

→ If driving, remain alert and look out for collapsed pavement or fallen rocks.

After a landslide

→ Flooding may occur after a landslide, so be alert.

→ Check your property (building) for any structural damage.

→ Replant (Replace) damaged ground immediately since soil erosion could caused by loss of ground cover.

→ Don't go into the landslided area as further landslide may occur.

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UNIT-6 Social issues and the environment

05 FEBRUARY Tuesday

36/329

From unsustainable to sustainable development

Unsustainable → not able to maintain at the current rate

Sustainable → able to maintain at the current rate

Unsustainable development means the developments which are cost of our life supporting system like air, water, soil and other natural resource

* To be sustainable, development must process both economical and ecological sustainability.

- Sustainable development can be done by the following ways.

1. To promote environmental education and awareness.
   Media can also be helpful in developing the love for environment.
   From childhood, we should develop a feeling of "love to earth"

Today is your day! Your mountain is waiting. So get on your way. – Dr. Seuss

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03/2019

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Tuesday	-	5	12	19	26
Wednesday	-	6	13	20	27
Thursday	-	7	14	21	28
Friday	1	8	15	22	29
Saturday	2	9	16	23	30
Sunday	3	10	17	24	31

FEBRUARY
Wednesday
06

06.week → 3R approach

37/328

Three R means Reduce, Reuse and Recycle. we should reduce excessive ~~amount~~ use of natural resource, but use them again and again.

→ Appropriate technology :- The technology should use less resource and produce minimum waste. All development should be eco-friendly.

→ To utilize resources in a proper limit which should not affect the biodiversity and ecosystem of the earth.

→ Economic growth of poor countries should be the Key of developed countries.

URBAN PROBLEM RELATED TO ENERGY

Urban areas are developing very fast. uncontrolled population and extending of urban areas causes energy problems.

~~As they~~ Energy problem day by day becoming serious.

MAR APR MAY JUN

Life must be measured by thought and action, not by time. – John Lubbock, Conductor

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07 FEBRUARY Thursday

38/327 Energy demand is higher than the production.

Cause of energy problems in urban areas are:

- Increasing use of energy for domestic and commercial purposes.
- Industrial plants using a large amount of energy
- Increasing in number of vehicles.
- Decreasing production of Hydro-electricity due to insufficient rains.
- For ~~higenic~~ hygienic lifestyle of rich people in urban areas.

Control measures

* By various awareness programme, and by controlling the urbanization we can control the energy related problems.

* Development of solar system, wind energy, bio-mass energy should be done.

* The Govt. should take care of energy theft.

Opportunities are usually disguised as hard work, so most people don't recognize them. – Ann Landers

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FEBRUARY
Friday
08

39/326

WATER CONSERVATION

Water is one of the most needed natural resource for human beings. Without water life is not possible.

Conservation of water is, therefore an absolute necessity of today.

The following steps should be taken for conservation of water.

→ ① Reuse and Recycling of water

We should use water again and again in industries by various techniques. The heated water in thermal power plants should re-use after cooling.

→ ② Efficient distribution system. The distribution of river water should evenly processed by building dams and barrages. As the river water is flowing un-used to the sea.

→ ③ Improvement of under ground storage capacity.

→ ④ Desalination of sea water: If the salt content can removed from sea, it can be used.

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09 FEBRUARY Saturday

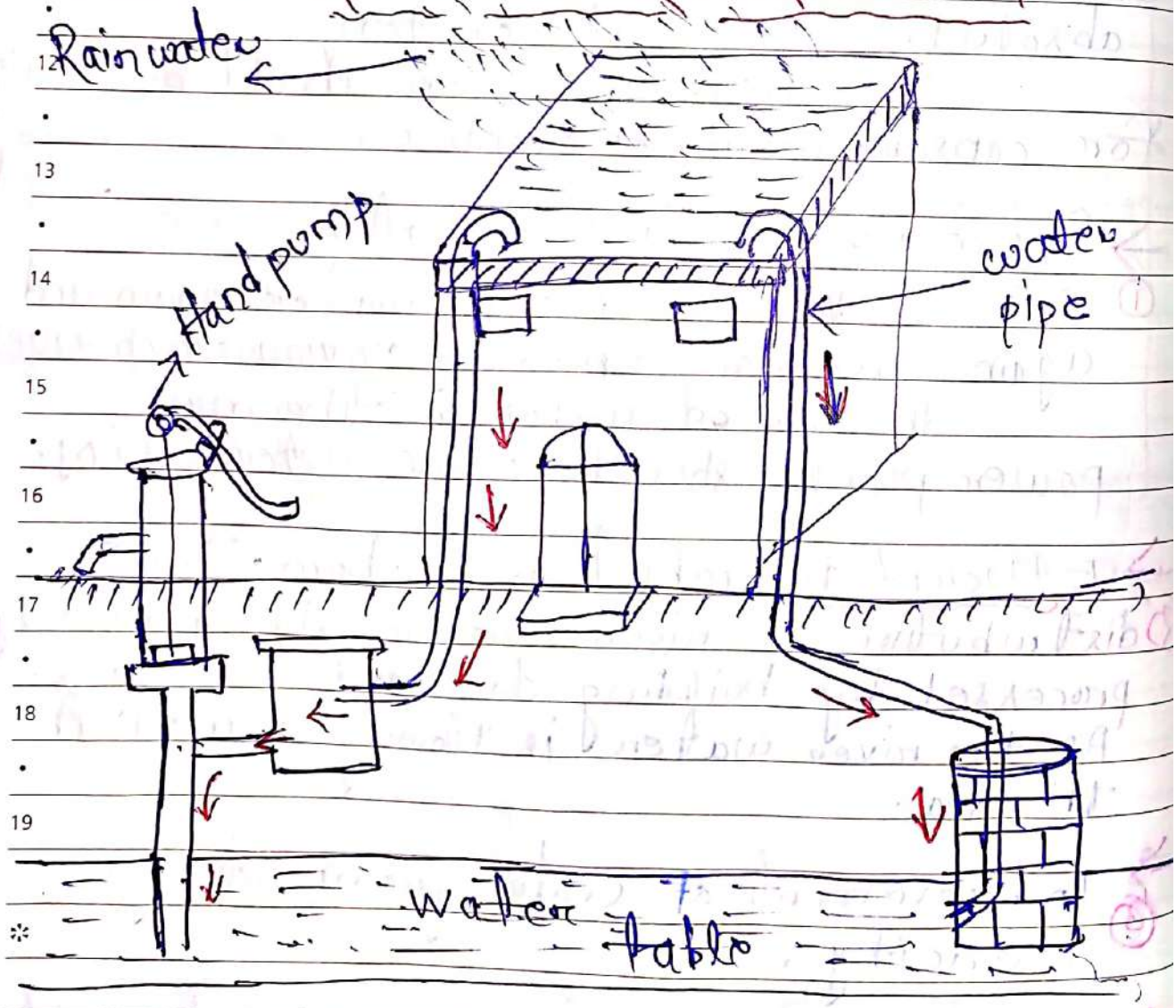
Monday	–	4	11
Tuesday	–	5	12
Wednesday	–	6	13
Thursday	–	7	14
Friday	1	8	15
Saturday	2	9	16
Sunday	3	10	17

40/325

⑤ Afforestation should be done at the hill slope to check loss of water.

⑥ Enhancement of storage capacity. can also helps to conserve water

RAIN WATER HARVESTING



The fruit of love is service, The fruit of service is peace, And peace begins with a smile. – Mother Teresa

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Rain water harvesting means collecting rain water on the roof of buildings and storing it underground for later use.

As shown in the figure the rain water is collected by roof top and then stored in the dug well by using pipes.

Then the stored water can easily used by handpumps.

* By this rain water harvesting increases water availability and also increases ground water level.

Objectives

→ To store excess water for future use
→ To improve physical and chemical quality of ground water.
→ To reduce storm water run-off.
→ To prevent salinity in coastal areas.
→ To restore ground water which depleted by over-exploitation.

Advantages

→ Rise in ground water levels
→ Increase in availability of grand water throughout the year.
→ Reduction in flood hazard.

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11 FEBRUARY Monday

Monday	-	4	11
Tuesday	-	5	12
Wednesday	-	6	13
Thursday	-	7	14
Friday	1	8	15
Saturday	2	9	16
Sunday	3	10	17

42/323 → Soil erosion also reduced.
→ Reduction in the use of energy.
→ Assuring sustainability of ground water resource.

WATERSHED MANAGEMENT

Watershed :- It is the land area which drains water to a common water bodies like river, lake, sea.

All lands on earth are part of one water-shed.

> Thus watershed is thus the land and water area, which contributes runoff to a common water body.

So this watersheds are now not safe due to uncontrolled unplanned land use like overgrazing, deforestation, mining soil erosion and industrilization.

Therefore, watershed management is necessary to save our environment, eco-system and food chain and for pollution control.

I am prepared to go anywhere, provided it be forward. – David Livingstone

Scanned by CamScanner

7. week

objectives

→ To increase agricultural production.
→ pollution control
→ over exploitation of resources should be minimized.
→ flood control
→ wild life preservation.
→ Erosion control.

Various measures in watershed management

→ Scientific mining and quarrying must be done in watershed areas.
→ Water harvesting in watersheds to be used in dry season in low rainfall area.
→ To promote soil binding plants
→ Contour cropping, terracing should be used to minimize run-off and soil erosion.
→ Farmers and tribal people in should participate in watershed management programmes.

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13 FEBRUARY Wednesday

Monday		4	11	02
Tuesday		5	12	
Wednesday		6	13	
Thursday		7	14	
Friday	1	8	15	
Saturday	2	9	16	
Sunday	3	10	17	

07

44/321

CLIMATE CHANGE

1st of all you have to know the difference between weather and climate

weather	climate
→ short term	→ long term
→ limited area	→ ~~wide area~~ area wide
→ Can change rapidly	→ seasonal changes.
→ difficult to predict	→ Measured over long spans of time. (35 years)

Climate change is natural phenomenon and has been occuring since the formation of earth.

Climate change is a change in the average weather of a given area or region. This includes temperature wind patterns and precipitation.

Causes of climate change

→ Deforestation
→ Increase vehicle
→ Industries
→ Forest fire
→ chemical fertilizer
→ Methane emission from animal

Very little is needed to make a happy life; it is all within yourself, in your way of thinking. – Marcus Aurelius

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Effects of climate change

45/320

→ Extinction of species.
→ Acidification of ocean.
→ Adverse effect on health.
→ Oxygen depletion.
→ Increase in ~~temp~~ of water bodies.
→ Various natural disasters.

Solution ⇒ → Using solar energy.
→ Using Hydropower, wind energy
→ By using Eco-friendly transport
→ Afforestation.

GLOBAL WARMING :-

→ The average global temperature is 15°C.

The lower most layer of atmosphere (troposphere) traps the heat by natural and artificial process due to some gases called Green house gases. So the increase in temperature of the environment ~~due to~~ is known as GLOBAL WARMING.

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15 FEBRUARY Friday

Monday	–	4	11	02/
Tuesday	–	5	12	18
Wednesday	–	6	13	19
Thursday	–	7	14	20
Friday	1	8	15	21
Saturday	2	9	16	22
Sunday	3	10	17	23
				24

07. We

46/319 Various green house gases which results global warming are.

→ Carbon dioxide (CO_2)

→ Ozone (O_3)

→ Methane (CH_4)

→ Nitrous oxide (N_2O)

→ Chlorofluorocarbons (CFC_s)

→ Water vapours.

→ change in temperature by more than 2°C is dangerous for various eco-systems.

ACID RAIN

Acid rain means the presence of excessive acids in rain water.

Acid rain is a mixture of acids mainly H_2SO_4 and HNO_3.

$$H_2O + SO_3 \longrightarrow H_2SO_4$$

$$H_2O + NO_2 \longrightarrow HNO_3$$

Art attracts us only by what it reveals of our most secret self. – Jean-Luc Godard

Scanned by CamScanner

FEBRUARY
Saturday
16

47/318

H_2SO_4 (sulphuric acid) is the main source of acid rain which contribute (60-70 %) and HNO_3 (Nitrogen oxide) contributes (30-40 %).

So due to these acids the pH value of water get lowered fro 1 to 5.

Effects of acid rain

→ Reduction in fish population.
→ Blue algae and many bacteria are killed
→ Acid rain also destroys the crop field.
→ Acid rain corrodes house, monuments, statues, bridges etc.
→ It affects the structure of marble which is known as "Stone-leprosy."

MAR
APR
MAY
JUN

A man is the sum of his actions, of what he has done, of what he can do, nothing else. – Mahatma Gandhi

Scanned by CamScanner

17 FEBRUARY
Sunday

Monday	-	4
Tuesday	-	5
Wednesday	-	6
Thursday	-	7
Friday	1	8
Saturday	2	9
Sunday	3	10

48/317

OZONE LAYER DEPLETION

ozone layer

Ozone (O_3) is a naturally occuring gas found through out atmosphere.

The deposition of ozone gas is max^m at 15-30 Km height from the earth surface and formed a layer.

This layer is known as "Ozone layer".

The thickness of the ozone layer is measured by Dobson unit (DU).

The avg thickness of ozone layer is 230 DU.

* The ozone layer strongly absorbs or blocks the dangerous Ultra-Violet (UV) rays radiates from the sun and protect the life on earth.

People may doubt what we say, but they always believe what we do. – Don Galer

Scanned by CamScanner

03/2019

Monday	-	4	11	18	25
Tuesday	-	5	12	19	26
Wednesday	-	6	13	20	27
Thursday	-	7	14	21	28
Friday	1	8	15	22	29
Saturday	2	9	16	23	30
Sunday	3	10	17	24	31

FEBRUARY
Monday

18

08.week

49/316

* So ozone layer depletion means the reduction of the amount of ozone gas (O_3) in stratosphere.

* Industries that manufacture things like soap, solvent, Air conditioner, Refrigerators etc emits a dangerous gas known as (CFCs) (Chlorofluorocarbons).

* The ozone depletion begins when CFCs get broken by UV rays.

The breaking action releases chlorine atoms. and one chlorine atom can break 100,000 ozone molecule.

Thus ozone layer depletion occurs.

* Other gases like → Methyl chloroform
→ Hydrofluorocarbons
also causes ozone depletion.

MAR APR MAY JUN

The gem cannot be polished without friction, nor man perfected without trials. – Chinese proverb

Scanned by CamScanner

19 FEBRUARY Tuesday

Monday
Tuesday
Wednesday
Thursday
Friday 1
Saturday 2
Sunday 3

50/315 Effects of ozone layer depletion 08

→ Skin cancer, sunburn

→ Blindness, eye diseases.

→ Reduction in crop production.

→ Wood, plastic, rubber material get ~~effect~~ degraded by UV radiation

prevention

→ Limit private vehicle.

→ Eco-friendly household.

→ Avoid using pesticide.

→ Ban on CFCs emitting products.

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NUCLEAR ACCIDENTS AND HOLOCAUST

↳ (Nuclear War)

Japanese towns of Hiroshima and Nagasaki

→ The 1st atom bomb was exploded about 580 metres high above ground level in Hiroshima on Aug 6, 1945.

At least 100,000 people were reported killed in Hiroshima.

→ The 2nd atom bomb was exploded in Nagasaki and around 45 000 people were killed.

It destructs the buildings and destroyed the area of 15 Km.

→ The atom bomb in Hiroshima used Uranium-235

→ The " " " Nagasaki used plutonium-239

public suffered various disease in those area till a decade (10 years) in that area.

MAR APR MAY JUN

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1 metric ton : 1 ton = 1000 Kg

1 megaton = 10 lakh ton

= 100 crore Kg

21 FEBRUARY Thursday

Monday			
Tuesday	–	4	11
Wednesday	–	5	12
Thursday	–	6	13
Friday	–	7	14
Saturday	1	8	15
Sunday	2	9	16
	3	10	17

52/313

The first hydrogen bomb was exploded in Bikini island near australia.

In 1957 and 58 USA, Russia and great britain test 85 megatons of nuclear weapon which causes several damages to human beings and environment.

~~1 ton~~ The explosion at the Chernobyl power plant in Ukraine is the worst nuclear disaster in 1986.

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POPULATION EXPLOSION

A sudden growth or rapid rise in population which is over the limits is known as population explosion.

Eg

population of world in 1825 - 1 billion
In next 100 year - 2 billion
In next 35 year - 3 billion
In next 15 year - 4 billion
In next 12 year - 6 billion

And now around 7.2 billion people are available in the world.

Causes

→ Accelarating birth rate
→ decrease in death rate
→ Better medical facilities
→ Illiteracy.

Effects

→ Unemployment
→ Illiteracy
→ poor health.
→ generation of waste
→ Air/water pollution
→ ozone layer depletion
→ land degradation,

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population control

54/311

Planned - Reduce birth rate

- Education
- family planning
- Marriageable age.

FAMILY ~~WA~~ WELFARE PROGRAMME

→ This programe lunched in india in 1951 which was rammed "family planning programme"

→ In 1977 it was renamed as "family welfare programme"

→ India is the 1st country to do such programme.

→ The concept of welfare is basically related to quality of life.

→ It includes education, nutrition, health, empolyment,

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safe drinking water, shelter for everyone.

- It is centrally sponsered programme and states receive 100% assistance from central Govt.

VALUE EDUCATION

Education about something's or someone's value.

Anything which satisfies a human need, may be called a thing of 'value'

The various important values are.

(1) Religious value: It is defined as "faith in God"

→ Having faith in religious leaders, worshipping God, speaking truth.

(2) Social value It is defined as charity, Kindness, love, sympathy for the people, the service of mankind.

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25 FEBRUARY Monday

Humanity etc.

③ democratic Value

It is defined as respect for all, absence of discrimination
↳ (ଭେଦଭାବ ନ ରହିବା)

④ Aesthetic Value.
↳ appearence

It is defined as appreciation of beauty, drawing, painting, music dance, decoration, love for literature

⑤ Health value

It is considered to keep the body fit, do exercise, good food etc.

Reason can answer questions, but imagination has to ask them. – Ralph Gerard

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Resettlement and rehabilitation of people: its problem and concern:-

1st of all resettlement means "The settlement of people in a different place." And rehabilitation means: The action of restoring someone to health or normal life through some favourable action that has been damaged.

It is a well-known fact that both natural and human made disasters force people to move out of their land.

For example, the Orissa super-cyclone, latur and gujarat earthquake etc rendered thousands of people jobless.

The strategies formulated for resettlement and rehabilitation of those displaced by development projects can of course be equally applied to those displaced by natural and human made calamities. As far as our country's preparation for coping with the impact of natural and ~~human~~ human made calamities is concerned, we have begun to feel a little aware of negative impacts of such events because they are now occuring at frequent intervals.

As a result there are some institutional measure have been taken by the government by constituting committees at various levels. They

exist on paper and their immediate response to the actual events is yet to come in any significant manner.

But we can say, the beginning has been made and we need to further consolidate the initiatives already taken so that such measure can bring some relief to disaster victims by way of their resettlement and rehabilitation.

AIR (prevention and control of pollution) ACT.

The government of india passed the Air (prevention and control of pollution) Act in 1981 to clean up our air by controlling pollution.

It states that sources of air pollution such as industry, vehicles, powerplants etc are not permitted to release particulate matter, lead, carbon monoxide, sulfer dioxide and other toxic substances beyond a prescribed limit.

To ensure this, Pollution Control Boards (PCBs) have been set up by Govt to measure pollution levels in the atmosphere and at certain sources by testing the air.

This is measured in PPM. / ~~per cubic meter~~ or micrograms per cubic meter. /

Objectives of AIR act

- To provide for the prevention, control and abatement of air pollution.
- To provide for the establishment of central and state boards with a view to implement the act.
- To confer on the Boards the powers to implement the provisions of the act and assign to the Boards functions relating to pollution.

* The owner of the industry may penalized as upto three months of imprisonment and 5000 - 10,000 rupees per day during the pollution phase above standard level.

WATER (prevention and control of pollution) ACT

The government of india formulated Water (prevention and control of pollution) ACT in [1974] to prevent the pollution of water by industrial, agricultural and household wastewater that can contaminate our water sources.

The water act, 1974 with certain amendments in 1978 is an extensive legislation with more than 60 sections for the prevention and control of water pollution.

Among other things, the act provides for constitution of central and state boards for preventing water pollution, powers to take water samples and their analysis, discharge of sewage or trade effluents, minimum and maximum penalties, publication of names of offenders, offences by companies and Govt departments, water laboratories, analysis etc.

The main objective of the water act are to be provided for prevention, control and abatement of water pollution and the maintainance or restoration of the wholesome new of water.

The Central Govt and state govt have set up PCBs to monitor water pollut

Public awareness (about pollution)

pollution has been an increasing world issue since the american industrial revolution.

At least 5000 people die daily due to pollution related issues. Today, pollution is one of the global killers

So, the people among the world need to unite to work as one to conquer pollution problems, especially because the fact is that humans are at fault.

Govt. worldwide are a key opponent in reducing all forms of pollution.

The govt.'s main job is to keep the public informed of it's strides towards reaching a healthy and natural environment for its people.

Govt. programs should promote recycling and raise public awareness of the pollution threats.

Nearly 34% of waste sent to a landfill is recyclable and it is not enough.

So everyone needs to become aware of the solutions and percussions that can be taken to reduce amount of pollution.

The only way to prevent is to raise awareness about the issues caused by pollution.

Chapter - 7

Human population and the environment

population growth and variation among nations

Intro About 250 years ago, population existed relatively small numbers with limited technologies.

⟶ In the last two centuries developments have occured that have created environmental problems beyond nature's capacity.

⟶ Explosive growth of population creats environmental pressure. ~~because~~ and results from the difference betn the rate of birth and death.

Variation of population among nations

Different regions of the world find themselves at different stages of demographic transition from high to low mortality rate.

At present the world's population has crossed 7 billions.

This existing population is also not evenly distributed.

Less developed countries have 80% population while the developed countries have only 20%.

In 1800, the vast maylority of human population, i.e. 86% resided in Asia and Europe with 65% alone in Asia.

In 2011, the world had approx 700 crore population which is expected to increase up to 10 billion by the year 2050 if the same trend continues.

ENVIRONMENT and HUMAN HEALTH:

The interconnectedness of life on earth that nothing we put into the ecosystem simply disappears.

In other words, what we do to planet, we do to ourselves.

pollutants reach humans through food we eat, the water we drink and the air we breathe and endanger our health in both immediate and long-lasting ways.

Outdoor air pollution alone is associated with more than one million deaths and countless illnesses each year across the globe.

For example pesticides we use in crop fields are not degraded and they go directly into the food we eat, they can lead to cancer and antibiotic resistance in the people who eat them

polluted runoff from cities and farms flows into the rivers we draw much of our drinking water from and ends up in our bodies.

Likwise, polluted air have a great impact in human body, Heart diseases, cancer asthma results from polluted air.

Global warming then worsen pollution.

* So we can limit our exposure to toxins as much as possible, on the individual level by
- eating organic food.
- avoiding places and times of heavy air pollution.
- getting our water tested

and lastly we need to advocates who will stop pollution at its source.

Human Rights

~~The following human rights~~

All human beings have the fundamental right to an environment adequate for their health and well being.

Human rights mean access to the unspoiled natural resources that

enable survival, including land, shelter, food, water and air.

Human rights also consists of, the right to information and participation in decision making, freedom of opinion and expression and the right to resist unwanted development.

All these rights are equally important and they are all interdependent. people's livelihoods, their health, and sometimes their very existance depend upon the quality of and their access to the surrounding environment as well as the recognition of their rights to information, participation, security and redress.

Role of information technology in environment and human health.

SUNDAY 13

Information technology has a tremendous potential in the field of environment education and health as in any other field like business, economics, politics or culture.

A number of software have been developed for environment and health studies which are user friendly and can

help an early learners.

Database on environment system :- Database is the collection of interrelated data on various subjects.

The comprehensive database includes wildlife data base, conservation database, Forest cover data base etc. Database also available for various diseases. The following are the types of database.

(i) Nation Management Information System (NMIS)
(ii) Environmental Information System (ENVIS)
(iii) Geographical Information system (GIS)
(iv) The World Wide Web (WWW)

(a) NMIS :- NMIS of the department of science and technology has compiled a database on research and development.

(b) ENVIS :- The ministry of environment and forests, Govt of India has created an information system called ENVIS. It works for generating a network of database in areas like pollution control, coastal ecology, biodiversity

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3	4	5	6	7	8	9
10	11	12	13	14	15	16
17	18	19	20	21	22	23
24	25	26	27	28		

(c) GIS :- Satellite imageries provide us actual information about various physical and biological resources and also to some extent about their state of degradation in a digital form through remote sensing. By this we gather the information of water-logging deforestation, mineral reserve etc.

(d) WWW

With resources material on every aspect, class-room activities and digital files of photos, power point lecture, presentation, animation and quiz has proved to be extremely useful both for the students and the teachers of environmental studies.

Thank you :-

REFERNCES BOOKS:

Sl. NO.	Name of Authors	Titles of Book	Name of Publisher
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2.	D.D.Mishra	Fundamental concepts in Environmental Studies	S.Chand & Co-Ltd
3.	K.Raghavan Nambiar	Text book of Environmental Studies	SCITECH Publication pvt.Ltd
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GEOTECHNICAL ENGINEERING

SPINTRONIC TECHNOLOGY & ADVANCE RESEARCH (STAR)

DEPARTMENT OF CIVIL ENGINEERING

3RD SEMESTER

DIPLOMA
LECTURE NOTES ON
GEOTECHNICAL ENGINERRING
BY

SMRUTI SASWATI DASH

SYLLABUS

1

Introduction

1.1 Soil and Soil Engineering

1.2 Scope of Soil Mechanics

1.3 Origin and formation of soil

2

Preliminary Definitions and Relationship

2.1 Soil as a three Phase system.

2.2 Water Content, Density, Specific gravity, Voids ratio, Porosity, Percentage of air voids, air content, degree of saturation, density Index, Bulk/Saturated/dry/submerged density, Interrelationship of various soil parameters

3

Index Properties of Soil

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3.2 Specific Gravity

3.3 Particle size distribution: Sieve analysis, wet mechanical analysis, particle size distribution curve and its uses

3.4 Consistency of Soils, Atterberg's Limits, Plasticity Index, Consistency Index, Liquidity Index

4

Classification of Soil

4.1 General

4.2 I.S. Classification, Plasticity chart

5

Permeability and Seepage

5.1 Concept of Permeability, Darcy's Law, Co-efficient of Permeability,

5.2 Factors affecting Permeability.

5.3 Constant head permeability and falling head permeability Test.

5.4 Seepage pressure, effective stress, phenomenon of quick sand

6

Compaction and Consolidation

6.1 Compaction: Compaction, Light and heavy compaction Test, Optimum Moisture

Content of Soil, Maximum dry density, Zero air void line, Factors affecting Compaction, Field compaction methods and their suitability

6.2 Consolidation: Consolidation, distinction between compaction and consolidation.

Terzaghi's model analogy of compression/ springs showing the process of consolidation – field implications

7

Shear Strength

7.1 Concept of shear strength, Mohr- Coulomb failure theory, Cohesion, Angle of internal friction, strength envelope for different type of soil, Measurement of shear strength;- Direct shear test, triaxial shear test, unconfined compression test and vane-shear test

8

Earth Pressure on Retaining Structures

8.1 Active earth pressure, Passive earth pressure, Earth pressure at rest.

8.2 Use of Rankine's formula for the following cases (cohesion-less soil only)

(i) Backfill with no surcharge, (ii) backfill with uniform surcharge

9

Foundation Engineering

9.1 Functions of foundations, shallow and deep foundation, different type of shallow and deep foundations with sketches. Types of failure (General shear, Local shear & punching shear)

9.2 Bearing capacity of soil, bearing capacity of soils using Terzaghi's formulae & IS Code formulae for strip, Circular and square footings, Effect water table on bearing capacity of soil

9.3 Plate load test and standard penetration test

Chapter 1: Introduction

1.1 Soil and Soil Engineering

Definition of soil: The term 'soil' in soil engineering is defined as unconsolidated material, composed of soil particles, produced by the disintegration of rocks. The voids space between the particles may contain air, water or both. The soil particles may contain organic matter.

Definition of soil mechanics:

- The term 'soil mechanics' was coined by Dr. Karl Terzaghi in 1925, who is also known as the father of soil mechanics.
- According to Terzaghi, soil mechanics is the application of the laws of mechanics & hydraulics to engineering problems dealing with sediments & other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rook, regardless of whether or not they contain an admixture of organic constitute.
- **Soil mechanics is therefore, a branch of mechanics which deals with the action of forces on soil and with the flow of water in soil.**

Definition of soil engineering:

- Soil engineering is an applied science dealing with the applications of principals of soil mechanics to practical problems.
- It has a much wider scope than soil mechanics, as it deals with all engineering problems related with soils. It includes site investigations, design and construction of foundations, earth-retaining structures and earth structures.

Definition of geotechnical engineering:

- Geotechnical is a broader term which includes soil engineering, rock mechanics and geology.
- Sometimes geotechnical engineering is used synonymously with soil engineering.

1.2 Scope of Soil Mechanics:

Soil Mechanics is a basic subject and its scope will never end because all structures are built on soil and for buildings and structures to perform well and for a long time, soil tests should be done so that to know about the properties of soil and its characteristics.

1.3 Origin and formation of soil:

- In a broad sense, soil may be thought of an incidental material in vast geological cycle which has been going on continuously for millions of years of geological time.

- The geological cycle consists of 3 phases, Erosion, Transportation and deposition & Earth Movement.

a) Erosion Phase:

- The cycle starts with erosional phase in which there is degradation of exposed rock by weathering process.
- The weathering process may be
  i. Physical weathering
  ii. Chemical weathering

i. Physical weathering :

- The physical weathering process may be
  - Erosion of rock caused by the action of wind, water, glaciers.
  - Disintegration caused by alternate freezing and thawing in cracks in the rock.
- The resulting soil particles retain the same composition as that of parent rock.
- Their shape can be indicated by terms such as angular, rounded, flat and elongated.
- Gravel and sand fall into this group.

ii. Chemical weathering:

- The chemical process results in changes in the mineral form of parent rock due to the action of water.
- Chemical weathering results in the formation of group of crystalline particles of colloidal size (< 2μ) known as clay mineral.
- If the products of rock weathering are still located at the place where they originated, they are called residual soil.

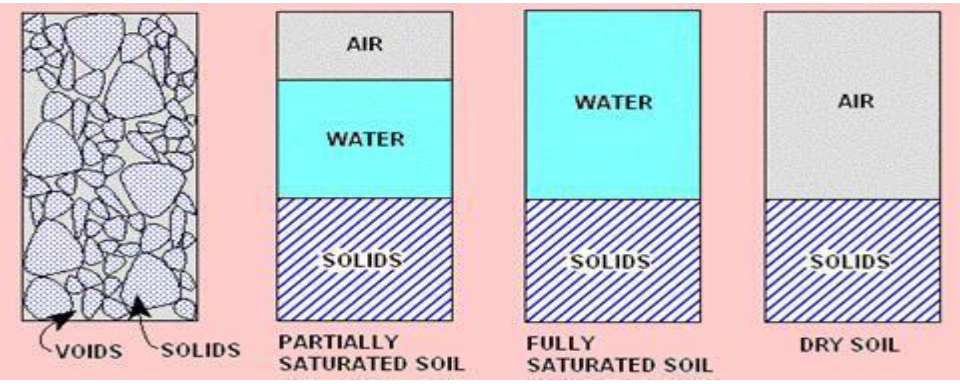
b) **Transportation/Deposition:**

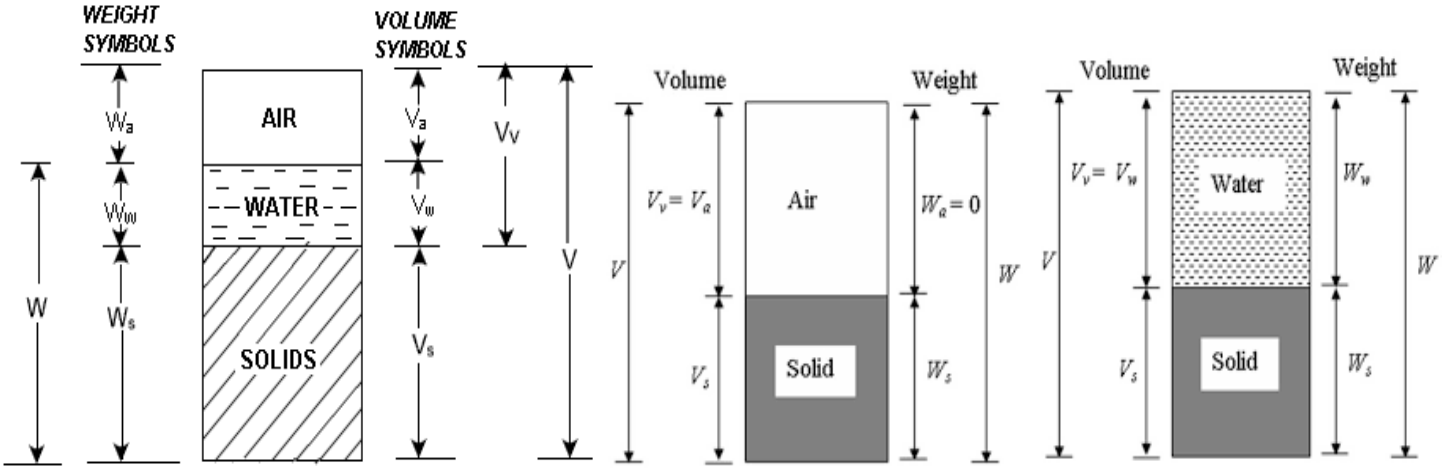
- In the second phase, the fragmented material is transported by agent such as wind, water or ice to new locations.
- Soil transported from their origin by wind, water, ice or any other agency and has been deposited is called transported soil. They have generally small grain size, large amount of pores.
- According to the transporting agency, soils are classified as:
  - Alluvial deposit – deposited by river water.
  - Lacustrine deposit – deposited by still water like lakes.
  - Marine deposit – deposited by sea water.
  - Aeolian deposit – transported by wind.
  - Glacial deposit – transported by ice.
  - Colluvial soil – deposited by gravity. (e.g. talus)

Chapter 2: Preliminary Definitions and Relationship

2.1 Soil as a three Phase system.

- A soil mass consists of solid particles which form a porous structure. The voids in the soil mass may be filled with air, with water or partly with air and partly with water. The three constituents are blended together to form a complex material.
- However, for convenience, all the solid particles are segregated and placed in the lower layer of the three phase diagram.
- Likewise, water and air particles are placed separately, as shown in figure below.
- The 3-phase diagram is also known as **Block diagram**.
- It may be noted that the 3-constituents cannot be actually segregated, as shown.
- A 3-phase diagram is an artifice used for easy understanding and convenience in calculation.
- Soil can be either two-phase or three-phase composition.
- Fully saturated soil and fully dry soil are two phase system.
- Partially saturated soil are three phase system.





Three phase system Two phase system Two phase system

Where

V_a= Volume of air W_a = Weight of air = 0

V_w = Volume of water W_w = Weight of water

V = Total volume of Soil mass W_s = Weight of soil solid

V_v = Volume of voids W = Total weight of soil mass

V_s = Volume of Solid W_{sat} = Saturated weight of soil mass

2.2 Important definitions:

1. Water content (w):

- Water content or moisture content of a soil is defined as the ratio of weight of water to weight of solids (dry weight) of the soil mass.

$$w = \frac{Ww}{Ws} \times 100 \; ; \; w \geq 0$$

- It is denoted by the letter symbol w and is commonly expressed as percentage i.e. 20%, 50% etc.
- The minimum value of water content is 0.
- There is no upper limit for water content.
- Generally fine grained soil have higher water content as compared to coarse grained soil.

2. Void ratio (e):

- Void ratio (e) is the ratio of the volume of voids (Vv) to the volume of soil solids (Vs), and is expressed as a decimal.

$$e = \frac{V_v}{V_s}$$

- There is no upper limit of void ratio in soil suspension.
- Void ratio of fine grained soil are generally higher than those of coarse grained soil.
- Size of void in coarse grained soil are generally larger than that in fine grained soil.

3. Porosity (n):

- Porosity (n) is the ratio of the volume of voids to the total volume of soil (V), and is expressed as a percentage.

$$n = \frac{V_v}{V} x\,100$$

- $V = V_V + V_S$, $V_V = V_W + V_a$
- The porosity of soil cannot exceed 100% hence it has a upper limit of 100% or 1.
- Both porosity and void ratio are measured of denseness or looseness of soil.

Note:

Total volume V is a variable quantity. But, since solids are incompressible, V_s remain invariant in the total volume V of the soil.

4. Degree of saturation (S):

- Degree of saturation of soil mass is defined as the ratio of volume of water in the voids to volume of voids.
- $S = \frac{V_w}{V_v}$, $0 \leq S \leq 100$
- For a fully saturated soil mass $V_v = V_w$, hence for the saturated soil mass S = 100% .
- For fully dry soil $V_w = 0$, hence for a fully dry soil mass s = 0%
- For partially saturated soil mass degree of saturation of soil mass varies between 0 – 100%, which is most common condition in nature.

5. Percentage Air voids (n_a):

- Percentage air voids (n_a) is the ratio of the volume of air to the total volume.
- $n_a = \frac{V_a}{V} \times 100$

6. Air content (a_c):

- Air content (a_c) is the ratio of the volume of air (Va) to the volume of voids V_v.

$$a_c = \frac{V_a}{V_v}$$

- $n_a = \frac{V_a}{V} = \frac{V_a \times Vv}{V \times Vv} = \frac{Vv}{V} \times \frac{V_a}{Vv} = n \times a_c$

  $\boxed{n_a = n \times a_c}$

7. Bulk unit weight (γ_t/γ):

- Bulk unit weight of a soil mass is defined as the weight per unit volume.
- $\gamma_t = \frac{W}{V} = \frac{W_s + W_w}{Vw + Va + Vs}$
- It is generally expressed as KN/m^3, N/m^3, kgf/ KN/cm^3

8. Unit weight of Solid (γ_s):

- It is defined as the ratio of weight of solids to weight of volume of solids. It is also called **absolute unit weight** of soil mass.
- $\gamma_s = \frac{W_s}{V_s}$

9. Unit weight of water (γ_w):

- $\gamma_w = \frac{W_w}{V_w}$
- The value of γ_w changes with temperature but usually we take γ_w as 9.81 kN/m^3 which is at 4°C.

10. Dry unit weight (γ_d):

- It is defined as weight of soil solid (or weight of dry soil) per total volume of soil.
- Unit is KN/m^3, N/m^3, kgf/ KN/cm^3
- $\gamma_d = \frac{W_s}{V}$
- Dry unit weight is used as a measure of denseness of soil. A high value of dry unit weight indicates that more solids are packed in unit volume of soil hence a more compact soil.

11. Saturated unit weight (γ_{sat}):

- It is defined as the bulk unit weight of soil mass in saturated condition.
- $\gamma_{sat} = \frac{\text{Wt. of Saturated soil}}{\text{Volume of soil}}$

12. Submerged unit weight (γ_{sub} or γ'):

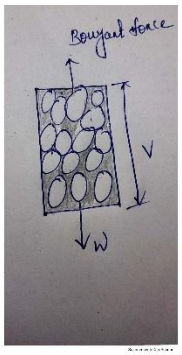
- It is defined as the weight of submerged soil per total volume of soil mass. When the soil exists below the ground water table, the soil mass is said to be in submerged condition.

- $\gamma_{sub} = \frac{\text{Wt. of Submerged soil}}{\text{Volume of soil}}$
- $\gamma' = \gamma_{sat} - \gamma_w$

Note:

Bulk unit weight = Saturated unit weight , (when soil is fully saturated)
Bulk unit weight = Dry unit weight, (when soil is completely in dry condition)

- When the soil exist below the ground water table, two forces are act on it.
- One is weight of soil, acting vertically downward and 2nd is buoyant force/weight acting vertically upward as shown in fig. below.



Buoyant weight/force = Submerged weight of soil

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

$\gamma' = \frac{\text{Wt. of Submerged soil}}{\text{Volume of soil}}$

Wt. of submerged soil = total weight of soil sample – weight of water displaced by the sample = Buoyant weight

$= W - \gamma_w \times V$

$\gamma' = \frac{W - \gamma_w \times V}{V} = \frac{W}{V} - \frac{\gamma_w \times V}{V} = \gamma_{sat} - \gamma_w$

13. Specific gravity of solids (G/G_s):

- It is defined as the ratio of the unit weight of solids (absolute unit weight of soil) to unit weight of water.
- $G_s = \frac{\gamma_s}{\gamma_w}$
- It is a unit less quantity.

- This is also known as Absolute specific gravity or Grain specific gravity.

14. Mass specific gravity (G_m):

- It is the ratio between the bulk unit weight of soil to unit weight of water
- $G_m = \frac{\gamma_t}{\gamma_w}$

5. Relative Density (D_r):

- $D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$
- e_{max} & e_{min} represents the soil in very dense and loose conditions respectively. e is the void ratio of natural soil.

Inter-Relations:

1. e.s = G.w

$$w = \frac{W_W}{W_s} = \frac{\gamma_W . V_W}{G_s . \gamma_W . V_s} = \frac{V_W}{G_s . V_s} = \frac{S . V_v}{G_s . V_s} = \frac{S . e}{G_s}$$

2. $W_s = \frac{W}{1+w}$
3. $V_s = \frac{V}{1+e}$
4. $e = \frac{n}{1-n}$
5. $n = \frac{e}{1+e}$
6. $\gamma_t = \frac{G+e.s}{1+e} \times \gamma_w$
7. $\gamma_{sat} = \frac{G+e}{1+e} \times \gamma_w$ (as S = 1)
8. $\gamma_d = \frac{G}{1+e} \times \gamma_w$ (as S = 0)
9. $\gamma_d = \frac{\gamma_t}{1+w}$

Chapter 3: Index Properties of Soil

- In this chapter, we shall describe the methods of determining those properties of soil which are used in their identification and classification.
- These include the determination of
  i. Water content (w)
  ii. Specific gravity (G)
  iii. Particle size distribution
  iv. Consistency limits
  v. In-situ density
  vi. Density index
- These properties are known as index properties.

i. Water content (w):

- Water content of soil is an important soil parameter which significantly influences the behavior of soil, particularly cohesive soils.
- Water content and unit weight changes during transportation and storage. Hence it is important to determine it before carrying out any other tests.
- Water content determination is also important because some physical state properties are calculated using water content following the practical measurements of others e.g. dry unit weight from bulk unit weight.
- The water content of a soil sample can be determined by any one of the following methods:
  a) Oven drying method
  b) Torsion balance method
  c) Pycnometer method
  d) Sand Bath method
  e) Alcohol method
  f) Calcium carbide method
  g) Radiation method

a) Oven drying method:

- The oven drying method is a standard laboratory method. This is very accurate method.
- The soil sample is taken in a small, non-corrodible, air tight container.

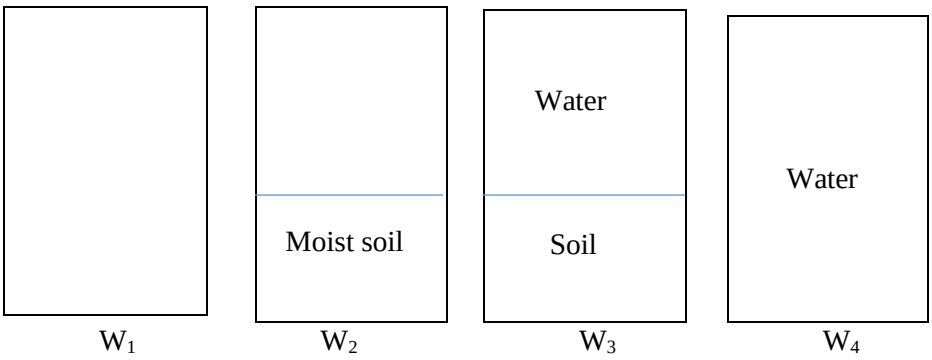
- The soil sample in container is then dried at temperature of 105-110^{o}C for 24 hour in laboratory. Above 110 ^{o}C, water of crystallization may lost.
- Water crystallization is the water in the molecular structure.
- For soil containing significant amount of organic matter, a temperature of 60^{o}C to 80^{o}C is recommended.
- If W_1 = weight of container, W_2 = wt. of container + wt. of moist soil and W_3 = wt. of container + wt. of dry soil, then water content is given by:

  $w = \frac{Ww}{Ws} = \frac{W2-W3}{W3-w1}$

b) Pycnometer method:

- A pycnometer is a glass jar of about 1 liter capacity and fitted with a brass conical cap means of a screw-type cover. The cap has a small hole of 6mm dia. At its apex.

- The pycnometer meter method for determination of water content can be used only if the specific gravity of solid (G) particle is known.
- First the weight of empty pycnometer is determined (W_1) in the dry condition. Then the sample of moist soil, is placed in the pycnometer and its weight with the soil is determined (W_2).
- The remaining volume of the pycnometer is then gradually filled with distilled water or kerosene. The weight of pycnometer, soil and water is obtained (W_3).
- Lastly the bottle is emptied, thoroughly cleaned and filled with distilled water or kerosene, and its weight taken (W_4).



- $W_w = W_2 - W_1 - W_S$

$$w = \frac{W_W}{W_S} = \frac{W_2 - w_1 - w_S}{W_S}$$

$$W_4 - W_1 = W_3 - W_1 - W_S + \frac{W_S \gamma_w}{G_S \gamma_w}$$

$$W_S = [(W_3 - W_4) \times G_S] / G_S - 1$$

$$w = [(W_2 - W_1) / [\{(W_3 - W_4).G_S\} / (G_S - 1)]] - 1$$

$$w = [\frac{W_2 - W_1}{W_3 - W_4} \times \frac{(Gs-1)}{Gs} - 1]$$

- Tish method is more suitable for cohesion less soil as in case cohesive soil removal of entrapped air is difficult.

ii) Specific gravity of soil particles:

- We know $G = \frac{\gamma_s}{\gamma_w} = \frac{\frac{W_s}{V_s}}{\frac{W_w}{V_w}}$
- If $V_s = V_w$, $G = \frac{W_s}{W_w} = \frac{M_s \times g}{M_w \times g} = \frac{M_s}{M_w}$
- Therefore Sp. Gravity of a soil particle is the ratio between mass of soil solid to mass of equivalent volume of water at same temperature.

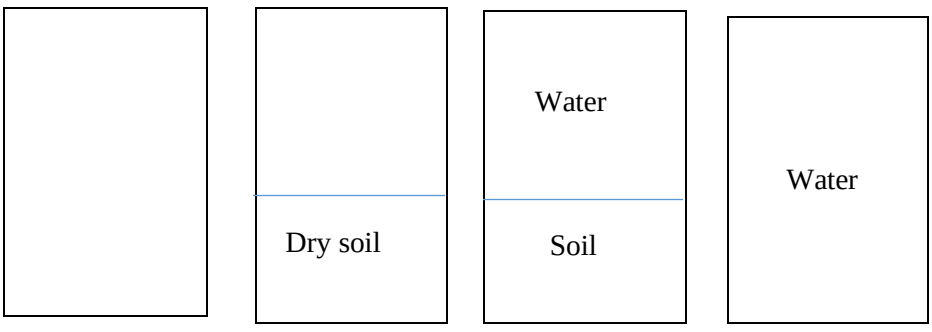
Determination of sp. Gravity:

Sp. gravity of soil is determined by 2- methods:

a) Density bottle method
b) Pycnometer method

Density bottle method is more accurate and suitable for all type of soil. But pycnometer method is suitable for coarse grained soil.

a) Density bottle method:



$G = \frac{M_s}{M_w} = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$

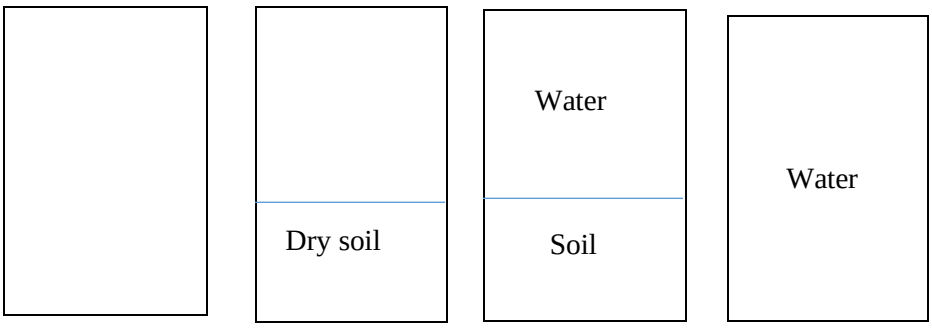
Procedure:

- A clean and empty density bottle is taken and weighted. Let the mass be M_1.
- Again the density bottle is poured with some dry soil whose sp. gravity is to determined and the whole assembly is weighted. Let the mass be M_2.
- Again the whole assembly is completely filled with water and weighted. Let the mass be M_3.
- Again the density bottle is cleaned and filled with water alone and weighted. Let the mass be M_4.
- Then, the sp. gravity $G = \frac{M_s}{M_w} = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)}$
- When kerosene is used in case of water, then the sp. gravity of soil $G = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} \times G_k$.

  Where G_k = sp. gravity of kerosene.
- Generally sp. gravity test is carried out on 27°C.
- Then sp. gravity of soil at t°C = $G_t{}^oC = \frac{G_{27°C} \times \gamma \text{ of distilled water at } 27°C}{\gamma \text{ of distilled water at } t°C}$.

b) Pycnometer method:

- This method is same as pycnometer method of water content determination with the difference that here dry soil sample is taken instead of moist soil sample as was taken in water contents determination.



W_1 W_2 W_3 W_4

Sp. gravity $G = \frac{\text{wt. of solid}}{\text{wt. of equivalent volume of water}}$

Wt. of solid = $W_2 - W_1$

Wt. of equivalent volume of water = $(W_4 - W_1) - (W_3 - W_2)$

$$G = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)}$$

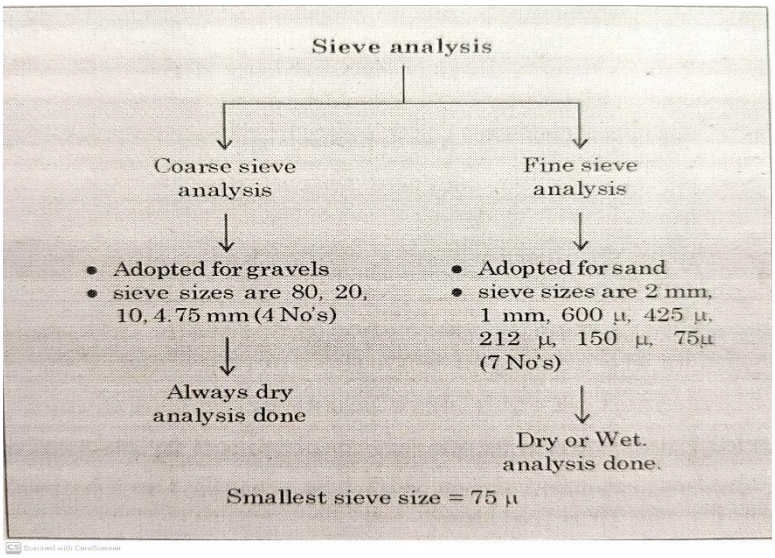
- Sp. gravity values are generally reported at 27°C.

iv) Particle size distribution:

- The classification of soils according to their size is known as particle size distribution.
- It is done by 2-methods.
    a. Sieve analysis
    b. Sedimentation analysis for particles of size less than 75µ

Particle Size	Type of Soil
> 300 mm	Boulder
300 – 80 mm	Cobbles

80 mm — 4.75 mm	gravel	Coarse Grained soil
4.75 mm — 2 mm 2 mm — 0.475 mm 0.475 mm — 75 µ	coarse sand medium sand } Sand fine sand	
75 µ — 2 µ	silt	Fine Grained soil
less then 2 µ	clay	



a) Sieve analysis:

- It is done for a soil which size lies more than 75µ.
- In this process sieves are places one over another in decreasing order of their aperture size.
- The sieve analysis is of 2 types
    i. Coarse sieve analysis; which consist of sieving of soil through 40mm, 20mm, 10mm and 4.75mm IS sieve.
    ii. Fine sieve analysis; which consist of sieving of soil though 2mm, 1mm, 600µ, 425µ, 212µ, 150µ, 75µ IS sieve.

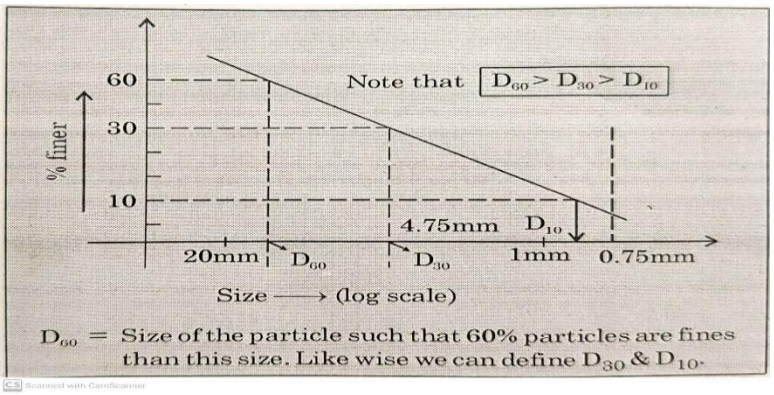
Procedure of sieve analysis:

- The soil sample is tested is dried, lumps are broken if necessary, and the sample is pass through the series of sieves by shaking.

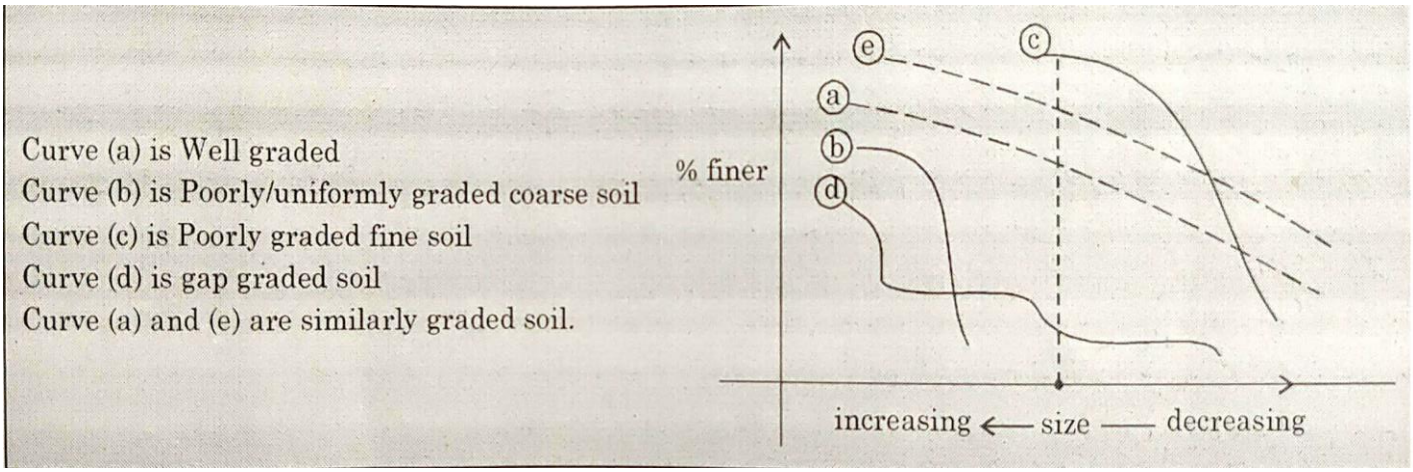
- The fraction retained on and passing 2mm IS sieve are tested separately.
- An automatic sieve shaker, run by an electric motor, may be used; about 10 to 15 minutes of shaking is considered as adequate.
- Larger particles are caught on the upper sieves, while the smaller ones is filter through to be caught on one of the smaller underling sieves.
- The material retained on any particular sieve should naturally include that retained on the sieves on the top of it, since the sieves are arranged with the aperture size decreasing from top to bottom.
- The weight of the material retained on each sieve is converted to a percentage of the total sample.
- The percentage material finer than a sieve size is obtained by subtracting this from 100.
- The materials passing the bottom most sieve, which is usually 75μ sieve, is used to conduct the sedimentation analysis for the fine fraction.
- If the soil is clayey in nature the fine fraction cannot be easily passed through the 75μ sieve in dry condition.
- In such case, the material is to be washed through it with water, until the wash water is fairly clean.
- The material which passes through the sieve is obtained by evaporation. This is called wet sieve analysis, may be required in case of cohesive granular soil.
- The resulting data are conventionally presented as a grain size distribution curve plotted on semi log co-ordinates, where the sieve size is on a horizontal logarithmic scale, and the percentage by weight of size smaller than a particular sieve size is on the vertical arithmetic scale.
- Logarithmic scales for the particle diameter give a very convenient representation of the sizes because a wide range of particle diameter can be shown in a single plot.

Sieve size	Total wt. of soil taken	Wt. retained on a particular sieve (gram)	Cumulative wt. retained	Cumulative %retained	% finer = 100 – cumulative %retained

- The resulting grain size distribution curve is as shown :



- Actually, sieve size is assumed to correspond to size of particle.
- The various types of curves obtained in sieve analysis are classified as follows:



- Well graded means soils of all sizes are present.
- Poorly graded/uniformly graded means soil of predominantly one size is only present.
- Gap graded means some of the soil particle size are missing.
- Position of the curve indicates **type of soil** whereas shape of the curve indicates **gradation**.
- As the slope of the curve decrease – gradation increase.
- D_{10} = effective size of Particle i.e. particle size which if present alone will cause the same effect as is caused by the soil.
- Grain size distribution curve is used to find out the following shape parameters.
    1. Co-efficient of uniformity (C_u) = $\frac{D_{60}}{D_{10}}$
    2. Co-efficient of curvature (C_c) = $\frac{D_{30}^2}{D_{60} \times D_{10}}$

If, $C_u = 1$ – Soil is perfectly uniformly graded. (Curve will be vertical)

$C_u > 4$ – well graded gravel

$C_u > 6$ – well graded sand

$1 \leq C_c \leq 3$ – well graded soil

- For well graded sand, $C_u > 6$, $1 \leq C_c \leq 3$
- For well graded gravel, $C_u > 4$, $1 \leq C_c \leq 3$

Larger the value of C_u, larger is the range of particles in soil.

Sedimentation analysis: -

- The soil particles less than 75μ size can be further analyzed for the distribution of various grain size of the order of silt and clay by 'sedimentation analysis' or 'wet analysis'.
- The soil fraction is kept in suspension in a liquid medium, usually water.
- The particles descend at velocities, related to their size, among other things.
- The analysis is based on **Stokes law.**
- As per this law, if a single sphere is allowed to fall in an infinite liquid medium without interference, its velocity first increase under the influence of gravity, but soon attain a constant value.
- This constant velocity, which is maintained indefinitely unless the boundary condition change, is known as the **terminal velocity.**
- Coarser particles tend to settle faster than finer ones.
- By Stokes law, the terminal velocity of the spherical particle is given by:
- V = $\frac{(\gamma_s - \gamma_l)D^2}{18\mu}$, where γ_s = unit weight of soil particles in N/m^3

  γ_l = unit weight of liquid in N/m^3, D – diameter of particles in meter, μ- dynamic viscosity in N.s/m^2. then V will be in m/s.
- We can also write V = $\frac{(G-1)gD^2}{18\vartheta}$, where $\vartheta -$ is kinematic viscosity in m^2/s and G- sp. gravity, $\vartheta = \frac{\mu}{\rho}$, unit- m^2/sec.
- Stocks law is considered valid for particle diameters ranging from 0.2 to 0.0002 mm.

Procedure of sedimentation analysis:

- About 50gm of dry soil sample passing through 75μ IS sieve is weighted accurately and is taken in a porcelain dish.
- About 50cc of dispersing agent is added to the soil sample.
- Then some quantity of distilled water is added to form a soil slurry and it is gently stirred using glass rod.
- The content of dish is then transferred to the cup of high-speed stirrer.
- Then the soil slurry is transferred to a 1000ml sedimentation jar.
- Then some water is added to the sedimentation jar, to make it 1000ml.
- Then the mouth of jar is close tightly with the pam of hand and jar is inverted several times to ensure uniform distribution of soil particles throughout the jar.
- Then jar is placed in a level surface and stope watch is started simultaneously.

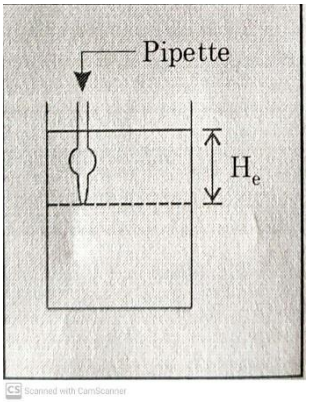
- At the end of different time intervals, t minutes (usually 1min, 2min, 4min, 8min, 16min, 30min, 1hr, 2hr, 4hr, 8hr, 16hr and 1day etc.) the size D mm of largest particle still in suspension at depth H_e cm and % finer N is determined by using hydrometer method or pipette method.

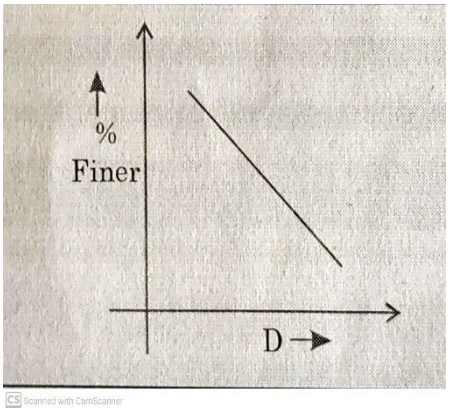
i. Pipette method:

- It is a laboratory method. A pipette, sedimentation jar, and a number of sampling bottles are necessary for the test.
- The method consists in drawing off 10ml samples of soil suspension by means of the sampling pipette from a standard depth of 10cm (i.e. H_e = 1cm) at various time intervals after the start of sedimentation (usually 1min, 2min, 4min, 8min, 16min, 30min, 1hr, 2hr, 4hr, 8hr, 16hr and 1day etc.).
- The pipette should be inserted about 20 seconds prior to the chosen instant and the process of the sucking should not be take more than 20 seconds.
- Each of the samples taken is transferred to a sampling bottle and dried in an oven. The concentration of all particles are present in a particular volume at time t = 0.
- If after time 't' sample is taken out in pipette from height H_e then all particles having settling velocity greater than $\frac{He}{t}$ will have settle below height H_e.
- At this height at time 't' sample collected will have the same concentration of particles of settling velocity less than $\frac{He}{t}$ as was there in the original soil suspension.
- If settling velocity of particle size D = $\frac{He}{t}$, then

  % finer than D = $\left(\frac{W_D}{\frac{W_d}{V}} \times 100 \right)$ %, where W_D = wt. of soil particles finer than size Dmm per ml, still is suspension at depth H_e at elapsed time interval t min., W_d = wt. of soil particles taken for sedimentation analysis, V = vol. of soil suspension.

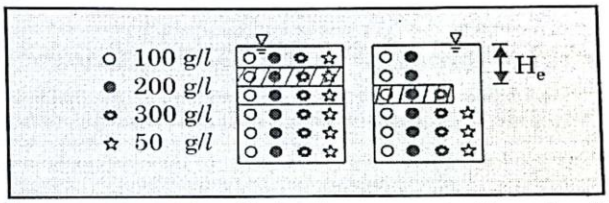
  Now, $\frac{(\gamma_s - \gamma_l)D^2}{18\mu} = \frac{He}{t}$, find 'D' from this formula corresponding to various time t and plot grain size distribution curve.





Notes: In pipette method, sample s collected from height H_e at various time intervals i.e. H_e is fixed.

Note: • Let there be 4-types of particles in the sample. Let in the original soil suspension before the start of sedimentation the concentration of various particles be as shown below



- If particle has settling velocity greater than H_e/t, then concentration of soil suspension collected at height H_e will be 600 g/l.

$$\text{Thus, \% finer than size } \star = \frac{\text{concentration of sample collected at time t from height } H_e}{\text{concentration of soil from original soil suspension}} \times 100 = \frac{600}{650} \times 100$$

- If particle ☆ has size D, then % finer than size D is $\frac{600}{650} \times 100$, where D is given by $\frac{H_e}{t} = \frac{(\gamma_s - \gamma_w)D^2}{18\mu}$

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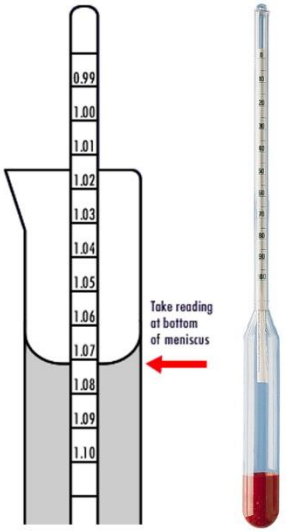
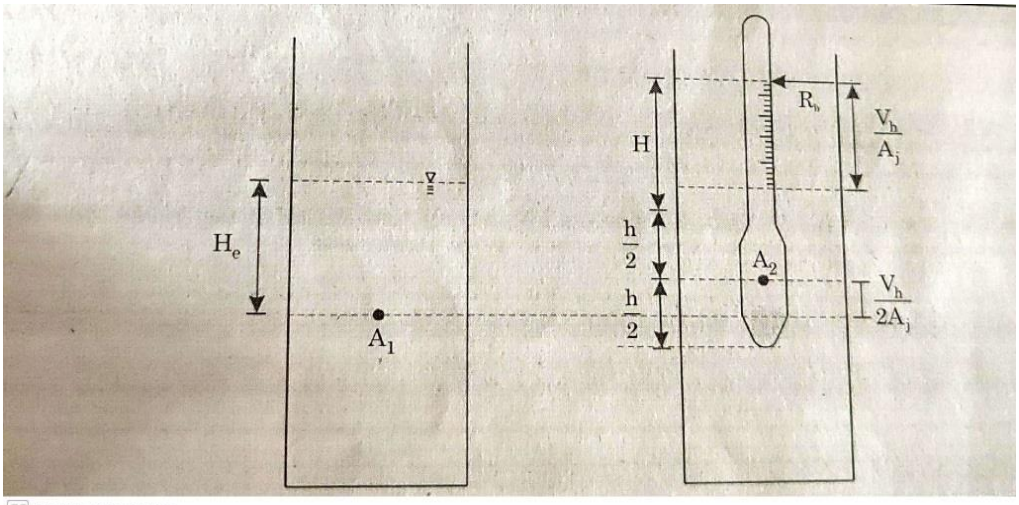
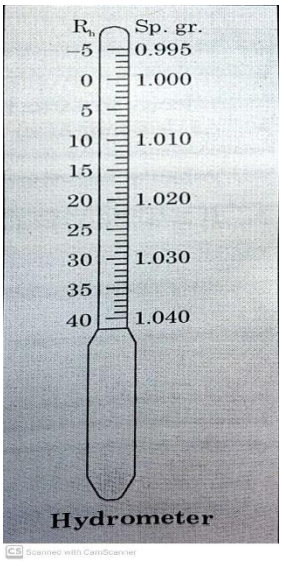


ii) Hydrometer method:

- The hydrometer method differs from the pipette analysis in that the weights of solids per ml in the suspension at a chosen depth at a chosen instant of time are obtained indirectly by reading the sp. gravity of the soil suspension with the aid of a hydrometer.
- **Hydrometer is the device which is use to measure the sp. gravity of liquids.**
- We have already discussed that % of finer than D = $\frac{wt.of\ the\ solid\ per\ cc\ at\ depth\ H_e\ at\ time\ t}{wt.of\ solid\ per\ cc\ in\ original\ soil\ suspension}$
- Where D is given by $\frac{(\gamma_s - \gamma_l)D^2}{18\mu} = \frac{He}{t}$





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- Thus, point at A_1 occupies the position A_2. A_2 is the center of volume of hydrometer.

  V_h = Volume of hydrometer

  A_j = Area of jar

  h = length of bulb in hydrometer
- Volume of bulb can be thought of as the total volume of hydrometer.
- Hydrometer measures density of soil suspension at depth H_e which is given by:

  $H_e = H + \frac{h}{2} + \frac{V_h}{2A_j} - \frac{V_h}{A_j}$
- H will correspond to hydrometer reading R_h. Thus, H_e will be related to R_h.

Procedure of hydrometer method:

- The hydrometer in inserted in the soil suspension prepared in 1000ml sedimentation jar at the end of different time interval, t min. (usually 1min, 2min, 4min, 8min, 16min, 30min, 1hr, 2hr, 4hr, 8hr, 16hr and 1day etc.) and the reading of the hydrometer i.e. R_h is noted.
- The reading of hydrometer R_h is corrected by using suitable correction. Then from the calibration chart the effective depth H_e cm. is found corresponding to hydrometer reading R_h.
- The size D mm. of largest particles still in suspension at depth H_e cm. at any elapsed time interval t min. is computed by using following equations.
- % finer than $D = \left(\frac{W_D}{\frac{W_d}{V}} \times 100 \right)\%$, $\frac{(\gamma_s - \gamma_l)D^2}{18\mu} = \frac{He}{t}$

  where W_D = wt. of soil particles finer than size Dmm per ml, still is suspension at depth H_e at elapsed time interval t min., W_d = wt. of soil particles taken for sedimentation analysis, V = vol. of soil suspension.

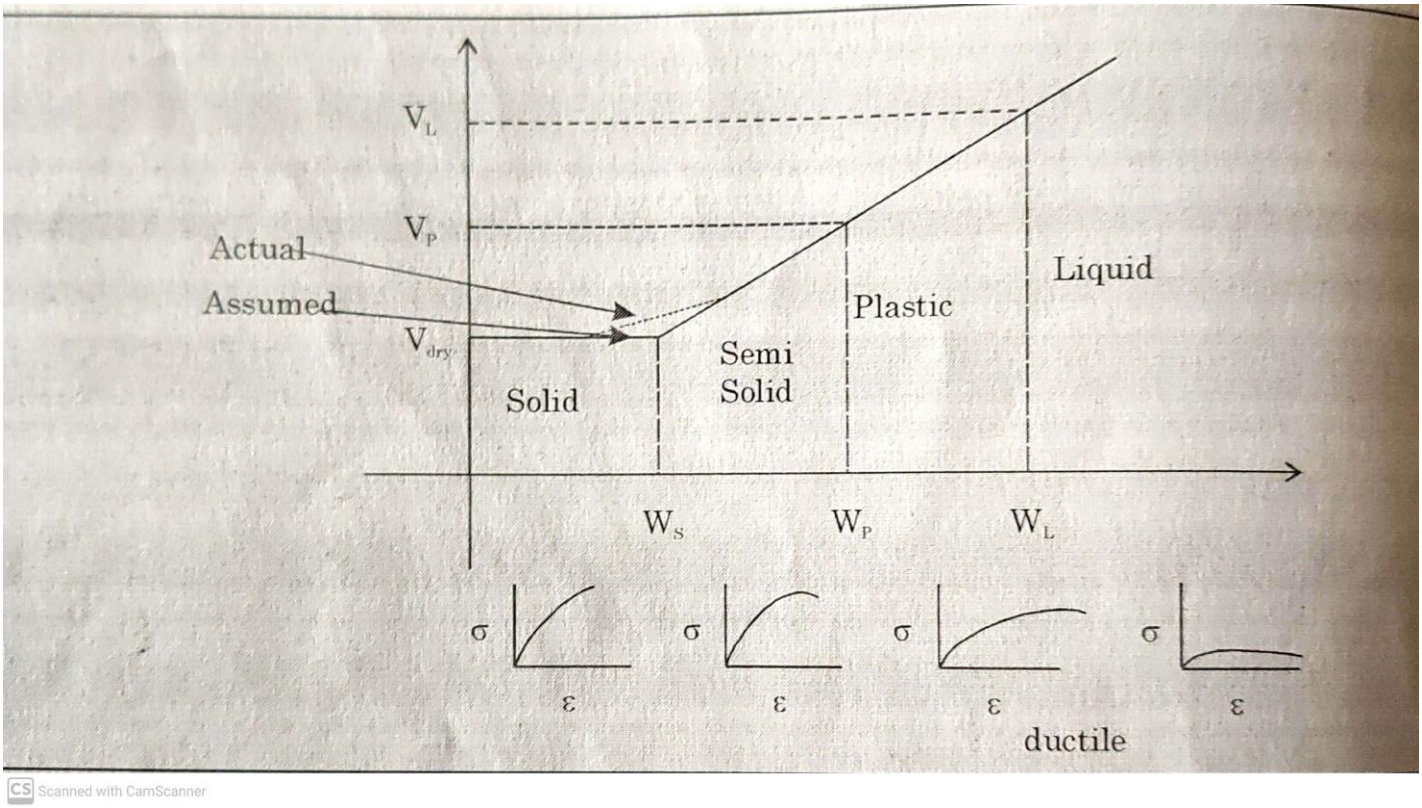
Limitation of Stocks' law:

- Stocks' law is applicable for spherical particles only. But the fine clay particles are not spherical in shape. So the concept of equivalent diameter may be used.
- It is assumed that every particle settles freely without interference, in an infinity liquid medium. The sedimentation analysis is conducted in a one-liter jar, the depth being finite; the walls of the jar could provide a source of interference to the free fall of particles near it. The fall of any particle is affected by the presence of adjacent particles; thus, the fall may not be really free. However it is assumed that the effect of these sources of interference is insignificant if suspension is prepared with about 50gm of soil per litter of water.

- The soil particles of soil specimen may have different values of specific gravity. But in computation avg. value of sp. gravity may use.

Consistency of soil:

- Consistency represents the relative ease with which a soil be deformed.
- This term is mainly used for clayey soil and is related to water content i.e. how with change in water content the consistency of soil changes.
- Atterberg classified the consistency in 4-stages. Behaviour of soil is different in different stages.
    1. Solid stage
    2. Semi solid stage
    3. Plastic stage
    4. Liquid stage



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W_L = liquid limit water content

W_P = Plastic limit water content

W_S = Shrinkage limit water content

V_L = Volume of soil at liquid limit

V_P = Volume of soil at plastic limit

V_{dry} = Volume of soil at shrinkage limit

Chapter-3; Index properties of soil

Consistency of soils:-

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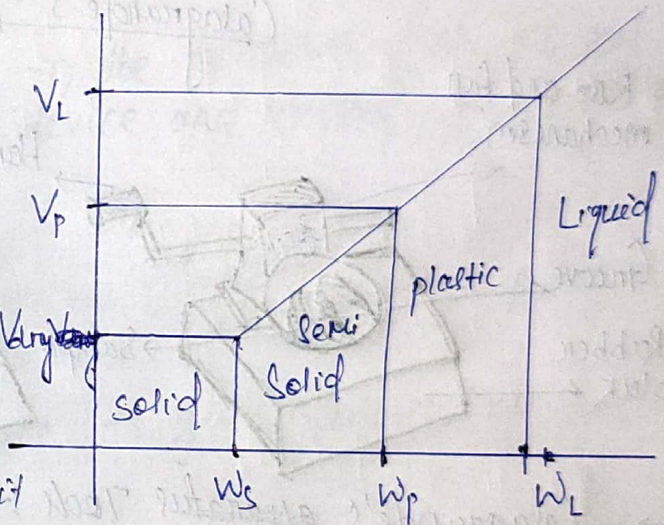

W_L = liquid limit water content
W_p = plastic limit water content
W_s = Shrinkage limit water content
V_L = Volume of soil at liquid limit
V_p = Volume of soil at plastic limit
V_{dry} = Volume of soil at shrinkage limit

From the above figure :- $\frac{V_L - V_p}{W_L - W_p} = \frac{V_p - V_{dry}}{W_p - W_s}$

Notes

Naturally existing soil has water content between W_L & W_p

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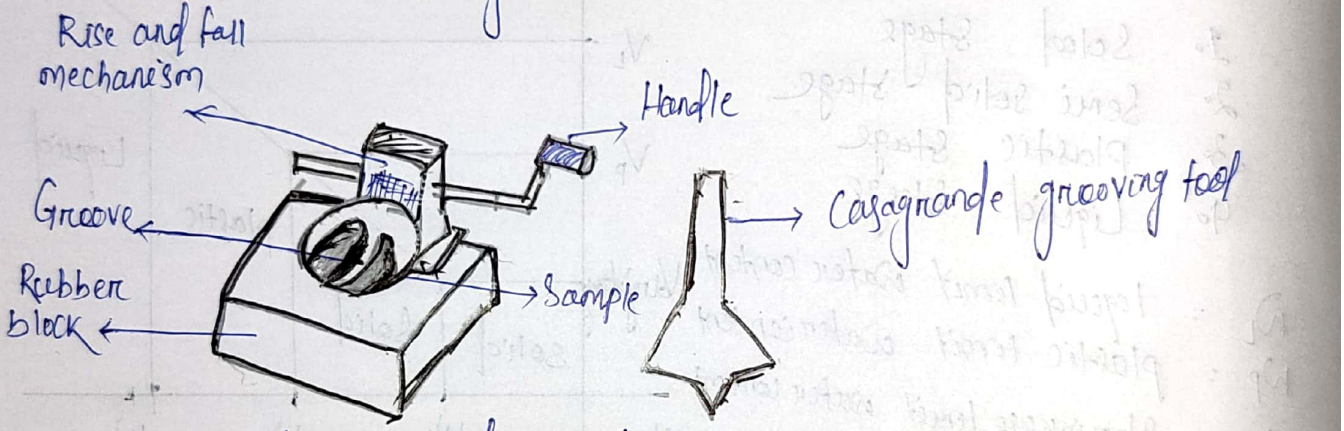
Liquid Limit

→ Min water content at which soil has a tendency to flow is called liquid limit/water content.

→ 'or'

It is the min^m water content at which a groove of 2mm made with standard grooving tool is filled by 25 no. of blows in Casagrande's apparatus.

Casagrande's apparatus



→ Casagrande's ~~apparatus~~ Tools :-

Soil is taken and water is added and the soil put inside Casagrande's apparatus. A groove of 2mm size is cut and the apparatus is given blows over a rubber pad No. of blows required to close the 2mm grove is noted. water content at which 25 blows closes the groove is called liquid ~~lotw~~ limit.

Determination of liquid limit :-

→ liquid limit can be determined using standard grooving tools and casagrandes apparatus.

→ The apparatus consists of a brass cup which can be raised and lowered to fall on a base by means of a cam operated by a handle. The height of fall of cup is adjusted by adjusting screw.

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→ The standard grooving tool is of 2mm wide at bottom, 13.6mm at top, 10mm depth.

Test procedure:-

(i) About 100gm. of soil sample passing through 425μ IS sieve is taken in a porceline dish.

(ii) Some quantity of distilled water is added to it and thoroughly mixed to form a soil paste of uniform colour.

(iii) The height of fall of cup of liquid limit device is adjusted to 1cm.

(iv) A portion of soil paste in the porceline dish is put in the cup of liquid limit device and levelled by means of spatula.

(v) Then a groove is made on the soil pat in the cup by using standard grooving tools.

(vi) Then the liquid limit device is given blows by cam operated by handle.

(vii) The no. of blows required to close the groove is noted.

(viii) This procedure is repeated to the same soil sample by changing the water content and finding corresponding no. of blows.

(ix) Then a graph is plotted between water content and no. of blows by taking water content on ordinate (y-axis) in natural scale against no. of blows as abscisa in (x-axis) logarithmic scale.

(x) From the graph the water content corresponding to 25 no. of blows is found out which is known as liquid limit of soil.

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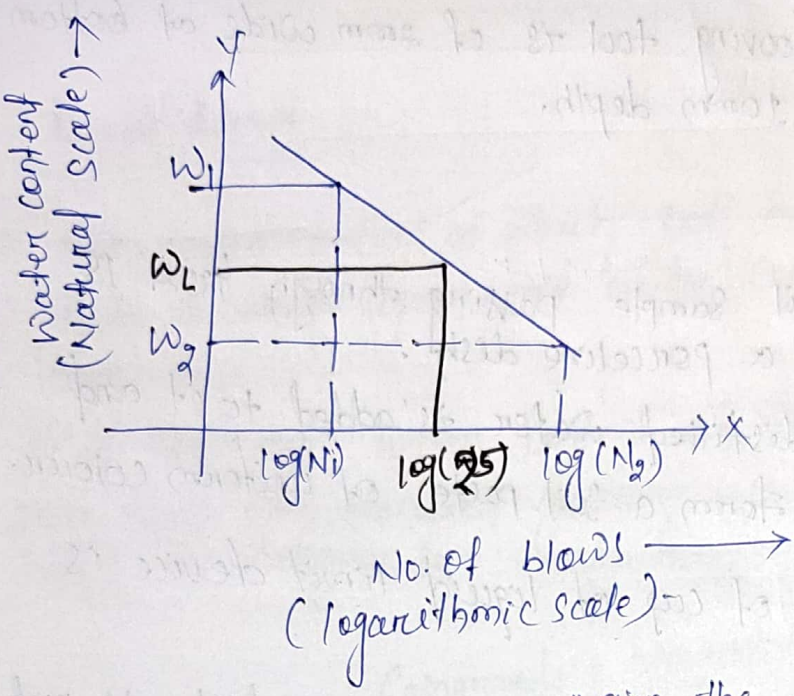

→ Slope of this graph will give the flow index

i.e $I_f = \frac{w_1 - w_2}{\log(N_2) - \log(N_1)} = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$

Plastic limit (w_p)

→ It is the min^m water content at which soil mass is in plastic stage, is called plastic limit water content.

'or'

It is the water content in which a soil mass can be rolled into a thread of 3mm dia. shows the sign of cracking.

Determination of plastic limits:

Procedure:

(i) About 30gm of soil sample is passing through 425μ of IS sieve is taken and some quantity of distilled water is added to it and thoroughly mixed to form a soil paste which can be rolled by the palm of hand.

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(ii) A small portion of the ball is then rolled on a smooth plate in a thread of 3mm. dia and thread is looked for the sign of cracking.

(iii) If no cracks is seen, the thread is picked up and again rolled into a ball bet^{n} palms to reduce water content.

(iv) The ball is then rolled to a thread of 3mm. dia.

(v) This steps are repeated until the thread shows sign of Cracking.

(vi) A portion of thread is taken for water content determination which gives the plastic limit.

Shrinkage limit (w_s)

→ It is the max^{m} water content at which further reduction in water content does not cause any reduction in the volume of soil sample is called shrinkage limit water content.

→ When water content is reduced below w_s, the particles are so closely spaced that volume reduction will not take place and the void space starts getting occupied by air instead of water.

→ It is the min^{m} water content at which soil is saturated.

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Determination of shrinkage limit:-

Test procedure:-

(i) About 50gm. of soil sample passing through 425μ IS-sieve is taken in a porceline dish, distilled water is added to it and mixed thoroughly to form a soil paste of slightly flowing consistency.

(ii) The shrinkage dish (non-corrodable cup) of 45mm dia & 15mm height) is weighted after coating inner side with a thin layer of grece or oil.

(iii) The shrinkage cup will filled with the soil paste in 3 layers.

(iv) The surface of soil is levelled and outer side of cup is cleaned.

(v) The mass of wet soil pat with shrinkage cup is found and this is deducted from ~~mass~~ weight of shrinkage cup to get weight of wet soil pat (W_1)

(vi) The wet soil pat is allowed to dry in air for some time and then kept in a thermostatically controlled oven and dried for 24hrs. at 105°C - 110°C.

(vii) After oven drying the weight of oven dried soil (W_2/W_s) and vol. of oven dried sample (V_s) is found out. Then the shrinkage limit is given by;

$$W_s = \frac{(W_1 - W_2) - (V_1 - V_s)\gamma_w}{W_s}$$

V_1 = vol. of wet soil pat or original soil volume.

V_s = dry vol. of soil

W_1 = weight of wet soil pat or original wt. of soil.

W_2 or W_s = dry wt. of soil.

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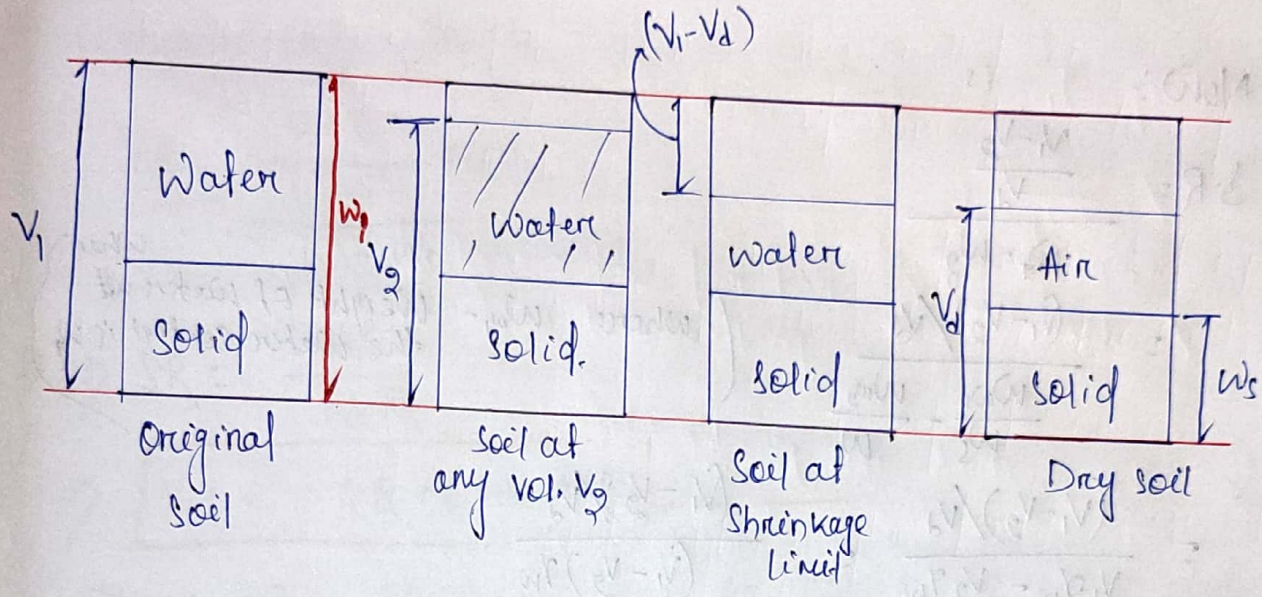

Now, Water content: $\frac{W_w}{W_s}$

→ At Shrinkage limit, $W_w = W_1 - W_s - (V_1 - V_d)\gamma_w$

∴ **Shrinkage limit $(W_s) = \frac{W_1 - W_s - (V_1 - V_d)\gamma_w}{W_s}$**

Shrinkage limit can also determine by :-

$$W_s = \frac{1}{(G_m)_{dry\ state}} = \frac{1}{G_f}$$

$(G_m)_{dry\ state}$ - Mass Sp. gravity at dry state.

Shrinkage Ratio (S.R.)

→ Shrinkage ratio is defined as vol. change in soil above Shrinkage limit expressed as percentage of dry soil per unit change in water content above Shrinkage limit.

$$S.R = \frac{\left(\frac{V_1 - V_2}{V_d}\right) \times 100}{W_1 - W_2}$$

W_1 = Water content at vol. V_1
W_2 = " " at vol. V_2
V_d = Dry vol. of soil sample.

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Now :-

$$S.R = \frac{\frac{V_1 - V_2}{V_d}}{w_1 - w_2}$$

$$= \frac{(V_1 - V_2)/V_d}{\frac{w_{w_1}}{w_s} - \frac{w_{w_2}}{w_s}}$$

(where w_{w_1} - Weight of water when the water content is w_1)

$$= \frac{(V_1 - V_2)/V_d}{\frac{V_1 \gamma_w - V_2 \gamma_w}{w_s}} = \frac{(V_1 - V_2)/V_d}{\frac{(V_1 - V_2)\gamma_w}{w_s}}$$

$$\Rightarrow S.R = \frac{(V_1 - V_2)}{V_d} * \frac{w_s}{(V_1 - V_2)\gamma_w}$$

$$= \frac{w_s}{V_d \gamma_w}$$

$$\therefore S.R = \frac{\gamma_d}{\gamma_w}$$

$$S.R = \frac{\text{Dry density}}{\text{Density of water}}$$

Volumetric Shrinkage (V.S)

$$V.S = \frac{V_1 - V_d}{V_d} \times 100$$

→ It is the reduction in vol. of soil mass expressed as % age of its dry vol.

$$\therefore S.R = \frac{\frac{V_1 - V_2}{V_d} \times 100}{w_1 - w_2}$$

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we also can write

$$S.R = \frac{\frac{V_1 - V_d}{V_d} \times 100}{w_1 - w_s}$$

$$\Rightarrow S.R = \frac{v.s}{w_1 - w_s}$$

$$\Rightarrow \boxed{V.S = S.R * (w_1 - w_s)}$$

Where w_s = Shrinkage limit or Shrinkage limit water content

Indices:-

→ The different types of indices are :-

(i) Plasticity Index (I_p)
(ii) Liquidity Index (I_L)
(iii) Consistency Index (I_c)
(iv) Flow Index (I_f)
(v) Density Index (I_d)
(vi) Toughness Index (I_t)

(i) Plasticity Index (I_p) :-

→ It is difference bet" liquid limit (L.L) & plastic limit (P.L) of a soil

$I_p = L.L - P.L$

$$\boxed{I_p = w_L - w_p}$$

(ii) Liquidity Index (I_L)

→ It is the ratio bet" difference of natural water content & plastic limit to the plasticity index.

$$\boxed{I_L = \frac{w - w_p}{I_p}} \Rightarrow \boxed{I_L = \frac{w - w_p}{w_L - w_p}} \quad (\because I_p = w_L - w_p)$$

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When $I_L = 0$

$\Rightarrow \frac{w - w_p}{I_p} = 0$

$\Rightarrow w = w_p$ ($\because$ natural water content is equal to plastic limit water content)

So the soil is in plastic state

When $I_L = 1$

$\Rightarrow \frac{w - w_p}{I_p} = 1$

$\Rightarrow w - w_p = w_L - w_p$

$\Rightarrow w = w_L$

So, the soil is in liquid state.

Consistency index (I_c) :-

→ It is the ratio betn difference of liquid limit & natural water content and plasticity index.

$$I_c = \frac{w_L - w}{I_p}$$

$$I_c = 1 - I_L$$

($\because 1 - \frac{w - w_p}{w_L - w_p} = \frac{w_L - w}{I_p} = I_c$)

When $I_c = 0$

$\Rightarrow w_L = w$

So, the soil is in liquid state.

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When $I_c = 1$, then $\frac{w_L - w}{I_p} = 1$

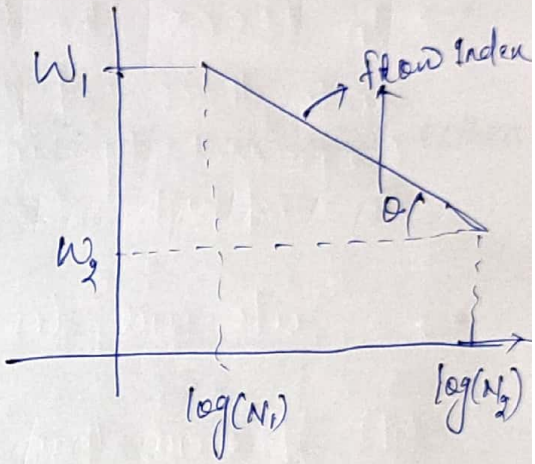
$\Rightarrow w_L - w = w_L - w_p$

$\Rightarrow w = w_p$ ——

∴ So, the soil is in plastic state.

Flow Index (I_f) :-

→ It is the slop of the flow curve



$$I_f = \frac{w_1 - w_2}{\log(N_2) - \log(N_1)}$$

$$\therefore \boxed{I_f = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}}$$

Where N_1 = No. of blows corresponding to water content w_1

N_2 = No. of blows corresponding to water content w_2.

Toughness Index (I_t) :-

→ It is the ratio betⁿ plasticity index to flow index.

$$\boxed{I_t = \frac{I_p}{I_f}}$$

Density Index (I_d) :-

→ It is the ratio betⁿ difference of void ratio of a soil sample at it's loosest state (e_{max}) & natural (e) state to, difference of void ratio of that sample at its loosest state (e_{max}) & densest state (e_{min}).

$$\boxed{I_d = \frac{e_{max} - e}{e_{max} - e_{min}}}$$

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Chapter-4 Classification of soil :-

4.1 → A soil classification means sorting of soil into group, which will show similar behaviour.

→ Classification is done on the basis of grain size distribution and plasticity chart.

→ For classifications we use symbols, which are used either in suffix or prefix.

Soil type	Prefix	Subgroup	Suffix
Gravel	G	well graded	W
Sand	S	poorly graded	P
clay	C	clayey	C
Silt	M	silty	M
Organic	O	liquid limit < 35	L (low compressibility)
peat	Pt	35 < Liquid limit < 50	I (intermediate ")
		liquid limit > 50	H (High ")

Ex:- GW → Well graded gravel
GM → Silty gravel
CH → Clay of high compressibility.

Soil classification

(1) Unified Soil Classification System (USCS)

(2) American Association of State highway and transport officials (AASHTO) system.

(3) Indian Standard Soil classification System (ISSCS)

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Indian Standard Soil classification System:-

→ Indian Standard Soil classification System (ISSCS) adopted by Bureau of Indian Standards.

→ In ISSCS the fine grained soil are subdivided into three categories of low, medium and high compressiblity instead of two categories of low and high compressibility in USC system.

→ Soils are divided into three broad devisions:-

(1) If more than 50% of soil retained on 75μ IS seive or above; the soil is essentially said to be coarse-grained soil.

(2) If more than 50% of soil passes 75μ sieve then the soil is said as fine-grained soil.

(3) If the soil is highly organic and contains a large percentage of organic matter and particles of decomposed vegetation, it is kept in a separate category marked as (Pt)

→ In case of course grained soil if more than 50% of course fraction is passing 80mm sieve & retained on 4.75mm sieve then the soil is essentially said to be gravel.

→ If more than 50% of course fraction is passing 4.75mm sieve and retained on 75μ sieve, then the soil is essentially said to be sand.

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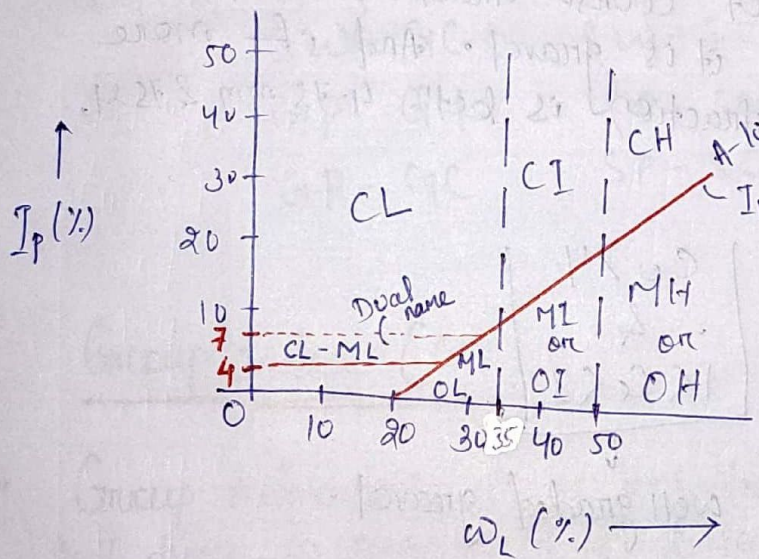
4.2 ~~classification~~

Plasticity chart:-

→ It is based on the values of liquid limit (w_L) and plasticity index (I_P), is provided in ISSCS to aid classification.

→ Depending on the point in the chart, fine soils are divided into clay (C), silt (M) or Organic soils (O).



Classification of course grained soil:-

(a) When fines are less than 5% of weight of the total soil (Fine means → passing through 75 μ sieve)

(b) fines > 12% of weight

(c) fines betⁿ 5 to 12%

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(a) fines < 5%

→ The various name gives as

GW → well graded gravel
GP → poorly graded gravel
SW → well graded sand
SP → poorly graded sand.

→ If more than 50% of course fraction is betn 80mm & 4.75 mm it is gravel. And if more than 50% of course fraction is betn 4.75 mm & 75 μ, then it is sand.

→

$C_u > 6$ & $1 < C_c < 3$	$C_u > 4$ & $1 < C_c < 3$
↓	↓
well graded sand	well graded gravel.

(b) Fines > 12%

→ The various name gives as

GM → silty gravel
GC → clayey gravel
SM → silty sand
SC → clayey sand.

→ In this case for the fines liquid & plastic limit test is done and I_p & W_L calculated and plot on plasticity chart.

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→ If the points gets plotted above A-line, the fines are clayey in nature & if points get plotted below A-line fines are silty in nature.

(C) Fines are betn 5-12%

→ When fines are betn 5-12%, dowels symbols are used. And the various name given as:-

GW-GM	SW-SM
GW-GC	SW-SC
GP-GM	SP-SM
GP-GC	SP-SC

Group Index (G.I)

→ Group index value is an arbitrary index assigned to the soil types in numerical eqn based on the percent fines, liquid limit & plasticity index.

→ The G.I value of soils vary in the range of 0 to 20

→ The higher the G.I value weaker the soil subgrade hence greater thickness of pavement required.

$$G.I = (0.2a + 0.005ac + 0.01bd)$$

$a = P - 35$ ≯ 40 (expressed as whole no betn 0-40)
$b = P - 15$ ≯ 40 (" " " 0-40)
$c = W_L - 40$ ≯ 20 (" " " 0-20)
$d = I_P - 10$ ≯ 20 (" " " 0-20)

P = %age finer (%age of soils passing from 0.075 mm sieve)
W_L = Liquid limit
I_P = Plasticity index = $(W_L - W_P)$

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Chapter-5 permeability and seepage

Introduction

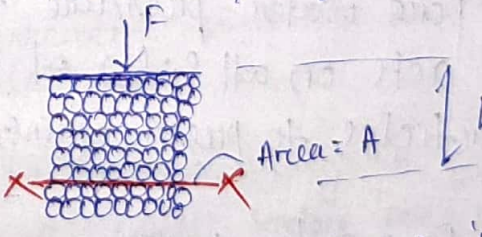
→ There are 2-types of interaction betn soil particles
(a) chemical interaction
(b) physical interaction

→ physical interaction is studied with help of effective stress; and effective stress concept was developed by Terzaghi and is applied to fully saturated soil and repeated 3-types of stresses.

(a) Total stress
(b) pore water pressure
(c) Effective stress

(a) Total stress

→ Total stress (σ) on a plane with in a soil mass is the force per unit area of soil mass transmitted in normal direction across the plane.



Total stress (σ) = $\frac{F}{A} + \frac{W}{A}$

where W = weight of the soil mass above the X-X plane in 'h' height.

$\Rightarrow \sigma = \frac{F}{A} + \frac{W * h}{A * h} = \frac{F}{A} + \left(\frac{W}{Vol}\right) * h$

$\Rightarrow \boxed{\sigma = \frac{F}{A} + \gamma * h}$ —

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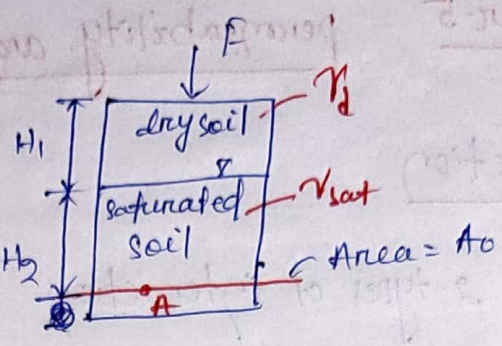
$$\sigma_A = \frac{F}{A_0} + \gamma_d H_1 + \gamma_{sat} H_2$$



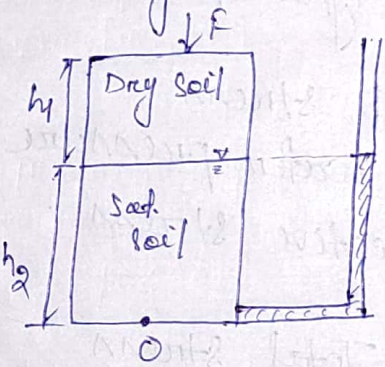
→ Total stress is a physical parameter which can be measured by suitable arrangement, such as by pressure - cell.

(b) pore water pressure (u)

→ It is the pressure of water filling the void space solid particles.

$$u = \gamma_w h_2$$

(A case when force 'F' has been acting on soil since long ago).



→ pore water pressure is also called as neutral stress because it acts on all sides of the particles, but does not cause particles to press against adjacent particles.

→ Pore water pressure at any point can be measured by inserting a stand pipe at the point under investigation and observing the height upto which the water rises in the stand pipe. (Thus it is a measurable quantity).

→ pore water pressure is measured using a piezometer or stand pipe.

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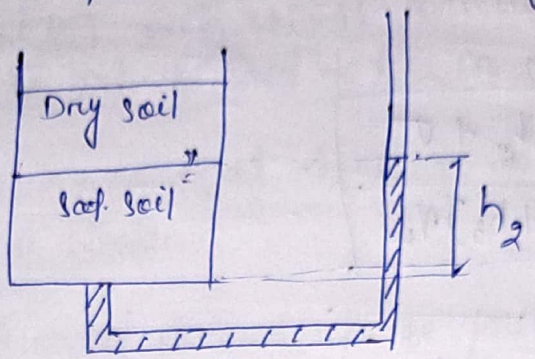
Effect of external load on pore water pressure.

Case-I (pore water pressure before any loading is applied)



→ pore water pressure initially before the application of loading is constant of value governed by the position of water table ($\gamma_w h_2$). This initial pressure is called static pore water pressure. As shown in the figure above, the water level rises in the stand pipe upto the same level as ground water Table.

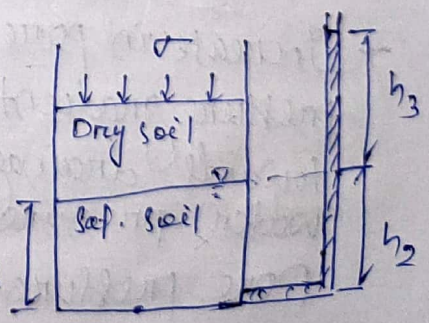
Case-II (pore water pressure immeadiatly after the application of external stress.)

→ As the external load is applied, soil particles immediatly tries to occupy a new position closer together.

→ But as water and soil are incompressible and if soil is laterally confined, this rearrangement will not be possible without expulsion of pore water.

→ Thus as water is resisting the particle rearrangement, pore water pressure is increased above the static ~~water~~ pore water pressure.

→ At $t=0$; i.e. just after application of load all the external load is taken up by pore water. This pore water pressure above static value is called excess pore water pressure; whose magnitude is equal to the external applied stress (σ).



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Total pore water pressure = Static pore water pressure + Excess pore water pressure.

at $t=0$

$$u = \gamma_w h_2 + \sigma$$

$$u = (h_2 + h_3)\gamma_w$$

where;

$$h_3 = \frac{\sigma}{\gamma_w}$$

→ Initially no part of the stress increment is taken up by the soil particles. This is possible for a fully saturated soil which is laterally confined such that volume changes occur entirely due to deformation of soil in vertical direction. (This cond^n will practically be met when thickness of soil layer is small compared to its area).

→ However, if lateral strain is possible than from the starting itself, some part of the stress increment is taken up by the soil particles and hence, initial excess pore water pressure will be lesser than the increment of external load.

Case-III : (long time after application of external load)

→ Increase in pore water pressure above static value causes pressure gradients and results in transient flow of pore water towards drainage face. This flow continues so long as the pore water pressure has not fallen down to $(h_2\gamma_w)$ i.e. hydrostatic pore pressure.

→ Reduction of excess pore water pressure as drainage takes place is called dissipation.

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→ When the dissipation is complete, soil is said to be in drained condition.

→ prior to dissipation, with excess pore water pressure at its initial values, the soil is said to be in undrained condition.

→ Drained condition means that there is no stress induced pressure in the pore water.

→ Soil remains saturated throughout the process of dissipation.

→ Thus as pore water pressure dissipated, external forces are transferred to the soil grains, resulting into rearrangement of soil grains and increase in effective stress.

→ When all the excess pore water has dissipated the effective stress increases by magnitude of stress increment.

→ When soil is subjected to reduction in total normal stress, soil does not rebound to the initial condition of reduced normal stress.

→ This is ~~due to~~ because particle rearrangement due to total stress increase is largely irreversible.

→ If the ~~soil~~ soil is having clay, there can be a significant expansion. As a result, pore water pressure will be reduced and excess pore water pressure will become negative.

→ Pore water pressure will gradually increase to its static value, flow taking place into the soil, effective normal stress decreases and volume increases. This process is called Swelling.

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Effective Stress :-

→ Effective stress is defined as equal to the total stress (σ) minus the neutral stress or pore water pressure (u).

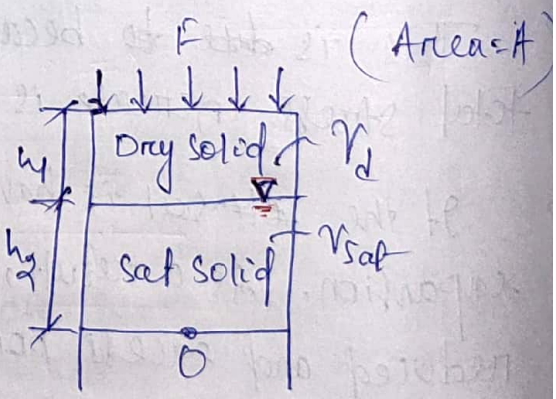
→ It is represented by symbol ($\bar{\sigma}$)

$$\bar{\sigma} = \sigma - u$$

→ Effective stress is the summation of forces transmitted through the contact bet^n soil grains per total area of the soil mass.

→ Effective stress is not a physical parameter i.e. it can not be measure using any equipment. It can only be derive.

Calculation of effective stress :-



$$\bar{\sigma} = \sigma - u$$

$$= \frac{F}{A} + \gamma_d h_1 + \gamma_{sat} h_2 - \gamma_w * h_2$$

$$\boxed{\bar{\sigma} = \frac{F}{A} + \gamma h_1 + \gamma_{sub} h_2}$$ ——

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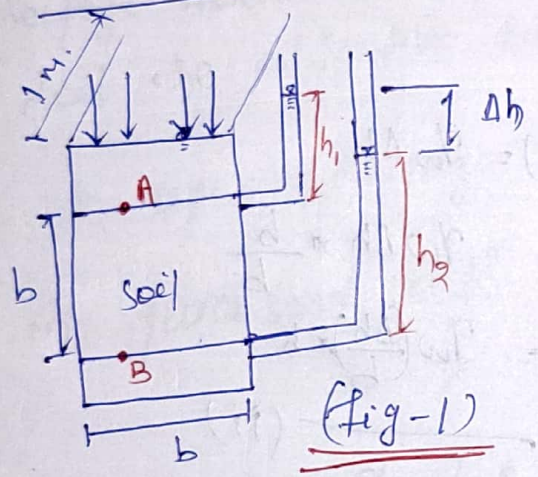
(5.4) Seepage pressure, effective stress, quick sand cond^n:-

Seepage:-

→ The process by which water flows through soil is called seepage.

(i) Downward Seepage:-

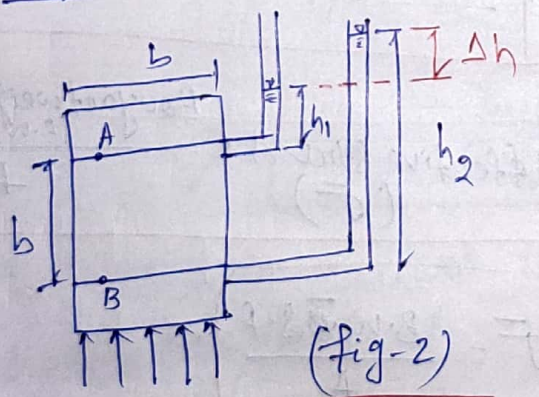


(fig-1)

Δh = loss in total head due to downward seepage in soil of length b.

Cross sectional area = $b * 1m.$
$= b\ m^2$

(ii) Upward seepage:-



(fig-2)

Δh = loss in total head due to upward seepage in soil of length b.

→ **Seepage force (S.F)**

S.F = γ_w * total head loss * Area

= $\gamma_w * \Delta h * (b * 1)$

$\therefore$ $\boxed{S.F = \gamma_w \Delta h b}$ ——

Seepage pressure (S.P) = $\frac{S.F}{Area} = \frac{\gamma_w \Delta h b}{(b*1)} = \gamma_w \Delta h$

$\therefore$ $\boxed{S.P = \gamma_w \Delta h}$ ——— (i)

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Hydraulic gradient :- (i)

$$i = \frac{\text{head loss occured during seepage}}{\text{length over which head loss occured.}}$$

→ It is defined as head loss per length causing head loss.

$$\therefore \boxed{i = \frac{\Delta h}{b}} \quad \text{——}$$

Now seepage pressure (S.P) = $\gamma_w \Delta h$

$$\Rightarrow S.P = \gamma_w \Delta h * \frac{b}{b}$$

$$= \gamma_w \left(\frac{\Delta h}{b}\right) * b$$

$$\therefore \boxed{S.P = \gamma_w i b} \quad \text{——} \quad (ii)$$

$$\therefore \boxed{S.P = \gamma_w \Delta h = \gamma_w i b} \quad \text{——}$$

For downward seepage ; Effective stress ($\bar{\sigma}$) = $\dfrac{\text{Bouyant weight (B.W)} + \text{Seepage force (S.F)}}{A}$

For upward seepage ; $\bar{\sigma} = \dfrac{B.W - S.F}{A}$

= From fig-1 & fig-2

$B.W = \gamma_{sub} * b * \text{Area}$; $S.F = (\gamma_w i b) * \text{Area}$

$$\therefore \bar{\sigma} = \frac{(\gamma_{sub} * b) * A \pm (\gamma_w i b) * A}{A}$$

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$$\bar{\sigma} = \gamma_{sub} b \pm \gamma_w i b$$ ——

Quick sand cond^n:-

→ In case of upward seepage flow, if the upward seepage force becomes equal to the bouyant weight of soil the effective stress in the soil becomes zero.

→ In upward seepage flow $\bar{\sigma} = \gamma_{sub} b - \gamma_w i b$ ——(1)

in quick sand cond^n

$\Rightarrow$ bouyant weight = seepage force

$\Rightarrow \gamma_{sub} b - \gamma_w i b = 0$

$\Rightarrow \gamma_{sub} b = \gamma_w i b$

$\Rightarrow$ $i_{cr} = \frac{\gamma_{sub}}{\gamma_w}$ ——

→ The hydraulic gradient is also known as critical hydraulic gradient represented by a symbol i_{cr}.

$$i_{cr} = \frac{\gamma_{sub}}{\gamma_w}$$ ——

→ When upward flow is taking place at critical hydraulic gradient, a soil such as sand looses all its shearing strength because effective stress become zero.

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→ This cond^n is called quick sand cond^n or boiling of sand because surface of sand looks as if it is boiling.

Note

→ quick sand cond^n is not a type of sand, it is a hydraulic cond^n

$$\therefore i_{cr} = \frac{\gamma_{sub}}{\gamma_w} \quad \text{---}$$

$$\gamma_{sub} = \frac{G-1}{1+e}\gamma_w \quad \text{---}$$

$$\Rightarrow i_{cr} = \frac{G-1}{1+e}\gamma_w * \frac{1}{\gamma_w}$$

$$\therefore \boxed{i_{cr} = \frac{G-1}{1+e}} \quad \text{---}$$

(5.1) Concept of permeability, Darcy's law, co-efficient of permeability :-

Permeability :-

→ permeability is the property of soil due to which water flow through the inter connected voids.

→ Coarse grained soils are regarded as highly permeable; where as fine grained soil (clay) is regarded as impermeable.

Darcy's law of permeability :

Statement :-

→ According to darcy's law, for laminar flow the velocity of liquid in soil is directly proportional to the

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"hydraulic gradient of soil".

$\therefore \quad V \propto i$

$\Rightarrow \boxed{V = Ki} \quad \text{---------- (1)}$

where K is the constant of proportionality known as co-efficient of permeability.

$\therefore \quad V = Ki$

$i = \frac{h}{L} = \frac{\text{head loss}}{\text{length}}$

$\Rightarrow V = K * \frac{h}{L}$

If $L = 1, h = 1$

then $V = K$ or $K = V$

So, the co-efficient of permeability is equal to the velocity of flow of liquid in a soil having unit head loss over unit length.

Discharge $(Q) = AV$ ($\because$ A = area of cross section of soil, V = velocity of liquid in soil)

$\Rightarrow Q = A(Ki)$

$\therefore \boxed{Q = AKi}$

limitation:-

- Darcys law is only applicable for laminar flow but in practice not all the flows are laminar.

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→ For laminar flow reynold's no. is less than 2000. But in wax experiment it is found that for validity of darcy's law reynold's no. should be less than 1.

(5.2) Factors affecting permeability:-

The different factors that affect permeability are:-

(i) Void ratio
(ii) properties of fluid
(iii) Size of soil particle
(iv) Soil structure
(v) presence of adsorbed water
(vi) presence of foreign matters
(vii) Degree of saturation.

(i) Void ratio:-

→ If void ratio will be more than there will be more void spaces so, the permeability will be more. So the permeability is directly proportional to void ratio.

(ii) properties of fluid:-

→ permeability is directly proportional to the density/unit weight of liquid & inversely proportional to the viscosity of the fluid.

$\therefore K \propto \frac{\gamma}{\mu}$

$\rightarrow \frac{K_1}{K_2} = \frac{\gamma_1/\mu_1}{\gamma_2/\mu_2} \therefore \boxed{\frac{K_1}{K_2} = \frac{\gamma_1 \mu_2}{\gamma_2 \mu_1}}$

μ = Viscosity of fluid.

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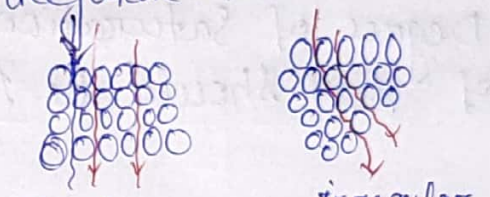
(iii) Size of the soil particle :-

→ The permeability of soil is directly proportional to square of size of soil particles.

$$K = C * (D_{10})^2$$

(iv) Soil structure :-

→ permeability will be more if the soil's are arranged in regular manner than irregular manner.



regular irregular

(v) presence of adsorbed water :-

→ Due to presence of adsorbed water the permeability of soil decreases, as the adsorbed water decreased the air voids.

(vi) presence of foreign matter :-

→ Due to presence of foreign matter the void spaces get filled so, the permeability of soil decreases.

(vii) Degree of saturation :-

→ The permeability of partially saturated soil is less than that of fully saturated soil.

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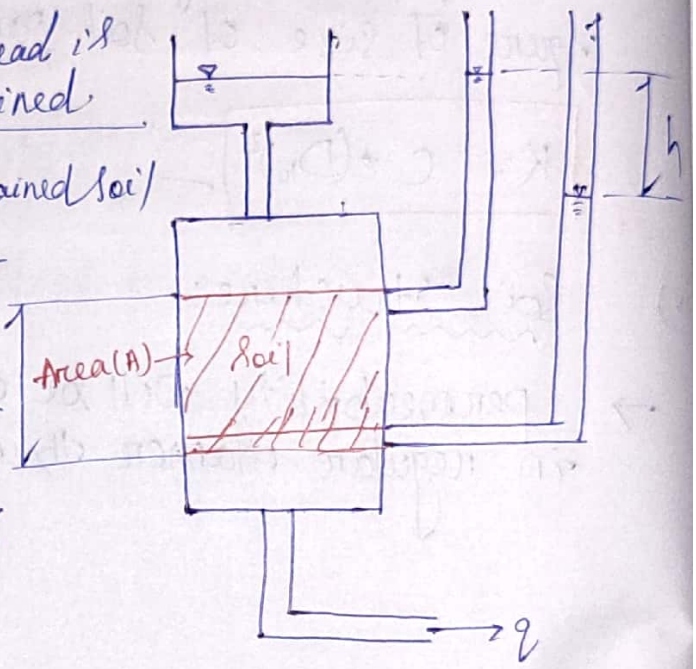
(5.3) Constant head permeability & falling head permeability

→ Coefficient of permeability for coarse grained soil is determined by means of constant head permeability test.

→ Degree of saturation of soil should be 100%.



NOW:-

Consider a cylindrical soil specimen of length(L) and cross-sectional area(A) is subjected to constant head permeability test.

Let h = head difference / head loss in length 'L'

K = co-efficient of permeability of soil

We know Discharge (Q) = AV

$= AKi$

$\Rightarrow Q = AK\frac{h}{L}$

$\Rightarrow \boxed{K = \frac{QL}{Ah}}$

L = length of soil specimen.

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Now $k = \frac{QL}{Ah}$

$\therefore Q = \frac{\text{Vol.}}{\text{time}} = \frac{V}{t}$

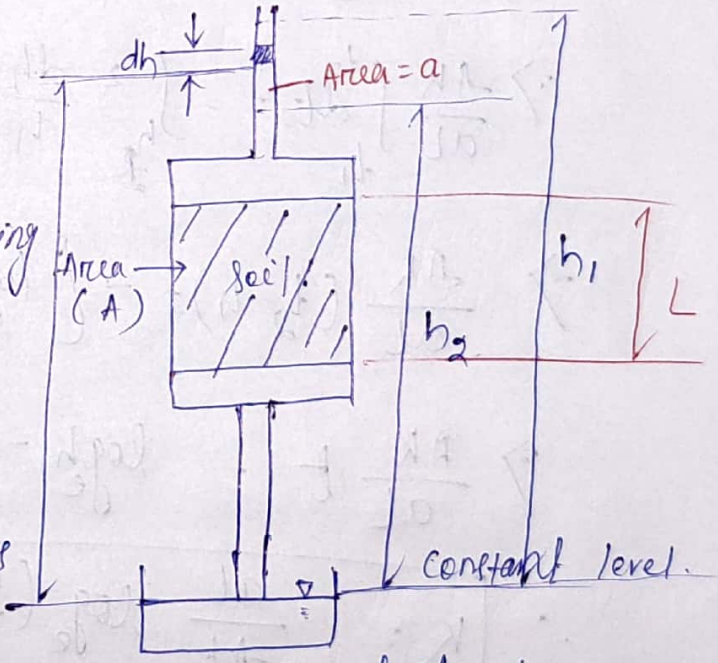
$\Rightarrow \boxed{k = \frac{VL}{tAh}}$ ——

Falling head permeability test:-

→ For fine soils, falling head / variable head method is used.

→ Consider a soil specimen of length (L) and cross section area (A), subjected to falling head permeability test.

a: Cross section area of stand pipe.



→ Here, the head of soil falls from h_1 at time instant t_1 to h_2 at time instant t_2 - Constant head chamber.

→ Let us consider the instant at which the head is 'h'. For very small time dt, the head fall by height dh. Let the discharge through the sample be Q. & discharge in stand pipe be 'q'

$\therefore q = -\left(\frac{a \cdot dh}{dt}\right)$ —(i) $\left(\because q = \frac{\text{Vol}}{\text{Time}}\right)$

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$Q = kiA$ — (ii)

∴ Discharge in stand pipe = discharge in soil

$$\Rightarrow -\left(a\frac{dh}{dt}\right) = kiA$$

$$\Rightarrow -a\frac{dh}{dt} = k\frac{h}{L}A$$

$$\Rightarrow a\,dh = -Ak\frac{h}{L}dt$$

$$\Rightarrow \frac{Ak\,dt}{aL} = \frac{-dh}{h}$$

$$\Rightarrow \frac{Ak}{aL}\int_{t_1}^{t_2} dt = -\int_{h_1}^{h_2}\frac{dh}{h}$$

$$\Rightarrow \frac{Ak}{aL}(t_2 - t_1) = -\left[\log_e h_2 - \log_e h_1\right]$$

$$\Rightarrow \frac{Ak}{aL}t = \log_e h_1 - \log_e h_2 \qquad (\because t_2 - t_1 = t)$$

$$\therefore \boxed{k = \frac{aL}{At}\log_e\left(\frac{h_1}{h_2}\right)}$$

$$\boxed{k = \frac{2.303\,aL}{At}\log_{10}\left(\frac{h_1}{h_2}\right)}$$

Where h_1 = initial head
h_2 = final head
t = time interval = $t_2 - t_1$
L = length of soil.

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(6) Compaction and consolidation :-

(6·1) Compaction :-

→ Compaction of soil is the process of increasing the unit weight of soil by forcing the soil solids into a dense state & reducing the air voids.

→ Compaction leads to increase in shear strength and helps improve the stability and bearing capacity of soil. It also reduces the compressibility and permeability of soil.

→ This is achived by applying static or dynamic loads to the soil.

→ Compaction is measured quantitatively in terms of dry unit wt. (γ_d) of the soil.

Laboratory test for compaction :-

→ There are 2-types of lab test used for compaction of soil.

(i) Standard proctor test / Light compaction test :-
(ii) Modified proctor test / Heavy compaction test :-

(i) Standard proctor test / Light Compaction test :-

Aim of the experiment :- To determine the optimum moisture content (OMC) of soil mass.

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Apparatus required:-

(i) Cylindrical metal mould; with detachable base plate having internal dia. of 10.16 cm, height of 11.86 cm & internal vol. of 945 cc.

(ii) Collar of height 5 cm.

(iii) Rammer of mass 2.5 kg with a height of free fall of 30.48 cm.

Procedure:-

(i) About 3 kg of dry soil passing through 4.75 mm IS sieve is taken in a tray.

(ii) Then some quantity of water is added to the dry soil. The amount of water is taken as 4% for coarse grained soil & 8% for fine grained soil for initial trial.

(iii) The computed quantity of water is added to soil in the tray and mixed thoroughly by means of hand.

(iv) Then the mass of mould with base plate is found and is taken as M_1.

(v) Then the mould is filled with some quantity of wet soil in the tray and compacted with 25 uniformly distributed blows on the surface using the standard rammer. The compacted soil should be about 1/3 of height of mould.

(vi) Then the surface of compacted soil is scratched with a knife to ensure bond with the next layer.

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(vii) Then another layer (2nd layer) of soil is placed over the 1st layer in the same procedure prescribed above.

(viii) Care should be taken to see that the soil mass does not protrude more than 6mm into the collar fitted (extend out of space) above the cylinder.

(ix) Then the collar is removed from the top of the soil and the excess soil above the cylinder is wiped out by using a knife.

(x) Then the cylinder with wet soil mass and detachable base plate is weighted, let the mass be M_2.

(xi) Then from given vol. of cylinder the dry density of soil mass is found out as follows;

$$\text{Density } (\rho) = \frac{M}{V} = \frac{M_2 - M_1}{V_{cylinder}}$$

Then $\boxed{\rho_d = \frac{\rho}{1+w}}$ —

where w = water content of soil mass = % of water added to dry soil.

(xii) This procedure is repeated to the no. of soil samples by changing the value of water content.

(xiii) for each sample the dry density is found out for the corresponding water content.

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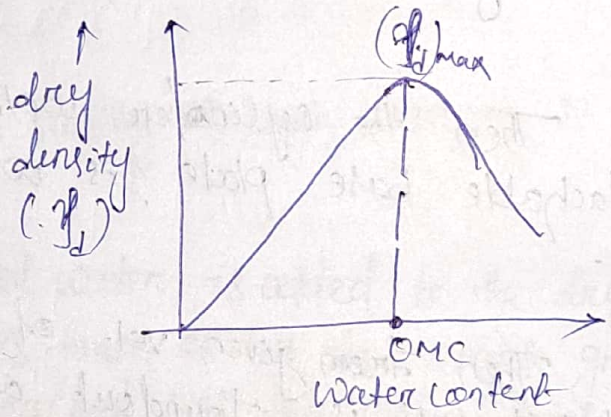
(xiv) Then a graph is plotted in betⁿ water content as abscissa (x-axis) & dry density as Ordinate (Y-axis)

(xv) From the graph the optimum moisture content of soil is found out.

Optimum moisture content:- (OMC)

→ The water content corresponding to maxᵐ dry density is called as optimum moisture content.



Conclusion:-

→ From the test it is concluded that initially the dry density increases with increase in water content. Then the dry density attends its maxᵐ value. Then the dry density decreases with increase in water content.

→ The water content corresponding to maxᵐ dry density is called as optimum moisture content (OMC).

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Zero air void line:-

→ The relationship betn water content & dry density of fully saturated soil is called as zero air void line.

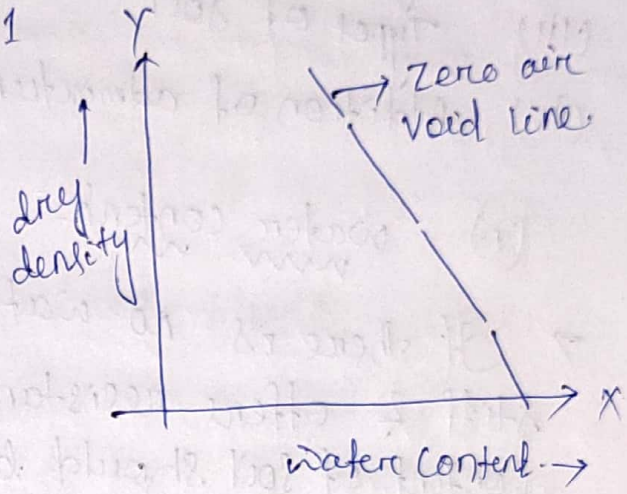
→ for zero air void line, $S_a = 1$

$$\therefore \rho_d = \frac{G \rho_w}{1+e} \quad \text{——— (i)}$$

$$es = Gw$$

$$\rightarrow e = Gw \quad (\because S = 1) \quad \text{——— (ii)}$$

from @ (i) & (ii)



$$\boxed{\rho_d = \frac{G \rho_w}{1+Gw}} \quad \text{———}$$

This is theoretical max^m dry density; which can not be achive practically on the field because what ever the type of compaction their are some amount of air void always present in the compacted soil.

(ii) Modified proctor test / Heavy compaction test:-

→ Weight of hammer = 4.54 kg
→ Height of fall = 18" = 457.2mm
→ volume of mould = 944 cc
→ Compacted on 5 layers with 25 blows in each layer.

procedure is same as light compction test.

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Factors affecting compaction:-

(i) Water content
(ii) Type of compaction
(iii) Types of soil
(iv) Addition of admixtures.

(i) water content:-

→ If there is no water in soil then, the soil becomes stiff & offers resistance to compaction. So, the water content of soil should be adjusted to OMC for quick compaction.

(ii) Types of ~~soil~~ compaction:-

→ Compaction will be quick in dynamic compaction than static compaction.

(iii) Type of soil:-

→ The coarse grained soil will be compacted easily and attempts max^m dry density than fine grained soil.

(iv) Addition of admixtures:

→ The soil will be compacted rapidly by the use of certain admixtures like flyash or terazyme

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Field compaction methods :-

In field, compaction is done by 2 methods

(a) Roller
(b) Rammer.

Rammer :-

→ It consists of a wooden shaft with an iron shoe attached with the bottom of shaft.

→ The wt. of iron shoe varries from 2-5 kg.

→ It is used for compaction of soil in confined areas like bridge, trench filling etc..

Roller :-

There are 4 types of roller,

(a) Smooth wheeled roller
(b) pnumatic tyred roller
(c) Sheep foot roller
(d) Vibratory Compactor or Vibratory roller.

Smooth wheeled roller :-

→ It consists of 3 wheels, 2 large wheels, in rear and one small wheel in front side.

→ It's total mass varries bet^{n} 2-15 mega gram
(1 mega gram = 1000 kg)

→ It is used ~~from~~ for compaction of granular base course in road.

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Pneumatic tyreed roller:-

→ It consist of 9-11 wheels
→ It's total mass varries from 5-200 megagram
→ It uses compressed air to develope required pressure.
→ It is used for compaction of cohessive as well as non-cohessive soil.

Sheep foot roller:-

→ It is similar to smooth wheeled rolled with some projections present on the surface of roller.
→ Brick ballast or water is used to increase the wt. of drum.
→ It is used for compaction of cohessive soil.

Vibrator roller or Vibrator compactor:-

→ It is of 2 types.
(i) Vibrator roller
(ii) Vibrator compactor
→ If vibration is induced on roller it is called vibrator roller.
→ If vibration is induced in plate then it is called vibratory compactor.
→ It is available in both smooth wheel type & pneumatic tyreed wheel type.

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→ It is used for compaction of granular base coarse which thickness varies from 0 to 30cm.

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Consolidation :-

→ Consolidation is a process of removal of water voids from soil without replacement of water voids by air voids.

(or)

→ Consolidation is the gradual reduction in the volume of fully saturated soil of low permeability due to dissipation of excess pore water pressure set up due to increase in total stress or otherwise.

→ The total settlement of soil is expressed as three components

(i) Immediate Settlement = S_{imm}
(ii) primary consolidation
(iii) Secondary Consolidation.

$$S_t = S_{imm} + S_{pri} + S_{2^o} \quad \text{---------- .}$$

Immediate Settlement :-

→ If soil is initially partially saturated, expulsion of air as well as compression of pore air may take place with the application of external load which is called Initial compression. It is a immidiate phenomenon.

1°- Consolidation :-

→ 1°- Consolidation occurs due to expulsion of excess pore water pressure generated due to increase in total stress.

→ It is a time dependent phenomenon.

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2°- consolidation :-

→ 2°- consolidation occurs after the end of 1°- consolidation.

→ It occurs at constant effective stress.

→ 2°- consolidation is thought to be due to gradual readjustment of clay particles into a more stable configuration following the structural disturbance caused by the decrease in void ratio.

Distinction betⁿ compaction and consolidation :-

Compaction	Consolidation
(i) It is the process of removal of air voids	(i) It is the process of removal of water voids.
(ii) It is a rapid process	(iii) It is a gradual process.
(iii) It can be carried out by laboratory	(iii) It is done in field.
(iv) In compaction curve the dry density increases initially with increase in water content. Then attain maxᵐ value at OMC. Then the dry density decreases with increase in water content.	(iv) In consolidation curve the void ratio decreases with increase in pressure.
(v) It is carried out in dry or partially saturated soil.	(v) It is carried out in fully saturated soil.

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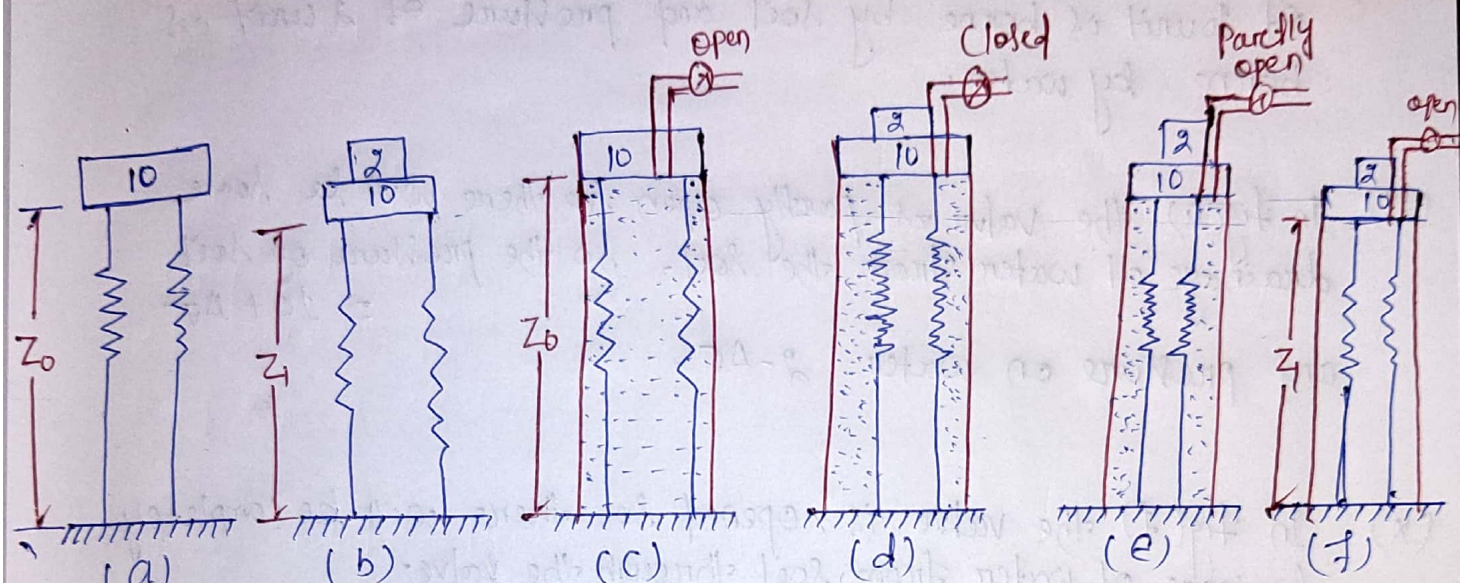
Terzaghi's model analogy of compression / Springs showing the process of consolidation :- (Spring analogy)



(i) fig shows a saturated soil mass taken in a container consist of soil particles forming of skeleton of soil and voids filled with water.

(ii) Here the skeleton formed of soil particles can be assumed to be replaced by a no. of springs and water filling voids in soil mass by the water filling the cylinder.

(iii) The compressive stress is caused by load applied on piston placed on top of spring.

(iv) An outlet valve is provided to control the drainage of water from the cylinder.

(v) Let, Z_0 be the length of spring under a pressure of 1 unit, as shown in fig (a).

(vi) Let, the length of spring decreases to a depth of Z_1 due to increase of pressure by 2 unit as shown in fig (b).

(vii) In fig (c) a spring is filled with water but no drainage takes place as the entire pressure of 1 unit, is taken by spring.

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(viii) In fig(d) an additional pressure of 2 unit acts and the valve is closed, so no drainage will takes place. So pressure of 10 unit is borne by soil and pressure of 2 unit is borne by water.

(ix) In fig(e) the valve is partly open so there will be some drainage of water from the soil. So the pressure on soil $= 10 + \Delta\sigma$

and pressure on water $= 2 - \Delta\sigma$.

(x) In fig(f) the valve is opened so, there will be completely drainage of water from soil through the valve.

Hence pressure on soil $= 12$.

& pressure on water $= 0$

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(9) Foundation Engineering:-

(9.1) Introduction:-

→ Footings are generally the lowermost supporting part of the structure known as sub-structure and are the last structural elements through which load is transferred to foundation, comprising soil/rock.

→ The structural elements transfer the applied loads from one part of the building to the other. These are in turn transmitted to the foundation which transfers it to the underlying soil/rock.

Functions of foundations:-

(i) Reduction of load intensity:-

Foundations distribute the load of the superstructure to a larger area so that total intensity of the load does not exceed the safe bearing capacity of soil.

(ii) Even distribution of load:-

foundation distribute the non uniform load of the super structure evenly to the subsoil.

(iii) provision of level surface:-

foundations provide a levelled and hard surface over which a super-structure can be built.

(iv) Lateral Stability:-

It anchors the super-structure to the ground thus imparting stability to the building.

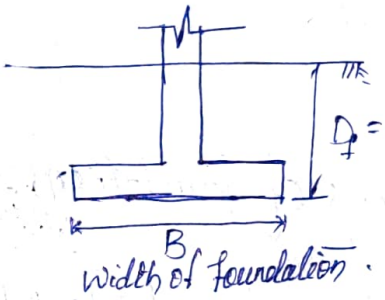
(v) Safety against undermining :-
It provides safety against undermining or scouring due to burrowing animals & flood water.

(vi) Protection against soil movements :-
Special measures prevent or minimise the distress (cracks) in superstructure, due to expansion or contraction of sub-soil.

Classification of foundation

As per Terzaghi :-

→ If $\frac{D_f}{B} < 1$ → The foundation is called shallow foundation.

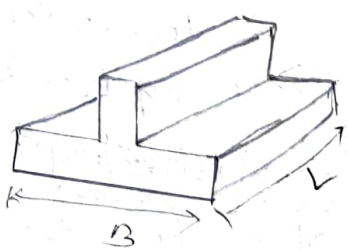


→ Shallow foundation transferes the load at smaller depth e.g.:- combined footing, Raft foundation, Isolated footing

→ If $\left(\frac{D_f}{B}\right) > 15$ → The foundation is called deep foundation e.g., pile foundation.

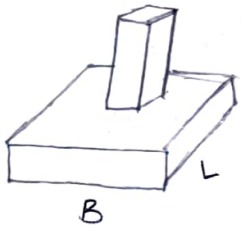
Types of footing :- (Shallow foundation) :-

1. Strip footing :-
These are also known as wall footing to support wall. [If $L >> B$] → Strip footing



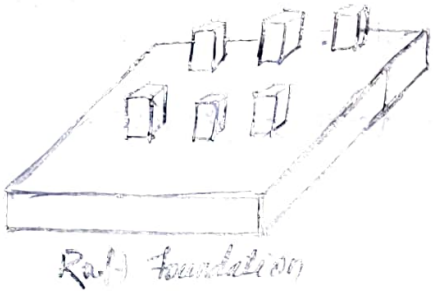
(2) Isolated footing:-

These are also known as spread footing. Isolated footing is used below the column.



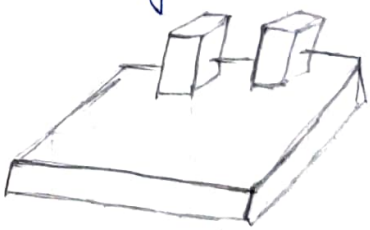
3. Raft/Mat foundation:-

These type of foundations are large continous footing which support all columns and walls of a structure and are constructed when soil is weak.



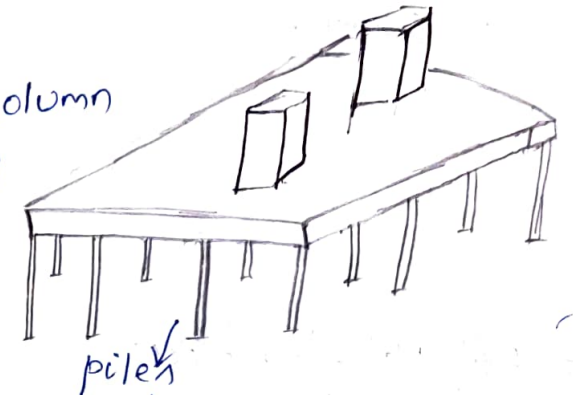
4) Combined footing:-

→ These footings are usally constructed due to space limitations and support two or more columns. They may be either rectangular or trapezoidal in shape.



Types of deep foundation:-

(1) Pile foundation:-



→ These are used to transmit heavey column loads to a group of pile joined at top by a pile cap.

→ These piles transmit the structural loads to the underlying soil through friction and bearing.

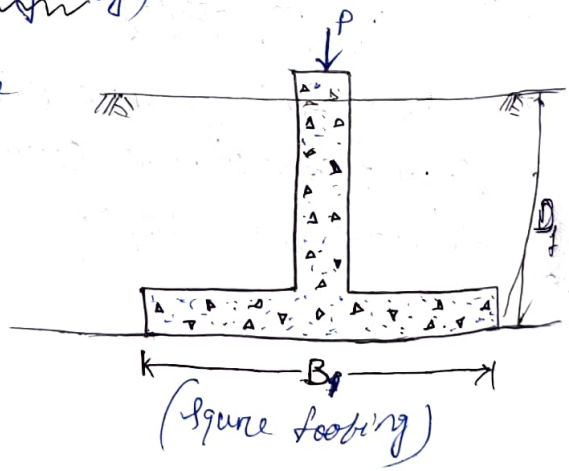
→ Such type of foundation system is usually adopted when the material below footing is too weak to support the structure and it becomes essential to transfer loads to better strata underlying weaker strata.

→ These foundations are very expensive.

Basic definitions:-

(1) Gross pressure or gross loading intensity (q_g)

→ It is the total pressure intensity at the base of footing.



$$q_g = \frac{P}{B^2} + \gamma D_f$$

(2) Net pressure intensity (q_n)

→ It is generally the loading intensity at the base of footing in excess of the load intensity that the soil was originally subjected to that causes deformation in soil.

Hence, net pressure intensity = $\frac{P}{B^2}$

$q_n = \left(\frac{P}{B^2} + \gamma D_f\right) - \gamma D_f$ (∵ with backfill)

$\therefore \boxed{q_n = q_g - \gamma D_f}$ ——

$\boxed{q_n = \frac{P}{B^2} - \gamma D_f}$ (∵ without back fill)

(3) Ultimate bearing capacity (q_u)

→ The maximum gross intensity of loading that the soil can support before it ~~fail~~ fails in shear is called ultimate bearing capacity (q_u)

(4) Net ultimate bearing capacity (q_{nu})

→ The max^m net intensity of loading at the base of the foundation that the soil can support before failing in shear.

$\boxed{q_{nu} = q_u - \gamma D_f}$ ——

(5) Net Safe bearing capacity: (q_{ns})

→ It is the max^m net pressure a soil can carry safely with out the risk of shear failure.

$\boxed{q_{ns} = \frac{q_{nf}}{F.O.S}}$ ——

(6) Gross Safe bearing capacity (q_s) or Safe bearing capacity (q_s)

→ It is the max^m gross pressure soil can carry safely with out the risk of shear failure.

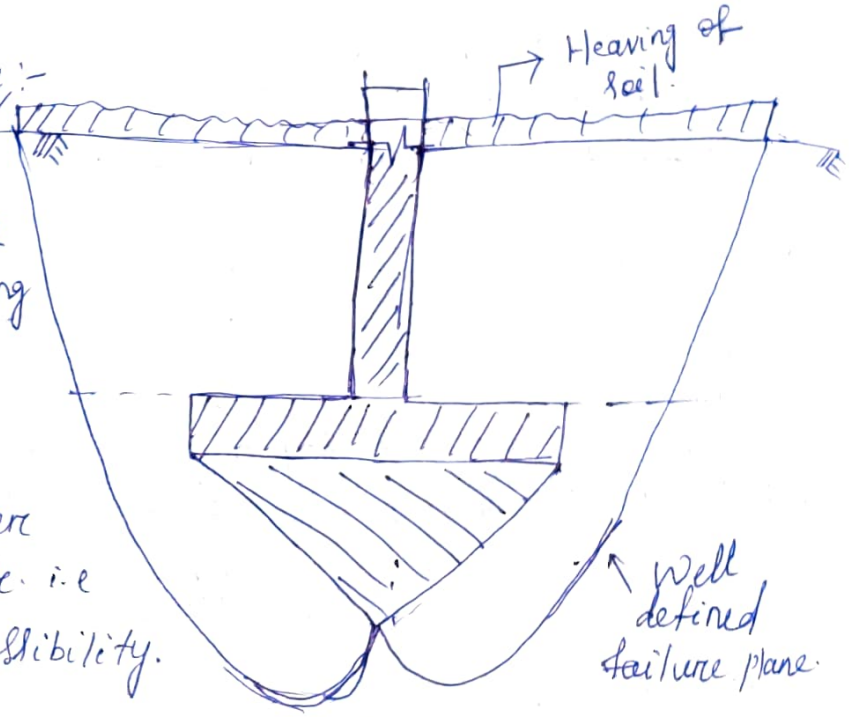
$\boxed{q_s = q_{ns} + \gamma D_f}$ ——

Types of failure :-

There are 3 types of bearing capacity failures of soil :-

(1) General Shear failure
(2) Local Sheare failure
(3) punching Shear failure.

(1) General Shear failure :-



→ General Shear failure occurs in soil possessing Brittle type shear Stress curve.

→ Dense Sand, Silt, over consolidated clay etc. i.e in soil of low compressibility.

→ In this case failure pattern is well defined and Sudden Shear failure is experienced with the bulging (Heaving) of ground Surface adjacent to foundation at both sides.

→ As the State of plastic equilibrium is reached initially in the soil arround the edge of footing and gradually spread downward and outwards.

→ ultimately, the State of plastic equilibrium is fully developed throughout the soil and the failure Surface is as shown in the above diagram.

→ Generally the general shear failure occure in the soil having relative density (D_r) > 70%

The characteristics of general shear failure are:-

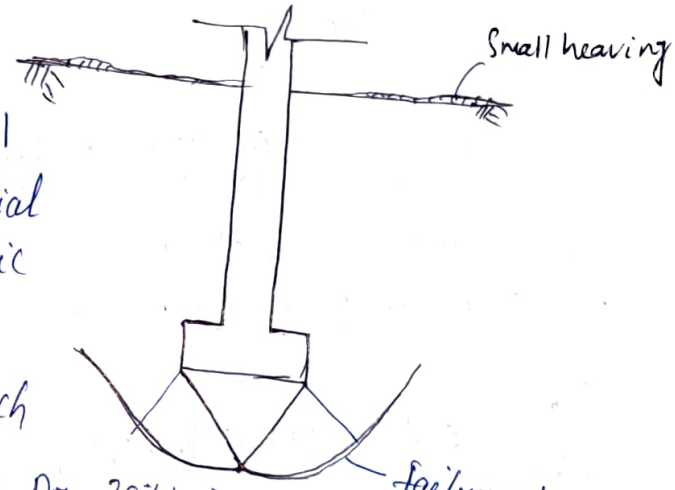
(i) It has well defined failure surface reaching upto ground surface.

(ii) Failure is accompanied by tilting of footing.

(iii) This failures occures in soils of low compressibility.

(2) Local Shear failure:-

→ In this failure there is significant compression of soil under the footing and only partial development of state of plastic equilibrium is reached.



→ The failure surface donot reach the ground surface.

→ It occures in the soil having Dr 30% to 70%.

The characteristics of local shear failure are,

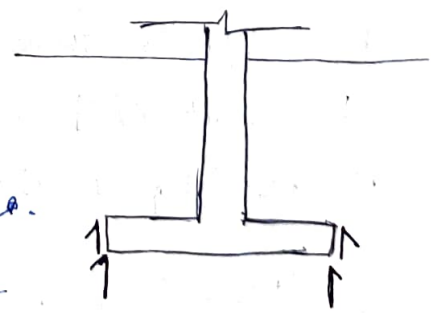
(i) The failure pattern is clearly defined only immediately below the footing.

(ii) There is no tilting of footing.

(iii) It occures in soils of high compressibility.

(3) Punching Shear failure:-

→ This failure occures when there is relatively high compression of soil under the footing accompanied by shearing in vertical direction around the edges of footings.



→ The characteristics of punching shear failure are:-

(i) No failure pattern is observed.

(ii) No tilting of footing.

(iii) It occures in soil of low compressibility

→ Generally occurs in very loose sand with relative density < 30%.

(9.2)

Terzaghi's theory of bearing capacity:-

Assumption :—

- The soil is homogeneous & isotropic.
- The shear strength is represented by Columb's eqn.
- The strip footing has a rough base.
- Failure zones do not extend above the horizontal plane through the base of footing.

Terzaghi's analysis of bearing capacity:-

- According to this theory the ultimate & Net ultimate bearing capacity of soil for strip footing is given by

$$q_u = cN_c + qN_q + \frac{1}{2}B\gamma N_\gamma \quad \text{—} \quad q_{nu} = cN_c + q(N_q - 1) + \frac{1}{2}B\gamma N_\gamma$$

where
cN_c = effect of cohesion
qN_q = Effect of over burden
$B\gamma N_\gamma$ = Effect of soil in the shearing zone

c = Cohesion.
$q = \gamma \times D_f$
D_f = depth of foundation.
B = width of Footing.
N_c, N_q, & N_γ are the bearing capacity factors.

(i) $N_c = (N_q - 1)\cot\phi$; (ii) $N_q = \frac{a^2}{2\cos^2(45 + \phi/2)}$
(iii) $N_\gamma = \frac{1}{2}\tan\phi\left[\frac{K_{P\gamma}}{\cos^2\phi} - 1\right]$

Where:-

K_{pr} = Passive earth pressure co-efficient

Where

$$a = e^{(3\pi/4 - \phi/2)\tan\phi}$$

Hence N_c, N_q & N_γ are the function of (ϕ) only. As K_p is also a function of ϕ. The value of bearing capacity factor increases as the value of ϕ increases.

In clayey soil :-

When $\phi = 0$,

$$\left.\begin{aligned} N_c &= 5.7 \\ N_q &= 1 \\ N_\gamma &= 0 \end{aligned}\right\} \text{ i.e in clayey condition.}$$

Modification of Terzaghi's Bearing capacity equation for various types of footing :-

(1) Square footing :-

$$q_{nu} = [1.3\, cN_c + q(N_q - 1) + 0.4 B\gamma N_\gamma]$$

(2) Circular footing :-

$$q_{nu} = [1.3\, cN_c + q(N_q - 1) + 0.3 B\gamma N_\gamma]$$

(3) Rectangular footing :-

$$q_{nu} = \left[\left(1 + 0.3\frac{B}{L}\right) cN_c + q(N_q - 1) + \left(1 - 0.2\frac{B}{L}\right)\left(\tfrac{1}{2} B\gamma N_\gamma\right)\right]$$

→ The above formula is for general shear failure.

→ If the failure is local shear failure then the above formula should be modified by N_c', N_q' & N_γ'

$$\tan\phi' = \frac{2}{3}\tan\phi$$

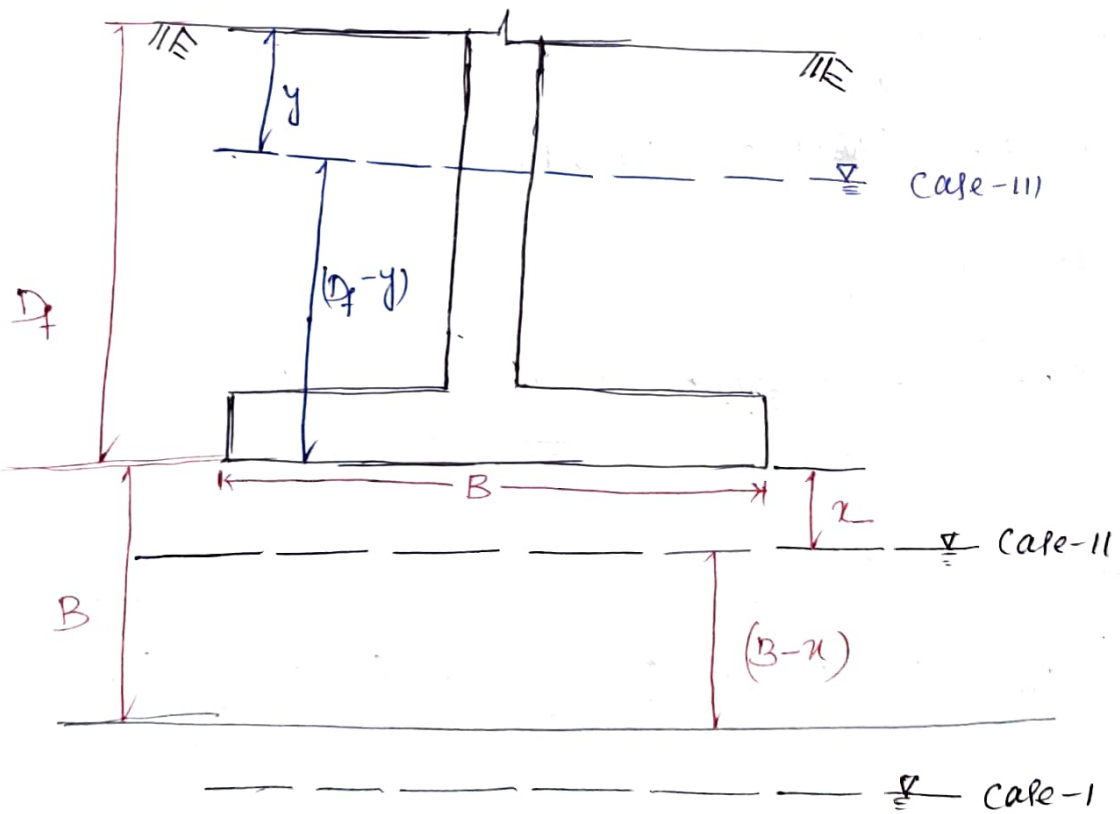
$$c' = \frac{2}{3}c$$

→ $$q_{nu} = c'N_c' + qN_q' + \frac{1}{2}B\gamma N_\gamma'$$

Note

for $\phi > 36°$; general shear failure is assumed & for $\phi < 29°$; local shear failure is assumed.

Modification of Terzaghi's equation for the location of water table :-



We know that Terzaghi's equation for strip footing is

$$q_{nu} = cN_c + q(N_q - 1) + \frac{1}{2} B\gamma N_\gamma$$

→ Soil in the zone from (D_f) to $(D_f + B)$ is affected by the shear failure.

→ For drained condition testing effective stress approach is used.

Case-1:- When water table is below depth $D = (D_f + B)$

$$q_{nu} = cN_c + \gamma_t D_f (N_q - 1) + \frac{1}{2} B \gamma_t N_\gamma$$

where γ_t = Bulk unit wt.

Case-2:- When water table is between depth $D = [D_f$ to $(D_f + B)]$

$$q_{nu} = cN_c + \gamma_t D_f (N_q - 1) + \frac{1}{2} [x \cdot \gamma_t + (B - x) \gamma_{sub}] N_\gamma$$

Where

$(x\gamma_t + (B - x)\gamma_{sub})$ is effective wt. of soil in the region from D_f to $(D_f + B)$ at depth $(D_f + B)$.

Case-3:-

When water table in in zone from depth (0 to D_f)

$$q_{nu} = cN_c + [\gamma_t \cdot y + (D_f - y)\gamma_{sub}] \cdot (N_q - 1) + \frac{1}{2} B \gamma_{sub} N_\gamma$$

Where $[\gamma_t y + (D_f - y)\gamma_{sub}]$ is the effective over burden wt. at the bottom of the foundation.

Shear Strength :-

(7.1)

→ Shear Strength is the capacity to resist shear stress.

→ Whenever a soil is loaded beyond its permissible limit then the shear failure of soil is occures.

Ex:- (i) Moment of wedge of soil behind the retaining wall.
(ii) Sinking of footing.

Shear Strength of soil :-

→ The resistance of soil to shear deformation is called as shear strength of soil.

→ The shear strength of soil depends upon :-

(i) Frictional resistance due to inter locking of soil particles.
(ii) Cohession bet^n the soil particles.

Mohr- Columb failure theory :-

→ According to this theory, "the shear strength of soil depends upon normal stress" as follows

$$\tau_f = c + \sigma \tan\phi$$

Where τ_f = Shear strength of soil
c = cohession
σ = normal stress
ϕ = angle of shearing resistance or angle of internal friction.

→ c & ϕ are not inherent properties of soil. These are related to the type of test and cond^n under which the testing is done.

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→ But after the development of effective stress concept, shear strength is express as

$$\tau = c' + \bar{\sigma} \tan\phi'$$

$\bar{\sigma}$ - effective stress

c' & ϕ' are effective stress shear strength parameter.

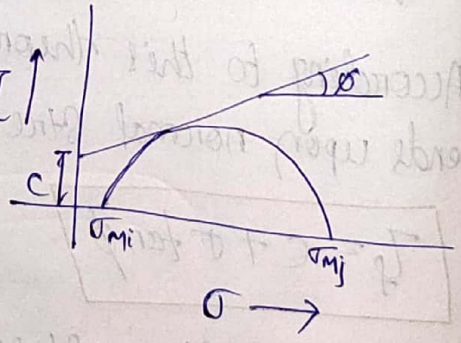
Cohesion :-

→ It is defined as the inter molecular force of attraction bet^n same particles.

(b) ~~Frictional cohesion~~ :-

Angle of internal friction

→ Angle of internal friction is a physical property of earth materials or the slope of linear representation of the shear strength of earth material.



Strength envelope :-

→ Combining- ~~rec~~ mohr-Columb Criteria it can be shown that if series of mohr circle is drawn corresponding to different test carried out on different specimen of soil upto failure, then common tangent to these circles represent mohr failure envelope or

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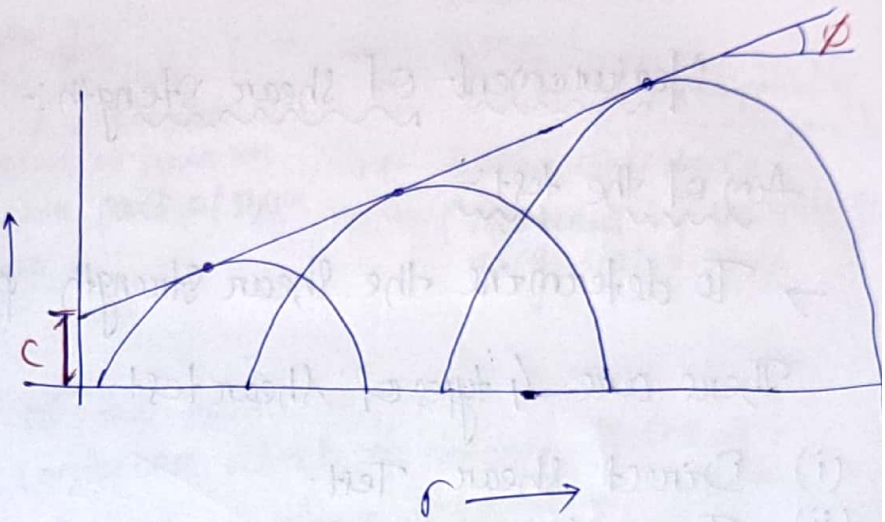
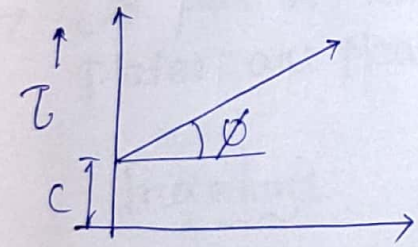
Strength envelop.

$\tau = c + \sigma \tan\phi$ —— $\tau \uparrow$

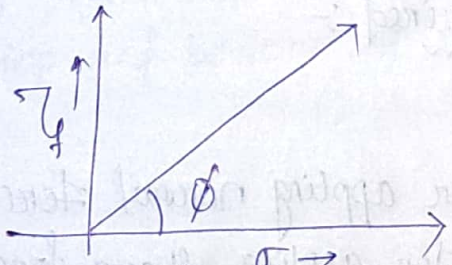


→ The relationship bet^n shear strength and normal stress of soil particle is called strength envelop.

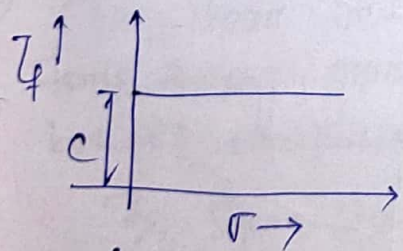
→ The strength envelop for different types of soil are as follows:-



(Cohesive soil)
(c-ϕ soil)



(Non-cohesive soil) →
(c=0)



(Purely cohesive soil)
(ϕ=0)

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Measurement of shear stength :-

Aim of the test :-

→ To determine the shear strength parameters (C & ϕ).

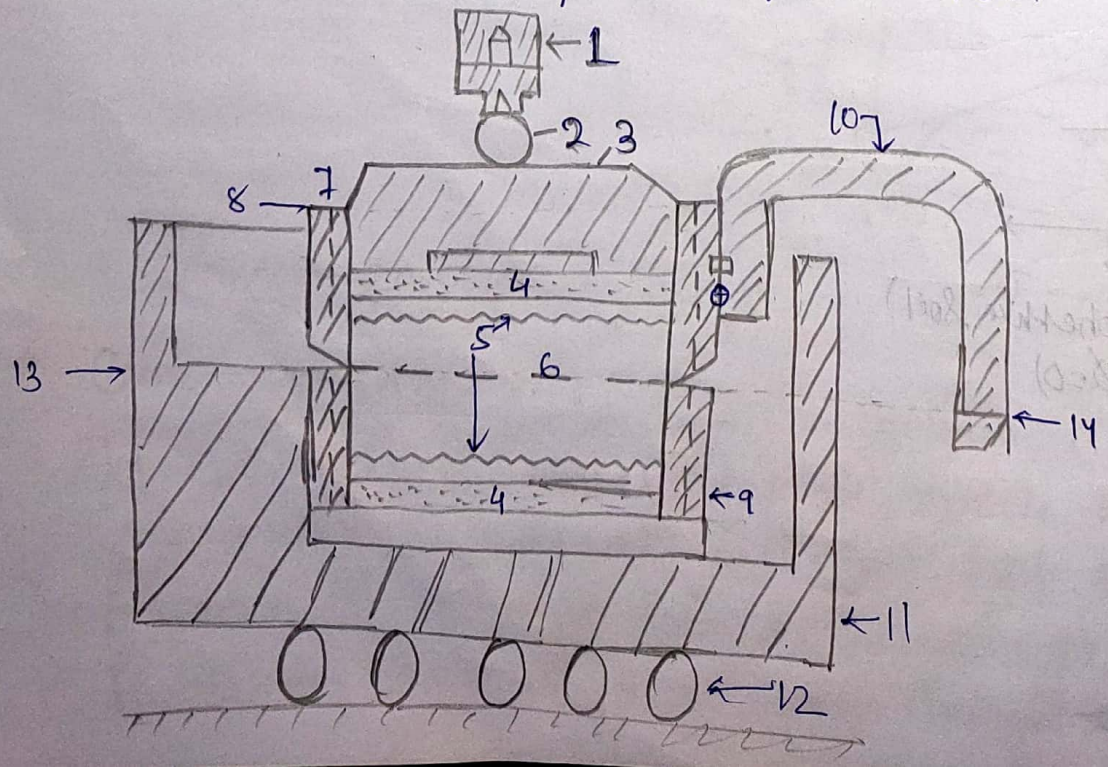
There are 4 types of shear test.

(i) Direct shear Test.
(ii) Triaxial compression test.
(iii) Unconfined compressive test.
(iv) Vane shear test.

(i) Direct shear test :-

Apparatus required :-

(i) Shear box.
(ii) Loading yolk for appling normal force.
(iii) Geared jack for appling shear force.
(iv) Facilities for measuring shear stress.
(v) Shear measurment and vertical deformation.



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1- loading yoke
2- Steel ball
3- loading pad
4- porous stones
5- metal grids
6- Soil Specimen
7- pins to fix two halves of shear box
8- upper part of shear box
10- U-arm
11- Container for shear box
12- Rollers
13- Shear force (applied by Jack)
14- Shear resistance. (measured by providing ring dial gauge)

→ The shear box consists of two halves, the lower halve is in contact with shear box container which is freely slides on rollers and to which the shear force is applied by means of geared Jack.

→ The Soil Specimen is placed on shear box such that it gets sheared on a horizontal plane exactly at its mid height.

→ A pair of metal grid plates and a pair of porous plates are placed on top and bottom of Soil.

Procedure:-

→ A normal stress 'σ' is applied on Specimen and is kept constant through out the test.

→ The shear Stress (τ) is caused by application of shear force through geared jack and is transmitted to the top of shear box is measured.

→ The shear stress is gradually increased until the Specimen fails. The shear stress (τ_f) corresponding to failure of Soil is noted.

→ The value of τ_f and σ are noted.

→ This test is repeated on 3 specimens by changing the value of σ & the corresponding value of τ_f are noted.

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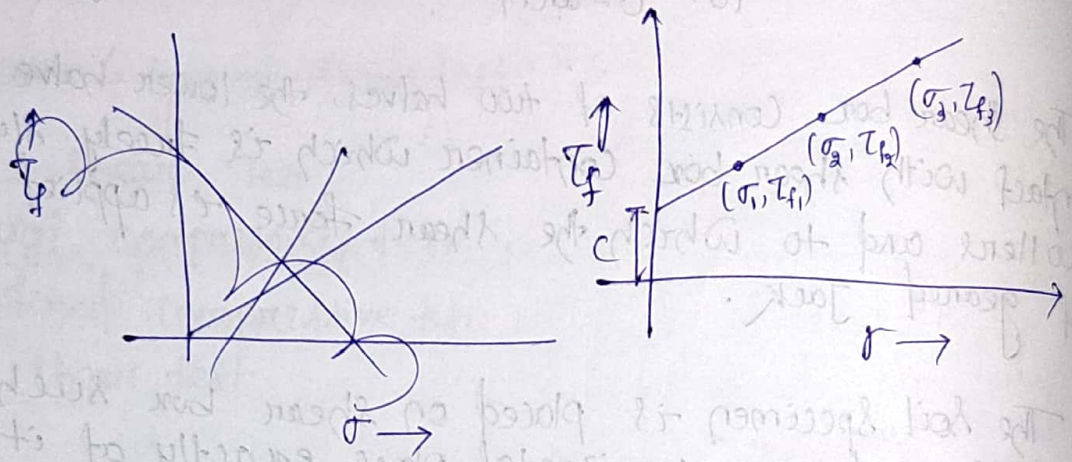
→ Then a graph is plotted by taking σ on x-axis & τ_f on y-axis.

→ Then from the graph the value of c & ϕ are computed.



Advantages of direct shear test:-

→ It is simpler than triaxial compression test.

→ Since the thickness of sample is small, quick drainage of pore pressure is possible.

Disadvantages of direct shear test:-

→ The failure plane is predetermined i.e. the specimen is not allowed to fail along the weakest plane.

→ The shear stress is not uniformly distributed being more at the edge and lesser at the centre.

→ Measurement of pore pressure is not possible.

→ Shear displacement causes reduction in area under shear. Corrected area should be used in computing normal & shear stresses.

→

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(ii) Triaxial Compression test:-

→ In this test the specimen is compressed by appling all the 3-type of principal stresses i.e $\sigma_1, \sigma_2, \sigma_3$.

→ In this test the soil specimen used in cylindrical in shape with length 2 to 2.5 times the diameter.

Apparatus required:-

(i) Triaxial cell

(ii) loading frame with accessories for appling gradually increasing axial load.

(iii) Provision for measuring axial force & axial displacement.

(iv) constant pressure system to apply and maintain constant cell pressure.

(v) pore pressure measuring apparatus.

(vi) vol. change gauge.

→ The triaxial cell consists of a high pressure cylindrical cell made up of transparent material.

→ An inlet for cell fluid and out-let for drainage of pore water is provided.

→ At the top an air release valve is provided to expale air from the soil.

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procedure:-

→ The soil specimen is kept inside the triaxial cell with porous plate at top and bottom.

→ The loading cap is placed on the top of porous plate.

→ The specimen is enclosed in a rubber membrane to prevent its constant with cell fluid.

→ After filling the cell with cell fluid usually water the required cell pressure (σ_3) is applied by means of constant pressure system.

→ The additional axial force called the deviator force is applied through the plunger.

→ The test is continued and additional axial force is increased till the specimen fails.

→ The deviator stress corresponding to deviator force at failure is noted.

→ This test is repeated on 3 soil specimens by changing the value of σ_3 on each specimen and corresponding values for deviator stress for diff. soil specimens are noted.

→ Then from the deviator stress (σ_d) and cell pressure (σ_3) the value of σ_1 is computed for diff. soil specimen as follow; $\boxed{\sigma_1 = \sigma_d + \sigma_3}$ —

where

σ_d = Deviator stress = $\frac{F}{A_c}$ (A_c = Corrected C/A)

→ Then by using σ_1 & σ_3 mohr's circle is drawn for each soil specimen.

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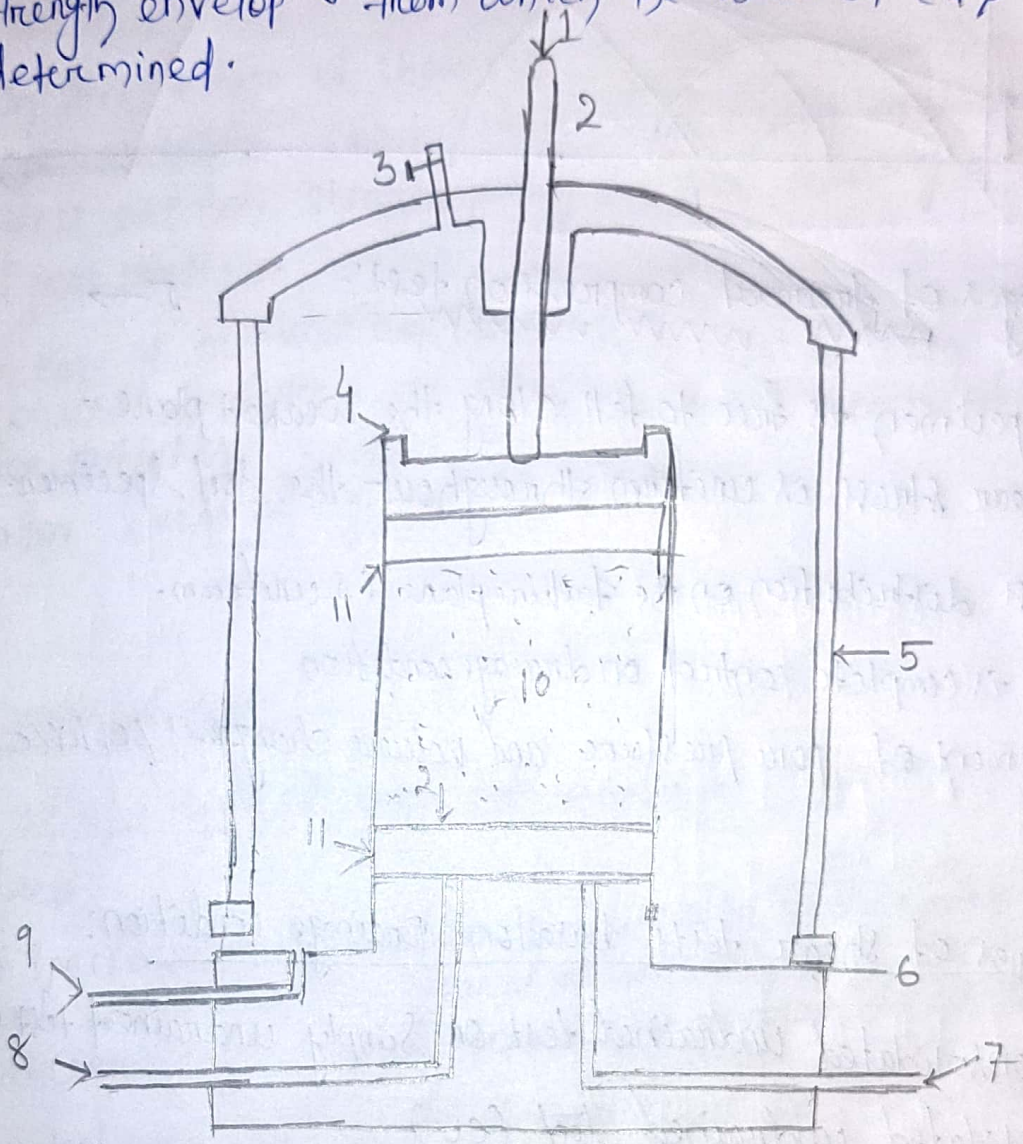
→ Then common tangent drawn on 3 mohr's circles gives the strength envelop from which the value of c & ∅ are determined.



1. Axial load (measure by proving ring) dial gauge
2. Loading ram
3. Air release valve
4. Top cap
5. perspex cylinder
6. Sealing ring
7. pore water outlet
8. Additional pore water outlet.
9. cell fluid inlet.
10. Soil specimen (enclosed in rubber membrane with 'O' rings at the ends)
11. porous discs

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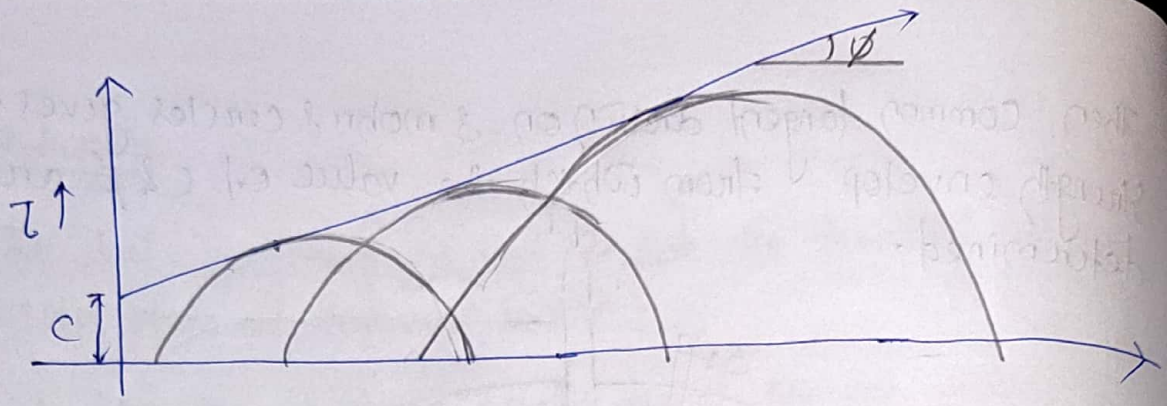

Advantages of triaxial compression test:-

→ The specimen is free to fail along the weakest plane.
→ The shear stress is uniform throughout the soil specimen.
→ The stress distribution on the failure plane is uniform.
→ There is complete control on drainage condition.
→ Measurement of pore pressure and volume change is possible.

Types of shear tests based on Drainage condition:-

(i) unconsolidated undrained test or simply undrained test (UU)
(ii) consolidated undrained test (CU)
(iii) Consolidated Drained test or simply drained test - (CD)

(i) Undrained test:-

→ Drainage is not permitted throughout the test.
→ In case of triaxial compression test drainage is not permitted during the application of both cell pressure and deviator stress.
→ This test is also called quick test.

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(ii) Consolidated undrained test:-

→ In this type of shear test the soil specimen is allowed to consolidate fully under initially applied stress and then sheared quickly without allowing dissipation of pore pressure.

→ In case of triaxial compression test the specimen is allowed to consolidate fully under applied cell pressure and then the pore water outlet is closed and the specimen subjected to increasing deviator stress at high rate of strain.

(iii) Drained test:-

→ In this type of shear test drainage is allowed throughout the test.

→ The specimen is allowed to consolidate fully under the applied initial stress and then sheared at low rate of strain giving sufficient time for the pore water to drain out at all stage.

→ The test may continue for several hours to several days.

Unconfined Compression test:-

→ unconfined compression test can be regarded as a special case of triaxial compression test in which no lateral pressure or confining pressure is applied, so $\sigma_2 = \sigma_3 = 0$.

→ The soil specimen is cylindrical in shape with length about 2 to 2.5 times its diameter.

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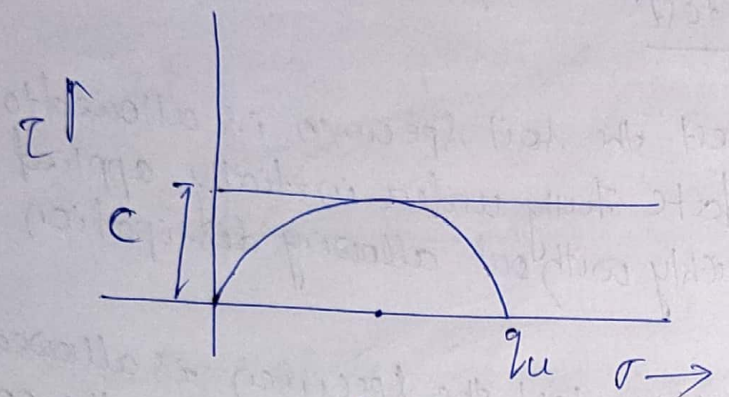

→ The max^m axial stress a soil can carry safely without shear failure is called as unconfined compressive strength.

→ It is denoted by q_u and is given by $\boxed{q_u = \frac{F}{A_c}}$

Where A_c = Corrected area of crossection.

F = unconfied axial load.

→ It is generally used to findout the strength of purly cohessive soil.

→ Here the axial stress on the soil specimen is increased till the specimen fails.

→ The axial stress, corresponding to failure of soil is noted which is equal to $\sigma_d = q_u$.

→ Then a mohrs circle is drawn by using σ_d.

→ The tangent to the mohrs circle gives the strength envelop.

→ From the strength envelop the value of c & ϕ are obtained.

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Vane Shear Test:

→ Vane shear test is a quick test used to determine undrained shear strength of cohesive soil.

→ The equipment essentially consist of four high tensile steel plates called vanes which are welded to the bottom end of a steel rod called the torque rod with an arrangement to measure the torque and rotation.

→ A typical arrangement consists of a calibrated torsion spring attached to the top of torque rod which is rotated by a combination of worm gear and worm wheel.

→ The vane shear test can be conducted both in the laboratory and in field.

→ A typical laboratory set of vanes has 20mm height, diameter of 12mm across vanes with blade thickness of 0.5 to 1mm.

→ The field set of vanes usually is 100 to 200 mm in height, 50 to 100mm in diameter across vanes with blade thickness of 2.5mm.

→ To conduct the test the vanes are gently pushed into the soil and the torque rod is rotated at a uniform rate of usually 1° per min.

→ The torque T corresponding to angle of rotation θ at uniform interval are noted.

→ Torque T is plotted as ordinate (Y-axis) against angle of rotation θ as abscissa (X-axis).

→ The torque T_f at failure is found and is used to calculated the shear strength τ_f :-

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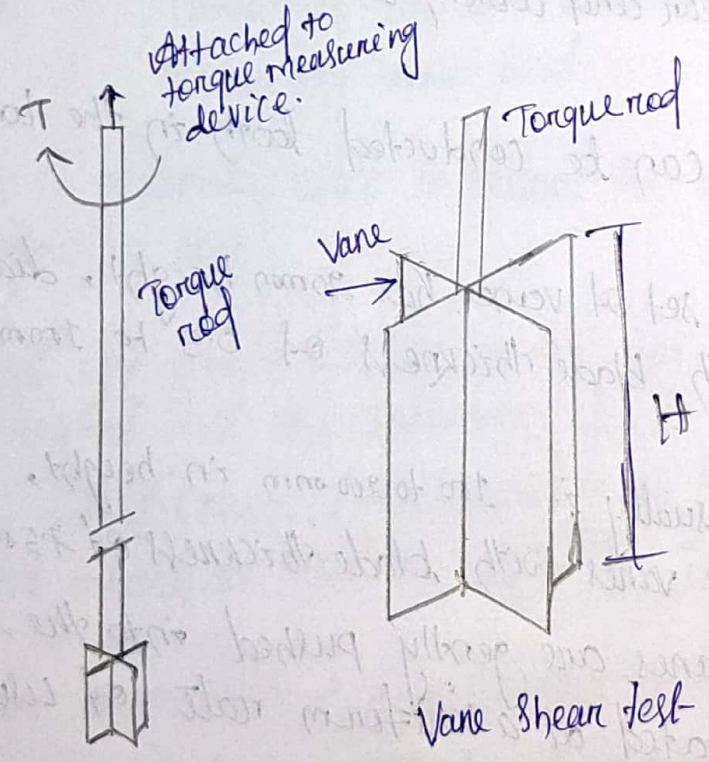
(i) $T_f = \pi d^2 \tau_f \left[\frac{H}{2} + \frac{d}{6}\right]$

When the vane is pushed with top end below the surface of soil so that both top and bottom ends takepart in shearing.

(ii) $T_f = \pi d^2 \tau_f \left[\frac{H}{2} + \frac{d}{12}\right]$

When the vane is pushed inside the soil with its top end flush with surface of soil so that only bottom end takepart in shearing.



Vane shear test

~~Lateral Pressure~~

Earth pressure on retaining structures

Introduction :-

→ Soil mass is stable when the slope of the surface of the soil mass is flatter than the safe slope. But at some places the space is limited, it is not possible to provide that slope and the soil is to be retained at a slope steeper than the safe one.

→ Therefore to retain this soil mass in a stable state a retaining structure is provided to provide the lateral support to the soil mass.

→ In the design of these retaining structure it becomes imperative to know the magnitude and line of action of earth pressure, where the earth pressure is the lateral force exerted by the soil on any structure retaining that soil.

8.1 Retaining wall

→ A structure which is used to hold back a soil mass is called a retaining structure.

Types of lateral earth pressure :-

→ Lateral earth pressure can be devided into 3 categories, depending upon the movement of retaining wall with respect to backfill soil.

(1) Earth pressure At Rest :- Wall does not move at all.
(2) Active earth pressure - Wall moves away from the backfill soil.
(3) Passive Earth pressure - Wall moves towards the backfill soil.

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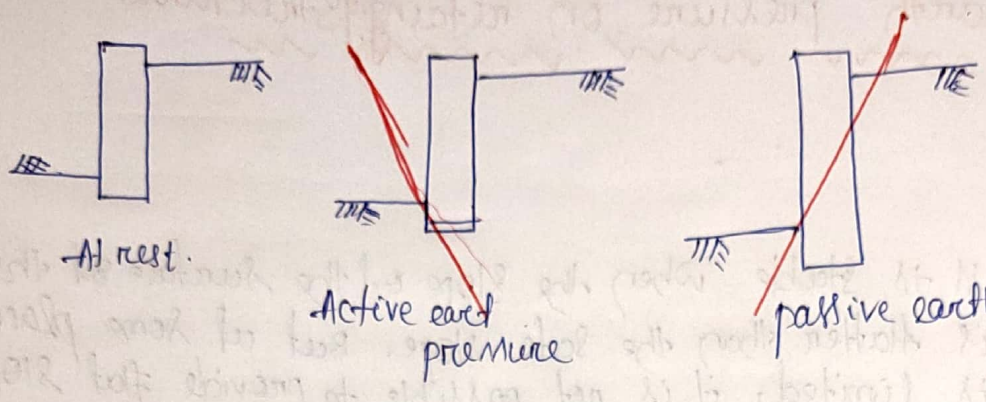

(1) Earth pressure at Rest :-

→ A soil element in its natural state at any depth 'z' below the ground surface is not subjected to any strain, the element in this condition is known as at rest condition.

→ **Earth pressure at rest (P_o) = $K_o \sigma_z$**

K_o = co-efficient of earth pressure at rest.

σ_z = Total stress or effective stress at the point on soil mass where we need to calculate earth pressure.

$K_o = \frac{\mu}{1-\mu}$, μ - poisson's ratio.

$= 1 - \sin\phi$, ϕ - angle of internal friction.

(2) Active earth pressure :-

→ It is the pressure exerted by the backfill to the retaining wall.

→ Due to this pressure the retaining wall tends to move outward.

→ The intensity of active earth pressure :-

$P_a = K_a \gamma H$

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Where, K_a = co-efficient of active earth pressure.
γ = unit wt. of back fill.
H = Height of retaining wall.

$$K_a = \frac{1-\sin\phi}{1+\sin\phi}$$

(3) Passive earth pressure :-

→ It is the pressure exerted by the retaining wall to the backfill.

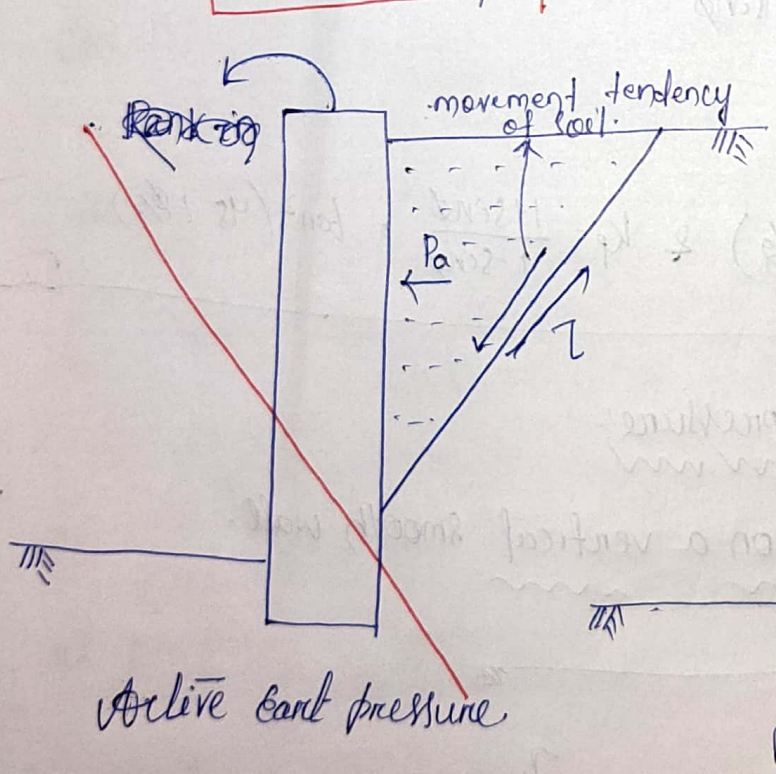
→ This pressure very rarely comes into play.

→ The intensity of passive earth pressure :-

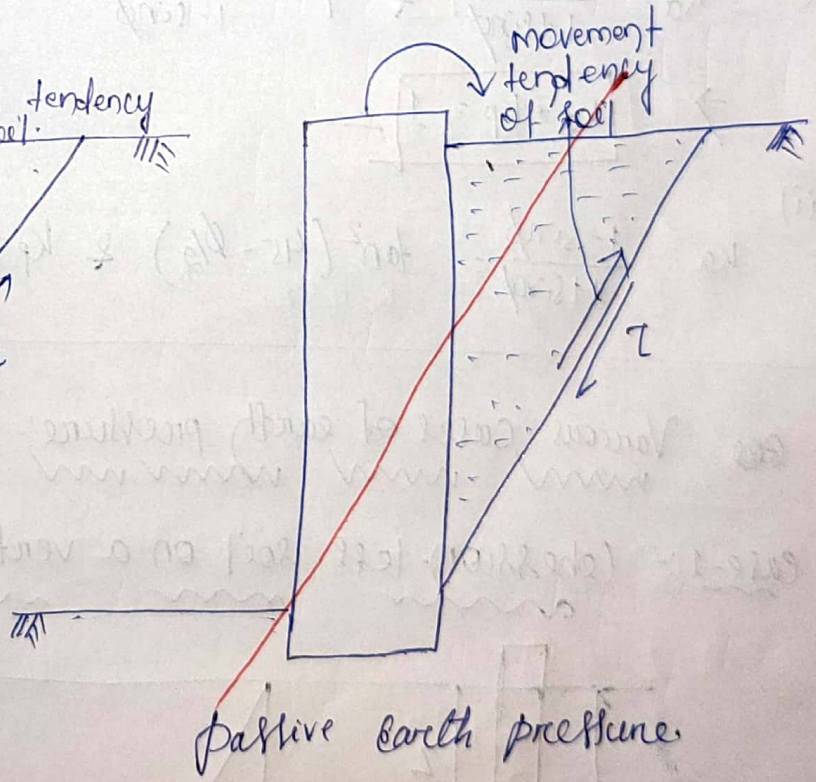
$$P_p = K_p \gamma \cdot H$$

Where K_p = co-efficient of passive earth pressure.

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



Active earth pressure



Passive earth pressure.

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Rankine's theory of earth pressure:-

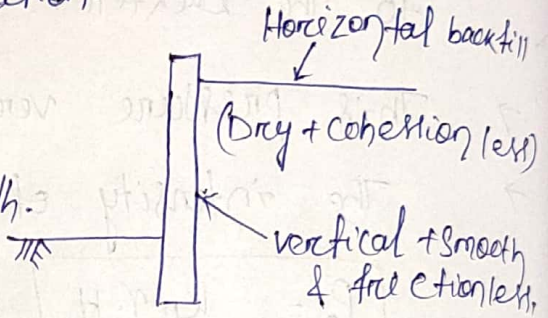
Assumptions:-

(i) Soil is semi-infinite, homogenous, isotropic, dry & cohesionless.

(ii) Soil is in the state of plastic condition at the time of active and passive pressure generation.

(iii) The backfill soil is horizontal.

(iv) Back of the wall is vertical and smooth.



(v)

Note

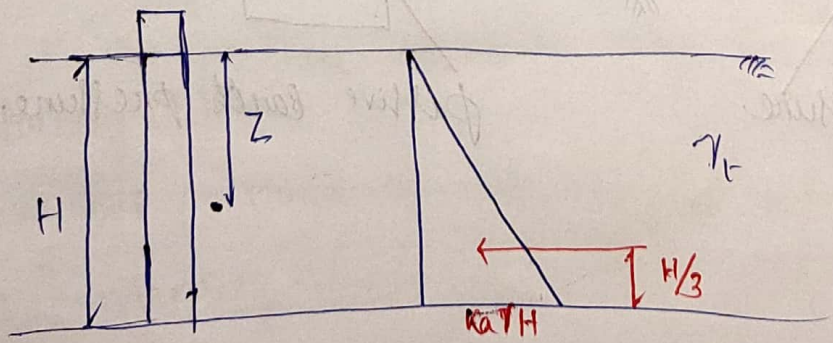
(i) $K_a = \frac{1-\sin\phi}{1+\sin\phi}$ & $K_p = \frac{1+\sin\phi}{1-\sin\phi}$

$\Rightarrow$ $\boxed{K_a * K_p = 1}$

(ii) $K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45-\phi/2)$ & $K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\phi/2)$

(vi) Various cases of earth pressure:-

Case-1:- Cohesionless soil on a vertical smooth wall.



at any depth Z = $P_a = K_a \gamma_t * Z$

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at $Z=0$, at $Z=H$

$P_a = 0$ $\qquad$ $P_a = K_a \gamma H$

Now; force or active thrust or total pressure due to active earth pressure per unit length of wall (F_a) = area of pressure prism.

i.e $F_a = \frac{1}{2} * K_a \gamma H * H$ (acting at $H/3$ from base)

$$\boxed{F_a = \frac{1}{2} K_a \gamma H^2}$$

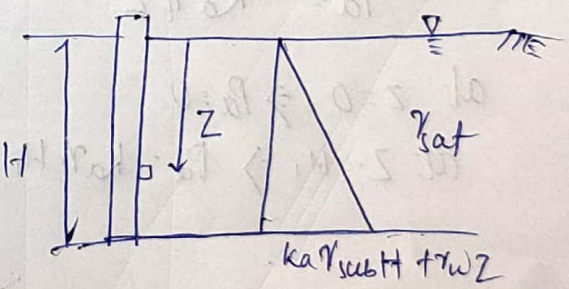
Similarly $F_p = \frac{1}{2} K_p \gamma H^2$, where F_p = total passive thrust

$F_o = \frac{1}{2} * K_o \gamma H^2$, where F_o = total pressure at rest.

Case-II :- Submerged cohesionless soil on vertical smooth wall :-

at any depth 'Z'

$P_a = K_a [\gamma_{sat} Z - \gamma_w Z] + \gamma_w Z$ ($\bar{\sigma}$ = total stress - pore water)



$\therefore P_a = K_a \gamma_{sub} Z + \gamma_w Z$

at $Z=0$

$\therefore P_a = 0$

at $Z = H$

$\therefore P_a = K_a \gamma_{sub} H + \gamma_w Z$

→ force or active thrust or total pressure due to active earth press. = $\frac{1}{2} * (K_a \gamma_{sub} H * H + \gamma_w H) * H$

$= \frac{1}{2} (K_a \gamma_{sub} H^2 + \gamma_w H^2)$ (acting at $H/3$ from base)

$\therefore \boxed{F_a = \frac{1}{2} (K_a \gamma_{sub} H^2 + \gamma_w H^2)}$

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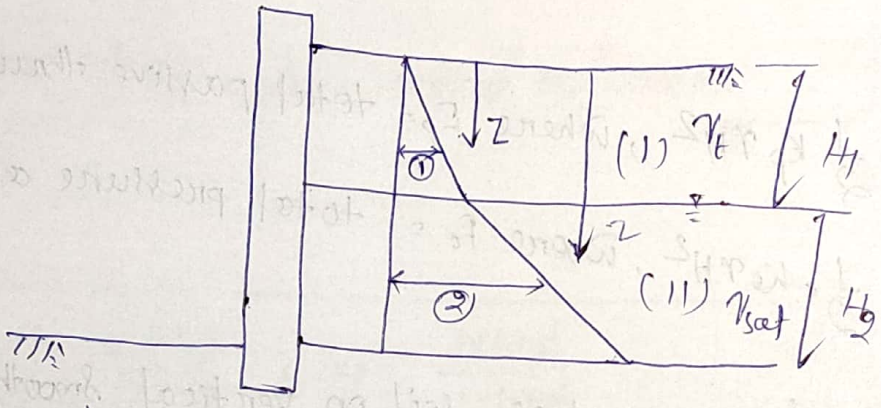
Similarly :-

$$P_p = K_p \gamma_{sub} H + \gamma_w H$$

$$F_p = \frac{1}{2}\left(K_p \gamma_{sub} H^2 + \gamma_w H^2\right)$$

Case-III :- Partialy Submerged cohession less soil on vertical smooth wall :-



Zone-I

$$P_a = K_a \gamma_t Z$$

at $Z = 0 \Rightarrow P_a = 0$

at $Z = H_1 \Rightarrow P_a = K_a \gamma_t H_1$

Zone-II

$$P_a = K_a \gamma_t H_1 + K_a \gamma_{sub}(Z - H_1) + \gamma_w (Z - H_1)$$

at $Z = H_1$

$$P_a = K_a \gamma_t H_1$$

at $Z = H_1 + H_2$

$$P_a = K_a \gamma_t H_1 + K_a \gamma_{sub} H_2 + \gamma_w H_2$$

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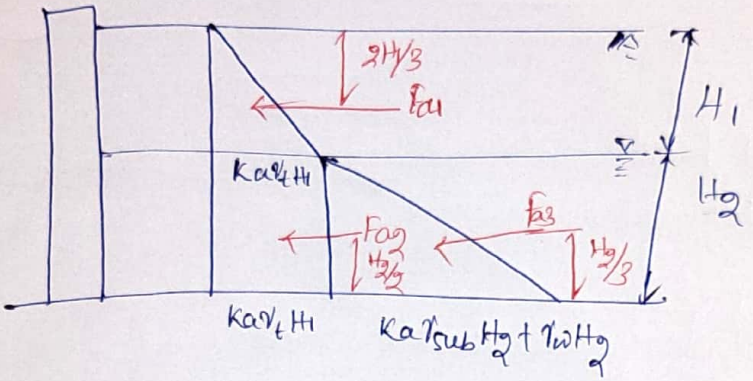

$F_a = F_{a1} + F_{a2} + F_{a3}$ —

∴ point of application of Resultant force from bottom of base $= \dfrac{F_{a1}\left(H_2 + H_2 - \frac{2H_1}{3}\right) + F_{a2}\left(\frac{H_2}{2}\right) + F_{a3}\left(\frac{H_2}{3}\right)}{F_{a1} + F_{a2} + F_{a3}}$

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(viii) In fig(d) an additional pressure of 2 unit acts and the valve is closed, so no drainage will takes place. So pressure of 1 unit is borne by soil and pressure of 2 unit is borne by water.

(ix) In fig(e) the valve is partly open so there will be some drainage of water from the soil. So the pressure on soil $= 1 + \Delta\sigma$

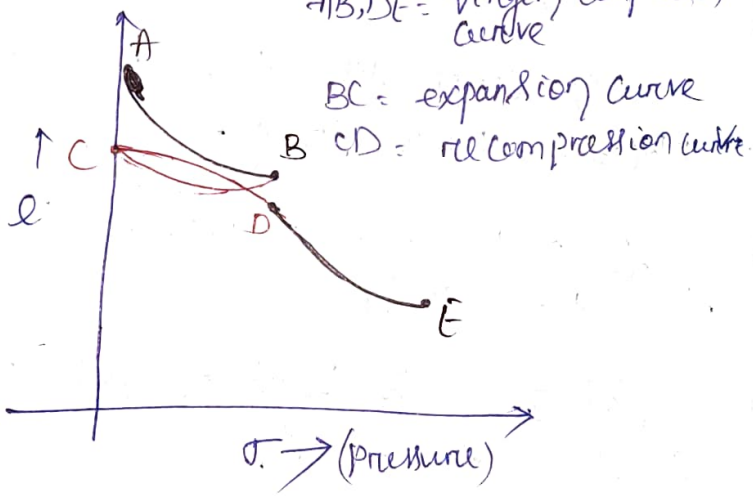
and pressure on water $= 2 - \Delta\sigma$.

(x) In fig(f) the valve is opened so, there will be completely drainage of water from soil through the valve.
Hence pressure on soil $= 1$ 2.
& pressure on water $= 0$

Field complications:-

pressure void ratio Curve or Consolidation Curve:-

→ The fig. shows the relationship bet^n void ratio & pressure of a remoulded soil specimen.



→ The Curve AB is obtained by increasing the effective pressure on soil due to which the void ratio of soil decreases considerable.

→ At point B, the pressure is removed. So, in both point BRC the pressure of soil decreases and void ratio of soil increases; however the soil will not attened the original void ratio.

→ In both C & D the pressure increases and the void ratio decreases slowly.

→ The pressure at point D is nearly equal to pressure at point B.

→ Again, after point D the void ratio considerable decreases with an increase of pressure.

→ Here the curve AB and DE are called as Virgin Compression curve and BC called as expansion curve and CD as recompression curve.

Terzaghi's assumption for 1D consolidation:-

(i) The soil is homogeneous and fully saturated.

(ii) The soil particles and water both are incompressible.

(iii) Darcy's law for flow of water will be applicable during consolidation.

(iv) Co-efficient of permeability is constant during consolidati

(v) The load is applied in one direction only and deformation occures in the direction of load only.

(vi) The deformation is due to entirely decrease in vol.

(vii) The drainage of pore water occures only in one direction.

Important formulae:-

(1) $e = e_0 - C_c \log\left(\frac{\sigma'}{\sigma_0'}\right)$

(2) $e = e_0 - C_s \log\left(\frac{\sigma'}{\sigma_0'}\right)$

(3) $T_v = \frac{C_v t}{d^2}$

(4) $C_v = \frac{K}{m_v \gamma_w}$

(5) $T_v = \frac{\pi}{4}(U^2)$, if $U \leq 60\%$ U in decimal

(6) $T_v = 1.781 - 0.933 \log(100 - U)$ if $U > 60\%$ U in % age

(7) for double drainage, $d = \frac{H}{2}$

(8) for single drainage, $d = H$

(9) $U = \frac{\Delta h}{\Delta H} \times 100$

σ' = final effective stress

σ_0' = initial effective stress.

T_v = Time factor

C_v = Co-efficient of consolidation

t = time interval

d = depth of drainage

U = degree of consolidation

H = thickness of soil layer.

m_v = co-efficient of vol. compressibility.

e_0 = initial void ratio

e = final void ratio.

C_c = compression index

C_s = swelling index.

Δh = settelment of any stage or time rate settelment.

ΔH = settelment at the end of consolidation.

REFERENCES BOOKS:

SLNO	NAME OF AUTHOR	TITLE OF BOOK	NAME OF PUBLISHER
1	Dr.B.C.Purnimia	Soil Mechanic & Foundation Engineering	laxmi Publication
2	Dr.K.R.Arora	Soil Mechanics & Foundation Engineering	Standard Publisher Distributors

HYDRAULIC & IRRIGATION ENGINEERING

SPINTRONIC TECHNOLOGY & ADVANCE RESEARCH

DEPARTMENT OF CIVIL ENGINEERING

4TH SEMESTER

DIPLOMA

LECTURE NOTE

HYDRAULIC & IRRIGATION ENGINEERING

BY

SMRUTI SASWATI DASH

Th2. HYDRAULICS & IRRIGATION ENGINEERING

Name of the Course: Diploma in Civil Engineering			
Course code:		Semester	4th
Total Period:	75	Examination	3 hrs
Theory periods:	5P/week	Class Test:	20
Maximum marks:	100	End Semester Examination:	80

A. RATIONALE

The course will be imparted in two parts. Primarily it aims to explain students the need of irrigation and components of the irrigation system which is covered in the second part of the course. The course aims to explain students the intricacies of irrigation engineering with reference to basic sciences relating to fluid mechanics and hydraulic machines. The essential components of fluid mechanics and hydraulic machines will be addressed in the first part of thecourse.

B. COURSE OBJECTIVES

On completion of the course students will be able to -

1. Define common fluid properties and interpret results from pressure measuringinstruments.
2. Realize the science behind fluid flow and compute fluid flow characteristics through notches, weirs, channels and pipes.
3. Realize the working principle of hydraulic pumps and evaluate their performance ingeneral cases.
4. Comprehend the need of irrigation
5. Determine cause and effect of water logging
6. Comprehend the purpose of irrigation system components and elaborate on these

C. TOPIC WISE DISTRIBUTION OF PERIODS

Chapter	Name Of Topics	Periods
PART: A (Hydraulics And Machines)		
1	Hydrostatics	12
2	Kinematics Of Fluid Flow	18
3	Pumps	05
Part: B (Irrigation Engineering)		
1	Hydrology	04
2	Water Requirement Of Crops	04
3	Flow Irrigation	07
4	Water Logging And Drainage :	02
5	Diversion Head Works And Regulatory Structures	08
6	Cross Drainage Works :	07
7	Dams	08

D. COURSE CONTENTS:

PART: A (Hydraulics)

1 HYDROSTATICS:

Properties of fluid: density, specific gravity, surface tension, capillarity, viscosity and their uses

Pressure and its measurements: intensity of pressure, atmospheric pressure, gauge pressure, absolute pressure and vacuum pressure; relationship between atmospheric pressure, absolute pressure and gauge pressure; pressurehead; pressure gauges.

Pressure exerted on an immersed surface: Total pressure, resultant pressure, expression for total pressure exerted on horizontal & vertical surface.

2 KINEMATICS OF FLUID FLOW:

Basic equation of fluid flow and their application: Rate of discharge, equation of continuity of liquid flow, total energy of a liquid in motion- potential, kinetic & pressure, Bernoulli's theorem and its limitations. Practical applications of Bernoulli's equation.

Flow over Notches and Weirs: Notches, Weirs, types of notches and weirs, Discharge through different types of notches and weirs-their application (No Derivation)

Types of flow through the pipes: uniform and non uniform; laminar and turbulent; steady and unsteady; Reynold's number and its application

Losses of head of a liquid flowing through pipes: Different types of majorand minor losses. Simple numerical problems on losses due to friction using Darcy's equation, Total energy lines & hydraulic gradient lines (Concept Only).

Flow through the Open Channels: Types of channel sections-rectangular,trapezoidal and circular, discharge formulae- Chezy's and Manning's equation, Best economical section.

3 PUMPS:

Type of pumps

Centrifugal pump: basic principles, operation, discharge, horse power & efficiency.

Reciprocating pumps: types, operation, discharge, horse power & efficiency

PART: B (Irrigation Engineering)

1 Hydrology

Hydrology Cycle

Rainfall: types, intensity, hyetograph

Estimation of rainfall, rain gauges, Its types(concept only),

Concept of catchment area, types, run-off, estimation of flood discharge by Dicken's and Ryve's formulae

2 Water Requirement of Crops

Definition of irrigation, necessity, benefits of irrigation, types of irrigation

Crop season

Duty, Delta and base period their relationship, overlap allowance, kharifand rabi crops Gross command area, culturable command area, Intensity of Irrigation,irrigable area, time factor, crop ratio

3 FLOW IRRIGATION

Canal irrigation, types of canals, loss of water in canals
Perennial irrigation
Different components of irrigation canals and their functions
Sketches of different canal cross-sections
Classification of canals according to their alignment, Various types of canallining
Advantages and disadvantages

4 WATER LOGGING AND DRAINAGE :

Causes and effects of water logging, detection, prevention and remedies

5 DIVERSION HEAD WORKS AND REGULATORY STRUCTURES

Necessity and objectives of diversion head works, weirs and barrages
General layout, functions of different parts of barrage
Silting and scouring
Functions of regulatory structures

6 CROSS DRAINAGE WORKS :

Functions and necessity of Cross drainage works - aqueduct, siphon, super-passage, level crossing
Concept of each with help of neat sketch

7 DAMS

Necessity of storage reservoirs, types of dams
Earthen dams: types, description, causes of failure and protection measures.
Gravity dam- types, description, Causes of failure and protection measures.
Spillways- Types (With Sketch) and necessity.

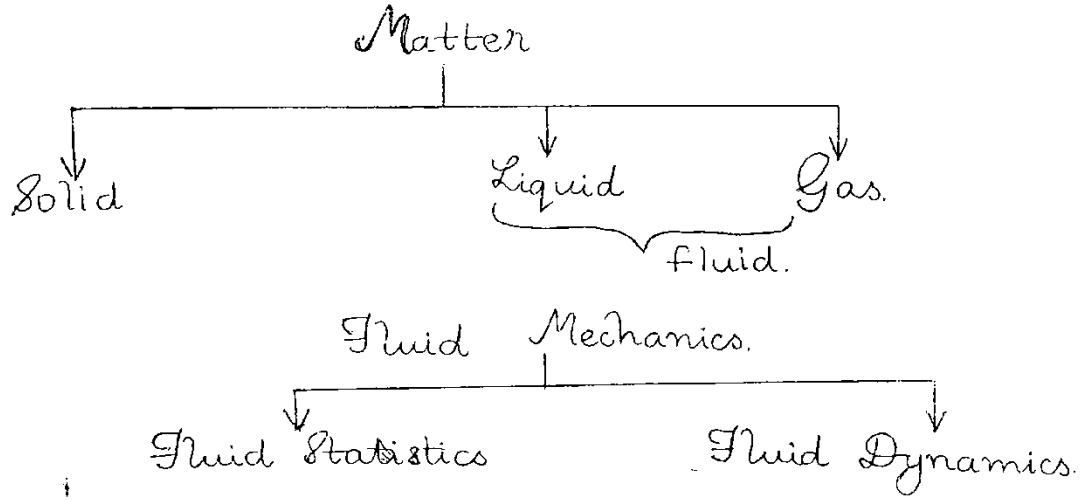
E. Syllabus Coverage up to Internal Assessment: Part A: Chapters 1, 2 & Part B: 1, 2

F. RECOMMENDED BOOKS

Sl. No	Name of Authors	Titles of Book	Name of Publisher
1	Modi & Seth	Fluid Mechanics & Hydraulic machines	Standard Book House
2	D.R. Biswal	Hydraulics & Fluid Mechanics	Kalyani Pbln
3	R.K.Rajput	A Text Book of Fluid Mechanics & Hydraulic machines	S.Chand

Sl. No	Name of Authors	Titles of Book	Name of Publisher
1	S.K.Garg	Irrigation Engineering & Hydraulics Structures	Khanna Publishers
2	N. N. Basak	Irrigation Engineering	TMH Publishing
3	S.K. Sharma	Irrigation Engineering & Hydraulic structures.	S. Chand Pbln

FLUID MECHANICS (8 marks)



A matter can exist either in Solid, Liquid or in Gaseous state. The liquid and the gaseous state, together it is called Fluid State. The solid state & the fluid state together exhibit different characteristics because of differences in the arrangement of molecular structure.

In a given volume, a solid comprises large number of molecules and the attraction b/w the molecules (cohesive forces) is very large ∴ a solid posses a rigid and compact shape. and it has a definite volume.

In the same given volume, a liquid contains relatively less no. of molecules and the cohesive forces b/w the molecules is small and therefore in the liquid mass, the molecules will be in motion. and a liquid occupies the shape of the container in which it is contained.

For the same given volume, a gas contains much less no. of molecules and the attractive forces b/w the molecules is very small and ∴ the gas molecules are in continuous random motion and it occupies the total vol. of the container in which it is pla

$$\rho = \frac{m}{V} \qquad (kg/m^3)$$

Dimensional Formula $= ML^{-3}$

ρ_{H_2O} at $4^\circ C = 1000\ kg/m^3$

For the gas,

$P = \rho RT$ $\qquad$ $pV = mRT$.

$P \uparrow \quad \rho \uparrow$ $\qquad$ $P = \left(\frac{m}{V}\right)RT = \rho RT$.

$T \uparrow \quad \rho \downarrow$

For the liquids, no change of density wrt pressure and temp.

Any gas moving with mach number, $M < 0.3$, it is considered as an incompressible gas. Because for the incompressible, $d\rho = 0$

2. Specific Weight (Weight Density), w

$$w = \frac{\text{weight}}{\text{volume}} = \frac{W}{V} \qquad (N/m^3$$

$$w = \frac{W}{V} = \frac{mg}{V} = \left(\frac{m}{V}\right)g$$

$$\Rightarrow \boxed{w = \rho g}$$

Dimensional Formula $= ML^{-2}T^{-2}$.

Since weight density depends on 'g', hence it is not called an absolute quantity.

3. Specific Volume (v)

$$v = \frac{\text{Volume}}{\text{Mass}} = \frac{V}{m} \qquad (m^3/kg).$$

Specific volume term is always given to the gases.

Since for a given mass of the gas, it may occupy different volumes.

4. Specific Gravity, S

$$S_{fluid} = \frac{\text{Density of fluid}}{\text{Density of std. fluid.}} \times \frac{g}{g}$$

$$= \frac{\text{wt. density of fluid}}{\text{wt. density of std. fluid.}}$$

The standard fluid:- for liquids → water
for gases → air.

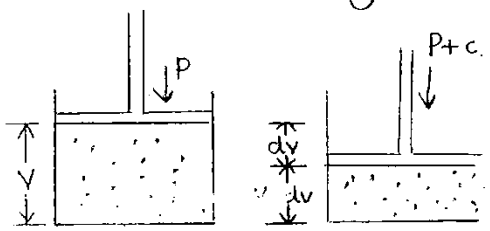
$S_{H_2O} = 1$; $S_{Hg} = 13.6$.

NOTE:

The term Relative Density is also sometimes used when the comparison is not made wrt to the standard fluid.

5. Bulk modulus of Elasticity (k) and Compressibility (β)

$$k = \frac{dP}{\left(\frac{-dv}{v}\right)} = \frac{P_2 - P_1}{-\left(\frac{V_2 - V_1}{V_1}\right)}$$



$V_1 = V$ $\quad$ $V_2 = V - dv$

$P_1 = P$ $\quad$ $P_2 = P + dP$

It is defined as the amount of incremental pressure (dP) required for the volume of gas, V decreases by an amount dv.

$$k = \frac{\text{compressive stress}}{\text{volumetric strain.}} = \frac{dP}{\left(\frac{-dv}{dv}\right)}$$

The -ve sign appears in the above eqn as with increase of pressure, volume decreases

* Compressibility (β)

It is defined as the property by virtue of which flu undergo a change of volume under the action of external pressure forces.

$$\therefore \beta = \frac{-\left(\frac{dv}{dv}\right)}{dp} = \frac{1}{k}$$

$$\rho = \frac{m}{V}$$

$$\rho \times V = m.$$

Differentiating on both sides,

$$\rho dv + V d\rho = 0. \qquad \text{(mass, m is constant)}.$$

$$-\frac{dv}{V} = \frac{\partial \rho}{\rho}$$

$$\therefore k = -\frac{dp}{\left(\frac{dv}{V}\right)} = \frac{dp}{\frac{d\rho}{\rho}}$$

$$\Rightarrow \boxed{k = \frac{\rho dp}{d\rho}}$$

For constant pressure process, $dp = 0$

$$\therefore k = 0.$$

For constant volume process, $dv = 0$

$$\therefore k = \infty$$

Isothermal Bulk modulus of Elasticity,

$$\frac{PV}{T} \ldots = C \qquad \text{(Characteristic Gas Equation)}$$

For isothermal process, T = constant.

$$\therefore PV = C.$$

$\therefore$ differentiating on both sides,

$$P dv + V dp = 0.$$

$$\frac{-dv}{V} = \frac{dp}{P}$$

$$k = \frac{dp}{-\left(\frac{dv}{V}\right)} = P \Rightarrow \boxed{k_{iso} = P}$$

Adiabatic bulk modulus of Elasticity.

$$\frac{PV}{T} = C.$$

$$PV^{\gamma} = C \quad \text{(adiabatic process)}$$

$$P\gamma V^{\gamma-1} dv + V^{\gamma} dp = 0.$$

$$\frac{\gamma P V^{\gamma-1}}{V} dv = -V^{\gamma} dp.$$

$$\gamma P \frac{dv}{V} = -dp.$$

$$\therefore \quad -\frac{dv}{V} = \frac{dp}{P} \times \frac{1}{\gamma}.$$

$$k = \frac{dP}{\frac{dP}{P} \times \frac{1}{\gamma}} = \gamma p.$$

$$k_{adi} = \gamma p. = \gamma \times k_{iso}$$

Units $= N/m^2$.

$\gamma \rightarrow$ adiabatic index.

$$\gamma = \frac{C_p}{C_v}$$

$C_p \rightarrow$ spec. heat at const. pr

$C_v \rightarrow$ sp. heat at const. volum

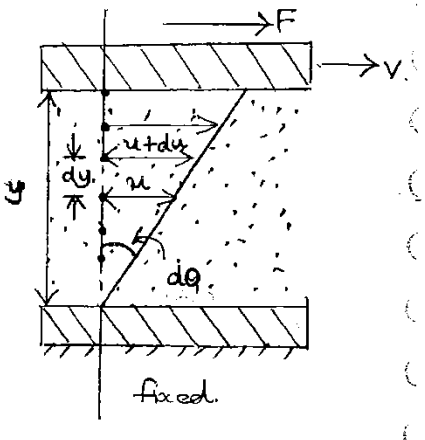
7th Aug,
THURSDAY

6. Viscosity.

It is the internal resistance offered by the one fluid layer for the movement of adjacent fluid layer. It is due to the inter molecular ~~movements~~ of exchange and the cohesive forces blw the molecu

$$V = \frac{d}{t}$$

$$d = V \times t.$$



For the fluid particle which is on the top plate, travelled a distance $= V \times dt$.

$$\tan d\theta = \frac{V \times dt}{y}.$$

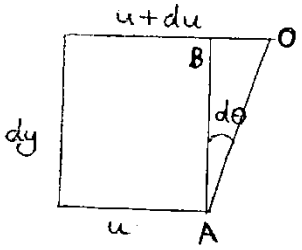
For small angles, $\tan d\theta \approx d\theta$.

$$d\theta = \frac{V \times dt}{y} \Rightarrow \boxed{\frac{d\theta}{dt} = \frac{V}{y}} \longrightarrow (1)$$

The term $\frac{d\theta}{dt}$ is called 'Rate of Angular deformation'

From ΔBAD,

$$\tan d\theta = \frac{du \cdot dt}{dy}$$



For small angles, $\tan d\theta \approx d\theta$

$$\therefore d\theta = \frac{du \cdot dt}{dy} \Rightarrow \frac{d\theta}{dt} = \frac{du}{dy} \longrightarrow (2)$$

$\frac{d\theta}{dt}$ → rate of shear strain (or) rate of angular deformation

As shear force, F ↑, rate of angular deformation, $\frac{d\theta}{dt}$ ↑

$$F \propto \frac{d\theta}{dt}$$

$$\tau = \frac{F}{A} \Rightarrow \tau \propto F \quad (A \rightarrow \text{area of plates is const.})$$

$$\therefore \tau \propto \frac{d\theta}{dt} \Rightarrow \boxed{\tau = \mu \frac{d\theta}{dt}} \longrightarrow (3)$$

where μ is the proportionality constant and is known as 'Coeffecient of Dynamic Viscosity'

$$\mu = \frac{\tau}{\frac{d\theta}{dt}}$$

If $\frac{d\theta}{dt} = 1$, then $\mu = \tau$.

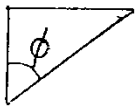
∴ coeffecient of dynamic viscosity is defined as the amount of shear stress required to produce unit rate of angular deformation or rate of shear strain.

Q

F, dt, A	F, dt, B
Fluid A	Fluid B

Which fluid has more viscosity?

$\frac{d\theta}{dt} < \frac{d\phi}{dt}$



$\therefore \mu_B < \mu_A$

(Since $\mu \propto \frac{1}{d\theta/dt}$)

From eqns ①, ② & ③,

$$\tau = \mu \frac{V}{y} = \mu \frac{du}{dy}$$

$$\boxed{\tau = \mu \frac{du}{dy}} \Rightarrow \boxed{\text{Newton's Law of Viscosity}}$$

The term $\frac{du}{dy}$ has units of $\frac{m/s}{m} = s^{-1}$

The term $\frac{du}{dy}$ is called 'Velocity Gradient'

* If $\mu = 0$, then $\tau = 0$, the fluid is called an 'Ideal Fl

The fluid which satisfies Newton's Law of Viscosity is called 'Newtonian Fluid'.

∴ for Newtonian Fluid, $\tau = \mu\left(\frac{du}{dy}\right)$

Eg: Petrol, diesel, water etc.

The fluids which does not obey Newton's Law of viscosity is called Non-newtonian Fluid.

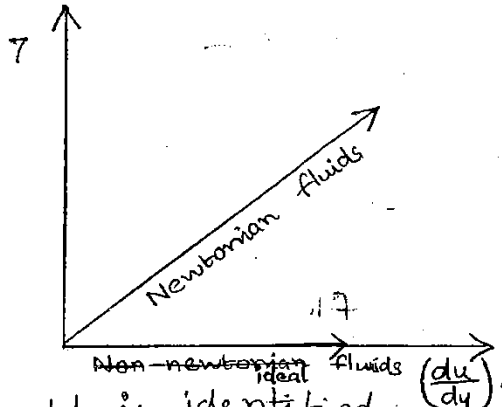
The study of Non-Newtonian fluids is called "Rheology"

Eg: Blood, milk, clay etc.

The relationship between shear stress and velocity gradient for Non-Newtonian fluids is:

$$\tau = A\left(\frac{\partial u}{dy}\right)^n + B.$$

where A → flow consistency index,
B → flow behaviour index,
n → index.



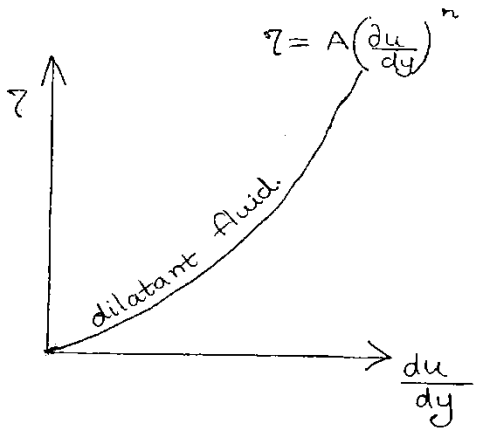
If $n = 1$, $A = \mu$, $B = 0$ ⇒ then the model is identified as Newtonian model.

Classification of Non-Newtonian Fluids:

1. Dilatant Fluid. ($n>1$, $B=0$)
(Shear thickening fluids)

$$\tau = A\left(\frac{du}{dy}\right)^n$$

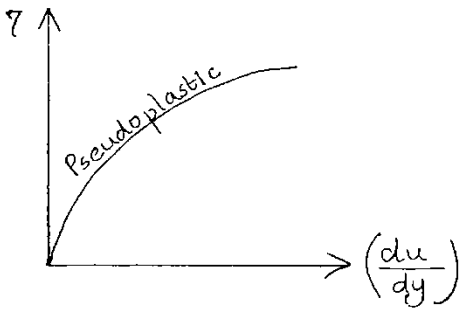
$$= \underbrace{A\left(\frac{du}{dy}\right)^{n-1}}_{\text{Apparent Viscosity } (\mu_{app})}\left(\frac{du}{dy}\right)$$



Eg: Milk

2. Pseudo plastic Fluid. ($n<1$, $B=0$)
(Shear thinning fluids)

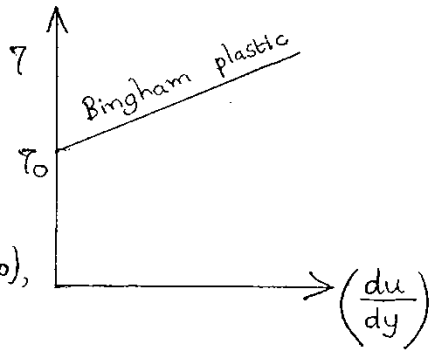
Eg: Blood.



3. Bingham Plastic Fluid ($n=1$, $B\neq 0$)

$$\tau = A\left(\frac{du}{dy}\right) + B$$

Eg: Toothpaste.



For these fluids, certain min. yield stress (τ_0), is required for the deformation.

4. Thixotropic Fluids.

For certain non-Newtonian fluids, the viscosity depends on time They are called 'Thixotropic' & 'Rheopectic' fluids.

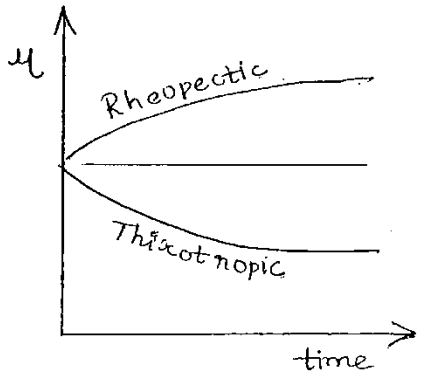
For thixotropic fluids, μ decreases with time.

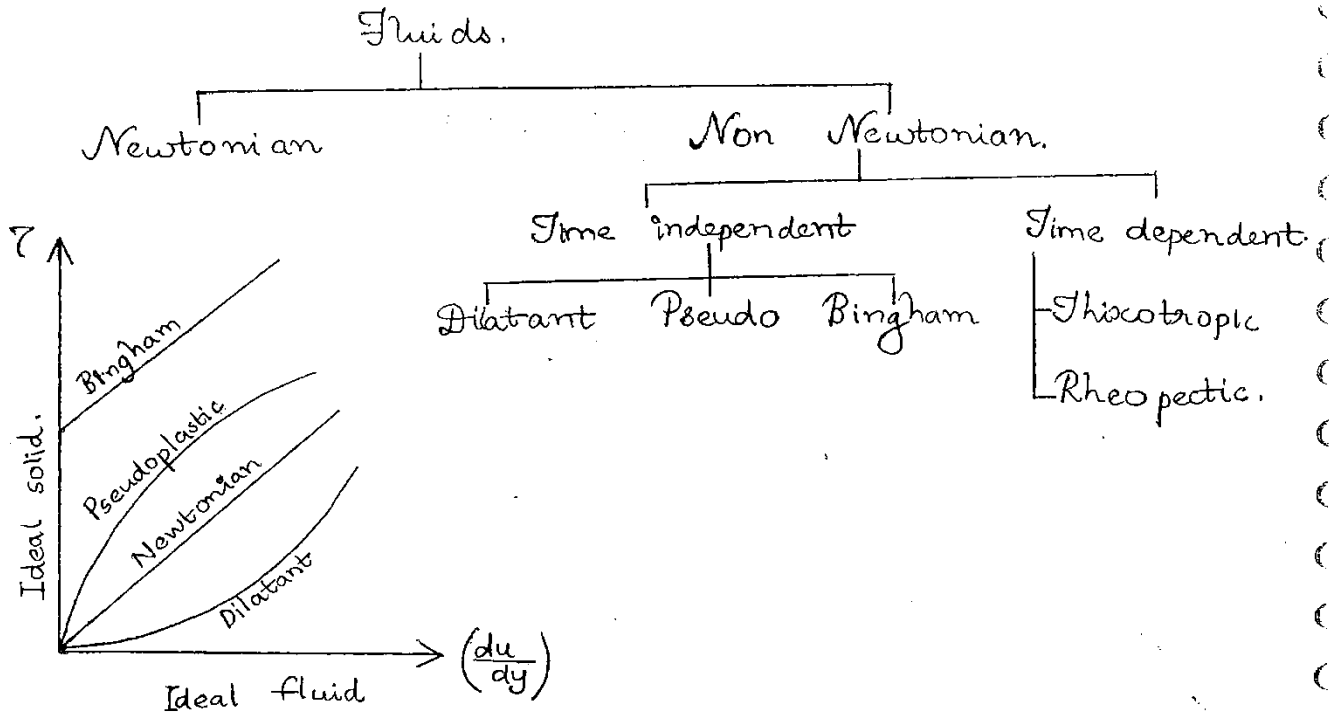
Eg: Lipstick.

5. Rheopectic Fluids

μ increases with time.

Eg: Gypsum solution in water





Units of μ:

$$\tau = \mu \left(\frac{du}{dy}\right)$$

$$\mu = \frac{\tau}{\frac{du}{dy}} = \frac{N/m^2}{s^{-1}} = \frac{Ns}{m^2} = Pa-s.$$

$$1\ Pa\text{-}s = \frac{N}{m^2} \times s = \frac{kg\ m s^{-2} \times s}{m^2} = \frac{kg}{m \times s} \quad (SI).$$

$$= \frac{g}{cm \times s}\ (c.g.s) \rightarrow \text{poise}$$

$$1\ \frac{kg}{ms} = \frac{1000\ g}{100\ cm \times s}$$

$$= \frac{10\ g}{cm \times s}$$

$$\therefore \boxed{1\ \frac{kg}{m.s} = 10\ \text{poise}}$$

$$1\ Pas = 1\ \frac{kg}{m \times s} = 10\ \text{poise}.$$

$$\boxed{1\ \text{poise} = 10^{-1}\ Pa\text{-}s}$$

* Kinematic Viscosity (γ)

$$\boxed{\gamma = \frac{\mu}{\rho}}$$

Units :- $\frac{kg}{\frac{m \times s}{kg/m^3}} = m^2/s$ (SI)

$= cm^2/s$ (C.G.S) → stoke

$\therefore 1\ m^2/s = 10^4$ stoke.

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Q.1. $V = 5.66\ m^3$, $m = 4765\ kg$.

$$\rho = \frac{m}{V} = \frac{4765}{5.66} = 841.87\ kg/m^3.$$

$$w = \rho g = 841.87 \times 9.81 = 8.26\ kN/m^3.$$

$$G = \frac{\rho_f}{\rho_{H_2O}} = \frac{841.87}{1000} = \underline{0.842}$$

Q.2 $\mu = 0.012$ poise.

R.D $= 0.79$

$$S_f = \frac{\rho_f}{\rho_{H_2O}}$$

$$\rho_f = 0.79 \times 1 = 0.79\ g/cm^3$$

$$\gamma = \frac{\mu}{\rho_f} = \frac{0.012}{0.79} = \underline{0.0152}\ \text{stokes}$$

Q.3. $u = 5 \sin(5\pi y)$

$\mu = 5$ poise $= 5 \times 10^{-1}$ Pa-s

$\tau = \mu \frac{du}{dy}$ (valid for laminar flow).

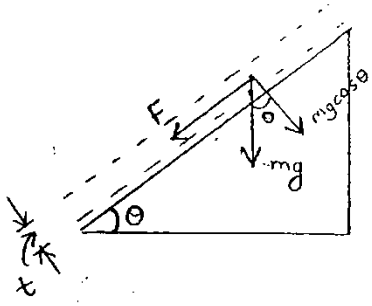
$= 0.5 \times 5 \times 5\pi \cos(5\pi \times 0.05)$

$= \underline{27.76\ N/m^2}$

Q.4

$$\tau = \mu \frac{du}{dy}$$

$$F = mg\sin\theta$$

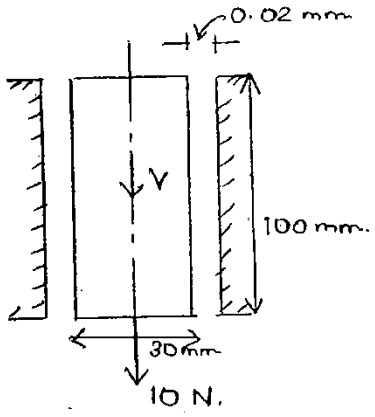


$$\tau = \frac{F}{A} = \frac{mg\sin\theta}{A}$$

$$= \frac{\rho \times V \times g\sin\theta}{A} \qquad \left\{\frac{V}{A} = t\right\}$$

$$\tau = \rho g t \sin\theta$$

Q.5 $\mu = 6$ poise $= 6\times10^{-1}$ Pa-s.



$$\tau = \mu \times \frac{du}{dy}$$

$$= \mu \times \frac{V}{y}$$

$$\tau = \frac{F}{A} = \frac{10\ N}{\pi d l} = \frac{10}{\pi\times30\times10^{-3}\times100\times10^{-3}}$$

$$= \frac{6\times10^{-1}\times V}{0.02\times10^{-3}}$$

$$\therefore V = 35.33 \text{ mm/s}$$

Q.6 $d = 350$ mm, $N = 200$ rpm, $\mu = 8$ poise $= 8\times10^{-1}$ Pa s.

$$P = \frac{2\pi NT}{60}$$

$$T = F \times r$$

$$F = \tau = \mu \frac{du}{dy}$$

$$\tau = 8\times10^{-1} \times \frac{\pi d N}{60\times0.002} = \frac{0.8\times\pi\times0.350\times200}{60\times0.002} = 1466\ N/m^2$$

$$\tau = \frac{F}{A} \Rightarrow F = 1466.076 \times \pi \times 0.35 \times 1$$

$$= 1612.03\ N$$

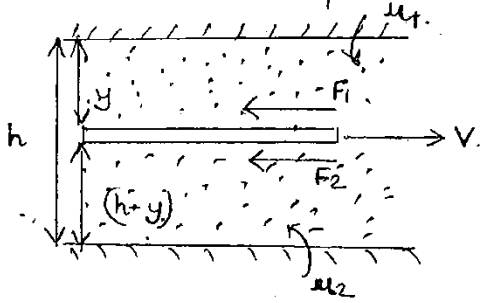
Torque, $T = F \times r$

$= 1612.035 \times \frac{0.35}{2} = 282.106$

$P = \frac{2\pi NT}{60} = \frac{2\pi \times 200 \times 282.106}{60} = 5908.4 \text{ W}$ (or $P = F \times V$ where $V = \frac{\pi dN}{60}$)

$= 5.908 \text{ kW}$

07 Let F be the total shear force acting on the thin plate moving with velocity, 'V'



$F = F_1 + F_2$ & $\frac{\partial F}{dy} = 0$.

$\tau_1 = u_1 \times \frac{du}{dy} = \frac{F_1}{A}$

$F_1 = \frac{A \times u_1 \times V}{y}$

Similarly,

$\tau_2 = u_2 \frac{du}{dy} = \frac{F_2}{A}$

$F_2 = \frac{A \times u_2 \times V}{(h-y)}$

$F = F_1 + F_2 = AV\left(\frac{u_1}{y} + \frac{u_2}{h-y}\right)$

For minimum F,

$\frac{dF}{dy} = 0$.

$-\frac{u_1}{y^2} + \frac{u_2}{(h-y)^2} = 0$.

$\frac{h-y}{y} = \sqrt{\frac{u_2}{u_1}} \Rightarrow \frac{y}{h-y} = \sqrt{\frac{u_1}{u_2}}$

08. $V_1 = 1000\ cm^3$ $V_2 = 995\ cm^3$

$P_1 = 1\ MN/m^2$ $P_2 = 2\ MN/m^2$

$$k = \frac{P_2 - P_1}{-\left(\frac{V_2 - V_1}{V_1}\right)} = \frac{2-1}{\left(\frac{1000-995}{1000}\right)} = \underline{\underline{200\ MPa}}$$

7. Vapour Pressure :

Consider a closed container in which a liquid is filled by leaving an empty space above the liquid surface. If the temperature is increased, the vapour bubbles will starts forming and leaves from the liquid surface and get collected in empty space. The stage'll reach where the rate of formation of the new vapour bubbles will be equal to the rate of collapsing of vapour bubbles into the liquid surface. Under this condition, the vapour phase is said to be in phase eqbm with the liquid.

The pressure exerted by the vapour upon the liquid surface is called 'Vapour Pressure'.

The partial pressure of water vapour is less than the vapour presoure, then no water is present in the lake.

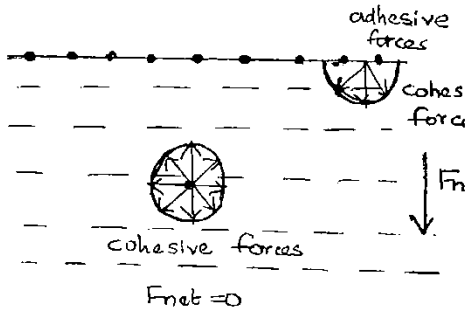
If the partial pressure of water vapour is equal to the vapour pressure then water is present in the lake.

In any fluid flow problems, at any section, the pressure should not fall less than or equal to vapour pressure, to avoid the cavitation.

If the pressare falls below the vapour pressure, then vapour bubbles will form and these will be carried along with flowing fluid into the high pr. regions where they will burst and it may damage the surfaces and the action is called the 'Pitting Action'

8. Surface Tension.

Tension exerted by the fluid at the surface is called Surface Tension.



Consider a molecule which is completely inside the liquid and it is surrounded by equal no. of molecules. ∴ the net force acting upon such molecule is zero.

If we consider another molecule which is at the surface, it is under the influence of attractive force by its own molecule and the gas molecules' attractive force.

The cohesive forces are greater, than the adhesive force and ∴ it is acted by a net downward vertical force. As well as, the surface molecules are also subj. to cohesive forces by the molecules which lies to the left and right of the molecule.
∴ the surface of the liquid is under tension. This tension per unit length is called 'Surface Tension'

Units: N/m (or) J/m².

Eg:- a) A small needle can float
b) Collection of dust particles on the liquid surface.
c) Small birds can sit in the ponds for drinking water.
d) Formation of liquid droplets.

NOTE: $\sigma_{water-air} = 0.0736$ N/m at 20°c.

Case 1: Liquid Droplet

Let d = diameter of droplet.
σ = surface tension
p = pressure in excess of atmosphere (gauge pressure)

Under equilibrium,

Pressure force = surface tension

$$p \times \frac{\pi}{4} d^2 = \sigma \times \pi \times d.$$

$$\boxed{p = \frac{4\sigma}{d}}$$

Case 2: Soap Bubble.

$$p \times \frac{\pi}{4} d^2 = \sigma (2 \times \pi \times d)$$

$$\boxed{p = \frac{8\sigma}{d}}$$

Case 3: Liquid Jet.

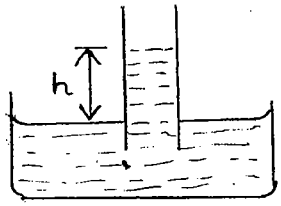
$$p \times l \times d = \sigma \times 2l.$$

$$\boxed{p = \frac{2\sigma}{d}}$$

Surface tension is due to cohesive forces. Surface tension decreases with increase in temperature. ∴ warm water is preferred for good washing of the clothes, thereby forming the liquid droplets into spherical form get minimised.

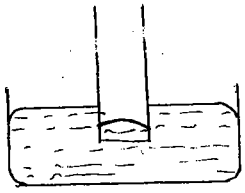
8th Aug, FRIDAY

9. Capillarity.



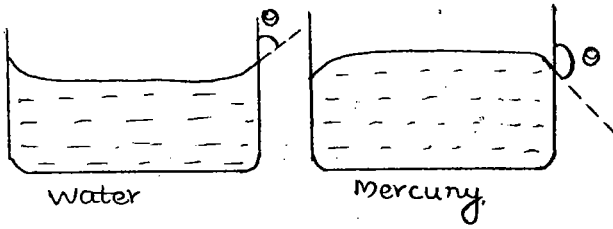
→ capillarity rise

→ adhesive > cohesive



→ capillarity depression

→ cohesive > adhesive



→ concave up

→ $\theta < 90^\circ$

→ wetted case

→ adhesive > cohesive

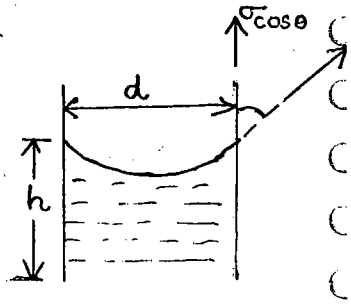
concave down

$\theta > 90^\circ$

not wetted case

cohesive > adhesive

The rise or fall of the liquid level in a narrow vertical tube when it's inserted in a liquid from the level of the liquid in a container



Under equilibrium,

Surface tension force = weight of liquid in the tube.

$$\pi d \times \sigma \cos\theta = \omega \times \frac{\pi}{4} d^2 \times h.$$

$$h = \frac{4\sigma \cos\theta \times \pi d}{w \times \frac{\pi d^2}{}}$$

$$\boxed{h = \frac{4\sigma\cos\theta}{w \times d} \quad \text{or} \quad \frac{4\sigma\cos\theta}{\rho g d}}$$

If $\theta > 90^\circ$, $\cos\theta$ = -ve, $\therefore$ h becomes -ve (capillarity fall)

As $d \uparrow$, $h \downarrow$

$\therefore$ for the tubes of manometer, to avoid the correctio for capillarity, the $d > 6$ mm is preferred.

For water, $\theta = 0^\circ$ For mercury, $\theta = 129^\circ$

$$h = \frac{4\sigma}{\rho g d}.$$

NOTE: Capillarity is due to both cohesion & adhesion.

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09. $d = 10^{-3}$ m, $\sigma = 0.07$ N/m.

$$p = \frac{4\sigma}{d.} = \frac{4 \times 0.07}{10^{-3}.} = 280 \text{ Pa} = \underline{0.28 \text{ kPa}}$$

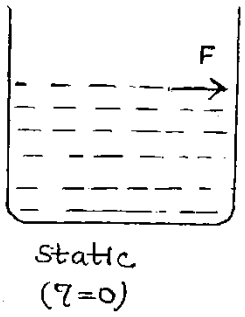
10. $d = 10 \times 10^{-3} = 10^{-2}$ m., $\sigma = 0.04$ N/m.

$$p = \frac{8\sigma}{d.} = \frac{8 \times 0.04}{10^{-2}} = \underline{32 \text{ N/m}^2}$$

Pressure Measurement

Intensity of Pressure:

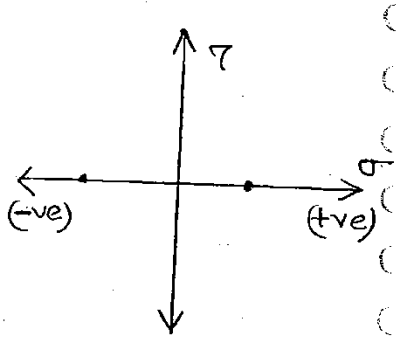
The pressure is defined as the normal force exerted by static fluid per. unit area.



Intensity of pressure, $p = \frac{\text{normal force}}{\text{area.}}$

Pressure is compressive in nature. (fluid is subjected to compressive forces by walls of the container)

The intensity of pressure if it is located on Mohr's circle, it has zero radius ($\because \tau = 0$)



Units: (i) $\frac{N}{m^2}$ (Pa)

(ii) kPa, MPa, GPa

(iii) mm of column of liquid.

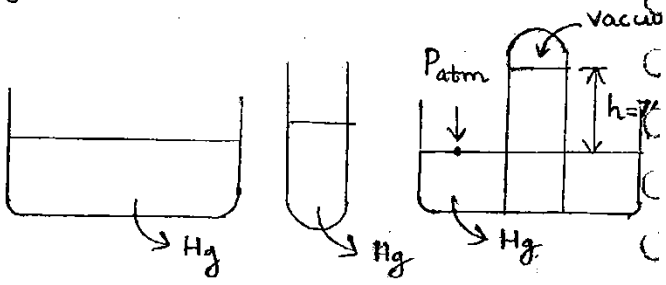
(iv) 1 bar = 10^5 Pa.

Types of Pressures:

1. Atmospheric Pressure (P_{atm})

It is defined as pressure exerted by the atmospheric air upon the surfaces

P_{atm} = 760 mm of Hg
= 10.33 m of H_2O



$$P_{atm} = P_{Hg} \times g \times h + P_v$$

$$= P_{Hg} \times g \times h$$

$$= 13.6 \times 1000 \times 9.81 \times 760 \times 10^{-3} \text{ Pa}$$

$$= 101.396 \text{ kPa.}$$

$$= 1.01396 \text{ bar.}$$

$$\approx 1.01325 \text{ bar}$$

$$\approx 1 \text{ bar.}$$

conducted by TORRICELL

(OR)

$$P_{atm} = P_{H_2O} \times g \times h_{H_2O}$$

$$= 1000 \times 9.81 \times 10.33.$$

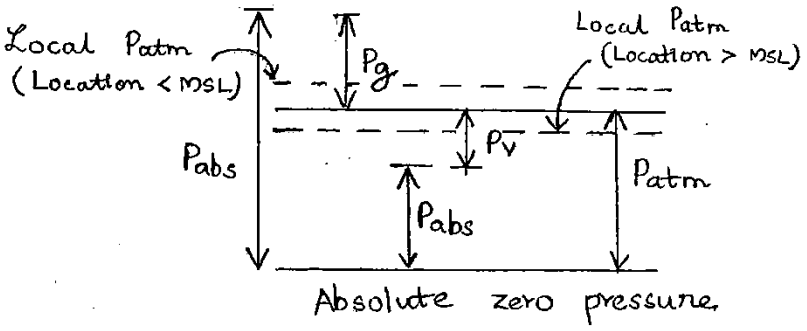
$$= 1.01337 \text{ bar (at MSL)}$$

2. Gauge Pressure

It is defined as the pressure measured by the gauge (instrument) in which atmospheric pressure is taken as datum. If the measured pressure is lower than atmospheric pressure, then it is called 'negative gauge pressure or vacuum pressure'.

3. Absolute zero pressure.

It is the pressure escorted by the absolute vacuum



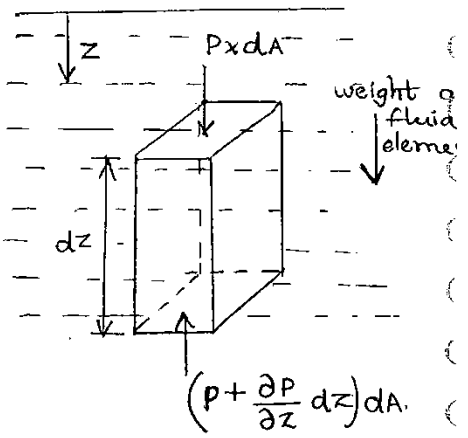
$$P_{abs} = P_{atm} + P_g$$

$$P_{abs} = P_{atm} - P_v$$

Hydrostatic Law:

It states that rate of increase of pressure in the vertically downward direction in a static fluid is equal to the specific wt. of the fluid.

$$\boxed{\frac{\partial P}{\partial z} = w}$$



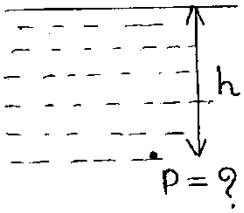
Under equilibrium,

$$\Sigma F_y = 0$$

$$-p \times dA + \left(p + \frac{\partial P}{\partial z} dz\right) dA - w = 0.$$

$$dz.dA\left(\frac{\partial P}{\partial z}\right) = w.dA.dz$$

$$\Rightarrow \boxed{\frac{\partial P}{\partial z} = w = \rho g}$$



$$\partial P = \rho g \, dz.$$

On integration,

$$p = \rho g z + c.$$

At $z = 0$, $p = P_{atm}$.

$$\therefore \boxed{p = \rho g z + P_{atm}.}$$

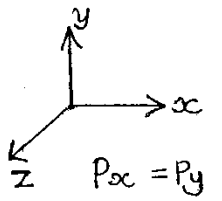
$$p - P_{atm} = \rho g z.$$

$$\boxed{\text{Gauge Pressure, } P_g = \rho g z}$$

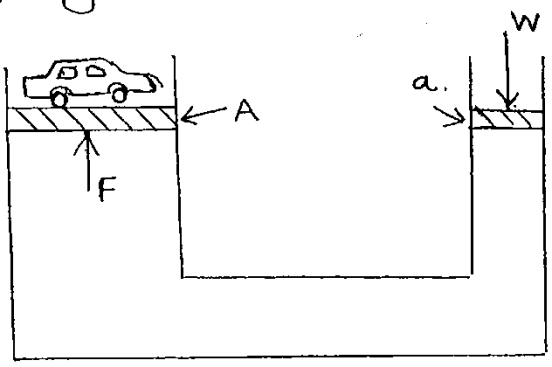
At distance 'h' from free surface, $p = \rho g h$

Pascal's Law:

It states that in a static fluid, the pressure acts equally in all the directions.



$P_x = P_y = P_z$



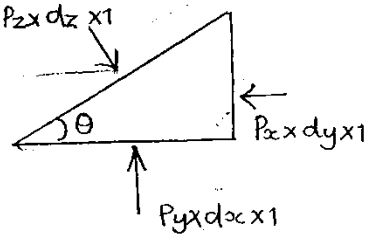
$\frac{W}{a} = \frac{F}{A}$; application of Pascal's Law (Hydraulic Lift)

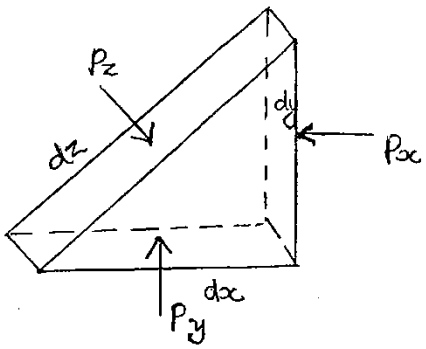
$\Rightarrow F = \left(\frac{A}{a}\right) W$

Consider a wedge shaped fluid element,

Consider unit element,

Resolve the forces along x-direction,





$$P_x \times dy \times 1 = P_z \times dz \times 1 \times \sin\theta. \Rightarrow P_x\, dy = P_z\, dz \sin\theta \rightarrow (1)$$

Resolve forces along y-direction,

$$P_y\, dx = P_z\, dz \sin\theta \cos\theta. \rightarrow (2)$$

$$\sin\theta = \frac{dy}{dz} \quad \therefore dy = dz \sin\theta$$

(1) becomes,

$$P_x\, dy = P_z\, dy$$

$$\Rightarrow P_x = P_z.$$

Similarly, (2) becomes,

$$P_y = P_z.$$

$$\therefore \underline{P_x = P_y = P_z}$$

Pressure Measurement Devices:

- Manometers
  - Simple
  - Differential.
- Mechanical Gauges.

a) Piezometer

b) U-tube manometer

c) Single column manometer

(i) Vertical column.

(ii) Inclined column.

→ Manometers:

These are the devices which are used to measure the pressure at any point in the pipe by balancing the column of the fluid by the different kind of the fluid

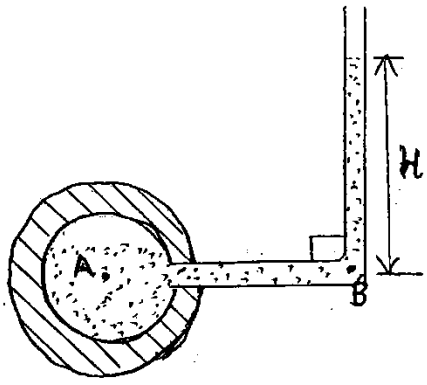
* Simple Manometers

These are the devices which contains a narrow tube whose one end is connected to the point where the pressure is to be measured and the other end is open to the atmosphere.

a) Piezometer.

$$P_A = P_B = \rho g H + P_{atm}$$

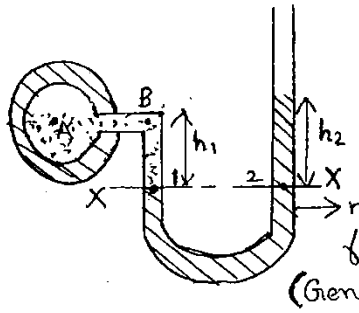
$\therefore P_A = \rho g H$ (gauge pressure)



Piezometer is used for measurement of moderate pressures

Piezometer is not useful for measurement of gas pressures since gases never form a free surface with the atmosphere.

b) U-tube Manometer



Gauge Pressure

(Generally Hg as it has (i) Low vap our press (ii) highest density (13600 kg/m³))

At datum line X-X,

$$P_1 = P_2$$

$$P_B + \rho g h_1 = P_2 g h_2 + P_{atm}.$$

$$P_A + \rho_1 g h_1 = P_2 g h_2 + P_{atm}.$$

$$P_A = P_2 g h_2 - \rho_1 g h_1 + P_{atm}.$$

$$\therefore \boxed{P_A = P_2 g h_2 - \rho_1 g h_1} \quad ; \text{ gauge pressure}$$

$$P_A = \rho_1 g h.$$

If pipe contains oil, (ρ_o)

$$P_A = \rho_o \times g \times h_o.$$

$$h_o = \frac{P_A}{\rho_o \times g} \quad m \text{ of oil.}$$

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If the pressure of oil given in terms of water column,

$$P_A = \rho_{H_2O} \times g \times h_{H_2O}.$$

$$h_{H_2O} = \frac{P_A}{\rho_{H_2O} \times g} = \frac{P_2 g h_2 - \rho_1 g h_1}{\rho_{H_2O} \times g} = \frac{P_2}{\rho_{H_2O}} \times h_2 - \frac{\rho_1}{\rho_{H_2O}} \times h_1$$

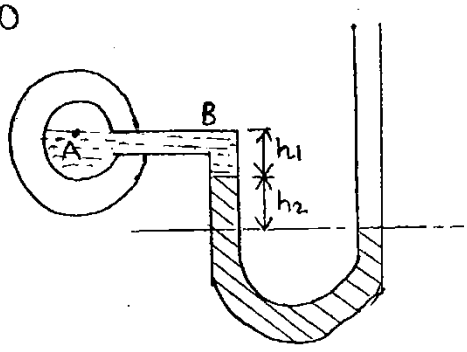
$$\boxed{h_{H_2O} = S_2 h_2 - S_1 h_1}$$

$$h_B + S_1 h_1 = S_2 h_2 + 0.$$

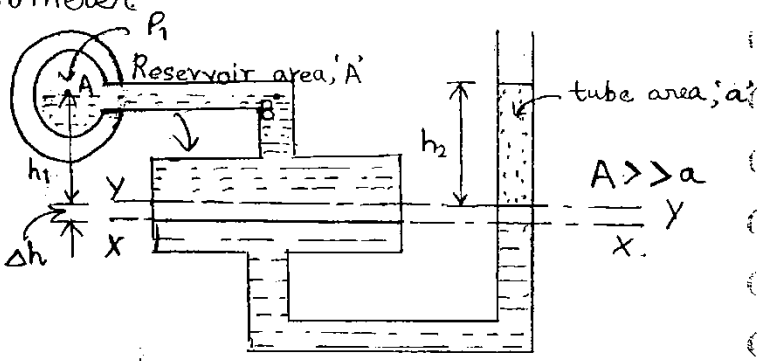
$$h_A + S_1 h_1 = S_2 h_2$$

$$\therefore \boxed{h_A = S_2 h_2 - S_1 h_1} \quad m \text{ of } H_2O$$

Only disadvantage is the measurement of h_1 & h_2. (two values)



c) Single Column Manometer



At datum line,

$$P_B + \rho_1 g (h_1 + \Delta h) = P_{atm} + \rho_2 g (h_2 + \Delta h)$$

$$P_A = (\rho_2 g h_2 - \rho_1 g h_1) + (\rho_2 - \rho_1) g \Delta h$$

For volumes equal,

$$A \times \Delta h = a \times h_2$$

$$\Delta h = \frac{a h_2}{A}$$

Since $a << A$, then Δh can be neglected.

$$P_A = \rho_2 g h_2 - \rho_1 g h_1 \quad \text{(gauge)}$$

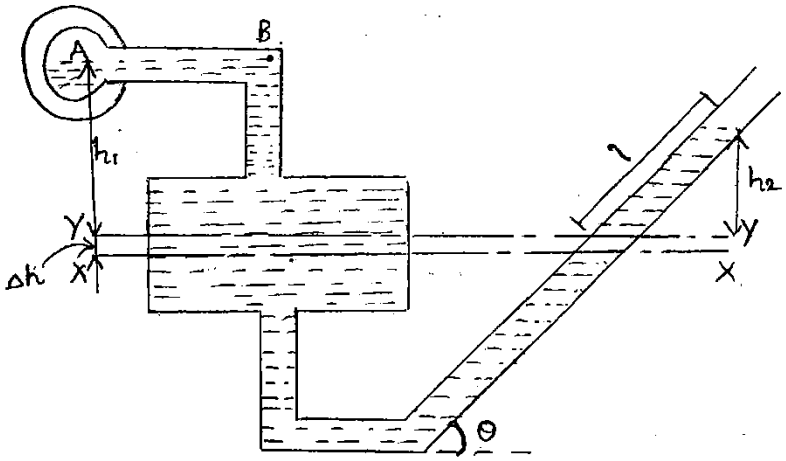
d) Single Column Inclined Manometer.

$P = h \rho g$ (h: vertical height)

$h_2 = l \sin\theta$.

As 'θ' increases, sensitivity decreases.

$$\therefore \text{Sensitivity} = \frac{1}{\sin\theta}$$



θ	Sensitivity.
0°	$\frac{1}{0} = \infty$
30°	$\frac{1}{1/2} = 2$
60°	$2/\sqrt{3} = 1.15$
90°	1

R-36.

Q.2 $S_o = 0.8$

$$P_{H_2O} = \rho_{H_2O} \times g \times h_{H_2O}$$

$$= 1000 \times 9.81 \times 80 \text{ Pa}.$$

$$P_{oil} = \rho_{oil} \times g \times h_o$$

$$= 1000 \times 0.8 \times 9.81 \times h_o.$$

$$P_{H_2O} = P_{oil}$$

$$\Rightarrow h_o = \underline{\underline{100 \text{ m}}}$$

Q.3. $P = 1000 + 0.008\, y^{3/2}$.

$$\frac{dP}{dz} = \rho g.$$

$$\int dP = \int \rho g \, dz.$$

$P = \rho g h \rightarrow$ incompressible flui (ρ const.)

$$P = g\left(1000 + 0.008\, y^{3/2}\right).$$

$$900 \text{ kPa} = g\left(1000 y + 0.008 \frac{y^{5/2}}{5/2}\right).$$

$$y = \underline{\underline{91.5 \text{ m}}}$$

Q.4. Pressure at the bottom $= 0.9 \times 10 \times 2 + 10 \times 4$

$= \underline{\underline{58 \text{ kN/m}^2}}$

$\gamma_w = 10$

$\gamma_{oil} = 10 \times 0.9$

$p = \gamma H$

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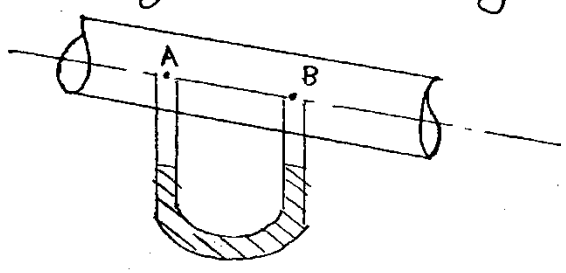
Q.5 **Differential Manometer:**

It is a device used to measure the difference of pressures blw any two points that may lie in the single pipe or in different pipe lines

$$P_A - P_B = ?$$

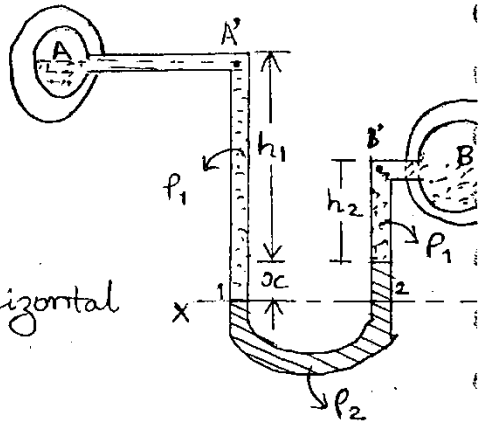


At datum line $x-x$,

$$P_1 = P_2$$

$$P_A + \rho_1 g(h_1 + x) = P_B + \rho_1 g h_2 + \rho_2 g x.$$

$$\boxed{P_A - P_B = \rho_1 g h_2 - \rho_1 g h_1 + \rho_2 g x - \rho_1 g x}$$



If the points A & B lie in the same horizontal line, then $h_1 = h_2$,

$$P_A - P_B = (\rho_2 - \rho_1) g x.$$

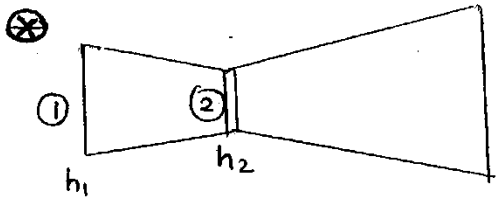
$$\therefore \quad h_A - h_B = \frac{(\rho_2 - \rho_1) g x}{\rho_1 g}.$$

$$= \left(\frac{\rho_2}{\rho_1} - 1\right) x.$$

$$= \left[\frac{\frac{\rho_2}{\rho_{H_2O}}}{\frac{\rho_1}{\rho_{H_2O}}} - 1\right] x$$

$$\Rightarrow \quad h_A - h_B = \left(\frac{S_2 - S_1}{S_1}\right) x$$

$$\boxed{\therefore \quad h_A - h_B = \left(\frac{S_m}{S_L} - 1\right) x}$$



Discharge of a venturimeter,

$$Q = \frac{a_1 a_2 \sqrt{2gh}}{\sqrt{a_1^2 - a_2^2}}$$

$$h = h_1 - h_2 = h_A - h_B.$$

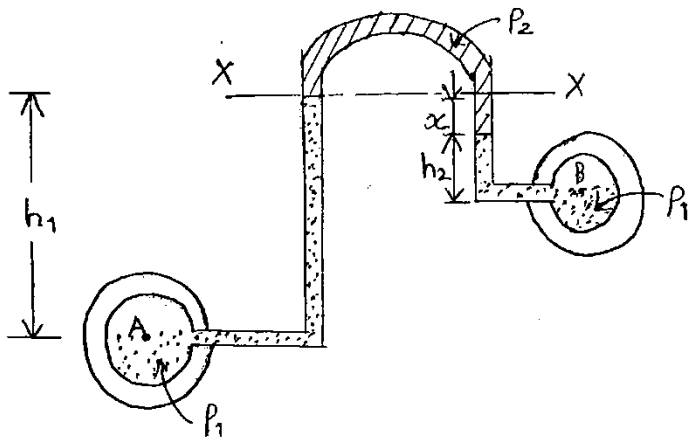
$S_{manometric\ fluid}$

$$= \left(\frac{13.6}{1} - x\right) = 12.6x \quad \Rightarrow \boxed{h = 12.6x}$$

S_{H_2O}

At the datum line,

$$P_A - \rho_1 g h_1 = P_B - \rho_1 g h_2 - \rho_2 g x$$



Q.05.

$$P_A + 0.8\times10^3 \times 9.81 \times 0.31 = P_B + 0.8\times10^3 \times 9.81 \times 0.13 + 13.6\times10^3 \times 9.81 \times 0.08$$

$$P_A - P_B = \underline{9.26} \text{ kPa}$$

Q.06. At the datum line,

$$P_M - 1000\times9.81\times0.09 = P_N - 1000\times9.81\times(0.12+0.06) - 0.83\times1000\times9.81\times0.03$$

$$P_M - P_N = -1127.169 \text{ Pa.}$$

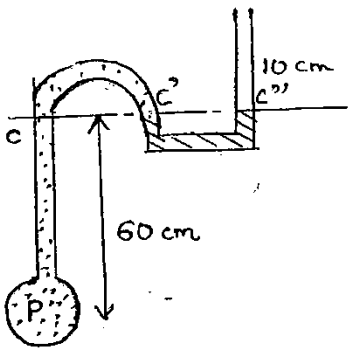
$$h_M - h_N = \frac{P_M - P_N}{0.8\times10^3\times9.81} = \underline{-13.84} \text{ cm}$$

Q.07. $P_{c''} = 13.6\times10^3\times9.81\times0.1 + P_{atm}$ (0)

$$P_c = P_{c'} = P_{c''} = 13.6\times10^3\times9.81\times0.1$$

$$P_p = P_c + 0.85\times10^3\times9.81\times0.6.$$

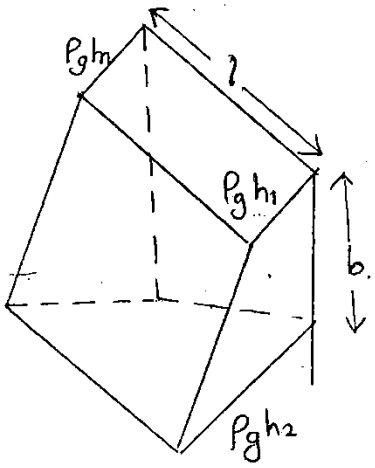
$$= 18.344 \text{ kPa.}$$

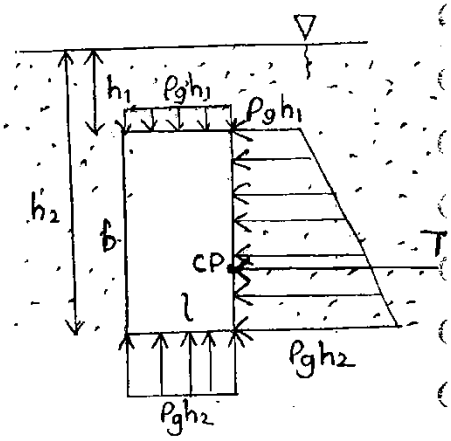


$$h_p = \frac{18.344\times10^3}{0.85\times10^3\times9.81} = \underline{2.2\text{m}}$$

→ Hydrostatic Forces on Submerged Surfaces.

Pressure Prism:





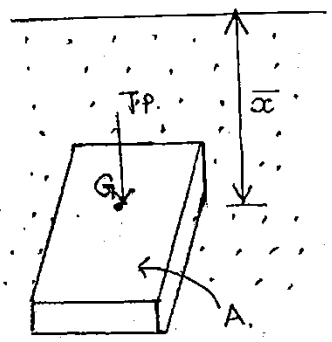
Total Pressure (TP):

Whenever a surface either ~~or~~ a plane or curve is completely submerged in the static fluid, the pressure force variations will take place across the surface. The resultant of all the pressure force variations is called 'Total Pressure'. It has units of 'force' (N).

Centre of Pressure (CP):

It is the point at which total pressure will act.

a) Horizontal Surface.



Let A → area of surface.

w → sp. wt. of liquid.

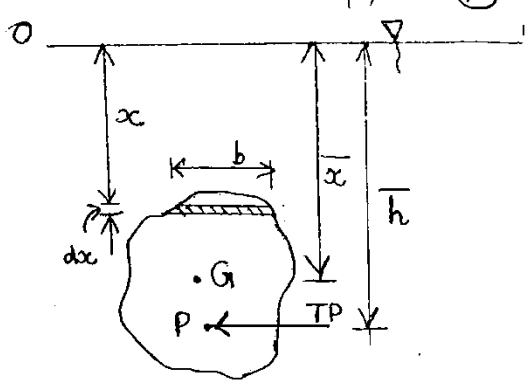
$\bar{x}$ → distance of C.G from free surface.

T.P = pressure × area.

$= \rho g \bar{x} \times A = \omega A \bar{x}$

$$\boxed{TP = \omega A \bar{x} \quad \& \quad CP = \bar{x}}$$

b) Vertical Surface



Pressure force acting on elemental area $= \rho g x . (b\,dx)$.

Sum of the pressure forces

$$= \int \rho g (b\,dx) \times x .$$

$$= \rho g \int (b\,dx) x \qquad \{(b\,dx)x = \text{first area moment}\}$$

$$TP = \rho g A \bar{x} = \omega A \bar{x}$$

"Sum of the moments of individual forces is equal to the moment caused by the resultant force" – Varignance Theore

Moment caused by pressure force acting on elemental area about free surface $= \text{Force} \times x$

$$= (P \times A) \times x$$

$$= \rho g x . b\,dx . x .$$

$$= \rho g (b\,dx) x^2$$

Sum of moments $= \int \rho g (b\,dx) x^2$

$$= \rho g \int (b\,dx) x^2 \qquad [(b\,dx)x^2 = \text{second area mome} = I]$$

$$= \rho g I_o$$

Parallel Axis Theorem:

$$\rho g I_o = \rho g (I_G + A \bar{x}^2) \longrightarrow (1)$$

Moment caused by TP about free surface $= TP \times \bar{h}$

$$= \omega A \bar{x} \times \bar{h} \longrightarrow (2)$$

From (1) & (2),

$$\rho g A \bar{x} \times \bar{h} = \rho g (I_G + A \bar{x}^2)$$

$$\therefore \bar{h} = \frac{I_G}{A \bar{x}} + \frac{A \bar{x}^2}{A \bar{x}}$$

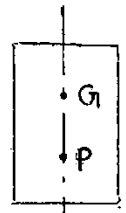
$$\left\{ \rho g A \bar{x} \times \bar{h} = \rho g I_o \Rightarrow \bar{h} = \frac{I_o}{A \bar{x}} \right.$$

$\bar{h} = \bar{x} + \frac{I_G}{A\bar{x}}$; $I_G \rightarrow$ MI of the surface about an axis passing through C.G

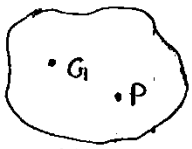
$\therefore \bar{h} > \bar{x} \Rightarrow$ C.P always lies below C.G

$$\bar{h} - \bar{x} = \frac{I_G}{A\bar{x}}$$

* For symmetrical surfaces, C.G & C.P lie along the same line.



Symmetrical



Unsymmetrical

* Volume of prism $= \frac{1}{2}(\rho g h_1 + \rho g h_2) \times b \times l.$

$= \frac{1}{2} \rho g (h_1 + h_2) \times b \times l.$

$\bar{x} = h_1 + \frac{b}{2} \Rightarrow \bar{x} = \frac{h_1 + h_2}{2}$

$= h_1 + \frac{h_2 - h_1}{2}$

$\therefore$ Volume of pressure prism $= \frac{1}{2}(\rho g h_1 + \rho g h_2) \times b \times l.$

$= \rho g \bar{x} \times A.$

$= \omega A \bar{x} = TP.$

$\therefore$ **Volume of pressure prism = TP.**

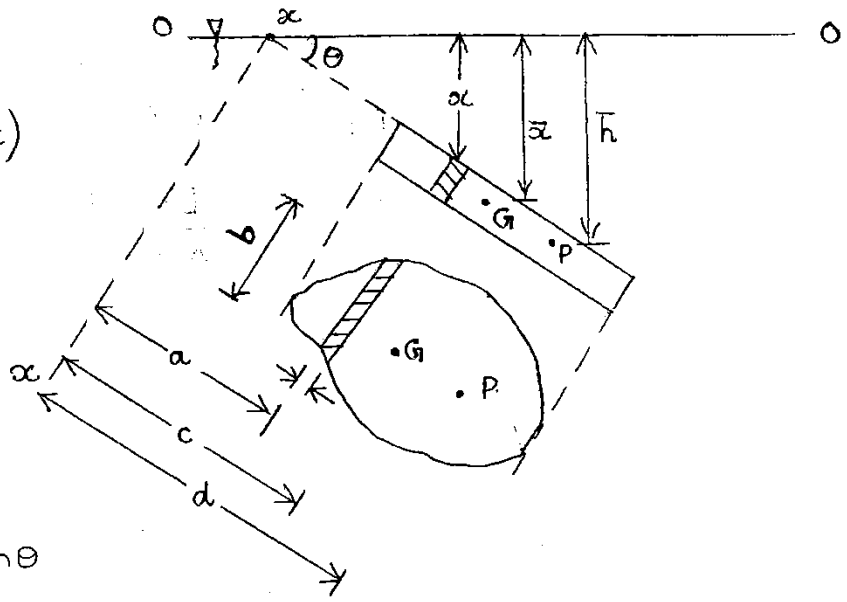
NOTE :

1. Volume of pressure prism represents the 'total pressure'
2. The centroid of the volume of the prism will be equal to the 'Centre of Pressure'

$$TP = \omega A \bar{x}$$

$$CP = \bar{x} + \frac{I_G}{A\bar{x}}$$

c) Inclined Surface.



Pressure force acting on elemental area $= \rho g x . (b dx)$

$$\sin\theta = \frac{x}{a} = \frac{\bar{x}}{c} = \frac{\bar{h}}{d}.$$

$$= \rho g (b\,dx)\, a \sin\theta.$$

Sum of pressure forces

$$= \int \rho g (b\,dx)\, a \sin\theta$$

$$= \rho g (\sin\theta) \int (b\,dx)\, a.$$

$$= \rho g \sin\theta \times A \times c$$

$$= \rho g \sin\theta \times A \times \frac{\bar{x}}{\sin\theta} = \rho g A \bar{x}$$

$$TP = wA\bar{x}$$

Moment of pressure force acting on elemental area

$$= \rho g x . (b dx) . a .$$

$$= \rho g (a \sin\theta)(b dx) a$$

Sum of the moments $= \int \rho g (b dx) a^2 \sin\theta .$

$$= \rho g \sin\theta \int (b dx) . a^2$$

$$= \rho g \sin\theta . I_{xx}.$$

$$= \rho g \sin\theta \left(I_G + Ac^2\right) \qquad c = \frac{\bar{x}}{\sin\theta}$$

$$= \rho g \sin\theta \left[I_G + A\left(\frac{\bar{x}}{\sin\theta}\right)^2\right] \longrightarrow (1)$$

Moment caused by total pressure about $xx-x$, $= TP \times d.$

$$= wA\bar{x} \times d \longrightarrow (2)$$

From (1) & (2),

$$wA\bar{x}\, d = wA\bar{x}\, \frac{\bar{h}}{\sin\theta} = \rho g \sin\theta \left(I_G + A\frac{\bar{x}^2}{\sin^2\theta}\right)$$

$$\bar{h} = \frac{I_G \sin^2\theta}{A\bar{x}} + \bar{x}$$

TP	$= \omega A \bar{x}$
CP	$= \frac{I_G \sin^2\theta}{A\bar{x}} + \bar{x}$

⊙ For horizontal surface, $\theta = 0^\circ$,

$$\bar{h} = \bar{x}$$

⊙ For vertical surface, $\theta = 90^\circ$,

$$\bar{h} = \bar{x} + \frac{I_G}{A\bar{x}}$$

Q.08. $TP = \omega A \bar{x}$

$= 1000 \times 9.81 \times \frac{\pi}{4} \times 4 \times 1 = \underline{30.76 \text{ kN}}$

Q.09. $TP = \omega A \bar{x}$

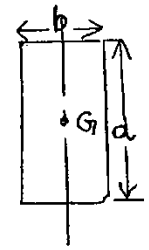
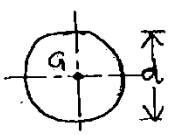
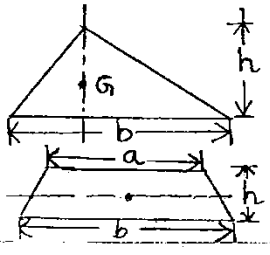
$= 1000 \times 10 \times 9 \times 1.5 = 135 \text{ kN}.$

Moment of hydrostatic thrust about top edge $= TP \times \bar{h}$

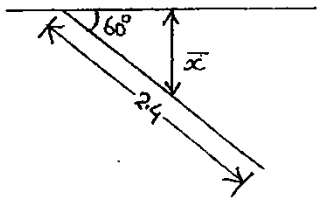
$= TP\left(\frac{I_G}{A\bar{x}} + \bar{x}\right)$

$= 135\left(\frac{3 \times 3^3/12}{9 \times 1.5} + 1.5\right)$

$= 135 \times 2 = \underline{270 \text{ kNm}}$

Shape	C.G from base	Area	I_G parallel to base
	$d/2$	$b \times d$	$bd^3/12$
	$d/2$	$\frac{\pi}{4}d^2$	$\frac{\pi}{64}d^4$
	$h/3$	$\frac{1}{2}bh$	$\frac{bh^3}{36}$
	$\left(\frac{2a+b}{a+b}\right)\frac{h}{3}$	$\frac{1}{2}(a+b)h$	$\frac{a^2+4ab+b^2}{36(a+b)} \times h^3$

10. $\bar{x} = 1.2 \sin 60$



$TP = \omega A \bar{x}$

$= 850 \times 9.81 \times 0.75 \times 2.4 \times 1.2 \sin 60$

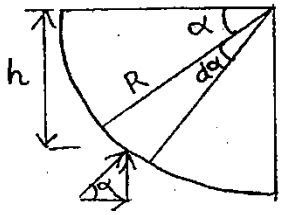
$= 15.598$ kN

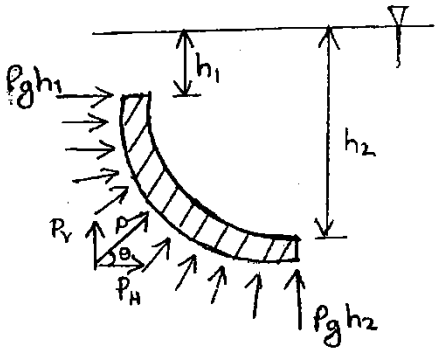
$CP = \bar{x} + \dfrac{I_G \sin^2\theta}{A\bar{x}}$

$= 1.2 \sin 60 + \dfrac{0.75 \times 2.4^3/12 \times \sin^2 60}{0.75 \times 2.4 \times 1.2 \sin 60}$

$= 1.385$ m

N. d) Curved Surface

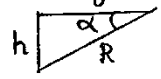




$dA = R d\alpha \times 1$

Pressure force acting on $dA = \rho g h \times dA$

$= \rho g h \times R d\alpha$



$\sin\alpha = \dfrac{h}{R}$

$h = R \sin\alpha$

∴ TP acting on $dA = \rho g (R\sin\alpha) R\, d\alpha = \rho g R^2 \sin\alpha\, d\alpha$

Horizontal component of TP $= \rho g R^2 \sin\alpha\, d\alpha \times \cos\alpha$

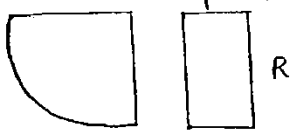
Sum of horizontal components $= \int_{\alpha=0}^{\pi/2} \rho g R^2 \sin\alpha \cos\alpha\, d\alpha$

$= \rho g R^2 \int \sin\alpha \cos\alpha\, d\alpha$

$= \rho g R^2 \left(\dfrac{-\cos 2\alpha}{2}\right)_0^{\pi/2}$

$= \dfrac{\rho g R^2}{2}$

$$= \rho g (R \times 1) \frac{R}{2} = \omega A \bar{x}$$



∴ Horizontal component of TP acting on the curved surface will be equal to the total pressure acting on the projected area of curved surface onto the vertical plane.

Vertical component of TP acting on $dA = \rho g R^2 \sin^2\alpha \, d\alpha$.

Sum of vertical component for entire surface

$$= \int \rho g R^2 \sin^2\alpha \, d\alpha.$$

$$= \rho g R^2 \int_0^{\pi/2} \sin^2\alpha \, d\alpha.$$

$$\int_0^{\pi/2} \sin^2\theta = \frac{\pi}{4}$$

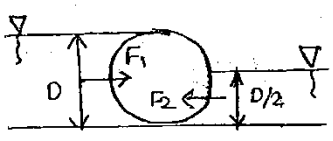
$$= \rho g \cdot \left(\frac{\pi R^2}{4} \times 1\right)$$

= ω × volume of liquid supported by curved surface.

= weight of liquid supported by curved surface upto free surface.

$P_H = \omega A \bar{x}$
P_V = W of liquid supported by curved surface.

11. Net horizontal force $= F_1 - F_2$



$$F_1 = \omega A \bar{x}$$

$$= \gamma \times D \times \frac{D}{2} = \frac{\gamma D^2}{2}$$

$$F_2 = \gamma \times \left(\frac{D}{2} \times 1\right) \times \frac{D}{4} = \frac{\gamma D^2}{8}$$

$$F_1 - F_2 = \frac{3\gamma D^2}{8}$$

3. BUOYANCY

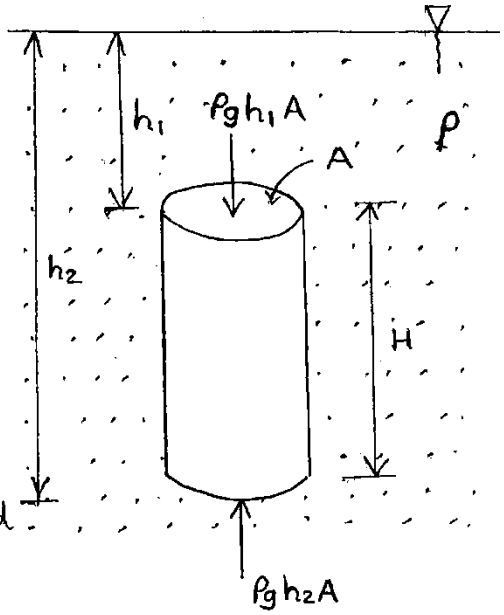
Buoyancy Force $= \rho g h_2 A - \rho g h_1 A$.

$= \rho g A (h_2 - h_1)$

$= \rho g A H = \rho g V_{body}$.

$= \rho g V_{fluid\ displaced}$

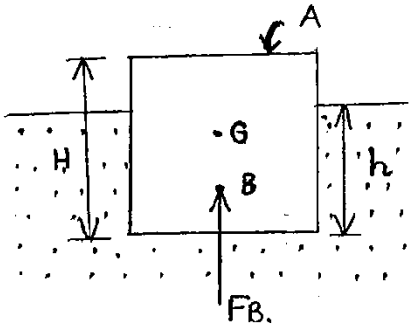
= Weight of fluid displaced

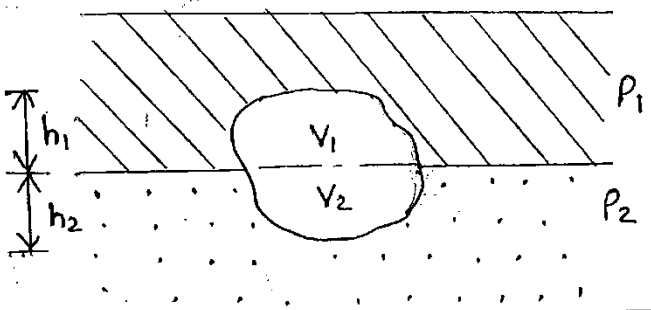


Whenever an object submerged partially or completely, it will be subjected to the net resultant vertical upward force and is known as 'Buoyancy force' or 'Force of Buoyancy'

The point at which the force of buoyancy acts is known a 'Centre of Buoyancy' (B). The Centre of Buoyancy is the centroic of the volume of fluid displaced (volume of body immersed)

Volume of body immersed $= A \times h$



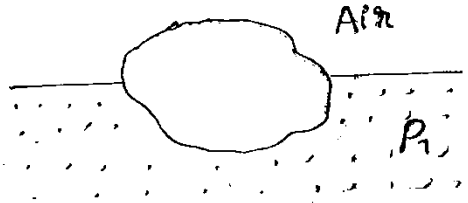


Buoyancy force $= \rho_1 g V_1 + \rho_2 g V_2$

At $p = 1$ bar, $T = 20 + 273 = 293$ K.

$p = \rho RT$

$\therefore \rho = \frac{p}{RT} = \frac{100}{0.287 \times 293} = 1.18\ kg/m^3$



F_B = weight of liquid displaced + weight of air displaced.

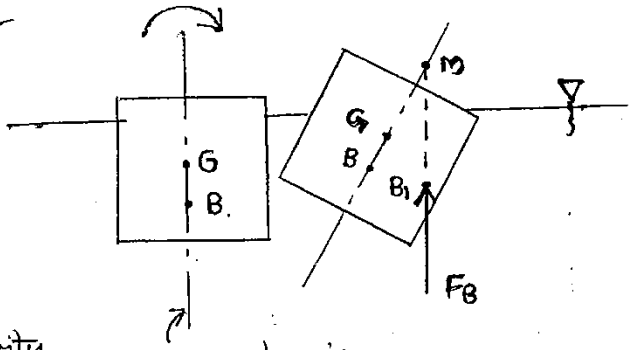
Since ρ of air is very small in magnitude, weight of air displaced will be neglected.

∴ For floating bodies,

F_B = weight of liquid displaced

→ Metacentre (M)

It is defined as the point about which the body will try to oscillate.

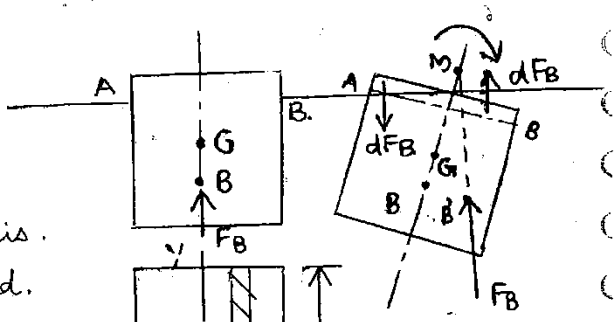


* Metacentric Height (GM)

It is the distance from metacentre to centre of gravity.

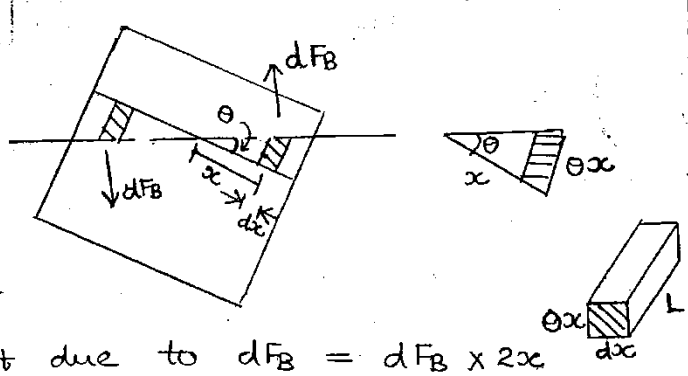
① Analytical Method.

$$GM = \frac{I_{Y-Y}}{\forall} - BG$$



I_{YY} → moment of inertia about Y-Y axis.

$\forall$ → volume of liquid body immersed.



Moment due to dF_B = $dF_B \times 2x$

$= \rho g \cdot \theta x \cdot dx \times L \times 2x$

$= \int \rho g \cdot \theta x \cdot dx \cdot L \times 2x$

$= \rho g \int 2x^2 \theta L \, dx$

$= \rho g \theta \int 2x^2 (L dx)$ — top surface. $L dx \rightarrow$ area of strip

$= \rho g \theta I_{yy}$ — $\int y^2 dA = I$

$I_{yy} \rightarrow$ moment of inertia @ top liquid level.

Moment caused by change of centre of buoyancy $= F_B \times BB_1$

$= F_B \times BM \times \theta$ — $\tan\theta = \frac{BB_1}{BM}$, $\tan\theta \approx \theta = \frac{B}{1}$

$\Rightarrow \rho g \theta I_{yy} = F_B \times BM \times \theta$ (?)

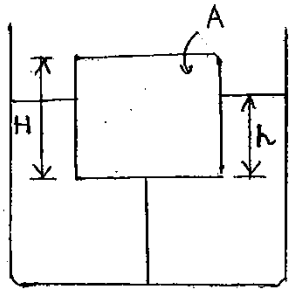
$BM = \frac{\rho g}{F_B} \times I_{yy}$ — $F_B = \rho g \forall$, $\forall \rightarrow$ volume of immersed body

$= \frac{I_{yy}}{\forall}$

$GM = BM - BG = \frac{I_{yy}}{\forall} - BG$

? Due to change of centre of buoyancy, F_B will induce a moment. This moment will be exactly equal in magnitude to the moment caused by the forces, dF_B.

Q A cylindrical body of c/s area, A, height H, density ρ_s is immersed to a depth h in a liquid of density, ρ and tied to the bottom with a string. Tension in the string is



$W + T = F_B$

$T = F_B - W$

$= \rho g h A - \rho_s g H A$

$\therefore T = (\rho h - \rho_s H) g A$.

When $F_B = W$, the object is freely floating.

$W > F_B$, the object will sink.

$F_B > W$, the body will move in upward direction to the point where $F_B = W$

For floating bodies, $T = 0$

$$\Rightarrow \rho h - \rho_s H = 0$$

$$\boxed{h = \frac{\rho_s}{\rho} H}$$; $h \rightarrow$ depth of body immersed in water

or $h = \frac{\text{R.D of solid}}{\text{R.D of fluid}} \times$ total height.

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01. $h = \frac{0.5}{1} \times 2 = 1$ m.

Volume of cube submerged under water $= 1 \times (2 \times 2) = \underline{4 m^3}$

02. $h = \frac{0.64}{1.025} \times 2 = \underline{1.248 \text{ m}}$

03 $h = \frac{3.4}{13.6} \times H$

$\frac{h}{H} = 0.25$

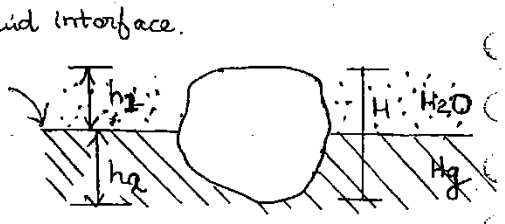
$\therefore$ Fraction of volume under mercury $= \underline{0.25}$

04. ~~Depth immersed in mercury~~ $= \frac{7.6}{13.6} \times 10 = 5.588$ ~~cm~~

~~Height of~~ $S_s = 7.5$, $S_{f1} = 13.6$, $S_{f2} = 1$

Let V be the volume of the solid, A be the c/s area of solid.

H = 10 cm



$F_B = (F_B)_{H_2O} + (F_B)_{Hg}$.

$(F_B)_{H_2O}$ = weight of water displaced

$= \rho g \times V_{H_2O}$

$= 1000 \times g \times A \times h_1 = 10 g h_1$

$(F_B)_{Hg}$ = weight of Hg displaced $= 13600 g \times A \times h_2 = 136 g h_2$

Total buoyancy force $= 1000gh_1 + 136gh_2$

At eqbm,

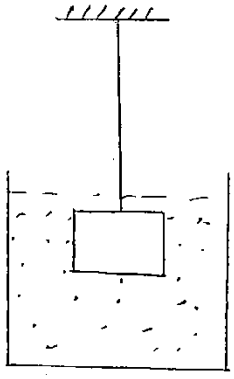
$$W = F_B.$$

$$\rho_s \times 0.1 \times 0.1 \times 0.1\, g = 10gh_1 + 136g(H - h_1).$$

$$7.6 = 10h_1 + 136(0.1 - h_1).$$

$$h_1 = \underline{\underline{4.76 \text{ cm}}}$$

5. $m = 12$ kg, $S_f = 0.8$, $S_s = 2.4$, $g = 10$ m/s



$W \downarrow$, $F_B \uparrow$, $T \uparrow$

$$W = F_B + T. \Rightarrow T = W - F_B.$$

$W = mg = 12 \times 10 = \underline{\underline{120 \text{ N}}}$

F_B = wt. of fluid displaced.

$= 0.8 \times 1000 \times 10 \times V_{body}$

$= 0.8 \times 1000 \times 10 \times \frac{12}{2.4 \times 1000}$ $\quad V = \frac{m}{\rho}$

$= 40$ N.

$$T = 120 - 40 = \underline{\underline{80 \text{ N}}}$$

6. $\rho_{CH_4} = 0.75$ kg/m^3

$\rho_{air} = 1.25$ kg/m^3

Let V be the volume of the balloon.

$(F_B)_{air}$ = weight of balloon with CH_4 + weight of man.

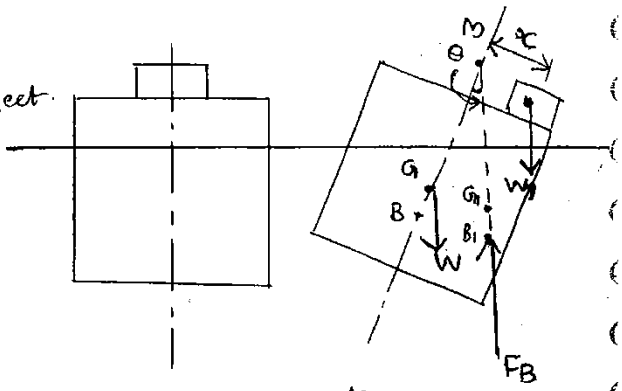
$$\rho_{air} g \cdot V_b = \rho_{CH_4} \times g \times V_b + 750 \text{ N}.$$

$$1.25 \times 9.81\, V_b = 0.75 \times 9.81 \times V_b + 750 \text{ N}.$$

$$V_b = \frac{750}{12.5 - 7.5} = \underline{\underline{150 \text{ m}^3}}$$

② Experimental Method.

Let w be the weight of object and w_1 be the weight of small object.

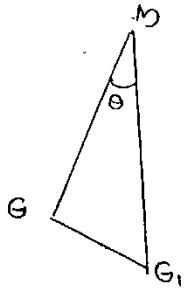


$$F_B \times GG_1 = w_1 \times x$$

$$w \times GG_1 = w_1 \times x.$$

$$w \times GM \times \tan\theta = w_1 \times x.$$

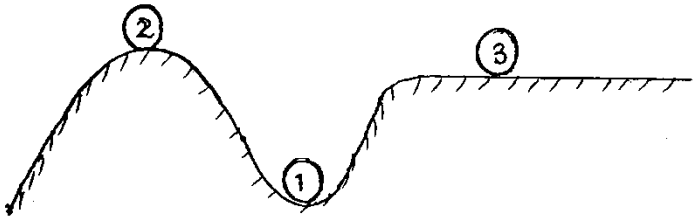
$$\therefore \boxed{GM = \frac{w_1 \times x}{w \tan\theta}}$$



$$\tan\theta = \frac{GG_1}{GM}$$

→ Conditions of Equilibrium

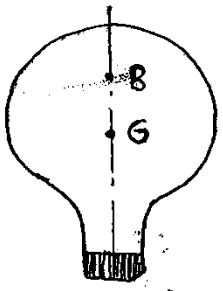
a) Submerged Bodies.

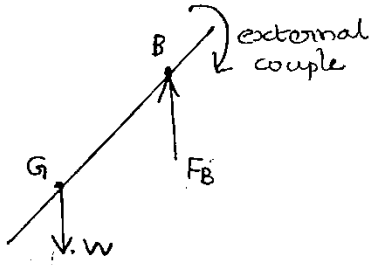


① → Stable Equilibrium

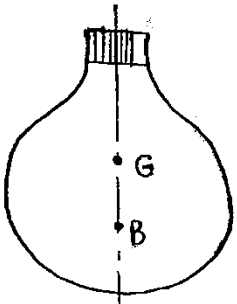
② → Unstable Equilibrium

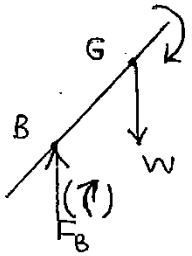
③ → Neutral Equilibrium.



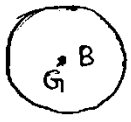


Stable Equilibrium.
(B is above G).



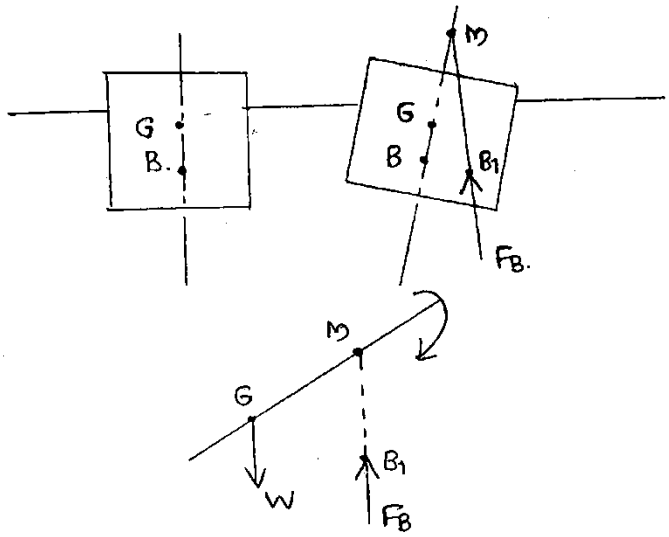


Unstable Equilibrium
(B is below G)

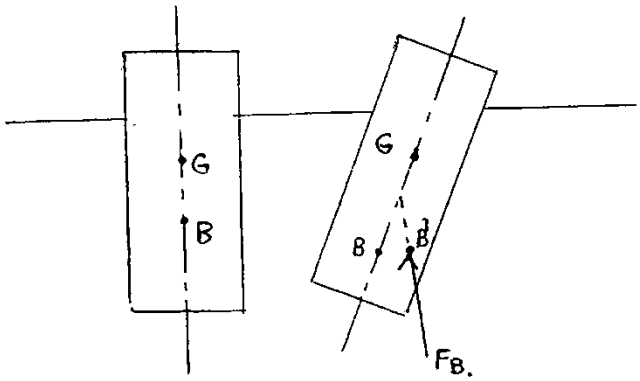


Neutral Equilibrium
(B coincides with G)

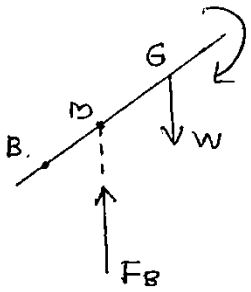
b) Floating Bodies



Stable Equilibrium
(M is above G)



Unstable Equilibrium
(M is below G)

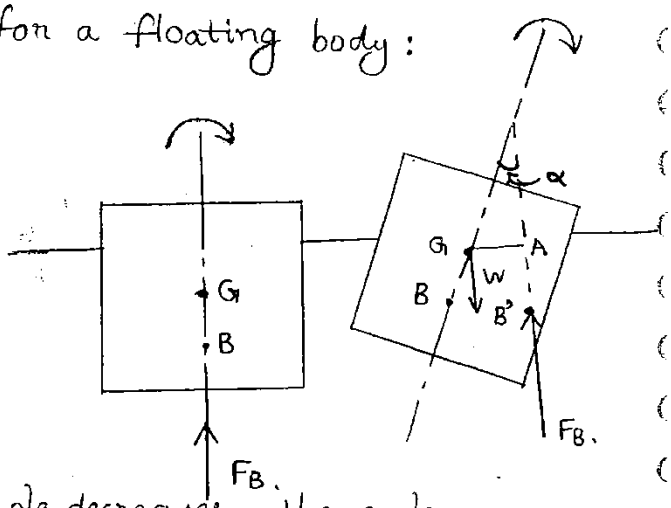


Neutral Equilibrium → M coincides with G

Time period of oscillation, $T = 2\pi \sqrt{\frac{K^2}{GM \times g}}$; GM → metacentric height.

When ship is loaded vertically as a stack, CG shifts upwards and chances of M coming below G are more, so for the safer side, GM is increased, but it cannot be increased randomly considering the comfort factors. $\left(T \propto \frac{1}{\sqrt{GM}}\right)$

→ Time period of Oscillation for a floating body:



Restoring couple $= F_B \times GA$

$= W \times GA$

$= W \times GM \times \sin\theta$

$= W \times GM \times \theta$.

$= W \times GM \times (-\theta)$

-ve sign indicates, restoring couple decreases the angle.

External couple $= I \times \alpha$

$= m \times k^2 \times \alpha$

$= \frac{W}{g} \times k^2 \times \frac{d^2\theta}{dt^2}$

Restoring couple = External couple.

$$-W \times GM \times \theta = \frac{W}{g} \times k^2 \times \frac{d^2\theta}{dt^2}$$

$$\frac{d^2\theta}{dt^2} = -\frac{GM \times g \times \theta}{k^2}.$$

$$\Rightarrow \boxed{\frac{d^2\theta}{dt^2} + \frac{GM\,g}{k^2}\theta = 0}$$

Soln. will be of form $\alpha \pm i\beta$.

Solution, $\theta = \left[C_1 \cos\sqrt{\frac{GM \times g}{k^2}}t + C_2 \sin\sqrt{\frac{GM.g}{k^2}}t\right]$ $\alpha = 0$, $\beta = \sqrt{\frac{GM\,g}{k^2}}$

Boundary Conditions:

(i) At $t = 0$; $\theta = 0$

(ii) At $t = \frac{T}{2}$; $\theta = 0$.

(i) $\Rightarrow$ $0 = C_1 + 0$ $\therefore C_1 = 0$.

(ii) $\Rightarrow$ $0 = C_2 \sin\left(\sqrt{\frac{GM.g}{k^2}} \times \frac{T}{2}\right)$

$$\sin \sqrt{\frac{GM.g}{k^2}} \cdot \frac{T}{2} = 0 = \sin n\pi \quad ; \quad n = 1, 2, 3 \ldots$$

$$\sqrt{\frac{GM.g}{k^2}} \cdot \frac{T}{2} = \pi .$$

$$\therefore \quad \boxed{T = 2\pi \sqrt{\frac{k^2}{GM.g}}} \quad ;$$

$k \rightarrow$ radius of gyration

$GM \rightarrow$ metacentric ht.

13th Aug,
WEDNESDAY

4. FLUID KINEMATICS

In the kinematics of fluid flow, the fluid characteristics like velocity, acceleration etc are studied without taking into consideration of the forces causing the motion.

The two methods by which the fluid flow problems can be analysed are:

(i) Lagrangian Method.

In this method, only a single fluid particle is considered for carrying out the study.

(ii) Eulerian Method

In this method, a point is considered in the fluid flow and study is carried out for those particles which are passing through given point. Only the Eulerian method is followed in all the fluid flow problems.

→ Types of Flow.

1. Steady & Unsteady flow

$$\frac{\partial P}{\partial t}_{(x,y,z)} = 0 \quad , \quad \frac{\partial V}{\partial t}_{(x,y,z)} = 0 \longrightarrow \text{Steady Flow}$$

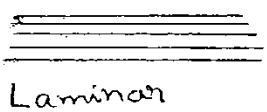
$$\frac{\partial P}{\partial t}_{(x,y,z)} \neq 0 \quad , \quad \frac{\partial V}{\partial t}_{(x,y,z)} \neq 0 \longrightarrow \text{Unsteady Flow}$$

2. Uniform & Non uniform flow.

$$\frac{\partial V}{\partial s} = 0 \longrightarrow \text{Uniform Flow}$$

$$\frac{\partial V}{\partial s} \neq 0 \longrightarrow \text{Non uniform Flow.}$$

3. Laminar & Turbulent flow.



Laminar

Turbulent.

Reynold's number, $R_e = \frac{\text{Inertia force, } F_i}{\text{Viscous force, } F_v}$

$F_i = m \times a$

$= m \times \frac{\text{velocity}}{\text{time}} = \frac{\rho \times \text{volume}}{t} \times \text{velocity}$

$= \rho \times Q \times \text{velocity} = \underline{\underline{\rho A V^2}}$

$F_v = \tau . A$

$= \mu \frac{du}{dy} \times A = \frac{\mu V A}{L}$

$\therefore Re = \frac{\rho A V^2}{\frac{\mu V A}{L}} = \frac{\rho V L}{\mu} \longrightarrow$ plate $= \frac{VL}{\gamma}$ $\left(\gamma = \frac{\mu}{\rho}\right)$

$= \frac{\rho V D}{\mu} \longrightarrow$ pipe $= \frac{VD}{\gamma}$

For a plate: (fluid is always a gas)

$Re < 5 \times 10^5 \longrightarrow$ Laminar

$5 \times 10^5 < Re < 10^7 \longrightarrow$ Transition

$Re > 10^7 \longrightarrow$ Turbulent

For the pipe:

$Re < 2000 \longrightarrow$ Laminar

$2000 < Re < 4000 \longrightarrow$ Transition

$Re > 4000 \longrightarrow$ Turbulent.

3. Compressible & Incompressible flow

$\partial \rho = 0 \longrightarrow$ Incompressible

$\partial \rho \neq 0 \longrightarrow$ Compressible.

4. 1D, 2D & 3D flows.

$u = f(x), \; v = 0, \; w = 0 \longrightarrow$ 1D

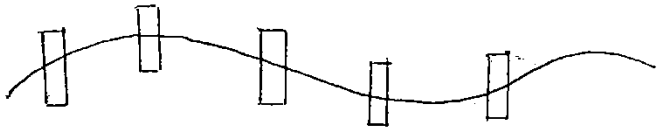
$u = f(x, y), \; v = f(x, y), \; w = 0 \longrightarrow$ 2D

$u = f(x, y, z), \; v = f(x, y, z), \; w = f(x, y, z) \longrightarrow$ 3D

5. Rotational & Irrotational Flows.

→ Rotational.

Fluid particles rotate about their mass centre.



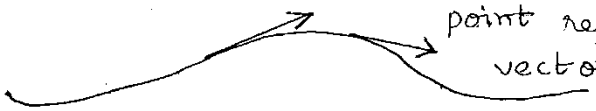
→ Irrotational

→ Rotational.

→ Types of Lines

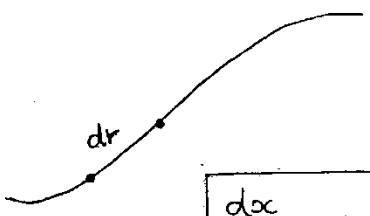
(i) Path Line. → path traced out by a fluid particle

(ii) Stream line → Imaginary line drawn in the flow field such that the tangent drawn at any point represents direction of velocity vector at that instant of time



NOTE:

- No flow takes place across the streamlines.
- No two streamlines cross each other.



$$\vec{dr} = dx\,\hat{i} + dy\,\hat{j} + dz\,\hat{k}$$

$$\vec{V} = u\,\hat{i} + v\,\hat{j} + w\,\hat{k}$$

$$\boxed{\frac{dx}{u} = \frac{dy}{v} = \frac{dz}{w}}$$ → Equation of Stream line

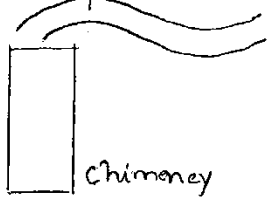
where u, v, w are velocity components along x, y & z directions

where V is the resultant velocity

(iii) Streakline

It is the locus or the path traced out by the fluid particles which have passed through the chosen point.

Eg: The path traced out by the chimney gases



NOTE:

⊙ For steady flow, the pathline, streamline and streakline are identical.

→ Velocity & Acceleration :-

Velocity vector, $\vec{V} = \vec{u}i + \vec{v}j + \vec{w}k$ where u, v, w are the velocity components along x, y & z direc

∴ Resultant velocity magnitude, $|V| = \sqrt{u^2+v^2+w^2}$

$u = f(x,y,z,t)$; $v = f(x,y,z,t)$, $w = f(x,y,z,t)$ ←

Acceleration along x-direction, $a_x = \frac{du}{dt}$

$$= \frac{\partial u}{\partial x}\cdot\frac{\partial x}{\partial t} + \frac{\partial u}{\partial y}\frac{\partial y}{\partial t} + \frac{\partial u}{\partial z}\cdot\frac{\partial z}{\partial t} + \frac{\partial u}{\partial t}\cdot\frac{\partial t}{\partial t}$$

$$\therefore \boxed{a_x = u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z} + \frac{\partial u}{\partial t}}$$

Similarly, $a_y = u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + w\frac{\partial v}{\partial z} + \frac{\partial v}{\partial t}$ ← local or temporal accele

$$a_z = u\frac{\partial w}{\partial x} + v\frac{\partial w}{\partial y} + w\frac{\partial w}{\partial z} + \frac{\partial w}{\partial t}$$

(first three terms: convective acceleratio)

∴ Acceleration vector, $\vec{a} = a_x\hat{i} + a_y\hat{j} + a_z\hat{k}$

Magnitude of resultant acceleration, $|a| = \sqrt{a_x^2 + a_y^2 + a_z^2}$

∴ Total Acceleration = convective + local.

Convective accl. is the acceleration produced because of change of velocity wrt change of space. $\left(\frac{\partial u}{\partial x}, \frac{\partial u}{\partial y}, \frac{\partial u}{\partial z}, \frac{\partial v}{\partial x}, \frac{\partial v}{\partial y}, \ldots\right)$

Local accl. is the acceleration produced because change of velocity wrt change of time $\left(\frac{\partial u}{\partial t}, \frac{\partial v}{\partial t} \text{ \& } \frac{\partial w}{\partial t}\right)$

Type of Flow	Convective	Local.
Steady & uniform	X	X
Steady & nonuniform	✓	X
Unsteady & uniform	X	✓
Unsteady & non-uniform	✓	✓

→ Discharge (Q)

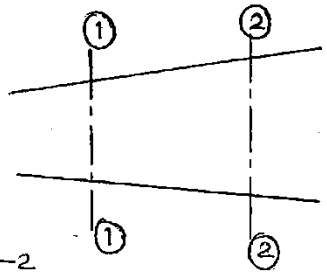
$$Q = \frac{\text{volume of fluid flowing}}{\text{time.}}$$

+ $\frac{L}{T}$

Unit: m^3/s

$$Q = \frac{A \times l}{t} = A \times \text{velocity}.$$

* Continuity Equation in 1-D :-



It is based on law of conservation of mass.

$$(\text{Mass flow rate})_{1-1} = (\text{mass flow rate})_{2-2}$$

$$\left(\frac{\text{mass}}{\text{time}}\right)_{1-1} = \left(\frac{\text{mass}}{\text{time}}\right)_{2-2}$$

$$\left(\frac{\text{Density} \times \text{volume}}{\text{time}}\right)_{1-1} = \left(\frac{\text{density} \times \text{volume}}{\text{time}}\right)_{2-2}$$

$$P_1 \times Q_1 = P_2 \times Q_2$$

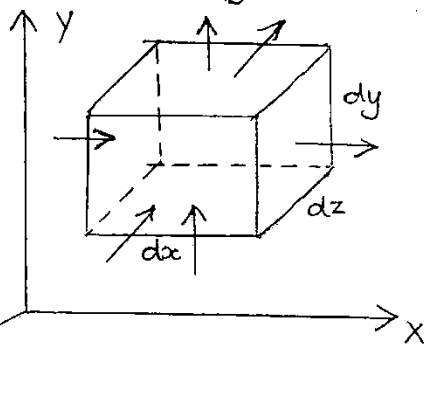
$$\boxed{P_1 A_1 V_1 = P_2 A_2 V_2}$$

For incompressible fluids, $P_1 = P_2$

$$\boxed{A_1 V_1 = A_2}$$

* Continuity Equation in Cartesian Co-ordinate System

$$\boxed{\frac{\partial \rho}{\partial t} + \frac{\partial \rho u}{\partial x} + \frac{\partial \rho v}{\partial y} + \frac{\partial \rho w}{\partial z} = 0}$$



Mass flow rate entering along x-direction $= \rho(dy\,dz)u$.

Mass flow rate leaving along x-direction $= \rho(dy\,dz)u + \left(\frac{\partial}{\partial x}(\rho u\, dy\, dz)\,dx\right)$ ← change of mass flow rate for a length of 'dx'

$=$

Mass flow rate stored $= \rho u\, dy dz - \left(\rho u\, dy dz + \frac{\partial}{\partial t}(\rho u dy dz\, dx)\right)$,

$$= -\frac{\partial}{\partial x}(\rho u\, dy\, dz)\, dx. \longrightarrow (1)$$

Mass flow rate stored because of flow taking place along y direction $= -\frac{\partial}{\partial y}(\rho v\, dx\, dz)\, dy. \longrightarrow (2)$

Mass flow rate stored because of flow taking place along z direction $= -\frac{\partial}{\partial z}(\rho z\, dx\, dy)\, dz. \longrightarrow (3)$

Total mass flow rate stored $= -\left(\frac{\partial}{\partial x}(\rho u\, dy dz)\, dx + \frac{\partial}{\partial y}(\rho v\, dx dz)\right.$
$\left. + \frac{\partial}{\partial z}(\rho z\, dx dy)\, dz\right) \longrightarrow (4)$

Change in mass of element per unit time $= \frac{\partial m}{\partial t}$

$$= \frac{\partial}{\partial t}(\rho dx.dy.dz). \hookrightarrow (5)$$

Equating (4) & (5),

$$-\left[\frac{\partial}{\partial x}(\rho u) + \frac{\partial}{\partial y}(\rho v) + \frac{\partial}{\partial z}(\rho w)\right] dx.dy.dz = \frac{\partial \rho}{\partial t}\, dx.dy.dz$$

$$\frac{\partial \rho}{\partial t} + \frac{\partial}{\partial x}(\rho u) + \frac{\partial}{\partial y}(\rho v) + \frac{\partial}{\partial z}(\rho w) = 0$$

For steady flow,

$$\frac{\partial \rho}{\partial t} = 0,$$

$$\frac{\partial}{\partial x}(\rho u) + \frac{\partial}{\partial y}(\rho v) + \frac{\partial}{\partial z}(\rho w) = 0$$

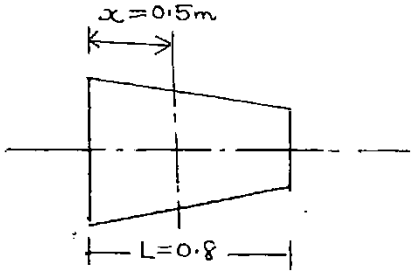
For steady, incompressible flow,

$$\rho\left(\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z}\right) = 0$$

$$\boxed{\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0}$$

P-61

2. $V = 2t\left(1 - \frac{x}{2L}\right)^2$



∂ Local acceleration $= \frac{\partial v}{\partial t}$

$$= \frac{\partial}{\partial t}\left(2t \cdot \left(1 - \frac{x}{2L}\right)^2\right)$$

$$= 2\left(1 - \frac{x}{2L}\right)^2 = 2\left(1 - \frac{0.5}{2 \times 0.8}\right)^2 = \underline{0.945\ m/s}$$

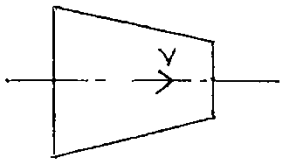
3. Convective acceleration $= a_x = u\frac{du}{dx} + v\frac{du}{dy}^{0} + w\frac{du}{dz}^{0}$ (flow in x direction)

$$= 2t\left(1 - \frac{x}{2L}\right)^2 \; 2t \times 2\left(1 - \frac{x}{2L}\right) \times \frac{-1}{2L}$$

$$= 2\times3\left(1 - \frac{0.5}{1.6}\right)^2 \times 2\times3\times2\left(1 - \frac{0.5}{1.6}\right) \times \frac{-1}{2\times2}$$

$$= \underline{-14.6\ m/s^2}$$

07. $A = 0.4 - 0.1x$



$\frac{dQ}{dt} = 0.12\ m^3/s$

Velocity, $V = \frac{Q}{A}$

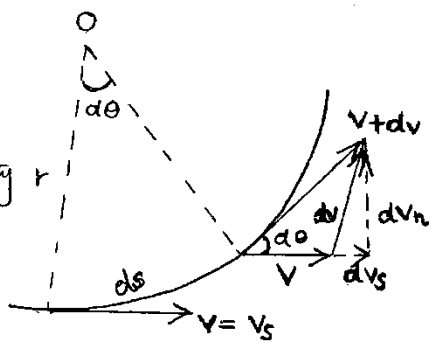
$\therefore$ Local acceleration $= \frac{dV}{dt} = \frac{d}{dt}\left(\frac{Q}{A}\right)$

$= \frac{1}{A}\frac{dQ}{dt}$

$= \frac{1}{0.4} \times 0.12 = \underline{\underline{0.3\ m/s^2}}$

Let 'V_s' be the tangential velocity

'V_n' be radial or normal velocity



$V_s = f(s, n, t)$

$V_n = f(s, n, t)$

Tangential acceleration, $a_s = \frac{dV_s}{dt}$

$= \frac{\partial V_s}{\partial s}\cdot\frac{\partial s}{\partial t} + \frac{\partial V_s}{\partial n}\cdot\frac{\partial n}{\partial t} + \frac{\partial V_s}{\partial t}\cdot\frac{\partial t}{\partial t}$ ... (1)

$$\boxed{a_s = V_s \cdot \frac{\partial V_s}{\partial s} + V_n \frac{\partial V_s}{\partial n} + \frac{\partial V_s}{\partial t}}$$

For steady flow $\frac{\partial V_s}{\partial t} = 0$

Normal or radial acceleration, $a_n = \frac{dV_n}{dt}$

$= \frac{\partial V_n}{\partial s}\cdot\frac{\partial s}{\partial t} + \frac{\partial V_n}{\partial n}\frac{\partial n}{\partial t} + \frac{\partial V_n}{\partial t}\frac{\partial t}{\partial t}$

$$\boxed{a_n = V_s \frac{\partial V_n}{\partial s} + V_n \frac{\partial V_n}{\partial n} + \frac{\partial V_n}{\partial t}}$$

For steady flow $\frac{\partial V_n}{\partial t} = 0$

For steady flow,

$a_s = V_s \frac{\partial V_s}{\partial s} + V_n \frac{\partial V_s}{\partial n}$ & $a_n = V_s \frac{\partial V_n}{\partial s} + V_n \frac{\partial V_n}{\partial n}$.

Since there is no flow across streamline, then $V_n = 0$

$$a_s = V_s \frac{\partial V_s}{\partial s} \quad \& \quad a_n = V_s \frac{\partial V_n}{\partial s}$$

$$ds = r\,d\theta \longrightarrow (1)$$

$$\tan d\theta = \frac{dV_n}{V} = d\theta \quad \text{(for small angles)} \longrightarrow (2)$$

From (1) & (2),

$$\frac{ds}{r} = \frac{dV_n}{V} = \frac{\partial V_n}{V_s} \qquad (\because V = V_s)$$

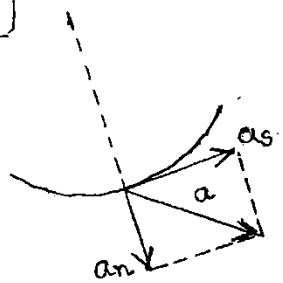
$$\frac{V_s}{r} = \frac{\partial V_n}{\partial s}$$

$$\therefore a_n = V_s \cdot \frac{\partial V_n}{\partial s}$$

$$= V_s \cdot \frac{V_s}{r}$$

Normal acceleration, $a_n = \frac{V_s^2}{r} = \frac{V^2}{r}$

$$a = \sqrt{a_s^2 + a_n^2}$$



NOTE:

- Tangential acceleration is because of change in the magnitude of the tangential velocity
- Normal or radial acceleration is because of the change in the direction of the tangential velocity.

4. $\frac{\partial V_s}{\partial s} = \frac{1}{3}$; $r = 9\,m$; $V_s = V = 3\ m/s$

$$a_n = \frac{V^2}{r} = \frac{3^2}{9} = 1\ m/s^2.$$

$$a_s = V_s \cdot \frac{\partial V_s}{\partial s} = 3 \times \frac{1}{3} = 1\ m/s^2.$$

$$a = \sqrt{a_n^2 + a_s^2} = \sqrt{1+1} = \underline{\underline{\sqrt{2}\ m/s^2}}$$

06. $r = 3m$; $V = 3\sin\theta$.

$$a_s = V_s \frac{\partial V_s}{\partial s}$$

But $\partial s = r\partial\theta$,

$$\therefore a_s = V_s \cdot \frac{\partial V_s}{r\partial\theta} = \frac{V_s}{r}\frac{\partial}{\partial\theta}(3\sin\theta)$$

$$= \frac{3\sin 45}{3} \times 3\cos 45$$

$$= \frac{3}{2} = 1.5\ m/s^2$$

→ Velocity Potential Function (ϕ)

$\phi = f(x,y,z,t) \Rightarrow u = -\frac{\partial\phi}{\partial x}$; $v = -\frac{\partial\phi}{\partial y}$; $w = -\frac{\partial\phi}{\partial z}$

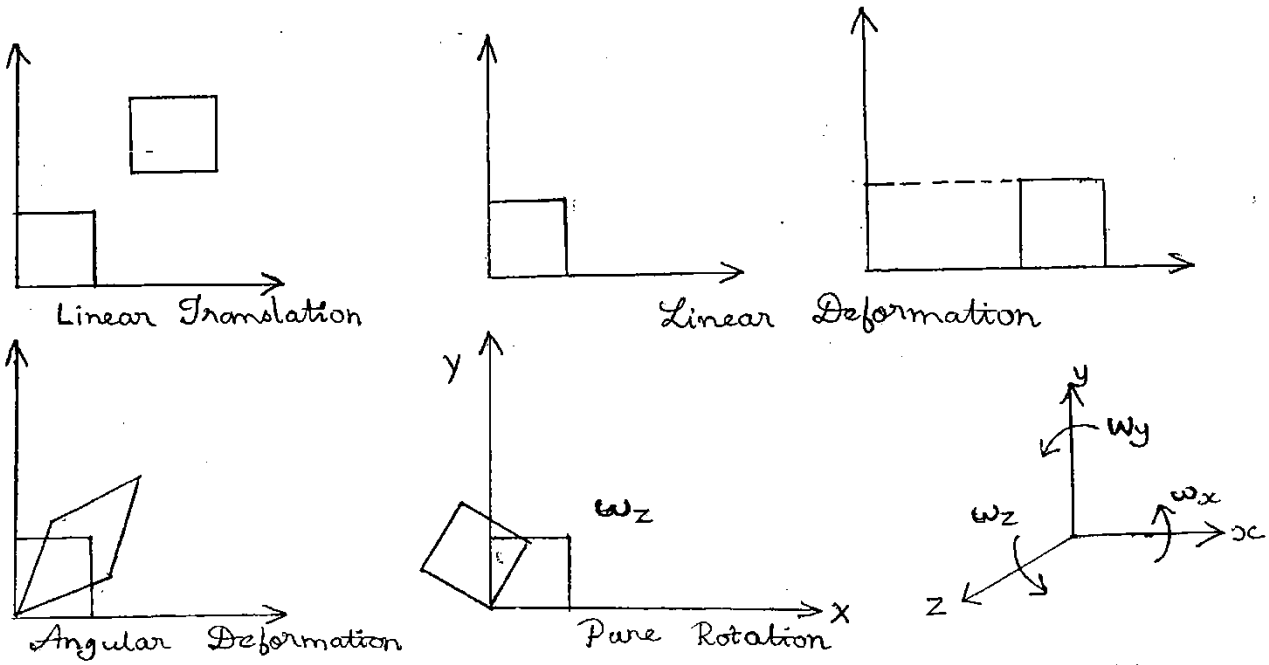
For steady incompressible flow,

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \quad \text{(continuity equation)}$$

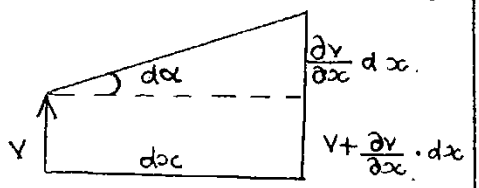
$$\frac{\partial}{\partial x}\left(-\frac{\partial\phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(-\frac{\partial\phi}{\partial y}\right) + \frac{\partial}{\partial z}\left(-\frac{\partial\phi}{\partial z}\right) = 0.$$

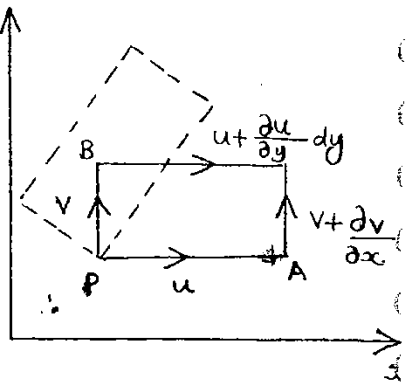
$$\boxed{\frac{\partial^2\phi}{\partial x^2} + \frac{\partial^2\phi}{\partial y^2} + \frac{\partial^2\phi}{\partial z^2} = 0} \rightarrow \text{Laplace Equation}$$

If ϕ exists for the flow, then it satisfies Laplace Equation



$$\tan d\alpha = \frac{\frac{\partial v}{\partial x} \cdot dx \cdot dt}{dx}$$





For small angles,

$\tan d\alpha = d\alpha$.

$$\frac{d\alpha}{dt} = \frac{\partial v}{\partial x}$$

Similarly,

$$\frac{d\beta}{dt} = \frac{-\partial u}{\partial y}$$ (-ve sign represents the element PB rotates in clockwise direction).

$$\omega_{PA} = \frac{d\alpha}{dt} = \frac{\partial v}{\partial x}; \quad \omega_{PB} = \frac{d\beta}{dt} = \frac{-\partial u}{\partial y}$$

Rotation about z-axis,

$$\omega_z = \frac{1}{2}(\omega_{PA} + \omega_{PB}).$$

$$= \frac{1}{2}\left(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}\right).$$

$$\omega_z = \frac{1}{2}\left(\frac{\partial}{\partial x}\left(\frac{-\partial \phi}{\partial y}\right) - \frac{\partial}{\partial y}\left(\frac{-\partial \phi}{\partial x}\right)\right)$$

$$= \frac{1}{2}\left(\frac{-\partial^2 \phi}{\partial x \partial y} + \frac{\partial^2 \phi}{\partial y \partial x}\right)$$

$$= 0 \quad \Rightarrow \quad \omega_z = \omega_y = \omega_z = 0$$

⊙ If the velocity potential fn exists, then the flow is irrotation

$$\omega = \frac{1}{2}\begin{vmatrix} + & - & + \\ i & j & k \\ \frac{\partial}{\partial x} & \frac{\partial}{\partial y} & \frac{\partial}{\partial z} \\ u & v & w \end{vmatrix}$$

$$\omega_z = \frac{1}{2}\left(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}\right)$$

$$\omega_y = -\frac{1}{2}\left(\frac{\partial w}{\partial x} - \frac{\partial u}{\partial z}\right) = \frac{1}{2}\left(\frac{\partial u}{\partial z} - \frac{\partial w}{\partial x}\right)$$

$$\omega_x = \frac{1}{2}\left(\frac{\partial w}{\partial y} - \frac{\partial v}{\partial z}\right)$$

→ Stream Function (ψ)

$$\psi = f(x, y, t).$$

$$v = \frac{\partial \psi}{\partial x} \quad ; \quad u = -\frac{\partial \psi}{\partial y}$$

⊙ Stream function exists only for 2-D flow.

For steady incompressible flow,

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0$$

$$\frac{\partial}{\partial x}\left(-\frac{\partial \psi}{\partial y}\right) + \frac{\partial}{\partial y}\left(\frac{\partial \psi}{\partial x}\right) = 0.$$

$$-\frac{\partial^2 \psi}{\partial x \partial y} + \frac{\partial^2 \psi}{\partial y \partial x} = 0 \quad \Rightarrow \psi \text{ satisfies continuity equa}$$

$$w_z = \frac{1}{2}\left(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}\right) = \frac{1}{2}\left(\frac{\partial}{\partial x}\left(\frac{\partial \psi}{\partial x}\right) - \frac{\partial}{\partial y}\left(-\frac{\partial \psi}{\partial y}\right)\right)$$

$$= \frac{1}{2}\left(\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2}\right)$$

If ψ satisfies Laplace equation, then $\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2} = 0$.

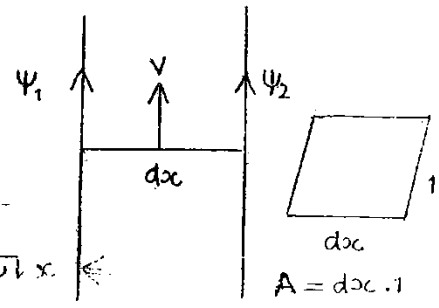
$\therefore w_z = 0. \Rightarrow$ flow is irrotational (not rotational).

Function	Continuity Equation	Laplace Equation	Rotational	Irrotatic
ϕ	✓	✓		✓
ψ	✓	✓		✓
		✗	✓	

$$Q = A \times V$$

$$= (dx \times 1) \times V$$

$$Q = V \cdot dx \quad \longrightarrow (1)$$



$$\frac{\partial \psi}{\partial x} = \frac{\partial \psi}{\partial x} \cdot \frac{\partial x}{\partial x} + \frac{\partial \psi}{\partial y} \cdot \frac{\partial y}{\partial x}$$

$$= \frac{\partial \psi}{\partial x} + \frac{\partial \psi}{\partial y} \cdot \frac{\partial y}{\partial x}$$

$$\partial \psi = \frac{\partial \psi}{\partial x} \cdot \partial x + \frac{\partial \psi}{\partial y} \cdot \partial y$$

$$\Rightarrow \partial \psi = v\, dx - u\, dy$$

Since flow taking place along y-direction,

$$\therefore u = 0$$

$$d\psi = v\, dx \longrightarrow (2)$$

From (1) & (2),

$$dQ = d\psi$$

or $$\boxed{dQ = \psi_2 - \psi_1}$$

NOTE:

⊙ The difference of magnitudes of the two stream fns will be equal to discharge per unit width.

01. $u = -x$, $v = 2y$.

$$u = -\frac{\partial \psi}{\partial y} \qquad v = \frac{\partial \psi}{\partial x}$$

$$\psi = +\int x\, dy$$

$$= |xy + c$$

$$\frac{dx}{u} = \frac{dy}{v} \Rightarrow \text{Equation stream line.}$$

$$\frac{dx}{-x} = \frac{dy}{2y}$$

Integrate on both sides,

$$-\int \frac{dx}{x} = \int \frac{dy}{2y}$$

$$-\log x = \frac{1}{2}\log y + \log c \qquad (1)$$

$$\Rightarrow x\sqrt{y} = c \qquad (1,1) \Rightarrow 1\sqrt{1} = c \Rightarrow x\sqrt{y} = 1$$

08. $\frac{\pi}{4} \times (5\times10^{-3})^2 \times 7.5\times10^{-3} = \frac{\pi}{4} \times (0.5\times10^{-3})^2 \times V_{blood} \quad (A_1V_1 = A_2V_2)$

$+ 20\times10^{-9}$

$V_{blood} = 0.65$ m/s

09. $\psi = x^2 - y^2$.

$$V = \sqrt{u^2+v^2}$$

$$a = \sqrt{a_x^2 + a_y^2}$$

$$a_x = u\cdot\frac{du}{dx} + v\frac{\partial u}{\partial y} + \omega\frac{\partial u}{\partial z}^{0} + \frac{\partial u}{\partial t}^{0}$$

$$a_y = u\cdot\frac{dv}{\partial x} + v\frac{\partial v}{\partial y}.$$

$$u = -\frac{\partial\psi}{\partial y} = 2y.$$

$$v = \frac{\partial\psi}{\partial x} = 2x.$$

$$\Rightarrow a_x = 2y\cdot 0 + 2x\cdot 2$$

$$= 4x.$$

$$a_y = 2y\cdot 2 + 2x\cdot 0$$

$$= 4y.$$

$$a = a_x\hat{i} + a_y\hat{j} = 4x\hat{i} + 4y\hat{j}$$

10. $\phi = x^2+y^2$

$$\frac{\partial\phi}{\partial x} = 2x \;;\; \frac{\partial^2\phi}{\partial x^2} = 2$$

$$\frac{\partial\phi}{\partial y} = 2y \;;\; \frac{\partial^2\phi}{\partial y^2} = 2$$

$$\frac{\partial^2\phi}{\partial x^2} + \frac{\partial^2\phi}{\partial y^2} = 4 \neq 0$$

$\phi = x^2 - y^2$

$$\frac{\partial^2\phi}{\partial x^2} = 2$$

$$\frac{\partial^2\phi}{\partial y^2} = -2$$

$$\frac{\partial^2\phi}{\partial x^2} + \frac{\partial^2\phi}{\partial y^2} = 0$$

11. $\psi = \frac{3}{2}(y^2 - x^2)$.

$Q = |\psi_2 - \psi_1|$.

$\psi_1 = \frac{3}{2}(3^2 - 1^2) = 12$.

$\psi_2 = \frac{3}{2}(3^2 - 3^2) = 0$

$\therefore Q = |0 - 12| = \underline{12 \text{ units}}$

12 $v = u\hat{i} + v\hat{j} = 3x\hat{i} + 4xy\hat{j}$.

$\omega_z = \frac{1}{2}\left(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}\right)$.

$= \frac{1}{2}(4y - 0) = \frac{1}{2} \times 4 \times 2 = \underline{4 \text{ rad/s}}$.

13. $\psi = x^2 - y^2$

$\frac{\partial \phi}{\partial x} = -u = -2$ $\qquad u = -\frac{\partial \psi}{\partial y} = +2y$

$\frac{\partial \phi}{\partial y} = -v$. $\qquad v = \frac{\partial \psi}{\partial x} = 2x$.

$\Rightarrow \frac{\partial \phi}{\partial x} = -2y$.

$\phi = -2yx + f(y)$ $\qquad \frac{\partial \phi}{\partial y} = -2x + f'(y)$.

~~At origin, $\phi = 0 \Rightarrow f(y) = 0$.~~ $\qquad -v = -2x + f'(y)$.

~~$\phi = -2yx$~~

$-\frac{\partial \psi}{\partial x} = -2x + f'(y)$ $\qquad \Rightarrow \phi = -2yx + c$.

$-2x = -2x + f'(y)$ $\qquad$ At $(0,0)$; $\phi = 0$,

$\therefore f'(y) = 0 \quad \therefore f(y) = c$. $\qquad 0 = 0 + c$

$\therefore \phi = \underline{-2xy}$

Lines of Constant velocity potential function:

$$\phi = c$$

$$d\phi = 0$$

But $\phi = f(x, y)$

$$d\phi = \frac{\partial \phi}{\partial x} dx + \frac{\partial \phi}{\partial y} . dy.$$

$$0 = -u dx - v dy.$$

$$\frac{dy}{dx} = \frac{-u}{v}$$

∴ Slope of constant velocity potential line is $\frac{-u}{v}$

Lines of Constant Stream Function:

$$\psi = c.$$

$$d\psi = 0$$

But $\psi = f(x, y)$

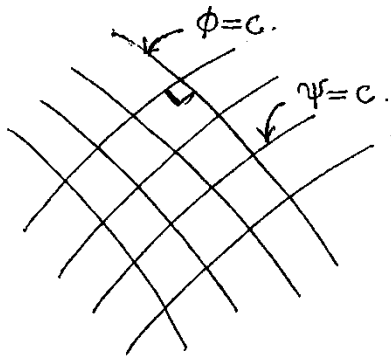
$$d\psi = \frac{\partial \psi}{\partial x} dx + \frac{\partial \psi}{\partial y} dy.$$

$$0 = v dx - u dy.$$

$$\Rightarrow \frac{dy}{dx} = \frac{v}{u}$$

∴ slope of constant stream function line is $\frac{v}{u}$.

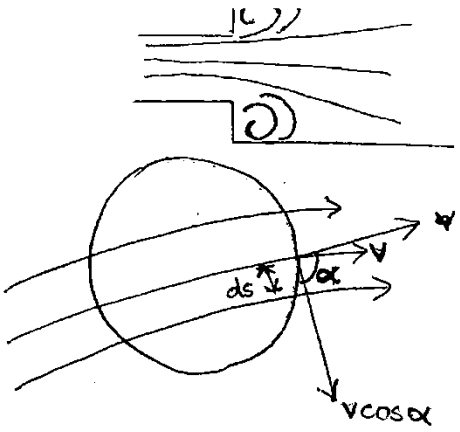
Product of slopes $= \frac{-u}{v} \times \frac{v}{u} = \underline{\underline{-1}}$



Flow net.

The line of constant velocity potential fn (ϕ) and line of constant stream function are orthogonal to each other.

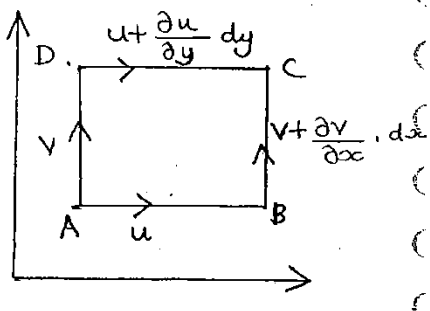
Circulation & Vorticity :



Circulation is defined as the flow of the fluid along a closed curve.

Mathematically, it is defined as the line integral of the tangential component of the velocity vector taken around a closed curve.

$$\text{Circulation, } T = \int_C V\cos\alpha \cdot ds$$



Eg:-

$$T_{ABCD} = u\,dx + \left(V + \frac{\partial V}{\partial x}dx\right)dy - \left(u + \frac{\partial u}{\partial y}dy\right)dx - V\,dy.$$

$$= u\,dx + V\,dy + \frac{\partial V}{\partial x}\cdot dx\cdot dy - u\,dx - \frac{\partial u}{\partial y}dx\cdot dy - V\,dy.$$

$$T_{ABCD} = \left(\frac{\partial V}{\partial x} - \frac{\partial u}{\partial y}\right)dx\cdot dy.$$

$$* \text{ Vorticity} = \frac{T}{A} = \frac{\left(\frac{\partial V}{\partial x} - \frac{\partial u}{\partial y}\right)dx\cdot dy}{dx\cdot dy} = \frac{\partial V}{\partial x} - \frac{\partial u}{\partial y}.$$

$$= 2\,\omega_z$$

$\therefore$ **Vorticity is twice the rotation**

Vortex Flow :

The flow of the fluid mass along a curved path is known as Vortex Flow.

If torque is required to rotate the fluid mass it is called Forced Vortex Flow. If no torque is required, then it is known as Free vortex Flow.

Rotating mass of fluid in a vertical cylinder (forced vortex flow)

In a free vortex flow, torque = 0

Torque = rate of change of angular momentum

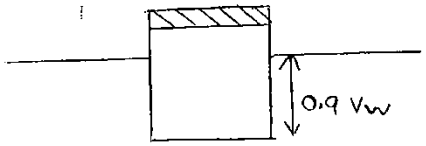
$$= \frac{\partial}{\partial t}(m \times v \times r)$$

$$\frac{\partial (mvr)}{\partial t} = 0.$$

$$mvr = c \Rightarrow \underline{\underline{vr = c}}$$

P-44

Q.18. F_B = weight of fluid displaced.



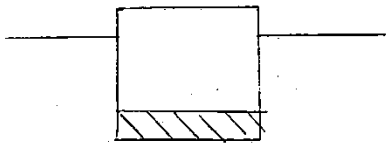
$$W = \rho_F \times g \times V_f.$$

$$mg = \rho_F \times g \times 0.9\, V_w$$

$$(m_w + m_l)g = 1000 \times 9.81 \times 0.9 \times \frac{m_w}{\rho_w}$$

$$(3.67 + m_l) = 1000 \times 0.9 \times \frac{3.67}{1000}$$

$$\Rightarrow m_l = 1.835\ kg$$



Q.19. $F_B = (F_B)_w + (F_B)_l.$

$$(m_w + m_l)g = (\rho_F \times g \times V)_w + (\rho_F \times g \times V)_l.$$

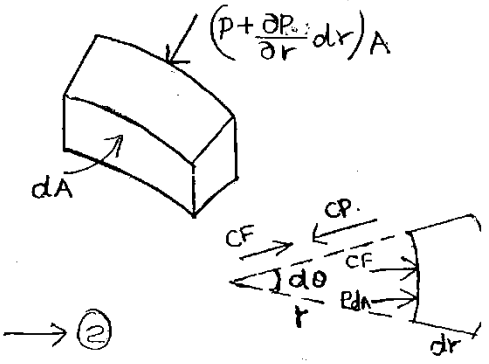
$$(3.67 + m_l)g = 1000 \times 9.81 \times 0.9\, V_w + 1000 \times 9.81 \times \frac{m_l}{\rho_l}$$

$$V_w = \frac{3.67}{600}$$

$$\underline{\underline{m_l = 2\ kg}}$$

* Equation of motion for Vortex flow:

Under equilibrium,



$$pdA - \left(p + \frac{\partial p}{\partial r} dr\right)dA + CF = 0$$

$$\therefore \quad CF = \frac{\partial p}{\partial r} dr\, dA \longrightarrow (1)$$

$$\text{But} \quad CF = \frac{mv^2}{r} = \frac{\rho dA \cdot dr \times v^2}{r} \longrightarrow (2)$$

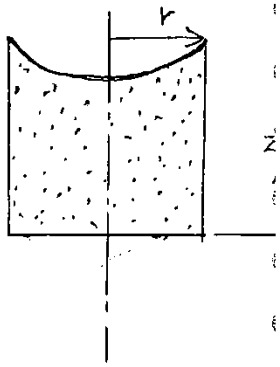
18TH Aug, MONDAY

From ① & ②,

$$\boxed{\frac{\partial p}{\partial r} = \frac{\rho v^2}{r}}$$

This equation represents the pressure gradient wrt. radius.

Also $\frac{\partial p}{\partial z} = -\rho g$.



$\therefore \quad p = f(r, z)$

$$dp = \frac{\partial p}{\partial r} dr + \frac{\partial p}{\partial z} . dz$$

$$\boxed{dp = \frac{\rho v^2}{r} dr - \rho g \, dz}$$

* Equation of motion for Forced Vortex Flow

We have $V = r\omega$

$$\therefore dp = \frac{\rho \times r^2 \times \omega^2}{r} dr - \rho g \, dz.$$

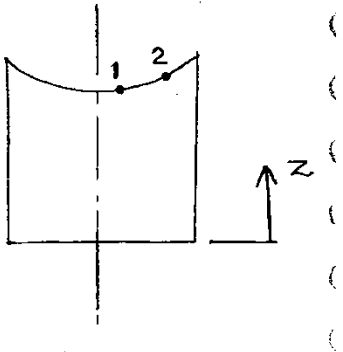
$$\int_1^2 dp = \int_1^2 \rho \omega^2 r \, dr - \int_1^2 \rho g \, dz$$

$$p_2 - p_1 = \left(\rho \omega^2 \frac{r^2}{2} \right)_1^2 - \rho g (z_2 - z_1)$$

$$= \frac{\rho \omega^2}{2} (r_2^2 - r_1^2) - \rho g (z_2 - z_1)$$

$$\Rightarrow \boxed{p_2 - p_1 = \frac{\rho}{2} (v_2^2 - v_1^2) - \rho g (z_2 - z_1)} \quad (\because v = r\omega)$$

⊙ If we consider two particles which are on the surface,



$$p_1 = p_2 = p_{atm}.$$

$$0 = \frac{\rho}{2} (v_2^2 - v_1^2) - \rho g (z_2 - z_1).$$

$$\Rightarrow \frac{v_2^2 - v_1^2}{2g} = z_2 - z_1$$

If point 1 is on the axis,

$$r_1 = 0 \Rightarrow r_1\omega = V_1 = \underline{\underline{0}}$$

$$\frac{V_2^2}{2g} = z_2 - z_1 \Rightarrow \frac{r_2^2\omega^2}{2g} = z_2 - z_1$$

Since z varies with square of radius, free surface of liquid in the forced vortex flow is parabolic

18th Aug,
MONDAY

DYNAMICS OF FLUID FLOW

Net force acting upon the fluid mass in the x direction is given by:

$$F_x = m \times a_x$$

where $m \longrightarrow$ mass of the fluid.

$a_x \longrightarrow$ acceleration in x direction.

The net force it comprises the following forces,

$$F_x = (F_g)_x + (F_p)_x + (F_t)_x + (F_v)_x + (F_c)_x$$

where $F_g \longrightarrow$ force due to gravity.

$F_p \longrightarrow$ force due to pressure.

$F_t \longrightarrow$ force due to turbulence.

$F_v \longrightarrow$ force due to viscosity.

$F_c \longrightarrow$ force due to compressibility.

If the forces due to gravity and pressure are taken into consideration in the fluid motion by neglecting all other forces, then the resulting equation of motion is called "Euler's Equation of Motion."

Assumptions:

① Fluid is inviscous ($F_v = 0$) $\Rightarrow$ ideal fluid

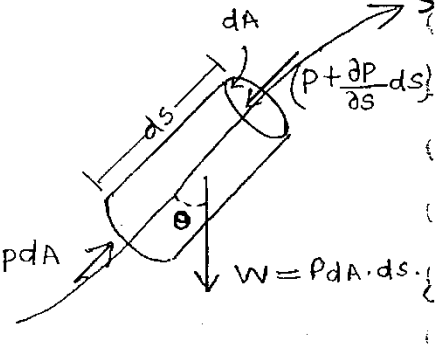
② Fluid is incompressible ($F_c = 0$)

③ Fluid is irrotational ($F_t = 0$) $\Rightarrow$ steady & laminar.

⊙ Consider a cylindrical fluid element in motion,

Net force acting on fluid mass

$= m \cdot a_s \quad (F_x = ma_x)$



$$pdA - \left(p + \frac{\partial p}{\partial s} ds\right) dA - w\cos\theta = m \times a_s$$

$$-\frac{\partial p}{\partial s} ds \cdot dA - \rho dA \cdot ds \cdot g\cos\theta = \rho dA \cdot ds \cdot a_s$$

$a_s = \frac{dv}{dt}$; $v = f(s,t)$.

$$\frac{dv}{dt} = \frac{\partial v}{\partial s} \cdot \frac{ds}{dt} + \frac{\partial v}{\partial t} \cdot \frac{dt}{dt}$$

$$a_s = v \cdot \frac{\partial v}{\partial s} + \frac{\partial v}{\partial t}$$

If flow is steady, then $\frac{\partial v}{\partial t} = 0$.

$$\therefore a_s = v \cdot \frac{\partial v}{\partial s}$$

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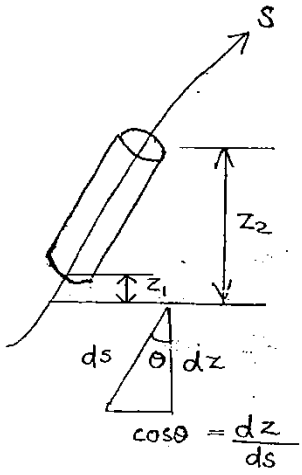
$$\therefore -\frac{\partial p}{\partial s} \cdot ds \cdot dA - \rho dA \cdot ds \cdot g\cos\theta = \rho dA \cdot ds \cdot v \frac{\partial v}{\partial s}$$

$$-\frac{\partial p}{\partial s} - \rho g\cos\theta = \rho v \frac{\partial v}{\partial s}$$

$$-\frac{\partial p}{\partial s} - \rho g \frac{\partial z}{\partial s} = \rho v \frac{\partial v}{\partial s}$$

$$\partial p + \rho v \partial v + \rho g dz = 0$$



EULER'S EQUATION OF MOTION

$$\boxed{\frac{\partial p}{\rho} + v\partial v + g dz = 0}$$

* Bernoulli's Equation of Motion

Bernoulli's eqn of motion is obtained by integrati Euler's eqn of motion.

$$\int \frac{\partial p}{\rho} + \int v dv + \int g dz = 0$$

$$\frac{p}{\rho} + \frac{v^2}{2} + gz = C$$

or

$$\boxed{\frac{p}{\rho g} + \frac{v^2}{2g} + z = C}$$

For any two points ① & ②,

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2$$

Assumptions:

(i) Fluid is ideal.

(ii) Fluid is irrotional.

(iii) Fluid is incompressible.

(iv) Flow is steady.

$\frac{P}{\rho g}$ → pressure head.

$\frac{V^2}{2g}$ → kinetic (or) velocity head.

z → datum (or) elevation head.

Each term in the Bernoulli's equation represent energy per unit weight of the fluid.

$PE = mgz$

$PE/\text{unit wt} = \frac{mgz}{mg} = z$

$KE = \frac{1}{2}mv^2$

$KE/\text{unit wt} = \frac{\frac{1}{2}mv^2}{mg} = \frac{v^2}{2g}$

Units: $\frac{\text{Energy}}{\text{weight}} = \frac{Nm}{N} = m$

$$\boxed{p + \frac{\rho v^2}{2} + \rho g z = \text{constant}}$$

Each term represents 'energy per unit volume' of fluid.

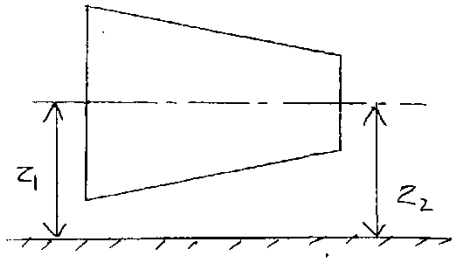
The term $\frac{\rho v^2}{2}$ is known as 'Dynamic Pressure'

⊙ For a real fluid,

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 + h_L$$

where h_L = head loss.

1. $\frac{P_1}{\rho g} = 10\,m$; $\frac{V_1^2}{2g} = 1\,m$; $d_1 = 100\,mm$.



$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2$$

$$10 + 1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g}$$

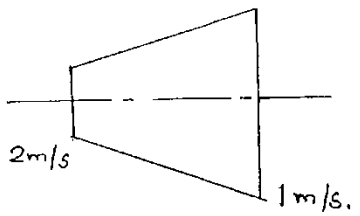
$$A_1 V_1 = A_2 V_2$$

$$V_2 = \frac{A_1 V_1}{A_2} = \frac{d_1^2 \times V_1}{d_2^2} = \frac{0.1^2 \times V_1}{0.05^2} = 4V_1.$$

$$11 = \frac{P_2}{\rho g} + \frac{16\,V_1^2}{2g}$$

$$\frac{P_2}{\rho g} = 11 - 16 \times 1 = \underline{-5\,m}$$

02. $z_1 = z_2$ (pipe is horizontal).



$$\frac{P_1}{\rho g}$$

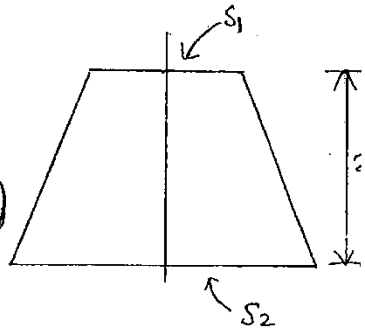
$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} = \frac{P_2}{\rho g} + \frac{V_2^2}{2g}.$$

$$\frac{P}{\rho g} + \frac{V^2}{2g} = C \qquad \boxed{V \uparrow,\ P \downarrow}$$

$$\frac{P_1 - P_2}{\rho g} = \frac{1^2}{2g} - \frac{2^2}{2g} = \frac{-3}{2g} = \underline{\frac{-1.5}{g}}$$

Difference in pressure heads $= \underline{\frac{1.5}{g}}$

03. $$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 + h_L.$$



$$2.5 + 1.27 + 2 = 5.407 + 0.203 + 0 + h_L \quad (\because z_1 = z_2 + 2)$$

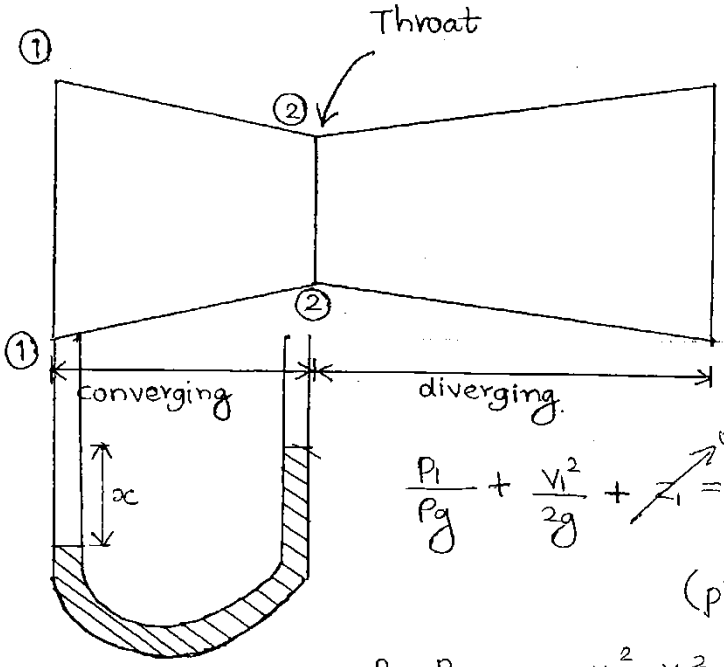
$$h_L = \underline{0.16\,m}$$

Flow is from a section at higher energy to that at lower energy. ie from S_1 to S_2

→ Applications of Bernoulli's Equation:

1. Venturimeter

It is used to measure the volume flow rate of fluid flowing in a pipe.



$A_2 < A_1$

$V_2 > V_1$

$P_2 < P_1$

$P_2 \not< P_v$ to avoid cavitation

Divergence angle should be very small and is about 5°-7° to avoid flow seperation.

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + \cancel{z_1}^{\,0} = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + \cancel{z}$$

(pipe is horizor

$$\frac{P_1 - P_2}{\rho g} = \frac{V_2^2 - V_1^2}{2g}$$

$$h = \frac{V_2^2 - V_1^2}{2g} \; ; \; h \rightarrow \text{difference of pressure head.}$$

$$V_2^2 - V_1^2 = 2gh. \longrightarrow (1)$$

$$a_1 V_1 = a_2 V_2$$

$$\therefore \; V_1 = \frac{a_2 V_2}{a_1} \longrightarrow (2)$$

From (1) & (2),

$$V_2^2 - \frac{a_2^2 V_2^2}{a_1^2} = 2gh$$

$$V_2^2 \left(\frac{a_1^2 - a_2^2}{a_1^2} \right) = 2gh$$

$$V_2^2 = \frac{2gh \cdot a_1^2}{a_1^2 - a_2^2}$$

$$\Rightarrow \; V_2 = \frac{a_1 \sqrt{2gh}}{\sqrt{a_1^2 - a_2^2}}$$

$$Q_{th} = \frac{a_1 \times a_2 \sqrt{2gh}}{\sqrt{a_1^2 - a_2^2}}$$

$h = \left(\frac{S_m}{S_L} - 1\right) x$; when manometric fluid is heavier than liquid flowing.

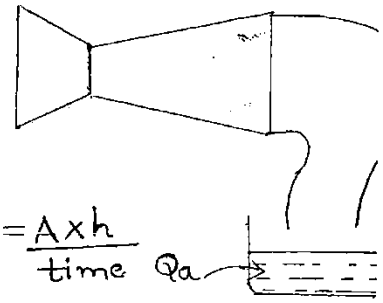
$= \left(1 - \frac{S_m}{S_L}\right) x$; when manometric fluid is lighter than liquid flowing.

$Q_a < Q_{th}$.

Coeffecient of Discharge, $C_d = \frac{Q_a}{Q_{th}}$

$\therefore Q_a = C_d \cdot Q_{th}$

$\Rightarrow Q_a = \frac{C_d \cdot a_1 \cdot a_2 \cdot \sqrt{2gh}}{\sqrt{a_1^2 - a_2^2}}$



$Q_a = \frac{A \times h}{\text{time}}$

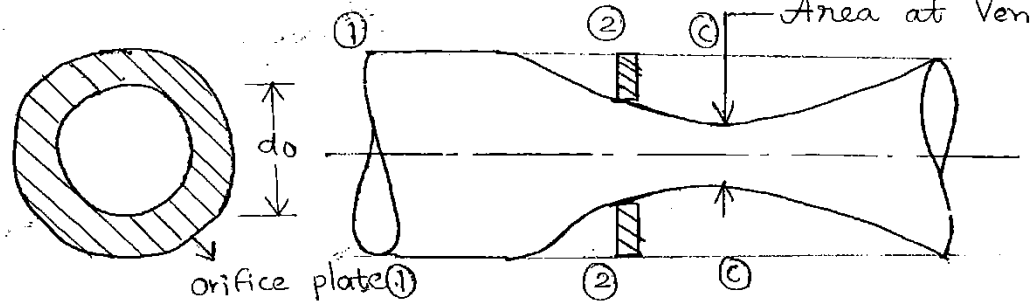
$C_d = 0.98$

time → time taken to rise 'h'

A → tank area

h → rise in tank.

2. Orificemeter



d_o → orifice diameter

a_o → orifice area.

Orifice meter is used to measure the volume flow rate.

$C_d = 0.62$

$$Q_a = \frac{C_d \cdot a_1 \cdot a_o \cdot \sqrt{2gh}}{\sqrt{a_1^2 - a_o^2}}$$

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3. Pitot Tube

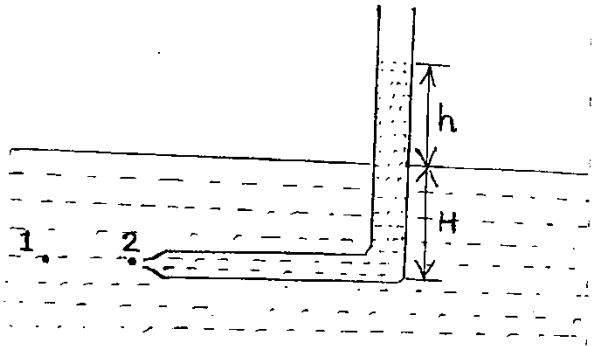
It is a device used to measure the velocity of flow.

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + \cancel{z_1}^{\,0} = \frac{P_2}{\rho g} + \cancel{\frac{V_2^2}{2g}}^{\,0} + \cancel{z_2}^{\,0}$$

$$\frac{P_1}{\rho g} = H \quad \& \quad \frac{P_2}{\rho g} = H + h.$$

$$H + \frac{V_1^2}{2g} = H + h + 0$$

$$\boxed{V_1 = \sqrt{2gh}}$$



2 - stagnation point ($V_2 = 0$

- measures the stagnatio pressure directly.

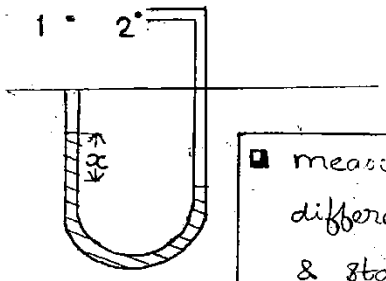
4. Pitot Static Tube.

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2$$

$$\frac{V_1^2}{2g} = \frac{P_2 - P_1}{\rho g} = x\left(\frac{S_m}{S_L} - 1\right)$$

$$V_1^2 = 2gx\left(\frac{S_m}{S_L} - 1\right)$$

$$\therefore \boxed{V_1 = \sqrt{2gx\left(\frac{S_m}{S_L} - 1\right)}}$$



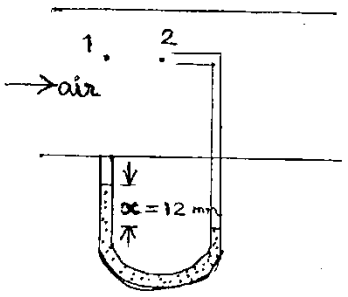
- measures the difference of sta & stagnation pre

$$\boxed{\begin{aligned} (V_1)_a &= C_v\sqrt{2gh} \\ (V_1)_a &= C_v\sqrt{2gx\left(\frac{S_m}{S_L} - 1\right)} \end{aligned}}$$

where $C_v \rightarrow$ coefficient of Pitot tube $\quad (V_a < V_{th})$

10. $C_v = 1$; $\rho_{air} = 1.2\ kg/m^3$

$$\frac{P_2 - P_1}{\rho g} = 12\ mm\ of\ H_2O = 12\left(\frac{S_{H_2O}}{S_{air}} - 1\right) \times 10^{-3}.$$



$$V_{air} = \sqrt{2gx}\sqrt{\frac{S_m}{S_L} - 1}$$

$$= \sqrt{2g \times 12 \times 10^{-3}\left(\frac{1}{1.2/1000} - 1\right)} = \underline{\underline{13.99\ m/s}}$$

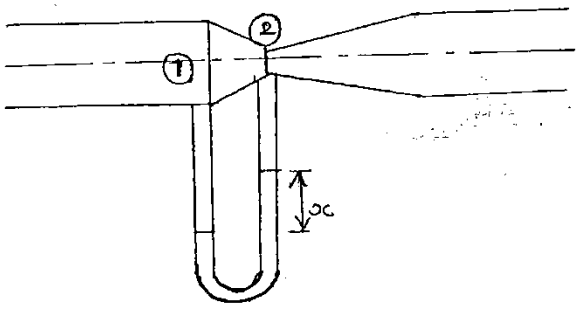
$$P_1 + \rho_{air} \times g \times y + \rho_{water} \times g \times x = P_2 + \rho_{air}(x+y)g.$$

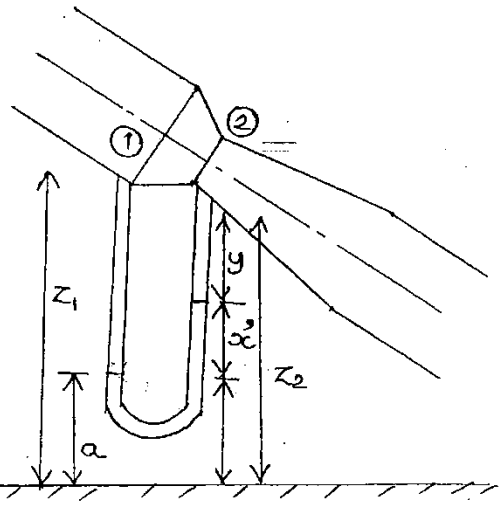
$$P_2 - P_1 = (\rho_{H_2O} - \rho_{air})g.x.$$

$$\frac{P_2 - P_1}{\rho_{air}\, g} = \frac{(\rho_{H_2O} - \rho_{air})gx}{\rho_{air} \times g.}$$

$$\frac{P_2 - P_1}{\rho_{air} \times g.} = \left(\frac{\rho_{H_2O}}{\rho_{air}} - 1\right)x.$$

9.





$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2$$

$$\frac{P_1 - P_2}{\rho g} = \frac{V_2^2 - V_1^2}{2g}.$$

$$h = \frac{P_1 - P_2}{\rho g} = x\left(\frac{S_m}{S_L} - 1\right)$$

Also $\frac{V_2^2 - V_1^2}{2g} = x\left(\frac{S_m}{S_L} - 1\right) \rightarrow ①$

$$P_1 + \rho_1 g(z_1 - a) = P_2 + \rho_1 g(z_2 - (x' + a)) + \rho_m g x'.$$

$$P_1 - P_2 = \rho_1 g z_2 - \rho_1 g x' + \rho_m g x' - \rho_1 g z_1$$

$$\left(\frac{P_1}{\rho_1 g} + z_1\right) - \left(\frac{P_2}{\rho_1 g} + z_2\right) = \left(\frac{\rho_m}{\rho_1} - 1\right)x'$$

$$= \left(\frac{S_m}{S_1} - 1\right)x'$$

$$\Rightarrow \frac{V_2^2 - V_1^2}{2g} = \left(\frac{S_m}{S_1} - 1\right)x' \rightarrow ②$$

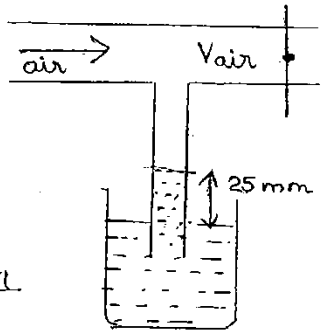
From ① & ②, $x = x'$.

∴ Manometer readings are independent of inclination of pipe. It depends only on rise of manometric fluid ($= x$).

∴ $h_1 = h_2 = h_3$

4. $\rho_{air} = 1.2\ kg/m^3$.

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g}^{0} + z_1^{0} = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2^{0} \quad (\text{not given})$$



$$\frac{\rho_{air} \cdot V_{air}^2}{2} = \rho_w g h_w$$

$$V_{air}^2 = \frac{2 \rho_w g . h_w}{\rho_{air}} = \frac{2 \times 1000 \times 25 \times 10^{-3} \times 9.81}{1.2}$$

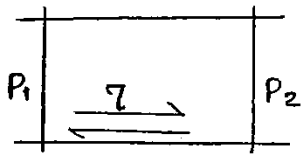
$$V_{air} = \underline{20.22}$$

8. Laminar (Viscous) Flow:

$$Re = \frac{\rho V D}{\mu} < 2000 \quad (\text{for pipe flow}).$$

when $\mu \uparrow$, $Re \downarrow$
$V \downarrow$, $Re \downarrow$
$\Rightarrow$ when a fluid has high viscosity and flowing at low velocities, then flow becomes laminar.

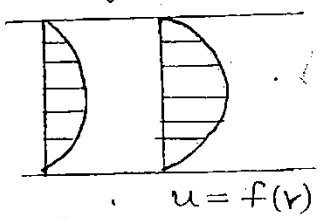
$P_2 < P_1$

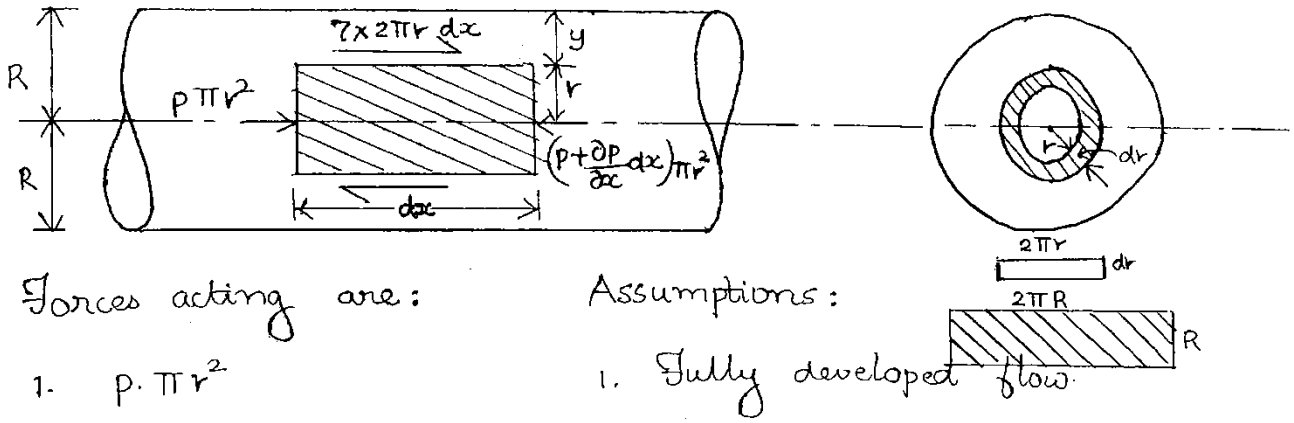


For a fully developed laminar flow,

$$u = f(r)$$

$$\frac{\partial p}{\partial x} = \text{constant}$$





Forces acting are:

1. $p \cdot \pi r^2$
2. $\left(p + \frac{\partial p}{\partial x} dx\right) \pi r^2$
3. $\tau \cdot 2\pi r \cdot dx$

Assumptions:

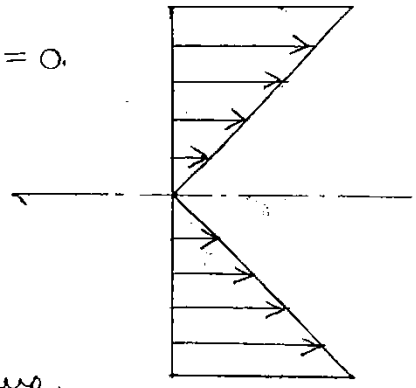
1. Fully developed flow
2. Steady and uniform ($a = 0$)

$$F_{net} = m \cdot a. = 0$$

$$p \times \pi r^2 - \left(p + \frac{\partial p}{\partial x} dx\right) \pi r^2 - \tau \times 2\pi r \times dx = 0.$$

$$\frac{\partial p}{\partial x} r = -2\tau$$

$$\boxed{\tau = -\frac{\partial p}{\partial x} \frac{r}{2}} \quad \rightarrow (1)$$



Shear Stress Distribution.

∴ Shear stress varies linearly with radius.

∴ At $r = 0$, $\tau = 0$.

$r = R$, $\tau = \tau_{max} = \tau_w$ (wall shear stress).

$y = R - r$

$dy = -dr$

$$\tau = \mu \frac{du}{dy} \quad \rightarrow (2)$$

$$= -\mu \frac{du}{dr}$$

From (1) & (2),

$$-\mu \frac{du}{dr} = \frac{-\partial p}{\partial x} \cdot \frac{r}{2}$$

$$du = \frac{1}{2\mu} \frac{\partial p}{\partial x} r \cdot dr$$

Integrating on both sides,

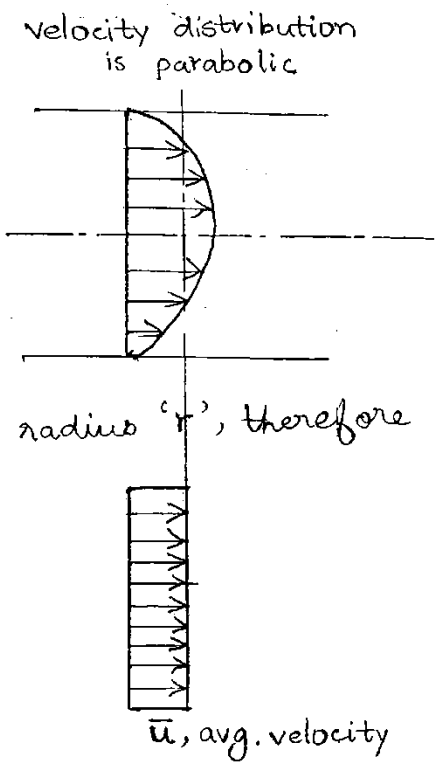
$$\boxed{u = \frac{1}{4\mu}\left(\frac{\partial P}{\partial x}\right) r^2 + C.}$$

At $r = R$, $u = 0$,

$$0 = \frac{1}{4\mu}\frac{\partial P}{\partial x}(R^2) + C.$$

$$\text{or} \quad C = -\frac{1}{4\mu}\frac{\partial P}{\partial x} R^2.$$

$$\therefore \boxed{u = \frac{-1}{4\mu}\frac{\partial P}{\partial x}(R^2 - r^2).}$$



Since velocity u varies with square of radius 'r', therefore velocity distribution is parabolic.

$$Q = A\,\bar{u}$$

$$\bar{u} = \frac{Q}{\pi R^2}$$

$$Q = \int_0^R dQ.$$

$$dQ = (2\pi r \cdot dr) \times u$$

$$= 2\pi r \cdot dr \left(\frac{-1}{4\mu}\frac{\partial P}{\partial x}(R^2 - r^2)\right)$$

$$\therefore Q = -\int_0^R \frac{2\pi}{4\mu}\left(\frac{\partial P}{\partial x}\right).(R^2 r - r^3)\,dr$$

$$= \frac{-\pi}{2\mu}\frac{\partial P}{\partial x}\left(R^2\frac{r^2}{2} - \frac{r^4}{4}\right)_0^R$$

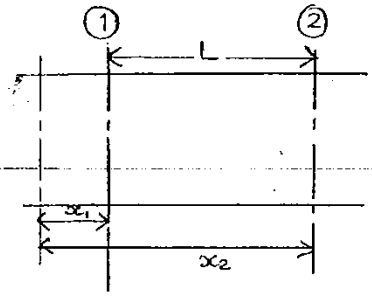
$$\boxed{Q = -\frac{\pi}{8\mu}\left(\frac{\partial P}{\partial x}\right) \cdot R^4}$$

$$\therefore \boxed{\bar{u} = \frac{Q}{\pi R^2} = -\frac{R^2}{8\mu}\left(\frac{\partial P}{\partial x}\right).}$$

Max. velocity (at $r=0$), $u_{max} = -\frac{1}{4\mu}\left(\frac{\partial P}{\partial x}\right) R^2$

$$\Rightarrow \boxed{\frac{u_{max}}{\bar{u}} = 2.}$$

Max. velocity which occurs at the centre is called as 'Centre Line Velocity' (u_{max})



$$\bar{u} = -\frac{1}{8\mu}\left(\frac{\partial P}{\partial x}\right) R^2$$

$$\int_1^2 \partial P = \int_1^2 -\frac{8\mu\bar{u}}{R^2} \cdot dx.$$

$$P_2 - P_1 = -\frac{8\mu\bar{u}}{R^2}(x_2 - x_1).$$

$$\boxed{P_1 - P_2 = \frac{8\mu\bar{u}L}{R^2} = \frac{32\mu\bar{u}L}{D^2}}$$

Head loss, $h_L = \frac{P_1 - P_2}{\rho g}$

$$\boxed{h_L = \frac{32\mu\bar{u}L}{\rho g D^2}} \rightarrow \text{Haigen Poiseuille Equation}$$

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2.01 $Q = ?$ $u_{max} = 1.5$ m/s ; $d = 40$ mm

$$Q = A \times \bar{u}$$

$$= \frac{\pi}{4} d^2 \times \frac{1.5}{2} = \frac{3\pi}{10000} \text{ m}^3/\text{s}$$

02. $d = 15$ cm $= 0.15$m ; $P_1 - P_2 = 100$ kPa ; $L = 10$ m.

$$\tau_w = -\frac{\partial P}{\partial x} \cdot \frac{r}{2} = \frac{100 \times 10^3}{10} \times \frac{0.15}{4} = 375 \text{ Pa}$$

3. $d = 0.1\ m,\ \tau = 250\ N/m^2,\ L = 10\ m.$

$$\tau = -\frac{\partial P}{\partial x}\frac{r}{2}$$

$$250 = \frac{\partial P}{10} \times \frac{0.05}{2}$$

$$\partial P = \underline{1 \times 10^5}\ N/m^2$$

04. $$\tau = \frac{\partial P}{\partial x}, \frac{r}{2}$$

$$= 2 \times 10^3 \times \frac{0.05}{4} = \underline{25\ Pa}$$

05. $Q = 800\ mm^3/s = 800 \times 10^{-9}\ m^3/s.$

$L = 2m,\ D = 0.5 \times 10^{-3}\ m,\ \frac{P_1 - P_2}{2} = 2 \times 10^6\ Pa.$

$$\bar{u} = \frac{Q}{\pi R^2} = \frac{800 \times 10^{-9}}{\pi \times \frac{(.05 \times 10^{-3})^2}{4}} = 4.07\ m/s.$$

$$\bar{u} = \frac{-R^2}{8\mu}\left(\frac{\partial P}{\partial x}\right).$$

$$\mu = \frac{\left(-0.5 \times 10^{-3}\right)^2/4}{8 \times 4.07}\left(\frac{2 \times 10^6}{2}\right) = 1.919 \times 10^{-3}\ NS/m^2.$$

06. $D = 0.2\ m,\ R = 0.1\ m.$

$$u_{max} = \frac{-1}{4\mu}\left(\frac{\partial P}{\partial x}\right) 0.1^2.$$

$$\frac{u_{max}}{u} = \frac{R^2}{R^2 - r^2}$$

$$1 = \frac{-1}{4\mu}\frac{\partial P}{\partial x} \times 0.1^2$$

$$u = \frac{-1}{4\mu}\frac{\partial P}{\partial x}\left(R^2 - r^2\right).$$

$$= \frac{1}{0.1^2}\left(0.1^2 - 0.05^2\right) = \underline{0.75\ m/s}$$

07 $d = 0.025\ m$, $\frac{\partial P}{\partial x} = 0.12\ kgf/cm^2 = \frac{0.12 \times 9.81}{10^{-4}} = 11772\ N/m^2$ (39)

$u = 2.34\ m/s.$

09 $\mu = 0.01\ poise = 0.01 \times 10^{-1}\ Pa\,s.$

$d = 0.01m$, $\bar{u} = 10\ mm/s = 0.01\ m/s.$

$$h_L = \frac{32 \mu \bar{u} L}{\rho g D^2}$$

$$h_L/1000 = \frac{32 \mu \bar{u}}{\rho g D^2}$$

$$\therefore h_L = \frac{32 \times 0.001 \times 0.01}{1000 \times 9.81 \times 0.01^2} = \underline{0.32\ m}$$

10. $h_L \propto \frac{1}{D^4}$

$$\frac{(h_L)_1}{(h_L)_2} = \frac{D_2^2}{D_1^2} = \frac{D_1^2}{\frac{D_1^2}{16}}$$

$$(h_L)_2 = \underline{16\,(h_L)_1}$$

$d_2 = \frac{d_1}{2}$; $Q_2 = Q_1$; $L_2 = L_1$

$$h_L = \frac{32 \mu \times \frac{Q}{A} \times L}{\rho g d^2}$$

$$\boxed{h_L = \frac{128 \mu Q L}{\rho g \times \pi \times d^4} \Rightarrow h_L \propto \frac{1}{d^4}}$$

12. $h_L = 2m$, $L = 10m$, $Re = 100$ (< 2000), $Q_2 = 2Q_1$

$Q = A \times \bar{u}$ (d is constant, Q↑ as $\bar{u}$ ↑).

$$h_L = \frac{128 \mu Q L}{\rho g \pi d^4} \qquad \rho, \mu$$

$h_L \propto Q$.

$$\frac{h_{L1}}{Q_1} = \frac{h_{L2}}{Q_2}$$

$$\frac{2}{Q_1} = \frac{h_{L2}}{2Q_1}$$

$$h_{L2} = 4m$$

13. $d = 0.02\,m$, $Re = 1600$ (laminar), $h_L = 0.2$, $Q = ?$, $\gamma = ?$

$$h_L = \frac{32 \mu \bar{u} L}{\rho g D^2}$$

$$\frac{h_L}{L} = \frac{32 \mu \bar{u}}{\rho g D^2}$$

$$= 32 \left(\frac{\mu}{\rho}\right) \frac{\bar{u}}{g d^2}$$

$$Re = \frac{\rho v d}{\mu}$$

$$Re = \frac{v d}{\gamma}$$

$$\Rightarrow \frac{\mu}{\rho} = \frac{\bar{u} d}{Re}$$

$$0.2 = 32 \times \left(\frac{\bar{u} \times d}{Re}\right)\left(\frac{\bar{u}}{g d^2}\right)$$

$$0.2 = \frac{32 \times \bar{u}^2}{1600 \times 9.81 \times 0.02}$$

$$\bar{u} = 0.0175 \quad 1.4\ m/s$$

$$Q = \frac{\pi}{4} D^2 \times \bar{u} = \frac{\pi}{4} \times 0.02^2 \times 1.4 = 4.4 \times 10^{-4}\ \text{cumecs}.$$

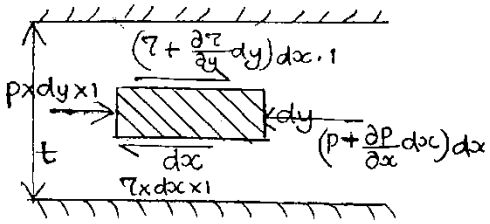
$$\gamma = \frac{\bar{u} d}{Re} = \frac{1.4 \times 0.02}{1600} = 1.75 \times 10^{-5}\ m^2/s = 1.75 \times 10^{-1}\ \text{stokes}$$

$$= 17.5\ \text{centi stokes}$$

1 stoke $= 10^{-4}\ m^2/s$

→ Flow between Two Parallel Plates

a) Both Plates are fixed



Consider unit width of plates,

Forces acting are:

1. $p \times dy \times 1$
2. $\left(p + \frac{\partial p}{\partial x} dx\right) dy \times 1$
3. $\tau \times dx \times 1$
4. $\left(\tau + \frac{\partial \tau}{\partial y} dy\right) dx \cdot 1.$

Flow is steady and uniform,

$$p \times dy \times 1 - \left(p + \frac{\partial p}{\partial x} dx\right) dy \times 1 - \tau \times dx \times 1 + \left(\tau + \frac{\partial \tau}{\partial y} \cdot dy\right) dx \cdot 1 = 0.$$

$$\frac{\partial \tau}{\partial y} = \frac{\partial p}{\partial x}.$$

For a laminar, $\tau = \mu \frac{du}{\partial y}.$

$$\frac{\partial}{\partial y}\left(\mu \frac{du}{dy}\right) = \frac{\partial p}{\partial x}.$$

$$\mu \frac{d^2u}{dy^2} = \frac{dP}{dx}$$

$$\Rightarrow \frac{\partial^2 u}{\partial y^2} = \frac{1}{\mu} \frac{\partial p}{\partial x}$$

Integrating on both sides,

$$\frac{\partial u}{\partial y} = \frac{1}{\mu} \frac{\partial p}{\partial x} \cdot y + C_1$$

Once again integrate,

$$u = \frac{1}{2\mu} \frac{\partial p}{\partial x} y^2 + C_1 y + C_2.$$

Applying boundary conditions,

(i) at $y = 0$; $u = 0$ &

(ii) at $y = t$; $u = 0$.

$$C_2 = 0 \quad ; \quad C_1 = \frac{-1}{2\mu}\frac{\partial P}{\partial x} \cdot t$$

$$\therefore \quad u = \frac{1}{2\mu}\frac{\partial P}{\partial x} y^2 + \frac{-1}{2\mu}\frac{\partial P}{\partial x} ty.$$

$$\boxed{u = \frac{-1}{2\mu}\left(\frac{\partial P}{\partial x}\right)(ty - y^2)}$$

Since u varies with square of y, therefore velocity distribution is parabolic.

$$Q = \int_0^t dQ.$$

$$dQ = (dy \times 1)u = -\frac{1}{2\mu}\left(\frac{\partial P}{\partial x}\right)(ty - y^2).dy.$$

$$Q = \int dQ = \frac{-1}{2\mu}\left(\frac{\partial P}{\partial x}\right)\left(\frac{ty^2}{2} - \frac{y^3}{3}\right)_0^t$$

$$\boxed{Q = \frac{-1}{12\mu}\left(\frac{\partial P}{\partial x}\right)t^3}$$

At $y = t$.

Average velocity, $\bar{u} = \frac{Q}{A} = \frac{\frac{-1}{12\mu}\left(\frac{\partial P}{\partial x}\right)t^3}{t \times 1} = \frac{-1}{12\mu}\left(\frac{\partial P}{\partial x}\right)t^2$

$$\boxed{\bar{u} = \frac{-1}{12\mu}\left(\frac{\partial P}{\partial x}\right)t^2}$$

At $y = t/2$, max. velocity occurs,

$$\boxed{u_{max} = \frac{-1}{8\mu}\left(\frac{\partial P}{\partial x}\right)t^2}$$

$$\boxed{\frac{u_{max}}{\bar{u}} = \frac{3}{2} = 1.5}$$

$$u = \frac{-1}{2\mu}\left(\frac{\partial P}{\partial x}\right)(ty - y^2)$$

At max. velocity,

$$\frac{du}{dy} = 0$$

$$\Rightarrow \quad t - 2y = 0 \text{ or } y =$$

$$\bar{u} = \frac{-1}{12\mu}\left(\frac{\partial P}{\partial x}\right) t^2$$

$$dp = \frac{-12\mu\bar{u}}{t^2} \cdot \partial x$$

$$\int_1^2 dp = -\int_1^2 \frac{12\mu\bar{u}}{t^2} \cdot \partial x$$

$$P_2 - P_1 = \frac{-12\mu\bar{u}}{t^2}(x_2 - x_1)$$

$$\boxed{P_1 - P_2 = \frac{12\mu\bar{u}L}{t^2}}$$

$$\boxed{\text{Head loss, } h_L = \frac{P_1 - P_2}{\rho g} = \frac{12\mu\bar{u}L}{\rho g t^2}}$$

11. $\rho = 880\ kg/m^3$, $\gamma = 7.4 \times 10^{-7}\ m^2/s$.

$\mu = \gamma\rho = 6.512 \times 10^{-4}\ Pa.s$.

$$\tau = \mu\frac{du}{dy} = 6.512 \times 10^{-4} \times 1000$$

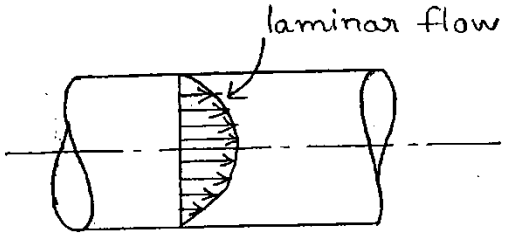
$$= 0.65\ Pa$$

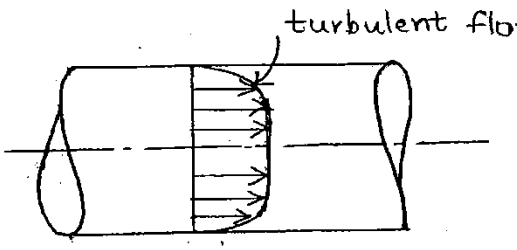
21 Aug, THURSDAY

→ Kinetic Energy Correction Factor (α)

α = $\frac{KE}{s}$ of fluid particles based on actual velocity / $\frac{KE}{s}$ of fluid particles based on average velocity.

$$\boxed{\alpha > 1}$$





$u \approx \bar{u} \Rightarrow \alpha \approx 1$

$$\frac{KE}{s} = \frac{1}{2}\left(\frac{m}{t}\right)v^2 = \frac{1}{2}\dot{m}v^2$$

KE/s based on actual velocity $= \int_0^R \frac{1}{2} P.dQ \times u^2$

$$= \int_0^R \frac{1}{2}\rho\,(2\pi r.dr \times u)\,u^2$$

$$= \pi\rho\left(\frac{-1}{64\mu^3}\right)\left(\frac{\partial P}{\partial x}\right)^3 \int r(R^2 - r^2)^3 dr$$

KE/s of fluid particles based on average velocity.

$$= \frac{1}{2} \rho Q . \bar{u}^2$$

$$= \frac{1}{2} \rho . A . \bar{u} . \bar{u}^2 = \frac{1}{2} \rho \pi R^2 \bar{u}^3$$

$$\alpha = \frac{\pi \rho \int_0^R r u^3 dr}{\frac{1}{2} \rho \pi R^2 \bar{u}^3} = \underline{\underline{2}} \quad \text{(for laminar flow)}$$

→ Momentum Correction Factor (α)

$$\beta = \frac{\text{momentum/s of fluid particles based on actual velo}}{\text{momentum/s of fluid particles based on average velo}}$$

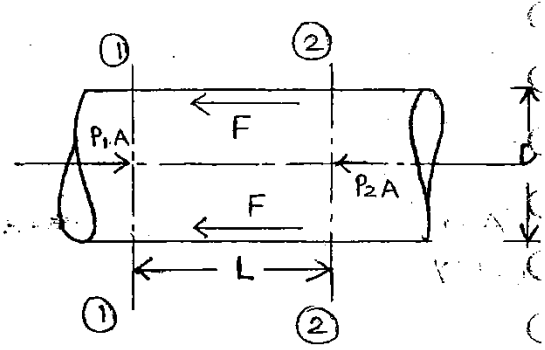
$$= \frac{4}{3} = 1.33$$

$\alpha = 2$
$\beta = 1.33$

→ Frictional Head Loss in a Pipe

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 + h_f$$

($V_1 = V_2$) ($V_1 = V_2$)



$$h_f = \frac{P_1 - P_2}{\rho g} \longrightarrow (1)$$

($V_1 = V_2$ as A is constan

Let F be the frictional force,

$$P_1 A_c - P_2 A_c - F = 0 \quad ; \quad A_c \rightarrow \text{C/s area.}$$

$$P_1 - P_2 = \frac{F}{A_c}$$

From experiments, William Froude derived the relationship,

$$F = f' A V^n \quad ; \quad n = 1.75 \text{ to } 2$$

$$A = \text{wetted area} = \pi D L$$

f' = frictional resistance per unit wetted area per unit velocit

$$\therefore F = f' (\pi D L) V^2$$

$$= f' (P . L) V^2 \quad ; \quad P \rightarrow \text{perimeter}$$

$$P_1 - P_2 = \frac{F}{A_C} = \frac{f'(\pi DL)\, v^2}{\pi/4\, D^2}$$

$$= \frac{4f'Lv^2}{D} \longrightarrow (2)$$

From (1) & (2),

$$h_f = \frac{P_1 - P_2}{\rho g} = \frac{4f'Lv^2}{\rho g D}$$

Let $\frac{f'}{\rho} = \frac{f}{2}$ ← coeffecient of friction.

$$\therefore \boxed{h_f = \frac{4fLv^2}{2gD}}$$

$4f = f \longrightarrow$ Friction factor.

$$\boxed{h_f = \frac{fLv^2}{2gD}}$$

DARCY'S WEISHBACH EQUATION.

Head loss in pipe flow, $h_L = \frac{32\mu vL}{\rho g D^2}$; v = avg. velocity.

$$h_f = h_L$$

$$\frac{fLv^2}{2gD} = \frac{32\mu vL}{\rho g D^2}$$

$$f = \frac{64}{\frac{\rho vD}{\mu}}$$

$$\therefore f = \frac{64}{Re.}$$

Coefficient of friction, $f = \frac{f}{4} = \frac{16}{Re.}$

Friction Factor $= \frac{64}{Re.}$
Coeffecient of friction $= \frac{16}{Re}$

$$\tau = -\frac{\partial P}{\partial x} \cdot \frac{r}{2}$$

$$\tau_\omega = -\frac{\partial P}{\partial x} \cdot \frac{R}{2} = -\frac{(P_2 - P_1)}{x_2 - x_1} \cdot \frac{R}{2}$$

$$= \left(\frac{P_1 - P_2}{L}\right) \cdot \frac{R}{2}$$

Wall shear stress, $\tau_\omega = \frac{(P_1 - P_2)D}{4L}$ $\longrightarrow$ ①

$$h_f = \frac{P_1 - P_2}{\rho g}$$

$$\therefore P_1 - P_2 = \rho g . h_f = \rho g . \frac{f L V^2}{2 g D}$$

$$= \rho \frac{f L V^2}{2D} \longrightarrow ②$$

From ① & ②,

$$\tau_\omega = \left(\frac{\rho f L V^2}{2D}\right) \cdot \frac{D}{4L}$$

$$= \frac{f V^2 . \rho}{8}$$

$$\sqrt{\frac{\tau_\omega}{\rho}} = \sqrt{\frac{f V^2}{8}}$$

Units of $\sqrt{\frac{\tau_\omega}{\rho}} = \sqrt{\frac{N/m^2}{kg/m^3}} = \sqrt{\frac{\frac{kg\,m}{s^2} \times \frac{1}{m^2}}{kg/m^3}} = \sqrt{\frac{m^2}{s^2}}$

$$= \frac{m}{s}$$

$$\therefore \sqrt{\frac{\tau_\omega}{\rho}} = V^* \text{ (Shear velocity)}$$

$$\boxed{V^* = \sqrt{\frac{f}{8}}\, V}$$

8. S.D = 0.8 ; μ = 1 poise = 10^{-1} PaS; D = 0.05 m ; V = 2 m/s.

$$\text{Friction factor} = \frac{64}{Re} = \frac{64}{\frac{\rho V D}{\mu}} = \frac{64}{\frac{0.8 \times 1000 \times 2 \times 0.05}{10^{-1}}}$$

$$= \underline{0.08}$$

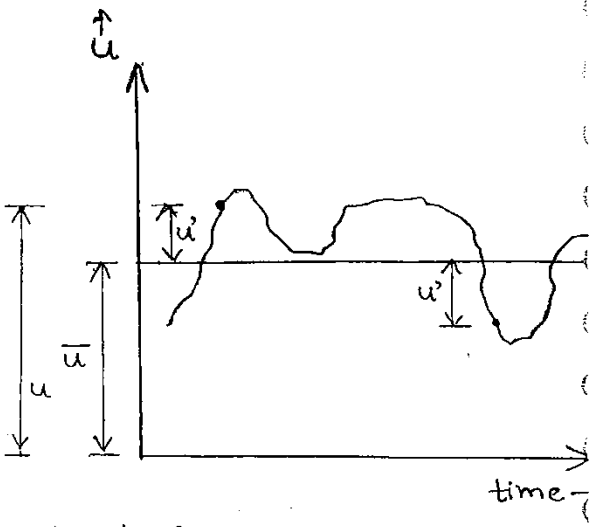
21st Aug,
THURSDAY

9. TURBULENT FLOW

$\bar{u}$ → average velocity.

u' → fluctuation component of velocity.

$$u = \bar{u} \pm u'$$



Similarly,

v' → fluctuation component of velocity in y-direction.

Total shear stress in turbulent flow $= \mu \frac{du}{dy} + \eta \frac{du}{dy}$

$$= \tau_v + \tau_t.$$

where η → eddy viscosity; it is very difficult to measure

Prandtle suggested, $\tau = \rho u' v'$ → Reynolds.

$$= \rho \bar{u}' \bar{v}'$$

l → Prandtle mixing length is the distance measured in the normal direction b/w the two fluid layers such that one lump of fluid particles can jump into the another fluid layer and there is no change of momentum of fluid particles along flow direction.

$l = ky$; k → von Karman constant $(= 0.4)$

$$u' = v' = l\left(\frac{du}{dy}\right)$$

$$\tau_t = \tau_{\text{turb}} = \rho u' v' = \rho l^2 \left(\frac{du}{dy}\right)^2$$

$$\boxed{\tau = \rho \times (0.4y)^2 \left(\frac{du}{dy}\right)^2}$$

$$\frac{\tau}{\rho} = (0.4y)^2 \left(\frac{du}{dy}\right)^2$$

$$\sqrt{\frac{\tau}{\rho}} = 0.4y \cdot \frac{du}{dy}$$

$$V_* = 0.4y \frac{du}{dy}$$

$$\Rightarrow \quad du = \frac{V_*}{0.4} \frac{dy}{y}$$

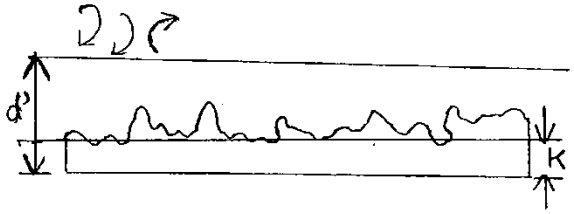
Integrating on both sides,

$$\boxed{u = 2.5\, V_* \ln y + C.}$$

∴ Velocity distribution in the case of turbulent flow is logarithmic.

→ Hydrodynamic Smooth & Rough Surfaces

Flow over a rough surface with protrusions (or irregularities)



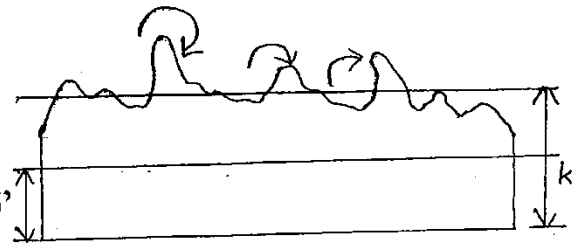
k → avg. height of protrusions.

δ' → laminar sublayer thickness.

δ' > k ⟶ smooth.

Laminar sublayer completely submerges the irregularities. All the turbelance will occur above this sublayer and turbulence will not touch the irregularities. So, a hydraulic smooth surface is formed.

δ' < k ⟶ rough surface.



$\frac{k}{\delta'} < 0.25$; k → Smooth

$\frac{k}{\delta'} \geq 6$; ⟶ Rough

$0.25 < \frac{k}{\delta'} < 6$; ⟶ Transition

$\frac{V_* k}{\gamma}$ → Reynold's Roughness number.

$\frac{V_* k}{\gamma} < 3$ → Smooth

$3 < \frac{V_* k}{\gamma} < 70$ → Transition

$\frac{V_* k}{\gamma} > 70$ → Rough

$$\boxed{\delta' = \frac{11.6\,\gamma}{V_*}}$$ From Nikuradses Experiment.

$$u = 2.5\, V_* \ln y + C$$

At $y = R$, $u = u_{max}$ (pipe) ; y is measured from wall.

$$u_{max} = 2.5\, V_* \ln R + C.$$

$$C = u_{max} - 2.5\, V_* \ln R.$$

$$\therefore \quad u = 2.5\, V_* \ln y + u_{max} - 2.5\, V_* \ln R$$

$$u - u_{max} = 2.5\, V_* \ln\left(\frac{y}{R}\right)$$

$$\boxed{\frac{u - u_{max}}{V_*} = 2.5 \ln\left(\frac{y}{R}\right)}$$

$$\frac{u_{max} - u}{V_*} = \frac{2.5 \log_{10}\left(\frac{R}{y}\right)}{\log_{10} e.} \quad ; \quad \log_b a = \frac{\log_x a}{\log_x b}$$

'PRANDTLE VELOCITY DEFECT EQUATION'

$$\boxed{\frac{u_{max} - u}{V_*} = 5.75 \log_{10}\left(\frac{R}{y}\right)}$$

At the pipe boundary, ie, at $y = 0$

$$u = 2.5\, V_* \ln 0 + C.$$

ie at $y = 0$; $u = -\infty$

if $a^x = N$

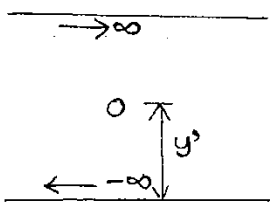
$x = \log_a N$

$\therefore \ln_e 0 = x$

$0 = e^x$

$\Rightarrow x = -\infty$

At $y = 0$, $u = -\infty$



ie at $y = y'$, u becomes 0.

$$0 = 2.5 V_* \ln y' + C.$$

$$C = -2.5 V_* \ln y'$$

$$u = 2.5 V_* \ln y - 2.5 V_* \ln y'$$

$$\frac{u}{V^*} = 2.5 \ln\left(\frac{y}{y'}\right)$$

$$\frac{u}{u^*} = 5.75 \log_{10}\left(\frac{y}{y'}\right) \rightarrow \text{valid for both smooth \& rough pipes.}$$

$y' = \frac{\delta'}{107}$ → smooth pipe.

$$\frac{u}{V^*} = 5.75 \log_{10}\left(\frac{y}{\delta'/107}\right)$$

$$= 5.75 \log_{10}\left(\frac{107 \times y}{\delta'}\right)$$

$$= 5.75 \log_{10}\left(\frac{107}{11.6} \times \frac{V_* y}{\gamma}\right)$$

$$= 5.75 \left(\log_{10}\left(\frac{107}{11.6}\right) + \log_{10}\left(\frac{V_* y}{\gamma}\right)\right)$$

$$\boxed{\frac{u}{V^*} = 5.55 + 5.75 \log_{10} \frac{V_* y}{\gamma}} \rightarrow \text{smooth pipes}$$

Similarly for rough pipes, $y' = \frac{k}{30}$

$$\boxed{\frac{u}{V^*} = 5.75 \log_{10}\left(\frac{y}{k}\right) + 8.5} \rightarrow \text{rough pipes}$$

Above equations gives the value of local velocity, u.

For mean velocity, $\bar{u}$

$$\boxed{\frac{\bar{u}}{V^*} = 1.75 + 5.75 \log_{10} \frac{V_* R}{\gamma}}$$

Smooth pipe

$$\boxed{\frac{\bar{u}}{V^*} = 4.75 + 5.75 \log_{10}\left(\frac{R}{k}\right)}$$

rough. pipe

$$\frac{u - \bar{u}}{V_*} = 5.75 \log_{10}\left(\frac{y}{R}\right) + 3.75$$ valid for Smooth & Rough pipes.

Q.1 Show that for a turbulent flow in a pipe, the ratio of u_{max} to $\bar{u}$ is $1.33\sqrt{f} + 1$.

$$\frac{u_0 - \bar{u}}{V_*} = 5.75 \log_{10}\left(\frac{y}{R}\right) + 3.75.$$ $\bar{u} = V$ = avg. velocity

At $y = R$, $u = u_{max}$,

$$\frac{u_{max} - V}{V_*} = 5.75 \log_{10}\left(\frac{R}{R}\right)^{0} + 3.75$$

$$\frac{u_{max} - V}{V_*} = 3.75.$$

$$V_* = \sqrt{\frac{f}{8}}\, V. \quad \text{(laminar flow)}.$$

$$\therefore \quad u_{max} - V = \left(3.75 \sqrt{\frac{f}{8}} \times V\right).$$

$$\frac{u_{max}}{V} - 1 = \frac{3.75\sqrt{f}}{\sqrt{8}}$$

$$\Rightarrow \frac{u_{max}}{V} = 1.33\sqrt{f} + 1 \quad ; \; f \rightarrow \text{friction factor}$$

$$\sqrt{\frac{\tau_w}{\rho}} = \sqrt{\frac{f}{8}}\, V$$

$$\tau_w = \frac{\rho f V^2}{8}$$

$$\therefore \tau_0 = \frac{\rho f V^2}{8}$$

Friction factor, $f = \frac{8\tau_0}{\rho V^2}$

Coefficient of Friction, $f = \frac{2\tau_0}{\rho V^2}$

Q.02. $V = 2$ m/s, $f = 0.02$

u_{max} = centre line velocity

$$\frac{u_{max}}{V} = \left(1.33\sqrt{f} + 1\right) = \left(1.33 \times \sqrt{0.02} + 1\right).$$

$$u_{max} = 2.376 \text{ m/s}$$

Q.03 diameter, $d = 0.3$ m, $Re = 10^6$, $f = 0.025$.

$$\delta = \frac{11.6\,\nu}{V_*}$$

$$V_* = \sqrt{\frac{f}{8}}\, V.$$

$$= \sqrt{\frac{0.025}{8}}\, V. = 0.056\, V.$$

$$Re = \frac{\rho V D}{\mu} = \frac{VD}{\gamma.}$$

$$\frac{V}{\gamma} = \frac{10^6}{0.3.}$$

$$\delta = \frac{11.6\,\gamma}{0.056V} = \frac{11.6 \times 0.3}{0.056 \times 10^6} = 6.21 \times 10^{-5}\ m.$$

$$= 0.062\ mm$$

4. $L = 100m$, $d = 0.1\ m$, $f = 0.02$, $h_L = 10\ m.$

$$\tau_\omega = \frac{\partial p}{\partial x}\frac{r}{2}$$

$$= \frac{\rho g h_L}{L} \times \frac{D}{4} = \frac{1000 \times 9.81 \times 10 \times 0.1}{100 \times 4.}$$

$$= 0.0245\ kPa$$

(OR)

$$\tau_\omega = \frac{f V^2 \rho}{8} \quad ; \quad h_L = \frac{f L V^2}{2gd.}$$

$$10 = \frac{0.02 \times 100 \times V^2}{2g \times 0.1}$$

$$\Rightarrow V^2 = 9.80665$$

$$\tau_\omega = \frac{0.02 \times 9.806 \times 1000}{8} = 24.516\ Pa = 0.024\ kPa$$

5. $k = 0.15\ mm$, $\tau = 4.9\ N/m^2$, $\nu = 0.01$ stoke

$$= 0.01 \times 10^{-4}\ m^2/s.$$

$$V_* = \sqrt{\frac{\tau_\omega}{\rho}} = \sqrt{\frac{4.9}{1000}} = 0.07\ m/s$$

$$\frac{V_* k}{\gamma} = \frac{0.07 \times 0.15 \times 10^{-3}}{0.01 \times 10^{-4}} = 10.5 \qquad 3 < \frac{V_* k}{\gamma} < 70$$

$\Rightarrow$ transition bound

(OR)

$$\delta' = \frac{11.6\,\nu}{V_*} = \frac{11.6 \times 10^{-6}}{0.07} = 0.166 \text{ mm.}$$

$$\frac{k}{\delta'} = \frac{0.15}{0.166} = 0.9.$$

$$0.25 < \frac{k}{\delta'} < 6 \implies \text{transition boundary.}$$

6. $d = 0.05$ m. $\qquad \frac{m}{t} = \pi$ kg/s

$\mu = 0.001$ Pas.

$\rho = 1000$.

$$\rho Q = \pi$$

$$Q = \frac{\pi}{\rho} \text{ m}^3/\text{s}$$

$$V = \frac{Q}{A} = \frac{\pi/\rho}{\pi R^2} = \frac{4}{1000 \times 0.05^2} = 1.6 \text{ m/s.}$$

$$Re = \frac{\rho V D}{\mu} = \frac{1000 \times 1.6 \times 0.05}{0.001} = 80000 > 2000 \implies \text{turbulent}$$

$$f_D = \frac{64}{Re} = \frac{64}{80000} = 8 \times 10^{-4} \qquad \frac{0.316}{(80000)^{1/4}} = 0.0187$$

$$h_f = \frac{f L V^2}{2 g D} = \frac{0.0188 \times 1 \times 1.6^2}{2 \times 9.81 \times 0.05} = 0.049 \text{ m}$$

Pressure drop per unit length $= \rho g h_f$

$= 0.049 \times 1000 \times 10$

$= \underline{\underline{480}}$ Pa/m

The friction factor in the case of laminar flow depends on Reynold's number. $\left(f = \frac{64}{Re}\right)$

whereas in the case of turbulence flow, friction factor depends on Reynolds number and avg. height of irregularities.

For smooth pipe :

$$f = \frac{0.316}{(Re)^{1/4}} \quad (Re < 10^5) \qquad f = \frac{0.032 + 0.221}{(Re)^{0.23}} \quad (10^5 < Re < 10^7$$

22nd Aug,
FRIDAY

FLOW THROUGH PIPES

→ Head Loss

(i) Major loss

occurs because of friction

$$h_f = \frac{fLV^2}{2gd}$$ (Darcy's Weishbach Equation)

$$f = \frac{64}{Re}$$ (Laminar Flow)

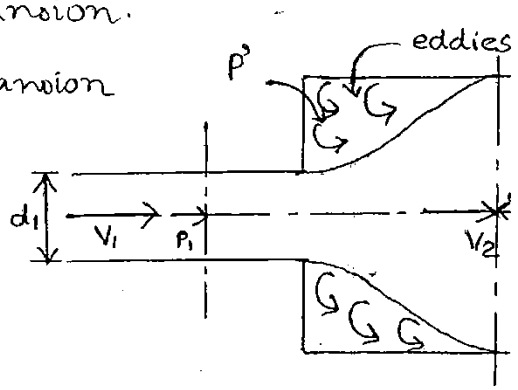
$$f = \frac{0.316}{(Re)^{1/4}}$$ (Turbulent flow in smooth pipe)

(ii) Minor Losses.

a) Sudden expansion of pipe.
b) Sudden contraction of pipe
c) Exit of the fluid from a pipe.
d) Inlet of the fluid to a pipe.
e) Pipe bend.
f) Obstruction in the pipeline.

→ Head loss due to sudden expansion.

Head loss due to sudden expansion is because of formation of eddies.



$$h_e = \frac{(V_1 - V_2)^2}{2g}$$

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + \cancel{z_1}^{0} = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + \cancel{z_2}^{0} + h_e$$

$$h_e = \frac{P_1 - P_2}{\rho g} + \frac{V_1^2 - V_2^2}{2g}$$

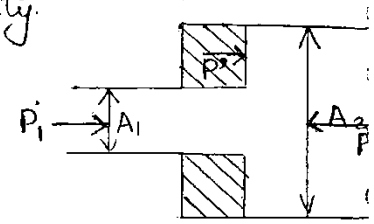
$A_1 < A_2 \Rightarrow V_1 > V_2$, then there is a acceleration.

$$\therefore F_{net} = m \times a$$

$$= m \times \frac{dv}{dt}$$

$$= \frac{\text{mass}}{\text{time}} \times \text{change of velocity}$$

$$= PQ\,(V_2 - V_1) \quad \longrightarrow (1)$$



But $F_{net} = P_1A_1 - P_2A_2 + P'(A_2 - A_1)$

From experiments, $p' = P_1$ (initial pressure)

$$\therefore F_{net} = P_1A_1 - P_2A_2 + P_1A_2 - P_1A_1$$

$$= (P_1 - P_2)\,A_2 \quad \longrightarrow (2)$$

Equating (1) & (2),

$$(P_1 - P_2)\,A_2 = PQ\,(V_2 - V_1)$$

$$P_1 - P_2 = PV_2\,(V_2 - V_1)$$

$$\frac{P_1 - P_2}{\rho g} = -\frac{V_1V_2}{g} + \frac{V_2^2}{g}$$

$$\therefore h_e = -\frac{V_1V_2}{g} + \frac{V_2^2}{g} + \frac{V_1^2 - V_2^2}{2g}$$

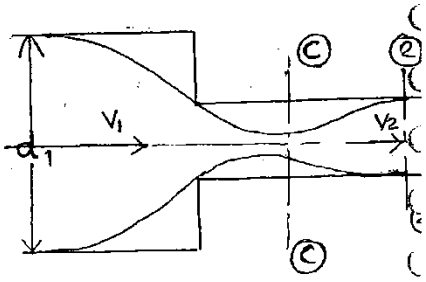
$$= \frac{V_2}{g}(V_2 - V_1) + \frac{(V_1 - V_2)(V_1 + V_2)}{2g}$$

$$= \frac{V_1 - V_2}{2g}\left(V_1 + V_2 - 2V_2\right) = \underline{\underline{\frac{(V_1 - V_2)^2}{2g}}}$$

→ Head loss due to sudden Contraction.

©–© : section at vena contraction

Head loss due to sudden contraction is because of expansion of fluid from area at vena contracta to small diameter pipe.



Let $V_c \rightarrow$ velocity of fluid at vena contracta.

$$\therefore \boxed{h_c = \frac{(V_c - V_2)^2}{2g}}$$

Coeffecient of contraction, $C_c = \frac{V_2}{V_c}$.

$$a_c V_c = a_2 V_2$$

Coeffecient of contraction, $C_c = \frac{a_c}{a_2} = \frac{V_2}{V_c}$.

$$h_c = \frac{V_2^2}{2g}\left(\frac{V_c}{V_2} - 1\right)^2$$

$$\boxed{h_c = \frac{V_2^2}{2g}\left(\frac{1}{C_c} - 1\right)^2}$$

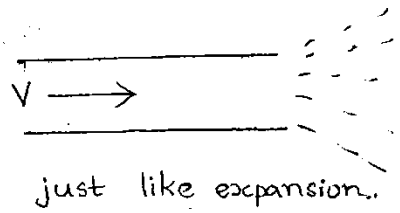
If C_c is not given, then,

$$\boxed{h_c = \frac{V_2^2}{2g}}$$

If C_c is given,

$$\boxed{h_c = \frac{0.5\, V_2^2}{2g}}$$

→ Head loss at the exit of Pipe.



just like expansion.

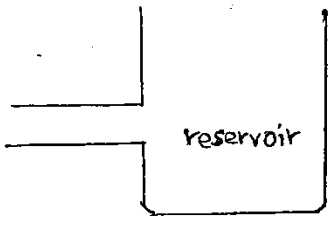
$$A_1 V_1 = A_2 V_2$$

$A_2 = \infty$; $V_2 = 0$.

$$h_o = \frac{(V_1 - V_2)^2}{2g}$$

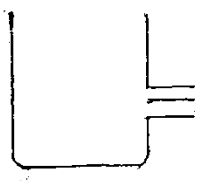
As $V_2 = 0$,

$$\boxed{h_o = \frac{V^2}{2g}}$$



→ Head loss at the inlet of the pipe

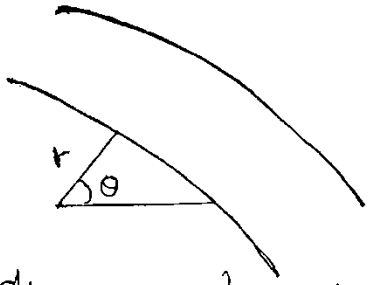
If C_c is being taken into consideration, then head loss due to sudden contraction, $h_c = 0.5 \frac{V_2^2}{2g}$.



$$\boxed{\text{Head loss at inlet of pipe, } h_i = 0.5 \frac{V_2^2}{2g}}$$

→ Head loss due to Pipe Bend.

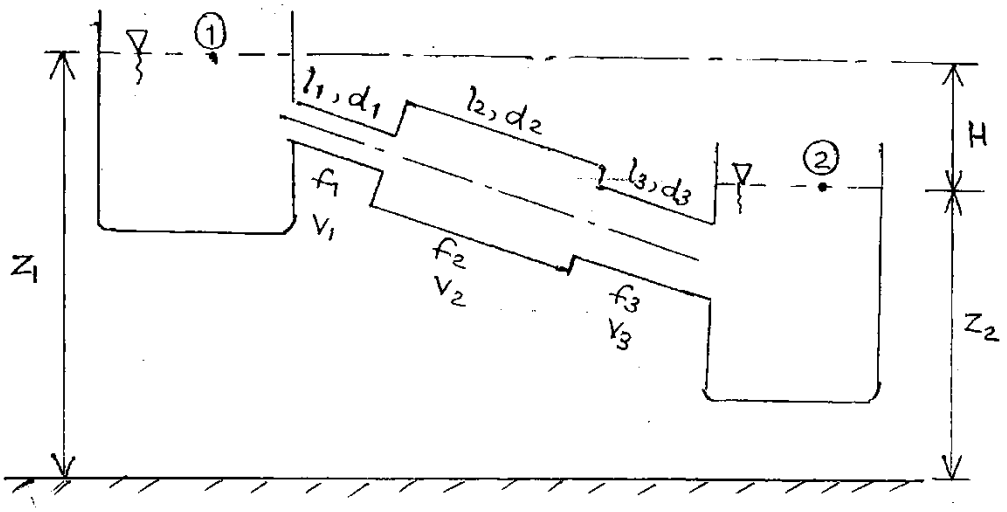
$$h_b = k \frac{V_b^2}{2g}$$



k → pipe bend coeffecient (depends on angle, radius)

→ Compound Pipe

(i) Pipes in Series



$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 + H_L$$

For large reservoirs, the velocity of fluid becomes zero.
($\because V_1 = V_2 = 0$).

$P_1 = P_2 = P_{atm}$

$$\therefore \quad h_L = z_1 - z_2$$

$$\boxed{h_L = H}$$

But h_L = major losses + minor losses.

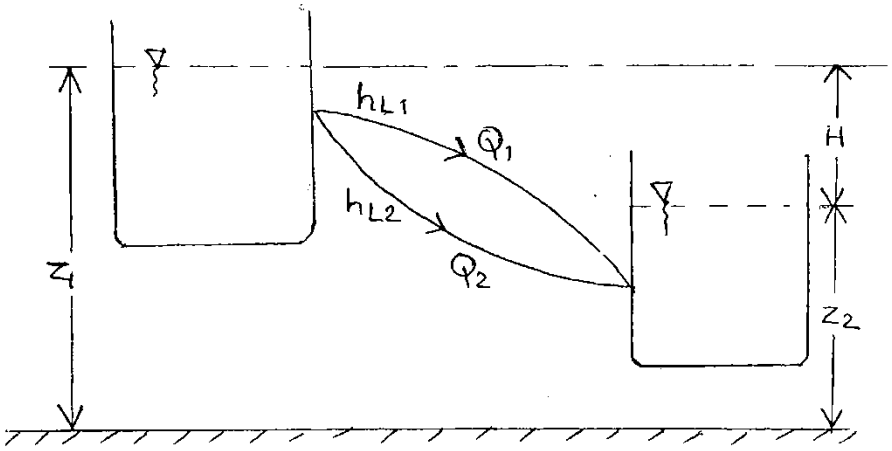
$$\therefore h_L = \underbrace{0.5 \frac{V_1^2}{2g}}_{(h_i)} + \underbrace{\frac{f_1 l_1 V_1^2}{2g d_1}}_{(h_f)} + \underbrace{\frac{(V_1 - V_2)^2}{2g}}_{(h_e)} + \underbrace{\frac{f_2 L_2 V_2^2}{2g d_2}}_{(h_f)}$$

$$+ \underbrace{0.5 \frac{V_3^2}{2g}}_{(h_c)} + \underbrace{\frac{f_3 l_3 V_3^2}{2g d_3}}_{(h_f)} + \underbrace{\frac{V_3^2}{2g}}_{(h_o)} = H$$

Neglect all minor losses,

$$H = \frac{f_1 l_1 V_1^2}{2g d_1} + \frac{f_2 l_2 V_2^2}{2g d_2} + \frac{f_3 l_3 V_3^2}{2g d_3}$$

(ii) Pipes in Parallel



$$Q = Q_1 + Q_2$$

$$h_{L1} = h_{L2}$$

If we consider major loss,

$$\frac{f_1 l_1 V_1^2}{2g d_1} = \frac{f_2 l_2 V_2^2}{2g d_2}$$

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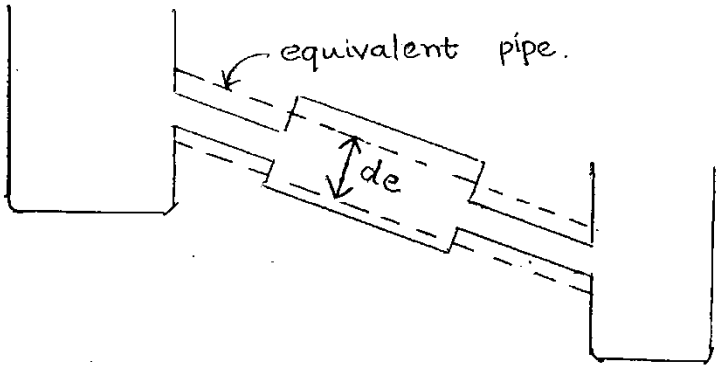
→ Equivalent Pipe

It is the pipe of uniform diameter that replaces a compound pipe such that it allows the same discharge and it maintains the same head loss.



Compound pipe requires couplings; which may result in leakages and demands regular maintenances. To avoid that equivalent pipes are used.

* Head loss in compound pipe

Assume $f_1 = f_2 = f_3 = f$.

Assume that equivalent pipe also have, f.

Head loss in the compound pipe,

$$h_L = \frac{f l_1 V_1^2}{2g d_1} + \frac{f l_2 V_2^2}{2g d_2} + \frac{f l_3 V_3^2}{2g d_3}$$

$$Q = A_1 V_1 = A_2 V_2 = A_3 V_3.$$

$$V_1 = \frac{Q}{A_1} = \frac{Q}{\frac{\pi}{4} d_1^2} = \frac{4Q}{\pi d_1^2}$$

$$\therefore V_1^2 = \frac{16 Q^2}{\pi^2 d_1^4}.$$

$$h_L = \frac{f l_1 \times 16 Q^2}{2g \times \pi^2 \times d_1^5} + \frac{f l_2 \times 16 Q^2}{2g \times \pi^2 \times d_2^5} + \frac{f l_3 \times 16 Q^2}{2g \pi^2 \times d_3^5} \longrightarrow (1)$$

Let $l_e \rightarrow$ length of equivalent pipe.

$$l_e = l_1 + l_2 + l_3.$$

$d_e \rightarrow$ diameter of equivalent pipe.

Head loss in equivalent pipe, $h_L = \frac{f l_e 16 Q^2}{2g \pi^2 \times d_e^5} \longrightarrow (2)$

$$(h_L)_{eq} = (h_L)_{comp.}$$

$$\frac{f l_e 16 Q^2}{2g \pi^2 d^5} = \frac{16 f Q^2}{2g \pi^2} \left(\frac{l_1}{d_1^5} + \frac{l_2}{d_2^5} + \frac{l_3}{d_3^5} \right)$$

DUPUIT'S EQUATION.

$$\boxed{\frac{l_e}{d_e^5} = \frac{l_1}{d_1^5} + \frac{l_2}{d_2^5} + \frac{l_3}{d_3^5}}$$

Dupuit's equation can be used to determine the equivalent diameter, de.

Q.01 $\gamma = 0.4$ stoke, $d = 8$ cm.

$Re = 2000$ (laminar)

$$2000 = \frac{\rho \times \bar{u}_{max} \times d}{\mu}$$

$$2000 = \frac{V \times 8 \times 10^{-2}}{0.4 \times 10^{-4}}$$

$$\bar{u} = V = 1 \text{ m/s}$$

$$u_{max} = 2\bar{u} = \underline{2 \text{ m/s}}$$

Q.02 $\gamma = 8 \text{ cm}^2/\text{s}$, $d = 8$ cm, $Q = 3200\pi \text{ cm}^3/\text{s}$.

$$V = \frac{Q}{A} = \frac{3200\pi}{\frac{\pi}{4} \times 8^2} = 200 \text{ cm/s}.$$

$$Re = \frac{\rho V d}{\mu} = \frac{200 \times 10^{-2} \times 8 \times 10^{-2}}{8 \times 10^{-4}} = 200 < 2000 \Rightarrow \underline{\text{laminar}}$$

Q.03 $h_L = \frac{fLV^2}{2gd} \Rightarrow h_L \propto \frac{1}{d^5}$.

$$\frac{(h_L)_A}{(h_L)_B} = \frac{(d_B)^5}{(1.2 d_B)^5} = \underline{0.402}$$

Q.04 $\frac{2l}{de^5} = \frac{l}{(0.1)^5} + \frac{l}{(0.2)^5}$ $\qquad l_e = l_1 + l_2 = l + l = \underline{2l}$

$$de = \underline{11.42 \text{ cm}}.$$

Q.05 For a parallel flow,

$$(h_L)_1 = (h_L)_2$$

$$\frac{f_1 L_1 V_1^2}{2g d_1} = \frac{f_2 L_2 V_2^2}{2g d_2}$$

$$\frac{f_1 L_1 Q_1^2}{2g d_1^5} = \frac{f_2 L_2 Q_2^2}{2g d_2^5}$$

Given $L_1 = L_2$; $d_1 = d_2$; $f_1 = 4 f_2$

$$\frac{4 f_2 \times L_1 \times Q_1^2}{2g d_1^5} = \frac{f_2 \times L_1 \times Q_2^2}{2g d_2^5}$$

$$\left(\frac{Q_1}{Q_2}\right)^2 = \frac{1}{4}.$$

$$Q_1 : Q_2 = 1 : 2$$

6. $$h_L = \frac{f L Q^2}{2g d^5}$$

$$Q^2 \propto d^5$$

$$\left(\frac{Q_1}{Q_2}\right)^2 = \left(\frac{2d}{d}\right)^5 = 32$$

$$\frac{Q_1}{Q_2} = 4\sqrt{2}$$

11. $d_2 = 2 d_1$.

$$h_e = \frac{(V_1 - V_2)^2}{2g}.$$

$$A_1 V_1 = A_2 V_2.$$

$$d_1^2 V_1 = d_2^2 V_2$$

$$V_2 = \left(\frac{d_1}{d_2}\right)^2 V_1.$$

$$= \frac{V_1}{4}.$$

$$h_e = \frac{\left(V_1 - \frac{V_1}{4}\right)^2}{2g} = \frac{V_1^2}{2g}\left(1 - \frac{1}{4}\right)^2.$$

$$\frac{h_e}{\frac{V_1^2}{2g}} = \left(1 - \frac{1}{4}\right)^2 = \frac{9}{16}$$

→ Equivalent Pipe for Pipes in Parallel.

$$Q = Q_1 + Q_2$$

If the two pipes have same diameter,

$$\Rightarrow \quad Q_1 = Q/2 \quad \& \quad Q_2 = Q/2$$

$$(h_L)_{eq} = (h_L)_{parallel}.$$

$$\frac{f l \times 16 Q^2}{2g \pi^2 \times de^5} = \frac{f l \times 16 \times (Q/2)^2}{2g \times \pi^2 \times d^5}$$

$$\frac{Q^2}{de^5} = \frac{Q^2}{4} \times \frac{1}{d^5}$$

$$de = (4)^{1/5} \times d.$$

NOTE:

If n pipes of equal diameter are connected in parall

$$\boxed{de = n^{2/5} d}$$

07. $de = 30\,cm$. ; $n = 2$.

$$d = \frac{de}{n^{2/5}} = \frac{30}{2^{2/5}} = 22.74 \text{ cm}$$

$d \approx 25$ cm

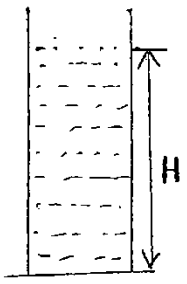
29th Aug
MONDAY

→ Power Transmission through Pipes

$$\text{Power} = \frac{\text{Work done}}{\text{Time}}$$

$$= \frac{\text{Force} \times \text{displacement}}{\text{Time}}$$

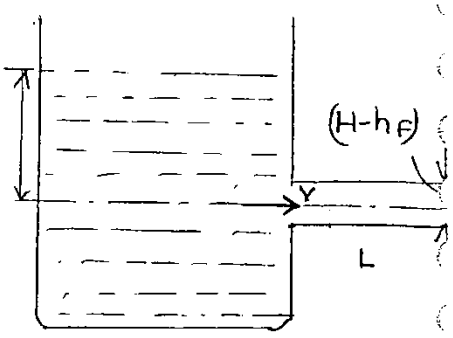
$$= \frac{\text{weight} \times H}{\text{Time}} = w \times \frac{\text{volume}}{\text{time}} \times H.$$



$$\therefore \quad \boxed{P = \rho g Q H}$$

Power at inlet of pipe = $\rho g Q H$

Let h_f be the frictional head loss in pipe.



Head available at outlet of pipe = $H - h_f$

Power at outlet of pipe = $\rho g Q (H - h_f)$.

Efficiency of power transmission = $\frac{\text{Power at outlet}}{\text{Power at inlet}}$.

$$= \frac{\rho g Q (H - h_f)}{\rho g Q H}.$$

$$\boxed{\eta = 1 - \frac{h_f}{H}}$$

⊙ Condition for Maximum Power Transmission

Power at outlet of pipe = $\rho g Q (H - h_f)$.

$$= \rho g \times \frac{\pi}{4} D^2 \times V \left(H - \frac{f L V^2}{2 g D}\right)$$

$$= \rho g \frac{\pi}{4} D^2 \left(HV - \frac{f L V^3}{2 g D}\right).$$

For maximum power,

$$\frac{dP}{dV} = 0.$$

$$H - \frac{3 f L V^2}{2 g D} = 0.$$

$$H - 3 h_f = 0.$$

$$\therefore \quad H = 3 h_f.$$

or $\boxed{h_f = \frac{H}{3}}$

Efficiency of max power transmission = $1 - \frac{h_f}{H} = 1 - \frac{1}{3}$

$$= \underline{\underline{66.66\%}}$$

08. $H = 99$ m.

$h_f = \frac{H}{3} = 33$ m. (for max. power).

Max power transmitted $= \rho g Q (H - h_f)$.

$= 10^4 \times 1 (99-33)$.

$= 660 \times 10^3$ W

$= \underline{660}$ kW

09. $Q = 100$ m^3/s, $H = 75$ m.

Power transmitted $= \rho g Q H$

$= 10^4 \times 100 \times 75$.

$= 75 \times 10^3$ kW.

$= 75 \times 10^3 \times 1.34$ hp

$= \underline{100576}$ hp

1 hp = 746 W

10. $Q = 0.1$ m^3/s, $H = 10$ m, $h_f = 5$ m.

$P_{(in)} = \rho g Q H$

$= 10^4 \times 0.1 \times 10 = 10$ kW. (power required to pump to the height).

$P_{(out)} = \rho g Q (H - h_f)$.

$= 10^4 \times 0.1 \times 5 = 5$ kW. (power required to overcome friction loss).

Total power required by pump $= \rho g Q (h_L + H)$.

$= 10^4 \times 0.1 (5+10)$

$= \underline{15}$ kW

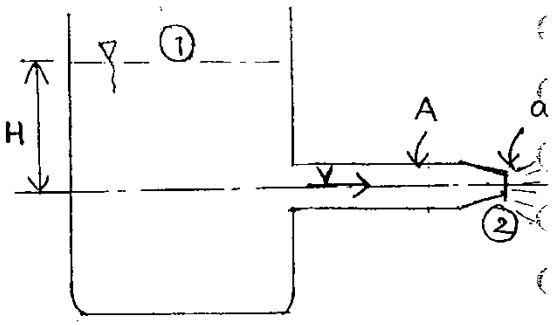
12 $h_L \propto \frac{1}{d^5}$.

$\frac{h_1}{h_2} = \frac{d_2^5}{d_1^5} = \left(\frac{d_1/2}{d_1}\right)^5 = \frac{1}{32}$

$h_2 = \underline{32} h_1$

→ Power Transmission Through Nozzle

$$\frac{P_1}{\rho g}^{P_{atm}} + \frac{V_1^2}{2g}^{0} + z_1 = \frac{P_2}{\rho g}^{P_{atm}} + \frac{V_2^2}{2g}^{\frac{v^2}{2g}} + z_2 + h_f.$$



$$\boxed{H = \frac{v^2}{2g} + h_f.}$$

$$h_f = \frac{fLV^2}{2g.D.}$$

$$H = \frac{v^2}{2g} + h_f. \frac{fLV^2}{2gD.}$$

$$AV = av.$$

$$V = \frac{av}{A.}$$

$$H = \frac{v^2}{2g}\left(1 + \frac{fL\,a^2}{DA^2}\right)$$

$$v^2 = \frac{2gH}{1 + \frac{fLa^2}{A^2D.}}$$

$$\therefore \quad \boxed{v = \sqrt{\frac{2gH}{1 + \frac{fLa^2}{A^2D.}}}}$$

Power at outlet of nozzle $= \frac{\text{KE of jet}}{\text{time}}$

$$= \frac{1}{2}\dot{m}v^2$$

$$= \frac{1}{2}(\rho a.v)v^2$$

$$= \frac{1}{2}\rho a v^3$$

Power at inlet of nozzle $= \rho g Q(H - h_f)$.

where $Q = AV = av$

$$= \rho g\, av\, \frac{v^2}{2g.}$$

Power at inlet of pipe $= \rho g Q H$.

$= \rho g a v . H$.

$$\eta = \frac{\text{power at outlet of nozzle}}{\text{power at inlet of pipe}}$$

$$= \frac{\frac{1}{2} \rho a v^3}{\rho g a v \times H} = \frac{v^2}{2gH}.$$

$$\boxed{\eta = \frac{v^2}{2gH} = \frac{1}{1 + \frac{fLa^2}{DA^2}}}$$

Power at outlet of nozzle $= \frac{1}{2} \dot{m} v^2$.

$= \frac{1}{2} \rho a v v^2$

$= \frac{1}{2} \rho a v \, 2g (H - h_f)$.

$= \frac{1}{2} \rho a v \, 2g \left(H - \frac{fLv^2}{2gD}\right)$.

$= \frac{1}{2} \rho a . 2g \left(Hv - \frac{fLa^2}{2gDA^2} v^3\right)$.

$H - h_f = \frac{v^2}{2g}$.

$av = AV$.

For max. power,

$$\frac{dP}{dV} = 0$$

$$H - \frac{fLa^2v^2}{2gDA^2} \cdot 3 = 0.$$

$$H - \frac{fLV^2}{2gD} \cdot 3 = 0$$

$$\therefore \quad H = 3h_f$$

$$h_f = \frac{H}{3}$$

$$\frac{v^2}{2g} + h_f = H = 3h_f.$$

$$\therefore \quad \frac{v^2}{2g} = 2\left(\frac{fLv^2}{2gD}\right)$$

$$\frac{v^2}{2g} = \frac{fL}{gD}\,\frac{a^2v^2}{A^2}$$

$$\frac{A^2}{a^2} = \frac{2fL}{D}$$

$$\Rightarrow \boxed{\frac{A}{a} = \sqrt{\frac{2fL}{D}}}$$

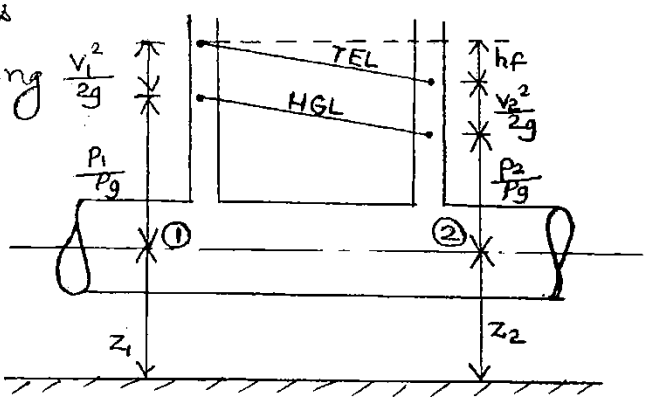
$$\frac{D^4}{d^4} = \frac{2fL}{D}$$

$$D^5 = 2fLd^4$$

$$\boxed{d = \left(\frac{D^5}{2fL}\right)^{1/4}}$$

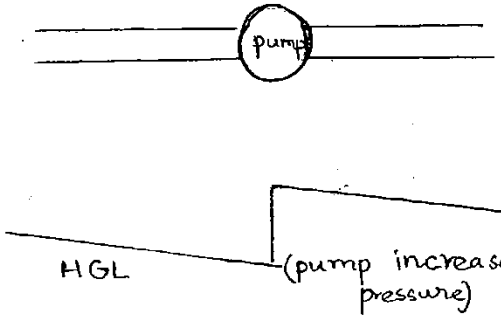
→ Total Energy Line (TEL) & Hydraulic Gradient Line (HGL)

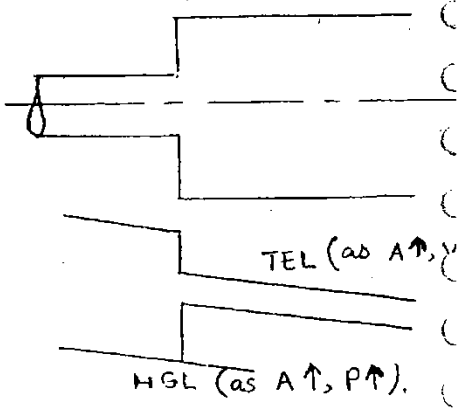
⊙ HGL is the locus of all points which will be obtained by taking the piezometric heads $\left(\frac{P}{\rho g} + z\right)$ along the length of the pipe in the direction of flow.



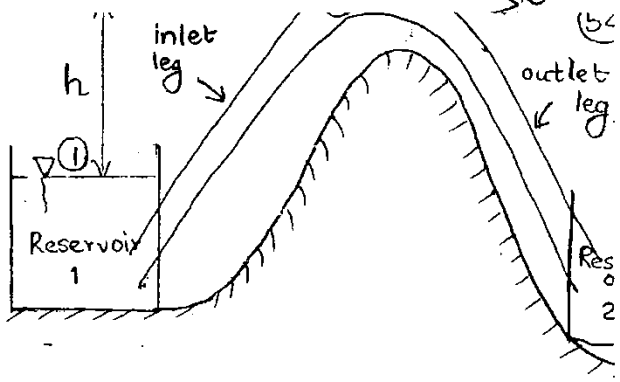
⊙ TEL is the locus of all points which will be obtained by taking the total energy $\left(\frac{P}{\rho g} + \frac{v^2}{2g} + z\right)$ along the length of pipe in the direction of flow.

⊙ TEL & HGL are seperated by kinetic head $\left(\frac{v^2}{2g}\right)$





→ Syphon



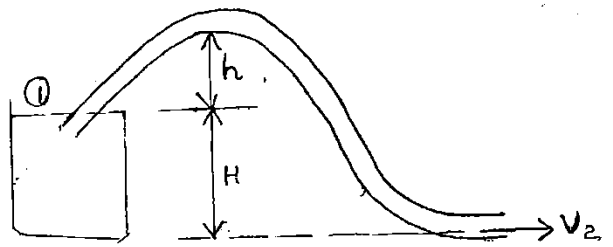
$$\frac{P_1}{\rho g}^{\text{Patm}} + \frac{V_1^2}{2g}^{0} + z_1^{0} = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + V_2$$

$$\frac{P_2}{\rho g} + \frac{V_2^2}{2g} + h = 0$$

$$\frac{P_2}{\rho g} = -\left(h + \frac{V_2^2}{2g}\right).$$

$$P_2 = -\rho g\left(h + \frac{V_2^2}{2g}\right) \quad \text{[vacuum pressure]}$$

The pressure at the summit point is less than atmospheric pr. and ∴ the liquid will be transferred from the reservoir to the summit point because of the pressure difference. The pressure at the summit point is reduced lower than 2.5 m of absolute pressure for water. Then the dissolved air in the water will get collected at the summit point and it leads to the air lock. and it will obstruct the continuity of flow.



$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2$$

$$\frac{V_2^2}{2g} = H.$$

$$\boxed{V_2 = \sqrt{2gH}}$$

The pressure at the summit pt. cannot be reduced to vapour pressure of water to prevent cavitation.

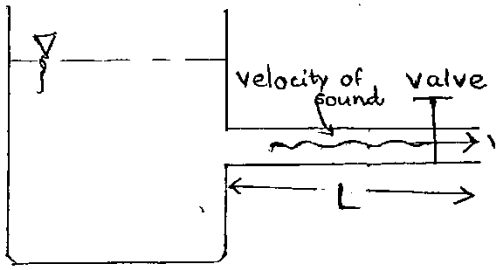
→ Water Hammering

Retarding force = pressure force.

$$m \times \left(\frac{V-0}{T}\right) = pA.$$

$$\frac{\rho \times \text{volume} \times V}{T} = pA.$$

$$\frac{\rho \times A \times L \times V}{T} = pA$$



$$\boxed{p = \frac{\rho L V}{T}}$$

> Incompressible flow
> Rigid pipe

T → time taken to reduce velocity.

■ When the pipe is rigid and fluid is compressible,

Loss in KE of water = Strain energy stored.

$$\frac{1}{2}m(V-0)^2 = \frac{p^2}{2K} \times A \times L$$

{Strain energy $= \frac{\sigma^2}{2E} \times$ volu

$$\frac{1}{2}\rho AL V^2 = \frac{p^2}{2K} AL$$

$$p^2 = \rho K V^2$$

Intensity of pressure, $p = V\sqrt{\rho K}$; Pipe rigid fluid compressible.

→ Velocity of Sound, $c = \sqrt{\frac{K}{\rho}}$

$$c^2 = \frac{K}{\rho}$$

$$p = V\sqrt{\frac{K}{c^2} \times K} \qquad \left(\because \rho = \frac{K}{c^2}\right)$$

$$\boxed{p = \frac{VK}{c}}$$

$$= V\sqrt{\rho \times \rho c^2}$$

$$\boxed{p = \rho V c.}$$

6. MOMENTUM EQUATION

→ Impact of Jets.

$$F_x = m \times a_x.$$

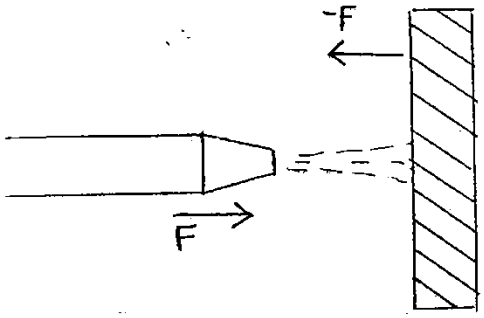
$$= m \times \frac{dv}{dt}.$$

$$F_x = \frac{d}{dt}(mv) \rightarrow \text{MOMENTUM EQUATION}$$

∴ Net force = Rate of change of momentum.

$F_x \cdot dt = d(mv)$ → Impulse Momentum Equation

impulse.



$-F = \frac{\text{mass}}{\text{time}}$ (Final velocity − Initial velocity).

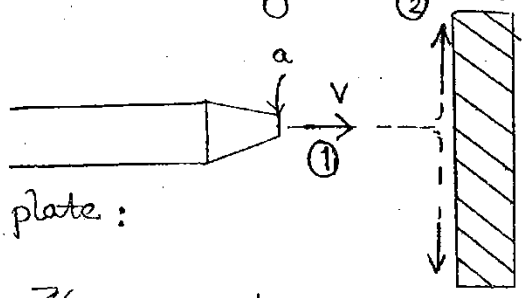
$$F = \frac{\text{mass}}{\text{time}} (\text{Initial velocity} - \text{Final velocity})$$

used to find the force exer by the jet on the obstructio

① Force exerted by the jet on the Stationery Flat Surfac

Let v be velocity of jet.

a be area of nozzle.



Force exerted by jet normal to the plate:

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2$$ (assume there are no losses over the plate)

$$V_1 = V_2 = V.$$

$$F = \frac{m}{\text{time}} (\text{initial velocity} - \text{final velocity}).$$

$$= \rho a V (V - 0)$$

(Final velocity component normal to the surface is zero)

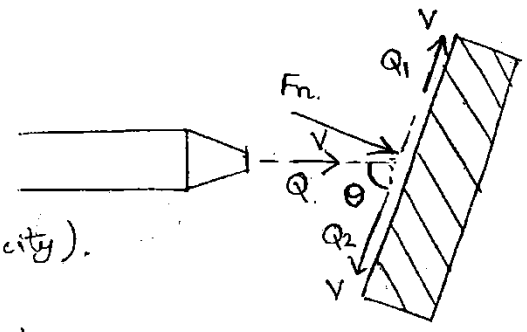
$$\boxed{F = \rho a V^2}$$

$F_x = F$

$F_y = 0$

∵ the plate is stationery, work done by the jet is zero.

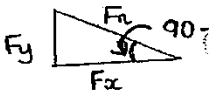
② Force exerted by the jet on a flat stationery inclined plate.



Force exerted by jet normal to the plate, $F_n = \frac{\text{mass}}{\text{time}}$ (ini. velo − final velocity).

$$F_n = \rho a V (V \sin\theta - 0)$$

$$\boxed{F_n = \rho a V^2 \sin\theta}$$



$F_x = F_n \cos\theta$

$= F_n \sin\theta$

$F_y = F_n \cos\theta$.

$$\boxed{F_x = F_n \sin\theta = \rho a V^2 \sin^2\theta}$$

$$\boxed{F_y = F_n \cos\theta = \rho a V^2 \sin\theta \cos\theta}$$

The net force acting parallel to the plate is zero,

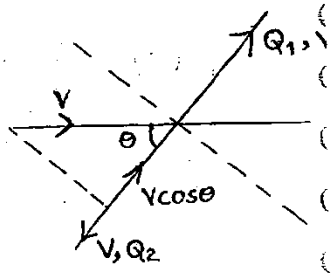
$$F_{net} = m \times a$$

$$0 = \frac{m}{\text{time}} (\text{initial velocity} - \text{final velocity})$$

$$= \rho Q V \cos\theta - (\rho Q_1 V - \rho Q_2 V)$$

$$Q \cos\theta = Q_1 - Q_2 \quad \rightarrow (1)$$

$$Q = Q_1 + Q_2 \quad \rightarrow (2)$$



$$\boxed{Q_1 = \frac{Q}{2}(1 + \cos\theta)}$$

$$\boxed{Q_2 = \frac{Q}{2}(1 - \cos\theta)}$$

P-95

Q.01 $F = \rho A V^2$

$= 1000 \times 10^{-4} \times 10^2$

$= \underline{10\,N}$

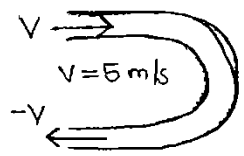
Q.02 $V = 5\ m/s$, $A = 10^{-4}\ m^2$

$F = \frac{mass}{time}$ (initial velocity − final velocity)

$= \rho Q\,(V - (-V))$.

$= 2\rho a V^2 = 2 \times 1000 \times 10^{-4} \times 25$

$= \underline{5\ N}$.



Q.03 $V = 10\ m/s$, $a = 5\ cm^2 = 5 \times 10^{-4}\ m^2$

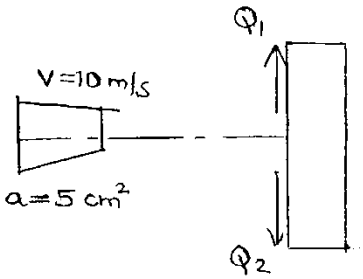
$Q = Q_1 + Q_2$

$= 2Q_1$

$aV = 2\,a_1 V_1$

$5 \times 10^{-4} \times 10 = 2 \times 2.5 \times 10^{-4} \times V_1$

$V_1 = \underline{10\ m/s}$



Q.04. Magnitude of force, $F = \rho a V^2$

$= 1000 \times 5 \times 10^{-4} \times 10^2$

$= \underline{50\ N}$

10th Aug, WEDNESDAY

③ Force exerted by the jet on the curved vane

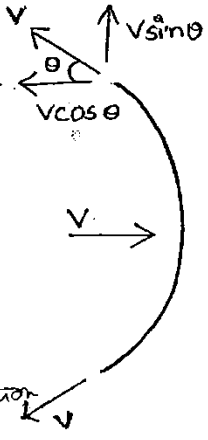
(i). Jet striking the vane at the centre

$F_x = \rho a V\,(V - (-V\cos\theta))$

$F_x = \rho a V^2\,(1 + \cos\theta)$

$F_y = \rho a V\,(0 - V\sin\theta) = -\rho a V^2 \sin\theta$

-ve sign indicates force exerted in vertically d/w direction



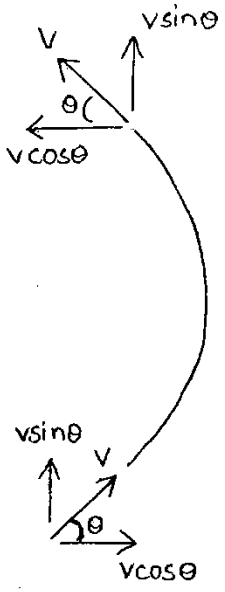
$\frac{WD}{s}$ = Force x $\frac{\text{displacement}}{s}$

= Force x velocity.

For the fixed vane, velocity of vane is zero.

$\therefore$ WD is zero.

(ii) Jet strikes tangentially.



$F_x = \rho Q (v\cos\theta - (-v\cos\theta))$

$= \rho Q (v\cos\theta + v\cos\theta)$

$= 2\rho a v^2 \cos\theta$.

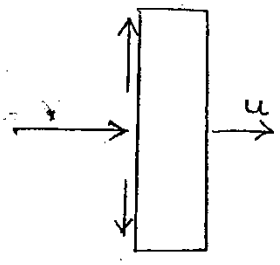
$F_y = \rho a v (v\sin\theta - v\sin\theta) = 0$

* MOVING VANES.

① Flat Vertical Plate

Let u be the velocity of the vane

In this case, the jet strikes the plate with relative velocity of $(v-u)$.



$F_x = \rho a (v-u)[(v-u)-0]$

$= \rho a (v-u)^2$

$F_x = \rho a (v-u)^2$

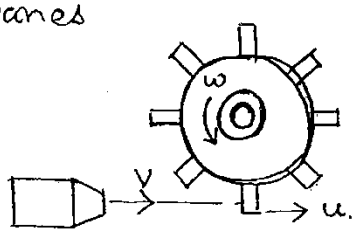
$\frac{W.D}{s} = F_x . u = \rho a (v-u)^2 \times u$.

⊙ If the jet is striking series of vanes

Mass of fluid striking the vane/s

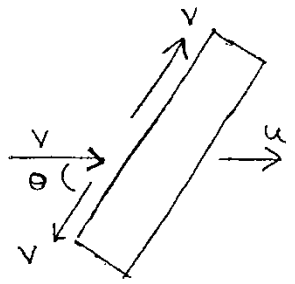
$= \rho a v$

$F_x = \rho a v[(v-u)-0] = \rho a v (v-u)$



$u = r\omega = \frac{\pi d N}{60}$

2. Inclined Plate



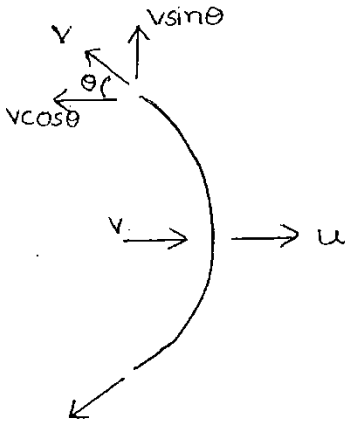
$$F_n = \rho a (v-u)[(v-u)\sin\theta - 0]$$

$$F_n = \rho a (v-u)^2 \sin\theta.$$

$$\boxed{F_{ox} = F_n \sin\theta = \rho a (v-u)^2 \sin^2\theta}$$

$$\boxed{F_y = F_n \cos\theta = \rho a (v-u)^2 \sin\theta \cos\theta.}$$

3. Curved Plate



$$F_x = \rho a (v-u)\,[(v-u) - (-(v-u)\cos\theta)]$$

$$\boxed{F_x = \rho a (v-u)^2 (1+\cos\theta)}$$

$$\frac{WD}{s} = F_x \cdot u$$

$$= \rho a (v-u)^2 (1+\cos\theta)\, u.$$

$$\eta = \frac{WD/s}{KE/s}$$

$$KE/s \text{ of jet} = \frac{1}{2}\left(\frac{m}{s}\right) v^2 = \frac{1}{2} \rho a v^3$$

$$\therefore \eta = \frac{\rho a (v-u)^2 (1+\cos\theta)\, u}{\frac{1}{2} \rho a v^3}$$

$$= \frac{2(v-u)^2 (1+\cos\theta)\, u}{v^3}.$$

For the max. efficiency, $\frac{d\eta}{du} = 0.$

$$\frac{d}{du}\left(\frac{2(1+\cos\theta)}{v^3} (v-u)^2 u\right) = 0$$

$$(v-u)^2 + 2(v-u)u \times -1 = 0.$$

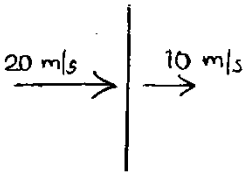
$$v - u - 2u = 0.$$

$$\Rightarrow \boxed{u = \frac{v}{3}}$$

For the η_{max}, the blade velocity will be one third of jet velocity.

05. $V = 20$ m/s, $u = 10$ m/s, $a = 0.01$ m^2, $\rho = 1000$ kg/m^3

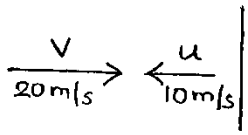
$$F = \rho a (V-u)^2$$
$$= 1000 \times 0.01 (20-10)^2 = \underline{\underline{1000\ N}}$$



06. $\frac{WD}{S} = \text{Power} = Fu = 1000 \times 10 = \underline{\underline{10\ kW}}$

07. $F = \rho a (V+u)^2$

$$= 1000 \times 0.01 \times (20+10)^2 = \underline{\underline{9000\ N}}$$



08. $A = 0.2 \times 10^{-4}$ m^2 $\quad V = 1 \times 10^{-2}$ m/s

$a = 0.07 \times 10^{-6}$ m^2

$$AV = aV$$
$$0.2 \times 10^{-4} \times 10^{-2} = 0.07 \times 10^{-6} \times V$$
$$V = 2.857 \text{ m/s}$$

$$F = \rho a V^2 = 1000 \times 0.07 \times 10^{-6} \times (2.857)^2$$
$$= 5.7142 \times 10^{-4} \text{ N}$$
$$= \underline{\underline{5.827 \times 10^{-5}}}\ kgf$$

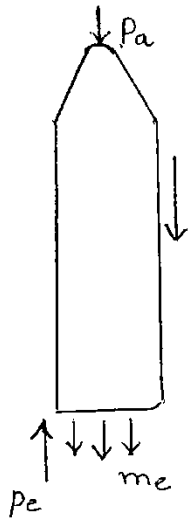
1 kgf = 9.81

09. $V = 20$ m/s, $u = 5$ m/s

$$F_1 = \rho a (V-u)^2 \qquad F_2 = \rho a V (V-u)$$
$$P_1 = \rho a (V-u)^2 \times u \qquad P_2 = \rho a V (V-u) \times u$$

$$\frac{P_1}{P_2} = \frac{\rho a (V-u)^2 \times u}{\rho a V (V-u) u} = \frac{V-u}{V} = \frac{20-5}{20 \times 5} = \frac{15}{20} = \underline{\underline{0.75}}$$

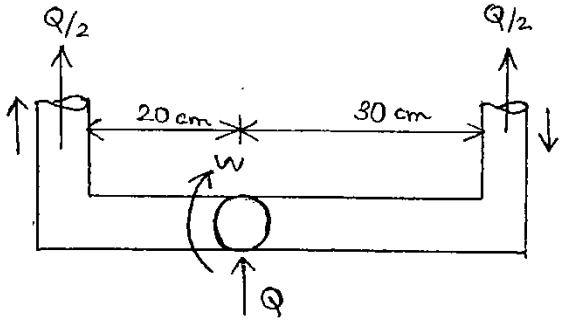
10. G = 1.2 ; Rocket Propulsion



M = fuel + oxidant.

⊙ Data insufficient

11. Velocity of jet from the nozzle $= \frac{\frac{Q}{2}}{A}$



$$= \frac{\left(\frac{144\times10^{-3}}{60}\right)/2}{1.2\times10^{-4}} = 10 \text{ m/s.}$$

Tangential velocity of arm end A $= r\omega$

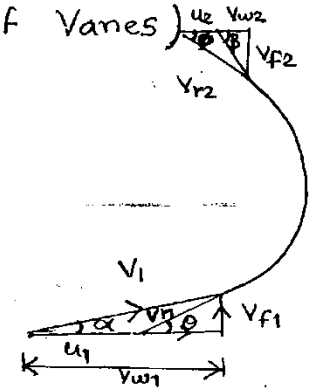
$= 0.2 \times 8 = 1.6$ m/s.

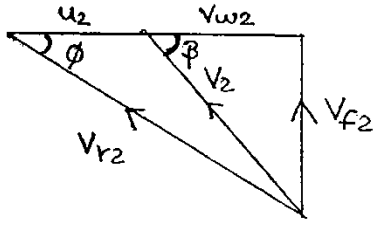
Tangential velocity of arm end B $= 0.3 \times 8 = 2.4$ m/s.

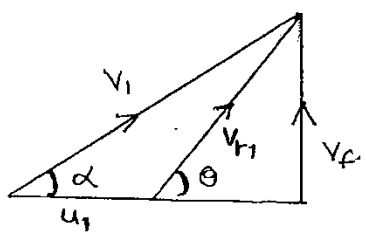
Absolute velocity of water at end A $= 10 + 1.6 = 11.6$ m/s.

Absolute velocity at end B $= 10 - 2.4 = 7.6$ m/s.

■ Unsymmetrical Curved Vane and the jet striking tangentially at one end and leaves tangentially (Series of Vanes)







$V_1 \rightarrow$ absolute velocity of jet

$V_{f1} \rightarrow$ velocity of flow

$V_{w1} \rightarrow$ velocity of whirl

$\alpha \rightarrow$ nozzle angle

$\theta \rightarrow$ inlet blade angle.

$\phi \rightarrow$ outlet blade angle

$u_1 \rightarrow$ blade velocity at inlet $= r_1 \omega$

$u_2 \rightarrow$ blade velocity at outlet $= r_2 \omega$

$$\frac{WD}{S} = \text{power} = \frac{2\pi N T}{60} = T \times \omega$$

$$T = \frac{\partial}{\partial t}(mvr) = \frac{\text{mass}}{\text{second}} \times \text{velocity} \times \text{radius}$$

Torque = rate of change of moment of momentum

Moment of momentum/s at inlet $= \rho a V_1 V_{w1} \times r_1$

Moment of momentum/s at outlet $= -\rho a V_1 V_{w2} \times r_2$.

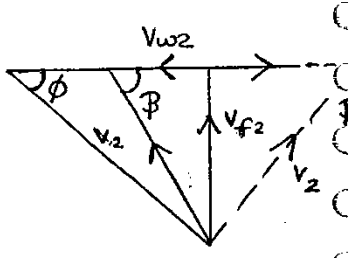
(same mass will be flowing out $= \rho a V_1$)

$$\text{Torque} = \rho a V_1 V_{w1} r_1 - (-\rho a V_1 V_{w2} r_2)$$

$$\frac{WD}{S} = T \times \omega$$

$$= \rho a V_1 V_{w1} \times r_1 \times \omega + \rho a V_1 V_{w2} \times r_2 \times \omega$$

$$= \rho a V_1 V_{w1} \times u_1 + \rho a V_1 V_{w2} \times u_2$$



$$\boxed{\frac{WD}{S} = \rho a V_1 (V_{w1} u_1 + V_{w2} u_2)}$$

⊙ If $u_1 = u_2 = u$.

$$\frac{WD}{S} = \rho a V_1 (V_{w1} + V_{w2}) u.$$

⊙ If $\beta = 90^\circ$, then $V_{w2} = 0$

$$\frac{WD}{S} = \rho a V_1 V_{w1} \times u.$$

⊙ If $\beta > 90^\circ$,

$$\frac{WD}{S} = \rho a V_1 (V_{w1} - V_{w2}) u.$$

$$\boxed{\frac{WD}{S} = \rho a V_1 (V_{w1} u_1 \pm V_{w2} u_2)}$$

$$\frac{WD/S}{\frac{\text{wt. of fluid}}{S}} = \frac{\rho a V_1 (V_{w1} u_1 \pm V_{w2} u_2)}{\rho a V_1 \times g}$$

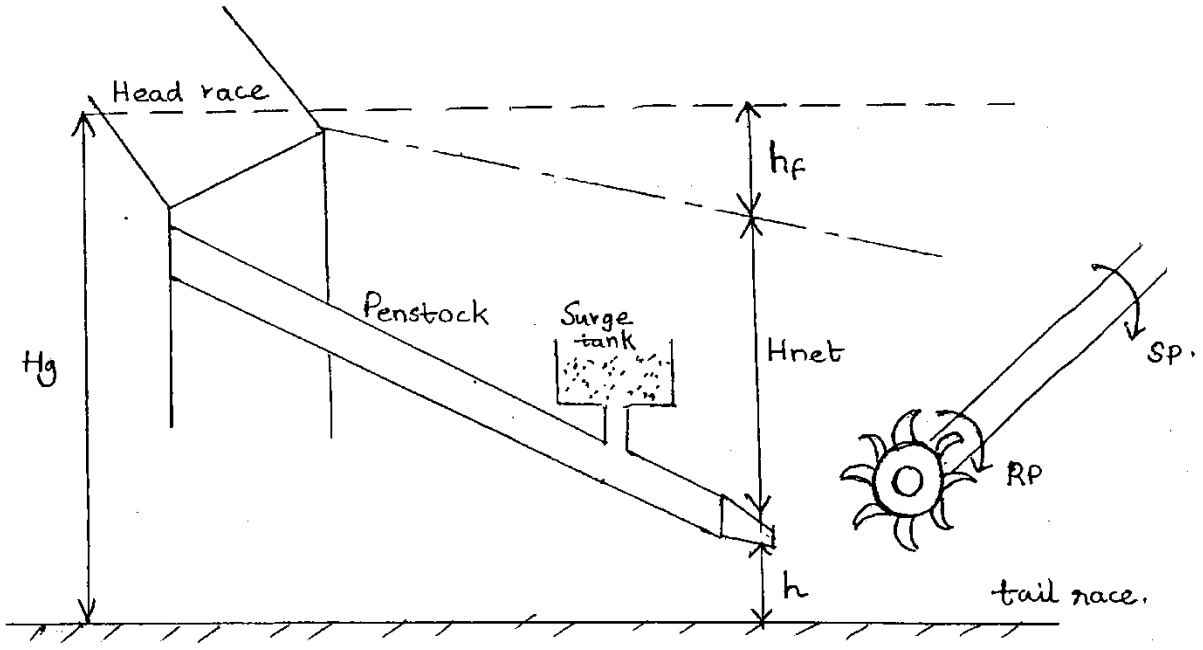
$$= \frac{V_{w1} u_1 \pm V_{w2} u_2}{g}$$

⊙ When $\beta = 90^\circ$, $V_{w2} = 0$

$$= \frac{V_{w1} u_1}{g} \quad \text{(Francis Turbine)}$$

11. TURBOMACHINERY

Turbine: hydraulic energy into mechanical energy



$H_g \rightarrow$ gross head.

$H_{net} \rightarrow$ net head $\Rightarrow H_{net} = H_g - h_f$ ($\because h$ is small).

⊙ Hydraulic efficiency, $\eta_h = \frac{R.P}{W.P}$; R.P → runner power, WP → water power.

$$RP = \rho a V_1 (V_{w1} u_1 \pm V_{w2} u_2) \; ; \; WP = \rho g Q H.$$

⊙ Mechanical efficiency; $\eta_m = \frac{SP}{RP}$; SP → shaft power

⊙ Overall efficiency, $\eta_o = \frac{SP}{WP}$

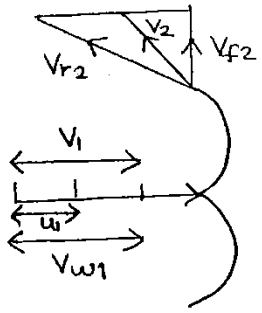
$$\boxed{\eta_o = \eta_h \times \eta_m}$$

$$= \frac{RP}{WP} \times \frac{SP}{RP} = \frac{SP}{WP}$$

→ Pelton Wheel.

- Turbine
  - Energy
    - Impulse (KE only)
    - Reaction (KE & pressure)
  - Speed
  - Head.

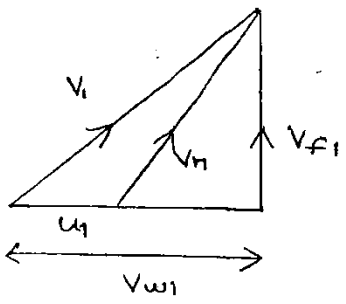
Pelton wheel is the low speed, high head tangential impulse turbine.



$\alpha = 0$

$u_1 = u_2 = u$

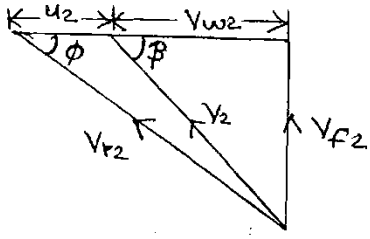
(Hemispherical bucket)



$$\frac{WD}{S} = \rho a V_1 (V_{w1} u_1 + V_{w2} u_2),$$

$$u_1 = u_2 = u.$$

$$= \rho a V_1 (V_{w1} + V_{w2}) u.$$



$$V_{w1} = V_1$$

$$V_{w2} = V_{r2} \cos\phi - u_2$$

If blade is smooth, then,

$$V_{r2} = V_{r1} = V_1 - u.$$

$$V_{w2} = (V_1 - u)\cos\phi - u.$$

$$\frac{WD}{S} = \rho a V_1 (V_1 + (V_1 - u)\cos\phi - u)\, u$$

$$= \rho a V_1 [(1+\cos\phi) V_1 - u(1+\cos\phi)] u$$

$$\boxed{\frac{WD}{S} = \rho a V_1 (V_1 - u)(1+\cos\phi) u}$$

If blade is having friction,

$V_{r2} = k \times V_{r1}$; $k \rightarrow$ blade friction coeffecient.

$$\frac{WD}{S} = \rho a V_1 (V_1 - u)(1 + k\cos\phi) u.$$

$$\eta = \frac{WD/S}{KE/S} = \frac{\rho a V_1 (V_1 - u)(1+\cos\phi) u}{\frac{1}{2} \rho a V_1^3.}$$

For max η, $\frac{\partial \eta}{du} = 0$;

$$(V_1 - u)1 + u(-1) = 0.$$

$$V_1 = 2u$$

$$\therefore \boxed{u = \frac{V_1}{2}}$$

For maximum efficiency, blade speed is equal to half of jet velocity.

1. $V_1 = C_v \sqrt{2gH}$
2. Blade speed ratio, $\phi = \frac{u}{\sqrt{2gH.}}$
3. Jet ratio, $m = D/d$; D → diameter of wheel, d → diameter of jet.
4. No: of buckets, $z = 15 + \frac{D}{2d.}$

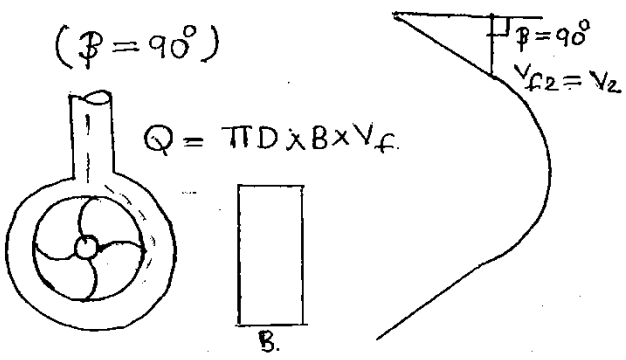
$$= 15 + 0.5\ m.$$

→ Radial Flow Turbines

a) Inward Flow ($\beta = 90^\circ$)

b) Outward Flow.

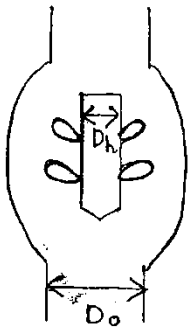
Radially enters
&
Axially leaves.



→ Axial Flow Turbines

a) Propeller — blades fixed to the hub.

b) Kaplan — adjustable vanes to the hub.



$$Q = \frac{\pi}{4}(D_o^2 - D_h^2)\, V_f.$$

267.

1. $T = 15915$ Nm

$N = 600$ rpm

$$P = T \times \omega = \frac{15915 \times 2\pi \times 600}{60} = \underline{1000 \text{ kW}}$$

2. $\eta_o = 80\%$; $Q = 50$ m³/s ; $H = 7.5$ m.

$$WP = \rho g Q H.$$

$$= 1000 \times 10 \times 50 \times 7.5.$$

$$SP = \eta_o \times WP.$$

$$= 0.8 \times 1000 \times 10 \times 50 \times 7.5 = 3000 \text{ kW}$$

$$= 4000 \text{ HP}.$$

5. $$\phi = \frac{u}{\sqrt{2gH.}}$$

$$V_1 = C_v \sqrt{2gH.}$$

If $C_v = 1$, $V_1 = \sqrt{2gH.}$; $V_1 \rightarrow$ actual velocity.

$$\phi = \frac{\text{bucket speed (or) blade speed}}{\text{theoretical velocity of jet}}$$

6. $$\phi = \frac{u}{\sqrt{2gH.}}$$

$$u = 0.46 \sqrt{2g \times 400} = 40.74 \text{ m/s}$$

$r\omega = 40.744$

$$r \times \frac{2\pi N}{60} = 40.744$$

$$N = \frac{40.744 \times 60}{2.25 \times \pi} = 345.84 \text{ rpm}$$

→ Specific Speed (N_s)

It is the speed of the turbine which is identical in shape, blade angles, geometrical dimensions, gate openings etc with actual turbine but it will work under unit head and develops unit power

$$N_s = \frac{N\sqrt{P}}{H^{5/4}} \quad ; \quad P \text{ in kW}$$

$$P = \frac{\rho g Q H}{1000}$$

$$P \propto QH$$

$$Q \propto D^2 \times V.$$

$$V \propto \sqrt{H}$$

$$u \propto V \propto \sqrt{H}$$

$$u \propto DN \quad (\because u = \pi DN/60)$$

$$D \propto \frac{u}{N} \propto \frac{\sqrt{H}}{N}$$

$$D^2 \propto \frac{H}{N^2}$$

$$Q \propto \frac{H}{N^2} \cdot \sqrt{H} \propto \frac{H^{3/2}}{N^2}$$

$$P \propto QH \propto \frac{H^{3/2}}{N^2} \times H.$$

$$\boxed{P \propto \frac{H^{5/2}}{N^2}}$$

$$P = \frac{K H^{5/2}}{N^2}$$

If $P=1$ & $H=1$, then $N=N_S$

$$1 = \frac{K \times 1}{N_S^2}$$

$$\Rightarrow N_S^2 = K$$

$$\therefore P = \frac{N_S^2 \times H^{5/2}}{N^2}$$

$$\therefore N_S = \frac{N\sqrt{P}}{H^{5/4}}.$$

N_S	Type of Turbine
8 to 30	Pelton wheel with single jet
30 to 55	Pelton wheel with two or more jets.
55 to 250	Francis Turbine.
250 to 880	Propeller or Kaplan.

3. $P = 8.1$ mw $= 8.1 \times 10^3$.

$H = 81$ m, $N = 540$.

$$N_S = \frac{N\sqrt{P}}{H^{5/4}} = \frac{540\sqrt{8100}}{81^{5/4}} = \frac{540 \times 90}{243} = \underline{\underline{200}}$$

$55 < N_S < 250 \Rightarrow$ Francis Turbine.

4. Scale ratio $= \frac{D_m}{D_p} = \frac{1}{2} = \frac{D_2}{D_1}$; $D_m \rightarrow$ diameter of model, $D_p \rightarrow$ diameter of prototype

$H_1 = 20$ m, $N_1 = 500$ rpm. $\quad \frac{DN}{\sqrt{H}} = C$.

$H_2 = 20$ m, $N_2 =$

$$\frac{D_1 N_1}{\sqrt{H_1}} = \frac{D_2 N_2}{\sqrt{H_2}} \Rightarrow \frac{500}{N_2} = \frac{D_2}{D_1} = \frac{1}{2} \Rightarrow N_2 = \underline{\underline{1000}}$$

→ Unit Quantities.

To compare the performance of turbines, the head'H' be reduced to unit quantity and the corresponding speed, discharg power are called unit quantities for the same size of turbine.

① Unit Speed, Nu.

$$u \propto N \propto \sqrt{H} \qquad \left(u = \frac{\pi DN}{60}\right)$$

$$N \propto \sqrt{H}.$$

$$N = k\sqrt{H}.$$

When $H = 1$, $N = Nu$.

$$Nu = k.$$

$$\boxed{Nu = \frac{N}{\sqrt{H}}}$$

② Unit Discharge, Qu.

$$Q \propto D^2 \times V$$

$$Q \propto V \propto \sqrt{H}.$$

$$Q = k\sqrt{H}.$$

If $H = 1$ m, $Q = Qu$

$$\boxed{Qu = \frac{Q}{\sqrt{H}}}$$

③ Unit Power (Pu)

$$P = \rho g Q H.$$

$$P \propto QH$$

$$P \propto \sqrt{H}.\ H \propto H^{3/2}$$

$$P = k H^{3/2}$$

If $H = 1$ m, $P = Pu$

$$\boxed{Pu = \frac{P}{H^{3/2}}}$$

10. BOUNDARY LAYER THEORY

Whenever a real fluid flows past the solid object, some of the fluid molecules will adhere to the surface of the solid and they will possess the velocity as that of the solid. This is known as 'no-slip condition'. If the solid object is stationery, then velocity of fluid particles which are on th surface have zero velocity. These molecules will retard the fluid molecules which are on the another layer. and therefor the velocity gradients will takes place in a narrow thin layer in the transverse direction and is known as the 'Boundary Layer.' In the boundary layer, the velocity will vary from zero to the free stream velocity.

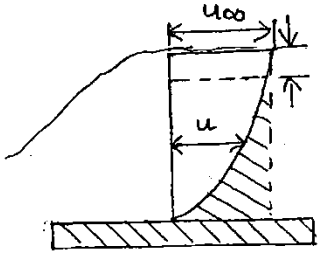
1. Boundary Layer Thickness (δ)

It is the thickness measured in the transverse direction from the surface in which the velocity will vary from zero to the 0.99 times of free stream velocity.

2. Displacement Thickness (δ^*)

It is the distance by which the solid object has to move in the transverse direction so that there is a compensation for loss of discharge due to the formation of boundary layer.

$$\delta^* = \int_0^{\delta} \left(1 - \frac{u}{u_\infty}\right) dy$$

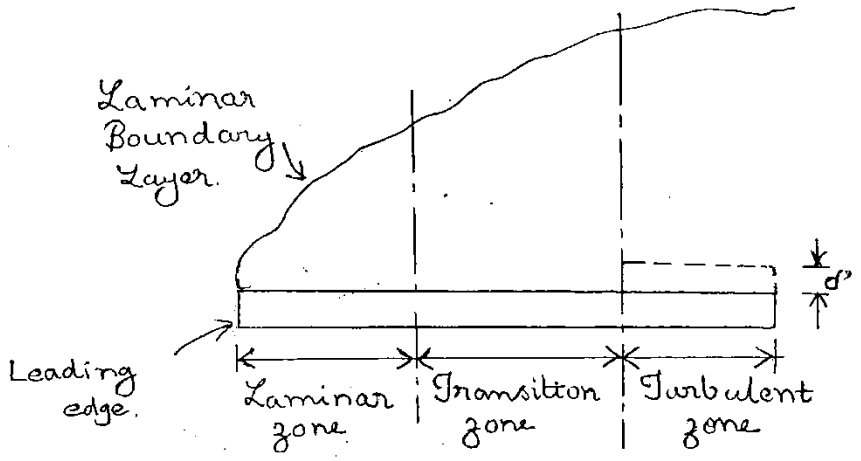


3. Momentum Thickness. (θ)

$$\theta = \int_0^{\delta} \frac{u}{u_\infty}\left(1 - \frac{u}{u_\infty}\right) dy.$$

4. Energy Thickness (δ^{**})

$$\delta^{**} = \int_0^{\delta} \left(1 - \frac{u^2}{u_\infty^2}\right) dy.$$



$\delta' \rightarrow$ laminar sublayer.

For a plate, $(Re)_{cr} = 5 \times 10^5$

$$Re = \frac{\rho u_\infty x}{\mu} \quad ; x \rightarrow \text{distance from leading edge upto which laminar zone exists.}$$

Q For a laminar velocity profile, $\frac{u}{u_\infty} = \frac{3y}{2\delta} - \frac{y^3}{2\delta^3}$, find momentum thickness (θ).

$$\theta = \int_0^{\delta} \frac{u}{u_\infty}\left(1 - \frac{u}{u_\infty}\right) dy.$$

$$= \int_0^{\delta} \left(\frac{3y}{2\delta} - \frac{y^3}{2\delta^3}\right)\left(1 - \frac{3y}{2\delta} + \frac{y^3}{2\delta^3}\right) dy.$$

$$= \int_0^{\delta} \left(\frac{3y}{2\delta} - \frac{9y^2}{4\delta^2} + \frac{3}{2}\frac{y^4}{\delta^4} - \frac{y^3}{2\delta^3} - \frac{y^6}{4\delta^6}\right) dy$$

$$= \left(\frac{3}{4}\delta - \frac{9}{12}\delta + \frac{3}{10}\delta - \frac{\delta}{8} - \frac{\delta}{28}\right)$$

$$= \frac{39}{280}\delta$$

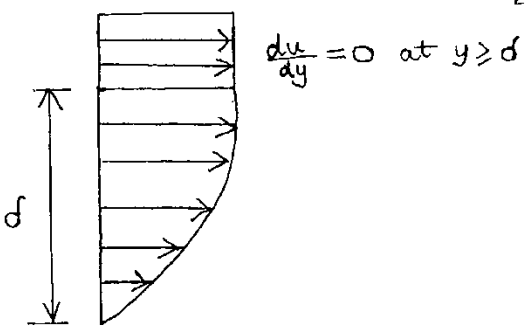
⊙ Boundary Conditions:

(i) At $x=0$, $\delta=0$.

(ii) At $y=0$, $u=0$

(iii) At $y=\delta$, $u=u_\infty$

(iv) At $y=\delta$, $\frac{du}{dy}=0$



Q For a laminar velocity profile $\frac{u}{u_\infty} = \frac{3y}{2\delta} - \frac{y^3}{2\delta^3}$, find

(i) Boundary layer thickness.

(ii) shear stress on the surface of plate.

$$\frac{\tau_0}{\rho u_\infty^2} = \frac{\partial \theta}{\partial x}$$

$$\tau_0 = \rho u_\infty^2 \cdot \frac{\partial \theta}{\partial x}.$$

$$= \rho u_\infty^2 \frac{\partial}{\partial x}\left(\frac{39}{280}\delta\right).$$

$$\tau_0 = \rho u_\infty^2 \frac{39}{280} \frac{\partial \delta}{\partial x} \longrightarrow (1)$$

$$\tau_0 = \mu \frac{du}{dy}.$$

$$= \mu \frac{d}{dy}\left(u_\infty\left(\frac{3y}{2\delta} - \frac{y^3}{2\delta^3}\right)\right)_{y=0}.$$

$$= \mu u_\infty \left(\frac{3}{2\delta} - \frac{3y^2}{2\delta^3}\right)_{y=0}$$

$$\tau_0 = \frac{3\mu u_\infty}{2\delta} \longrightarrow (2)$$

From (1) & (2),

$$\frac{3\mu u_\infty}{2\delta} = \rho u_\infty^2 \frac{\partial}{\partial x}\left(\frac{39}{280}\delta\right)$$

$$\frac{140\,\mu}{13\,\rho u_\infty} dx = 3\delta.\delta \Rightarrow \frac{140}{13} \times \frac{\mu}{\rho u_\infty} x = \frac{\delta^2}{2} + c.$$

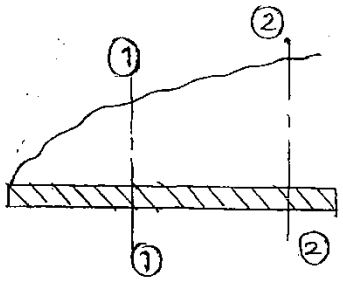
At $x = 0$, $\delta = 0$;

$\Rightarrow C = 0$

$$\delta^2 = \frac{280}{13}\,\frac{\mu x}{\rho u_\infty}$$

$$\delta = 4.64\sqrt{\frac{\mu x}{\rho u_\infty}} = 4.64\sqrt{\frac{x^2}{\frac{\rho u_\infty x}{\mu}}} \Rightarrow \delta = \frac{4.64\,x}{\sqrt{Re_x}}$$

$$\Rightarrow \boxed{\delta \propto \sqrt{x}}$$



As distance from leading edge increases ($x \uparrow$), velocity gradient decreases ($\frac{du}{dy} \downarrow$).

Shear stress, $\tau = \mu\left(\frac{du}{dy}\right)$ As $x \uparrow$:

(i) $\tau \downarrow$

(ii) $\delta \uparrow$

$\therefore \quad \tau \propto \frac{1}{\sqrt{x}}$ & $\delta \propto \sqrt{x}$.

Von Korman Momentum equation:

$$\boxed{\frac{\tau_0}{\rho u_\infty^2} = \frac{\partial \theta}{\partial x}}$$

$$\boxed{\delta \propto \sqrt{x} \Rightarrow \frac{\delta_1}{\sqrt{x_1}} = \frac{\delta_2}{\sqrt{x_2}}}$$

$$\boxed{\tau_0 \propto \frac{1}{\sqrt{x}}}$$

Blasius equation: $\boxed{\delta_{lam} = \frac{5x}{\sqrt{Re_x}}}$

01. $u_\infty = 120$ m/s, $x_{cr} = ?$

$u_\infty = 60$ m/s, $x_{cr} = 125$ mm.

$$(Re)_{cr} = 5 \times 10^5 = \frac{\rho u_\infty x}{\mu}$$

$\Rightarrow u_\infty \propto x.$

$$\frac{(u_\infty)_1}{(u_\infty)_2} = \frac{x_2}{x_1} \Rightarrow \frac{60}{120} = \frac{x_2}{125} \Rightarrow x_2 = \frac{125}{2} = 62.5 \text{ mm}$$

2. $\delta = \frac{5x}{\sqrt{Re_x}} \Rightarrow \delta \propto \frac{1}{\sqrt{Re_x}}$

$$\frac{\delta_1}{\delta_2} = \frac{\sqrt{Re_2}}{\sqrt{Re_1}}$$

$$= \sqrt{\frac{256}{100}} = \underline{\underline{1.6}}$$

3. $\frac{\delta_1}{\delta_2} = \frac{\sqrt{x_1}}{\sqrt{x_2}}$ ($\because \delta \propto \sqrt{x}$ for laminar boundary lay

$$\frac{2}{3} = \frac{\sqrt{x}}{\sqrt{x+1}}$$

$$\frac{x}{x+1} = \frac{4}{9}$$

$$x = \underline{\underline{0.8\,m}}$$

5. $\tau_0 \propto \frac{1}{\sqrt{x}}$.

$$\frac{(\tau_0)_1}{(\tau_0)_2} = \frac{\sqrt{x_2}}{\sqrt{x_1}} = \sqrt{\frac{4x_1}{x_1}} = 2$$

6. $\frac{u}{u_m} = 1.5\,\frac{y}{\delta}$; $\tau = \frac{K\mu u_m}{\delta}$

$$\theta = \int_0^\delta \frac{u}{u_m}\left(1 - \frac{u}{u_m}\right) dy$$

$$= \int_0^\delta \frac{1.5y}{\delta}\left(1 - \frac{1.5y}{\delta}\right) dy$$

$$= \int_0^\delta \left(\frac{1.5y}{\delta} - \frac{2.25\,y^2}{\delta^2}\right) dy.$$

$$= \frac{1.5}{2}\delta - \frac{2.25}{3}\delta = \underline{\underline{\frac{-\delta}{12}}}\ 0 \quad \{\text{question wrong}\}$$

$$\frac{\tau_0}{\rho u_\infty^2} = \frac{\partial\theta}{\partial x}$$

$$\tau_0 = \mu \frac{du}{dy}.$$

$$= \mu \left(\frac{1.5 y \times u_\infty}{\delta} \right) \qquad \frac{u}{u_\infty} = \frac{1.5 y}{\delta}.$$

$$\tau_0 = \frac{1.5 \mu u_\infty \cdot 1}{\delta} = \frac{k \mu u_\infty}{\delta}$$

$$\therefore \quad k = \frac{3}{2}$$

07 $$\frac{u}{u_\infty} = \frac{y}{\delta}$$

$$\delta^* = \int_0^\delta \left(1 - \frac{u}{u_\infty}\right) dy.$$

$$= \int_0^\delta \left(1 - \frac{y}{\delta}\right) . dy = \left(y - \frac{y^2}{2\delta}\right)_0^\delta$$

$$= \delta - \frac{\delta}{2} = \frac{\delta}{2}$$

$$\frac{\delta^*}{\delta} = \frac{\delta/2}{\delta} = \frac{1}{2}$$

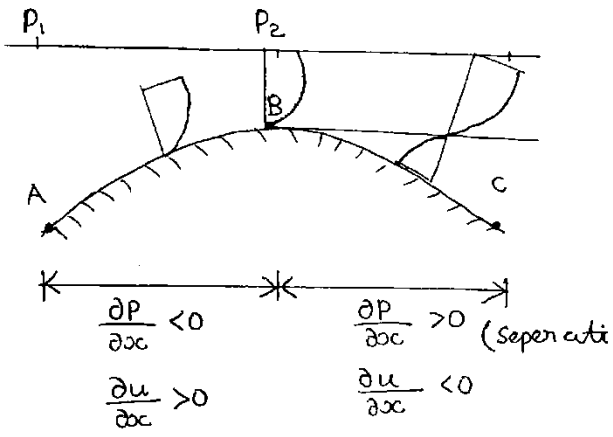
→ Boundary Layer Seperation.

$P_2 < P_1$

A to B → no seperation.

B to C → seperation takes place

B → point of seperation.



09. $(Re)_L = \frac{\rho u_\infty L}{\mu} = \frac{6 \times 1}{0.15 \times 10^{-4}} = 400000$

$\delta = \frac{4.64\, L}{\sqrt{(Re)_L}} = \underline{\underline{7.33 \text{ mm}}}$

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10. Shear stress at middle of plate:

Velocity profile is linear $\Rightarrow \frac{u}{u_\infty} = k \frac{y}{\delta}$.

$\tau \propto \frac{1}{\delta}$.

$\frac{\tau_1}{\tau_2} = \frac{\delta_2}{\delta_1} = \sqrt{\frac{x_2}{x_1}}$

$\frac{\delta_{L/2}}{\delta_L} = \sqrt{\frac{L/2}{L}} = \frac{1}{\sqrt{2}}$

$\delta_{L/2} = \frac{7.33}{\sqrt{2}} = 5.183$ mm.

$\frac{\mu}{\rho} = \nu$

$\tau_L = \mu \frac{du}{dy} = 0.15 \times 10^{-4} \times 1.226 \times \frac{6}{1} = 1.1034 \times 10^{-4}$.

$\frac{\tau_{L/2}}{\tau_L} = \frac{\partial_L}{\partial_{L/2}}$

$\tau_{L/2} = 1.034 \times 10^{-4} \times \frac{7.33}{5.183} = 1.56 \times 10^{-4}$

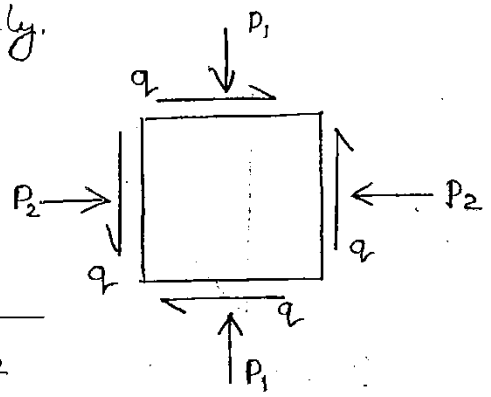
OPEN CHANNEL FLOW

→ Open Channel Flow vs. Pipe Flow.

Open Channel Flow	Pipe Flow
1. Open to atmosphere (pressure is zero)	1. Pressures other than zero (may be +ve or -ve)
2. Flow of open channel is gravitational., ie, from high level to low level	2. Pipe flow occurs from high total head to low total head It may be from low level to high level or high level to low le
3. Water surface is the HGL. TEL of open channel is above HGL, with an ordinate of corresponding velocity head.	3. In case of pipes, HGL may be above the pipe line or below t pipe line. Eg: syphon.
4. Velocity distribution is logarithmic	4. Velocity distribution may be parabolic (laminar) or almost rectangular (turbulent)
5. Shapes can be semi circular, trapezoidal, rectangular	5. Usually circular c/s

⊙ Atmospheric pressure is uniform on the free surface. According to Pascal's Laws, pressure is uniform in all directions at a point in absence of shear stress only.

∴ if a stress element is as shown,



for uniform pressure,

$$P_1 = P_2 = P$$

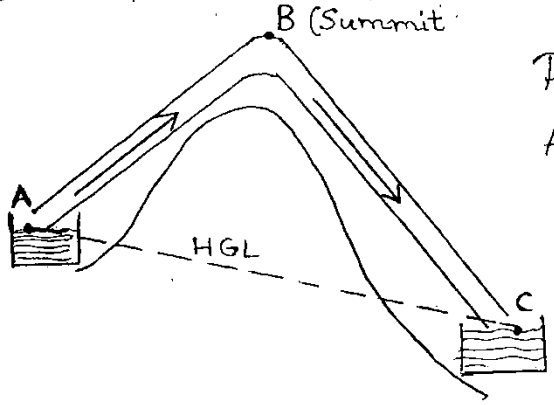
$$q = 0.$$

Radius of Mohr circle, $r = \sqrt{\left(\frac{P_1 - P_2}{2}\right)^2 + q^2}$

$$= 0.$$

∴ the Mohr Circle for uniform pressure at a point is a point on the normal stress axis.

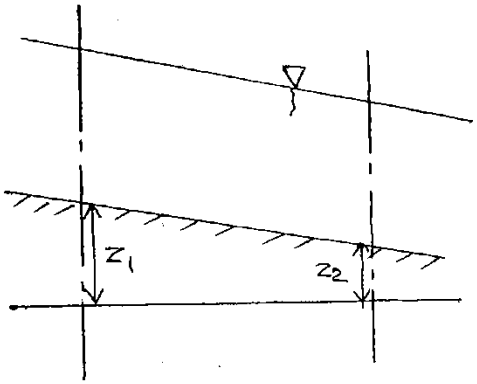
A free surface is subjected to uniform atmospheric pressure. Uniform pressure is possible only if shear stress is zero.



Pressure inside the syphon from A → B is negative.

and from B → A, pressure is positi

⇒ For a pipe flow, pressures other than zero.

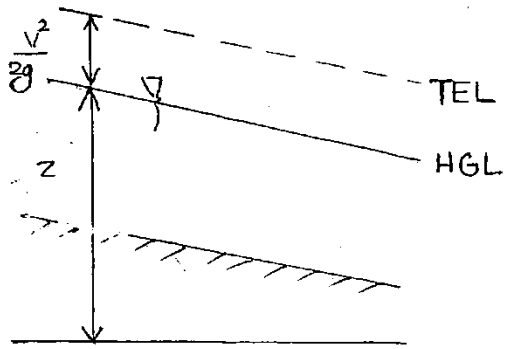


$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2 + h$$

For constant depth of flow, $V_1 = V_2$.

$$P_1 = P_2 = P_{atm}$$

$\therefore$ $z_1 > z_2$ (gravitational flow)

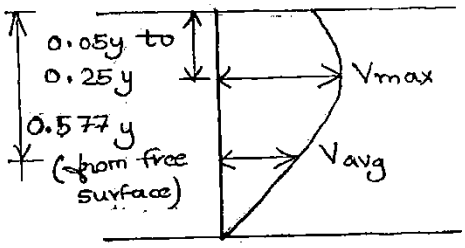


HGL = piezometric line.

$= \left(\frac{P}{\rho g} + z\right)$ line $= z$ line.

TEL of open channel flow

$= \left(\frac{V^2}{2g} + z\right)$

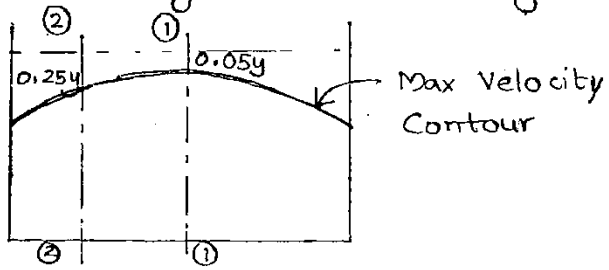


$Q = A \cdot V_{avg}$

$V_{avg} = \frac{V_{0.2y} + V_{0.8y}}{2}$ (2 pt. measurement velocities)

Current meter is used to measure velocities in Open channel fl

Velocity distribution of open channel flow is basically turbulent. All turbulent flows have logarithmic velocity distribution ($\propto y^{1/7}$)



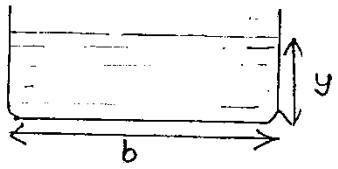
* Reasons for Velocity not Maximum at Free Surface.

(i) Water Currents. (wind forces)

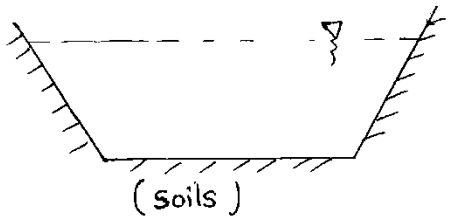
(ii) Surface Tension

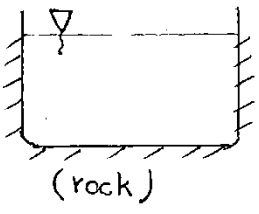
(iii) Boundary Friction

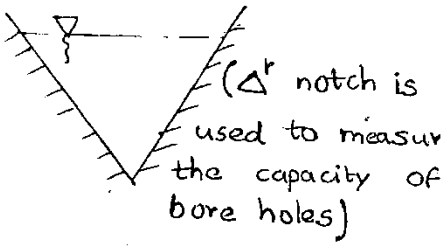
(iv) Aspect ratio. (y/b)

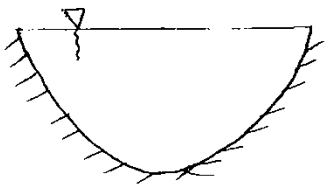


In shallow channels, where aspect ratio is less, max. velocity point lies at less depth from free surface compared to that of deeper channel with more aspect ratio.









Semi circular section is most economical

– Compared to other sections having sam area, semi-circle has least perimeter,

∴ cost of lining is the least for semi-circula channel sections.

→ Prismatic & Non Prismatic Channels.

If a channel has same slope, same size, same shape all along the length of channel, then its called Prismatic Channels. They are possible with artificial (lined) canals.

Shape, slope, size etc may change at diff. sections for a Non-prismatic channel. Eg:- Natural or unlined canals.

→ Classification of Open Channel Flow

(i) Steady Flow.

If the velocity of flow is constant at a given section at all times, it is called Steady flow.

∴ for steady flow, discharge (= AV) is also const. wrt time at a given section. ∴ $\frac{\partial Q}{\partial t} = 0$

In open channels, depth is important. ∴ for steady flow in open channels, $\frac{\partial y}{\partial t} = 0$.

* Unsteady Flow

$$\frac{\partial y}{\partial t} \neq 0.$$

It means the depth of flow, the velocity and discharge change from time to time at a given section.

(ii) Uniform Flow

If same velocity at all sections of an open chan at given time, it is called uniform flow.

$$\frac{\partial V}{\partial s} = 0$$

∴ depth of flow is constant at all sections at given time for uniform flow. ∴ $\frac{\partial y}{\partial x} = 0$

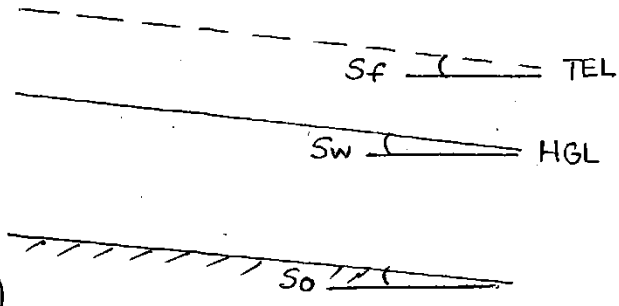
As depth is constant at all sections, water surface is parallel to channel bottom.

In uniform flow TEL, HGL and water surface are paralle to channel bottom

$S_o \rightarrow$ channel bed slope wrt horizontal.

$S_w \rightarrow$ water surface slope

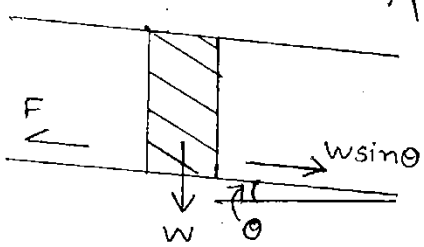
$S_f \rightarrow$ Energy slope (water friction line slope)



For uniform flow, $S_o = S_w = S_f$

⊙ Uniform flow is possible in wide rectangular channels.

→ Condition for Uniform Flow.



For equilibrium,

d/w force along the slope,

= u/w force due to friction along the slope.

$F_S = W\sin\theta$

∴ Driving force = Resisting force.

Driving force, F_D = component of weight of fluid resolved in the direction of channel bottom.

Resisting force, F_R = due to boundary friction.

→ Terms related to Uniform flow:

y_n = normal depth.

Depth corresponding to uniform flow = y_n.

S_n = normal slope.

Slope corresponding to uniform flow, = S_n.

Q_n = normal discharge.

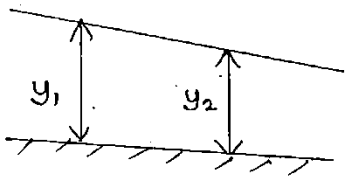
Discharge corresponding to uniform flow = Q_n.

For uniform flow, $S_o = S_n$

∴ $y_n = y_o$

→ * Non uniform Flow

Depth changes from section to section.



$\frac{dy}{ds} \neq 0$

Non Uniform Flow

↓ Gradually Varied Flow (GVF)

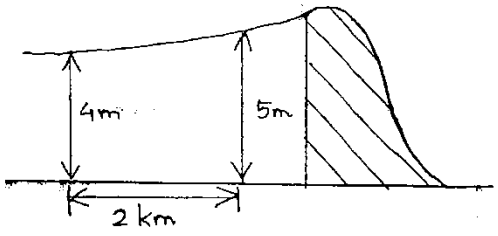
↓ Rapid / Suddenly Varied Flow (RVF)

Backwater curve is a gradually varied flow.

Head due to velocity of approach — Afflux.

Hydraulic Jump is an example of spatially rapidly varied flow.

Backwater Curve.



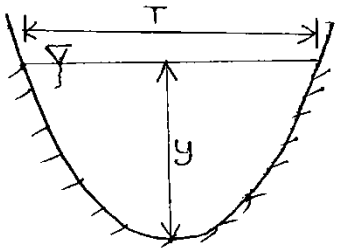
* Spatially Varied Flow (SVF):

If the flow changes from point to point and time to time, it is called Spatially Varied Flow.

In spatially varied flow, the flow or discharge is either added or subtracted to the main canal system.

Eg:- Spreading of irrigation water in the field.
Taking out water from different outlets in canal system.

→ Channel Geometry:



y = depth of flow
= free surface to deepest point

T = Top width of flow
= width of free surface

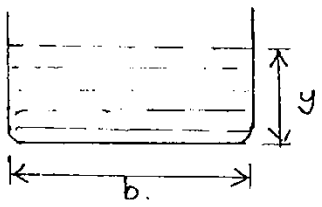
P = wetted perimeter
= solid boundary in contact with fluid.

A = wetted area
= c/s area of flow

R = hydraulic radius = hydraulic mean depth.
$= \frac{\text{wetted area}}{\text{wetted perimeter}} = \frac{A}{P}$

D = hydraulic depth $= \frac{A}{T}$

1. Rectangular Channels



$A = by$

$P = b + 2y$

$R = \frac{A}{P} = \frac{by}{b+2y}$

In case of wide rectangular channels, $y <<< b$. (Eg: rivers)

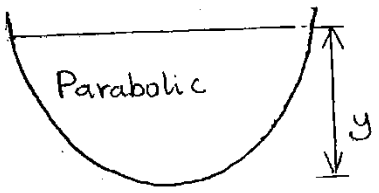
$$R \approx \frac{by}{b} \approx y.$$

$$D = \frac{A}{T} = \frac{by}{b} = y.$$

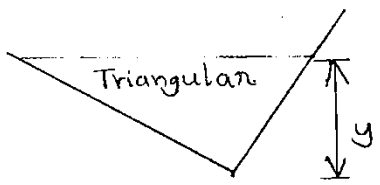
$T = b$
$A = by$
$R \approx y$
$D = y$

⊙ Physical meaning of Hydraulic Depth = Average depth.

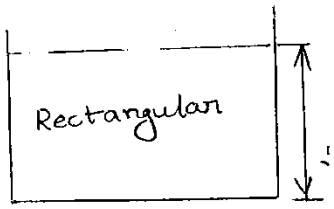
It cannot be applied in the case of closing shape of channels.



$D = \frac{2}{3}y$

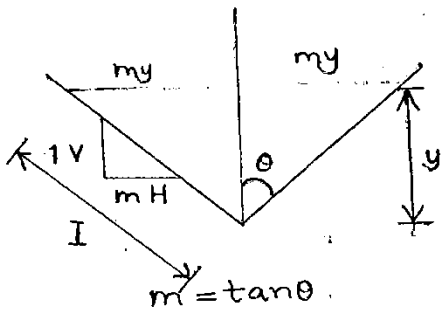


$D = \frac{y}{2}$



$D = y.$

2. Triangular Channels



$\theta \rightarrow$ semi vertex angle.

$m \rightarrow$ side slope

Top width, $T = 2my.$

Area, $A = \frac{1}{2} \times 2my \times y$

$= my^2$

$\therefore \quad A = y^2 \tan\theta.$

Wetted perimeter, $P = 2I.$

$= (2\sqrt{1+m^2})y$

$= 2y\sqrt{1+m^2}$

$\therefore P = 2y\sqrt{1+\tan^2\theta}$

$$R = \frac{A}{P} = \frac{my^2}{2y\sqrt{1+m^2}} = \frac{my}{2\sqrt{1+m^2}}$$

$$\text{Hydraulic depth} = \frac{A}{T} = \frac{my^2}{2my} = \frac{y}{2}$$

Hydraulic depth of triangle is independent of side slope.

If right angled triangle, $\theta = 45^0 \Rightarrow m = 1$

Area, $A = y^2$

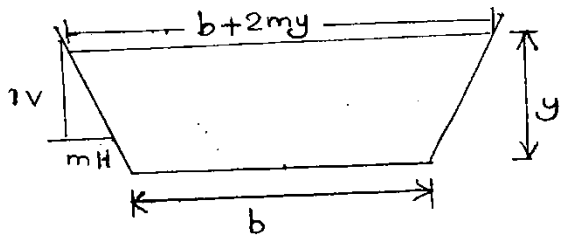
$P = 2y\sqrt{2}$

$$R = \frac{y^2}{2y\sqrt{2}} = \frac{y}{2\sqrt{2}}$$

$$D = \frac{y}{2}$$

$T = 2my$
$A = my^2 = y^2\tan\theta$
$P = 2y\sqrt{1+m^2}$
$R = \frac{my}{2\sqrt{1+m^2}}$
$D = \frac{y}{2}$

3. Trapezoidal Channels.



$$T = b + 2my$$

$$A = by + my^2$$

$$A = (b+my)y$$

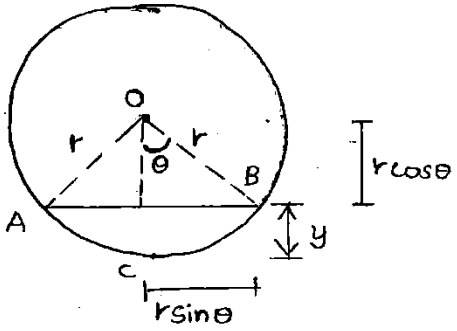
$$P = b + 2y\sqrt{1+m^2}$$

$$R = \frac{A}{P} = \frac{(b+my)y}{b+2y\sqrt{1+m^2}}$$

$$D = \frac{A}{T} = \frac{(b+my)y}{b+2my}$$

9th Sept, TUESDAY

4. Circular Channels



$$T = 2r\sin\theta$$

A = sector area OACB − Area of Δ^{le} OAB

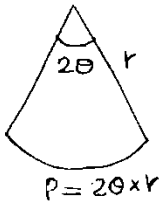
For $2\pi \rightarrow \pi r^2$

$2\theta \rightarrow r^2\theta$

$\therefore A = r^2\theta - \frac{1}{2} \times 2r\sin\theta \times r\cos\theta$

$= r^2\left(\theta - \frac{\sin 2\theta}{2}\right) = r^2\left(\frac{2\theta - \sin 2\theta}{2}\right)$

$= \frac{d^2}{8}(2\theta - \sin 2\theta)$



$P = 2r\theta = d\theta$

$R = \frac{A}{P} = \frac{d^2(2\theta - \sin 2\theta)}{8 \cdot d\theta}$

$R = \frac{d(2\theta - \sin 2\theta)}{8\theta}$

$D = \frac{A}{T} = \frac{d^2(2\theta - \sin 2\theta)}{8 \times 2r\sin\theta} = \frac{d(2\theta - \sin 2\theta)}{8 \cdot \sin\theta}$

⊙ If the circle is half full, $(2\theta = \pi)$.

Wetted area, $A = \frac{\pi r^2}{2}$

Wetted perimeter, $P = \pi r$

Hydraulic radius, $R = \frac{A}{P} = \frac{r}{2} = \frac{d}{4}$

$T = 2r\sin\theta$

$A = \frac{d^2}{8}(2\theta - \sin 2\theta)$

$P = 2r\theta = d\theta$

$R = \frac{d(2\theta - \sin 2\theta)}{8\theta}$

$D = \frac{d(2\theta - \sin 2\theta)}{8\sin\theta}$

⊙ Circular channel running full,

$$A = \pi r^2$$

$$P = 2\pi r \quad \Rightarrow R = \frac{r}{2} = \frac{d}{4}$$

$$D = \frac{\pi r^2}{0} = \infty \text{ (not defined)}$$

NOTE:

$$\text{Froude's number} = \sqrt{\frac{\text{Inertia force}}{\text{Gravity force}}} = \frac{V}{\sqrt{gD}}$$

Infact Froude's number and hence hydraulic depth, D are defined for gravitational flow only. Circular channel running full is not gravitational flow, it is pressure flow. Hence D not defined. for circular channel running full.

■ Perimeter of ellipse $= 2\pi \sqrt{\frac{a^2+b^2}{2}}$

a & b are semi-axis of ellipse.

Q. A circular channel of radius r was hit by a truck. Due to impact, c/s shape changed to an ellipse whose major axis is twice that of minor axis. Calculate the percentage change in hydraulic radius.

$$R_c = \frac{A}{P_c} \quad \& \quad R_e = \frac{A}{P_e}$$

$$\frac{R_c}{R_e} = \frac{P_e}{P_c} = \frac{2\pi \sqrt{\frac{a^2+4a^2}{2}}}{2\pi r} = \frac{9.934\, a}{2\pi r} \qquad \text{Say } b = 2a.$$

Assuming area remains the same before and after impacts,

$$\pi ab = \pi r^2$$

$$2\pi a^2 = \pi r^2$$

$$a^2 = \frac{r^2}{2}$$

$$\therefore \frac{Rc}{Re} = \frac{9.934a}{2\pi \times \sqrt{2}a} = 1.118$$

$$Re = 0.8944\, Rc.$$

% change in hydraulic radius = <u>10.55 %</u> (↓)

* Froude's Number; (Fr)

Fr < 1 ; Sub critical flow

Fr = 1 ; Critical flow

Fr > 1 ; Super critical flow

- For rectangular channel,

$$D = y$$

$$\therefore Fr = \frac{V}{\sqrt{gD}} = \frac{V}{\sqrt{gy}}$$

⊙ For critical flow,

$$Fr = Frc = 1$$

$$\therefore \frac{Vc}{\sqrt{g\,yc}} = 1$$

⊙ For subcritical flow, Fr < 1

$$\frac{V}{\sqrt{gy}} < 1.$$

Assume V = constant,

$$\therefore y > yc \text{ , then } Fr < 1$$

Flows with <u>greater depth</u> are subcritical.

Assume y = constant,

$\therefore \quad V < V_c \Rightarrow$ tranquil flow

Hence subcritical flows are 'Tranquil Flows'.

⊙ For Supercritical flow,

$$Fr > 1$$

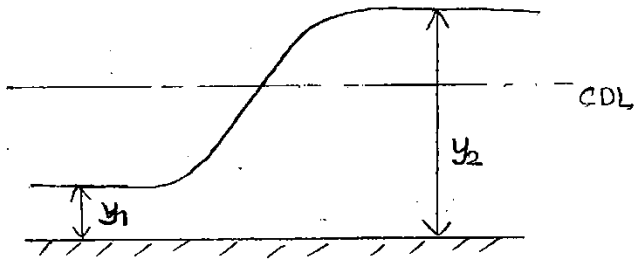
$$y < y_c$$

$$V > V_c$$

Hence supercritical flow occurs at shallow depths. If denominator y = const; $V > V_c$. As $V > V_c$, supercritical flows are also called 'Torrential flows'.

⊙ Hydraulic Jump occurs if supercritical flows meets subcritical flow.

Hydraulic ~~depth~~ jump occurs if a flow whose depth less than critical depth meets another flow whose depth more than critical depth.



– For triangular channel

$$Fr = \frac{V}{\sqrt{gD}}$$

$$= \frac{V\sqrt{2}}{\sqrt{gy}} \quad ; \quad D = \frac{y}{2}$$

– For parabolic channel,

$$D = \frac{2y}{3}$$

$$Fr = \frac{V\sqrt{3}}{\sqrt{2gy}}$$

- For Circular Channel,

(i) Running half

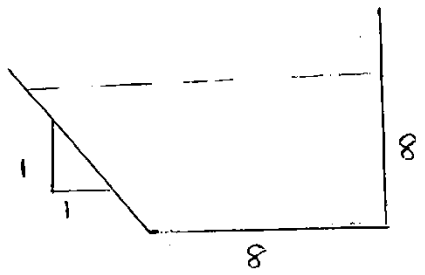
$$F_r = \frac{V\sqrt{4}}{\sqrt{\pi r g}} \qquad D = \frac{\pi r}{4}$$

$$Fr = \frac{2V}{\sqrt{\pi g r}}$$

- For trapezoidal channel,

$$Fr = \frac{V}{\sqrt{gD}} \quad ; \quad D = \frac{A}{T}$$

Q.



$Q = 10\ m^3/s$.

Find whether flow is critical, subcritical or supercritical.

$T = 16.$

$A = 8 \times 8 + \frac{1}{2} \times 8 \times 16$

$= 128$ ~~96~~

$D = \frac{128}{16} = \frac{96}{16} = \mathbf{6}$

$V = \frac{Q}{A} = \frac{10}{128} = 0.1042\ m/s$

$Fr = \frac{V}{\sqrt{gD}} = \frac{0.1042}{\sqrt{6g}} = 0.0136 < 1$

$\therefore$ the flow is subcritical.

→ Reynold's Number (Re)

$$Re = \frac{\text{Inertia force}}{\text{Viscous force}}$$

$$Re = \frac{\rho V L}{\mu}$$

In case of open channels, L = hydraulic radius.

$$\therefore Re = \frac{\rho V R}{\mu}$$

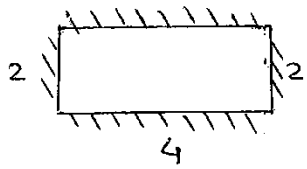
For circular pipe, $Re = \frac{\rho V d}{\mu}$ $\quad (d = 4R)$

For non-circular pipe, $Re = \frac{\rho V 4R}{\mu}$

Also, head loss due to friction in turbulent flow of non-circular pipes, $h_f = \frac{fLV^2}{2gd} = \frac{fLV^2}{2g(4R)}$

Q. A gas of viscosity, μ, density ρ flows through a rectangular duct of 4m x 2m. Its Re is ...? Assume velocity

$$Re = \frac{\rho V d}{\mu} = \frac{\rho V (4R)}{\mu}$$



$$R = \frac{A}{P} = \frac{4 \times 2}{2(4+2)} = \frac{2}{3}$$

$$Re = \frac{\rho V \times \frac{4 \times 2}{3}}{\mu} = \frac{8 \rho V}{3 \mu}$$

→ Classification of Open Channels based on Re

$Re < 500$; Laminar

$500 < Re < 2000$; Transitional.

$Re > 2000$; Turbulent

→ Boundary Shear Stress of Open Channels.

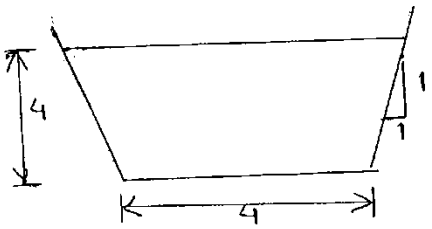
$$\tau_{avg} = \tau_{mean} = \gamma R S_o$$

where $\gamma \rightarrow$ wt. density

$R \rightarrow$ hydraulic radius

$S_o \rightarrow$ channel bottom slope

Q.



$A = (b + my)y = (4 + 1 \times 4)4 = 32 \text{ m}^2$

$P = b + 2y\sqrt{1+m^2} = 4 + 2 \times 4\sqrt{1+1}$

$= 15.31$

$$R = \frac{A}{P} = \frac{32}{15.31} = 2.08$$

$$\tau_{avg} = \gamma R S_o = 9810 \times 2.08 \times \frac{1}{1000} = \underline{\underline{20.5}} \text{ N/m}^2$$

'Preston Tube' is the device used for measuring boundary shear stress.

→ Uniform Flow Formulae :

(i) Chezy's formula

$$V = C\sqrt{RS}$$

where $C \rightarrow$ Chezy's constant

$C_{lined} \approx 75$

$C_{unlined} \approx 40$

'C' represents smoothness.

$V_{lined} > V_{unlined}$. Lined canals have less friction, more smooth compared to unlined canals. As $V \propto C$, C represents smoothness in fact.

Q. Arrange in increasing order of C

1. Sand 2. Boulder 3. Gravel 4. Silt

$2 < 3 < 1 < 4$

$C_{sea} < C_{river}$

* Dimensional formula for 'C':

$$C = \frac{V}{\sqrt{SR}} = \frac{L}{T\sqrt{L}} = \frac{\sqrt{L}}{T}$$

NOTE: Non-dimensional formula for $c = \frac{C}{\sqrt{g}}$

(ii) Manning's Formula.

$$V = \frac{1}{n} R^{2/3} S^{1/2};$$

$n \rightarrow$ Manning's rugosity coeffecient.

$$V \propto \frac{1}{n}$$

Manning's n represents roughness.

$$n_{lined} < n_{unlined}$$

$$n = \frac{R^{2/3} S^{1/2}}{V} = \frac{L^{2/3}}{L/T} = \frac{T}{L^{1/3}}$$

Dimensional formula : $[M^0 L^{-1/3} T]$

* Merits of Manning's Formula :

a) It is a simple formula.

b) As most of the channels are turbulent, it has experimental support compared to other formulae. ∴ it is more accurate.

$$Q = AV$$

$$\propto y \cdot R^{2/3} \quad \text{(mannings formula)}.$$

$$Q \propto y \cdot y^{2/3} \propto y^{5/3}$$

$$\Rightarrow \underline{\underline{Q \propto y^{5/3}}}$$

$$Q \propto y\sqrt{R} \quad \text{(Chezy's formula)}.$$

$$\propto y\sqrt{y}$$

$$\underline{\underline{Q \propto y^{3/2}}}$$

Q. If the depth of a channel increases by 20%, % increase in discharge.

$$\frac{Q_1}{Q_2} \propto = \frac{y_1^{5/3}}{y_2^{5/3}}.$$

$$\frac{Q_1}{1.2\,Q_1} = \frac{y_1^{5/3}}{y_2^{5/3}} \qquad \frac{Q_2}{Q_1} = \left(\frac{y_2}{y_1}\right)^{5/3} = (1.2)^{5/3}$$

$$y_2^{5/3} = 0.833\, y_1^{5/3}. \qquad Q_2 = 1.35\, Q_1 \Rightarrow \underline{35\%}$$

* Relation blw Chezy's 'C' & Manning's 'n':

$$V = C\sqrt{RS} = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\boxed{C = \frac{1}{n} R^{1/6}}$$

(iii) Stickler's Formula.

$$\boxed{n = \frac{d_{50}^{1/6}}{21.1}}$$

where d_{50} → size below which 50% of particles are finer (m)

Q. In calculating value of n, instead of taking d_{50} in m, by mistake it is considered as mm. The error in the value of n is.

$$n = \frac{(d_{50})^{1/6}}{21.1}$$

$$\text{Error} \propto (1000)^{1/6}$$

$$\frac{n_1}{n_2} = \frac{1^{1/6}}{1000^{1/6}}$$

$$n_2 = 3.16\ n_1$$

$$\% \text{ error} = \left(\frac{3.16 - 1}{1}\right) \times 100 = \underline{\underline{216\ \%}}$$

(iv) Meyerhoff's Modified Formula.

$$n = \frac{(D_{90})^{1/6}}{26}$$

$D_{90} \rightarrow$ size below which 90% particles are finer.

→ Conveyance of a Channel.

$$Q = K\sqrt{S}$$

whatever may be the formula, whether Chezy's or Manning's, $Q = K\sqrt{S}$

where K is called Conveyence of channel.

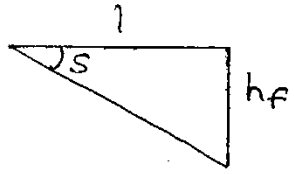
According to Manning's formula,

$$Q = A \cdot \frac{1}{n} R^{2/3} \sqrt{S}$$

$$K = \frac{AR^{2/3}}{n} = AC\sqrt{R}$$

* Relation b/w Chezy's C, Manning's n. & Darcy's f.

$$h_f = \frac{f l v^2}{2gd} = \frac{f l v^2}{2g.(4R)}$$



$$V = \sqrt{\frac{8g}{f}} \sqrt{R.\frac{h_f}{l}}$$

$$= \sqrt{\frac{8g}{f}} \sqrt{RS} = C\sqrt{RS}$$

$$\therefore \boxed{C = \sqrt{\frac{8g}{f}} = \frac{1}{n} R^{1/6}}$$

→ Hydraulically Efficient Channels

From a common man's perspective, a channel is said to be hydraulically efficient if it gives max. discharg

$$Q = AV$$

$$Q \propto V \quad ; \text{if } A \text{ is constant.}$$

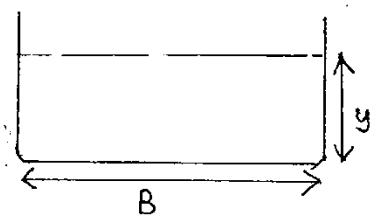
$$\propto C\sqrt{RS}$$

$$Q \propto \sqrt{R} \quad ; \text{ if } C \text{ is constant \& } S \text{ is constar}$$

$$Q \propto \frac{A}{P} \propto \frac{1}{P}$$

If channel is said to be hydraulically efficient, if it has max. discharge and min. wetted perimeter. for a given area, nature of boundary and longitudinal slope.

(i) Rectangular Channel.



$$A = By$$

$$P = B + 2y = \frac{A}{y} + 2y.$$

For min. wetted perimeter,

$$\frac{dP}{dy} = 0$$

$$\frac{-A}{y^2} + 2 = 0.$$

$$A = 2y^2$$

ie $By = 2y^2 \Rightarrow$ $\boxed{B = 2y}$

Q. Width of a rectangular channel 5m. Its hydraulic radius

$$b = 5 = 2y_e.$$

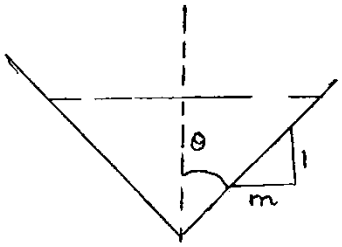
$$y_e = 2.5$$

$$R = 0.5\, y_e = \underline{\underline{1.25\ m}}.$$

(ii) Triangular Section

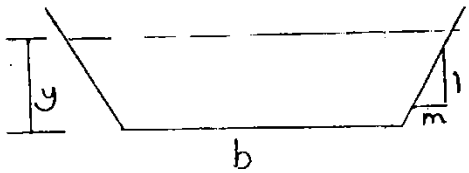
$$\theta = 45^\circ$$

Side slope, $m = 1$



∴ A right angle triangled is the efficient triangular section $\left(R = \frac{y}{2\sqrt{2}}\right)$

(iii) Trapezoidal Section



Condition 1: Assume m = const
y = variable

$$A = (b + my)y$$

$$P = b + 2y\sqrt{1+m^2}$$

$$\frac{dP}{dy} = 0.$$

$$\frac{d}{dy}\left(\frac{A}{y} - my + 2y\sqrt{1+m^2}\right) = 0$$

$$b + 2my = 2y\sqrt{1+m^2}$$

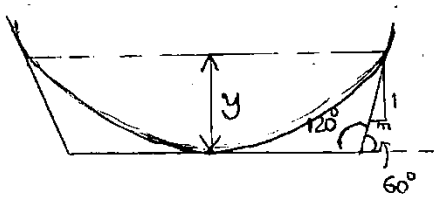
Top width = 2 x inclined side.

$$R = \frac{y}{2}$$

Condition 2: m (variable)
y (constant).

$$\frac{dP}{dm} = 0.$$

$$m = \frac{1}{\sqrt{3}}$$



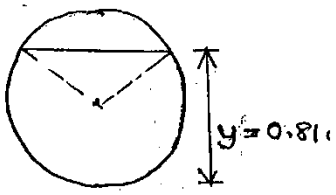
If centre of top width as origin, and depth of flow as radius, if a semi-circle is drawn, it will touch all the 3 sides tangentially; it means an efficient trapezoidal section is half of a regular hexagon.

(iv) Circular Channel Running Partially.

Condition 1: For max. velocity

$$y = 0.81\, d.$$

$$R = 0.3\, d.$$



condition 2: For max Q,

$$y = 0.93\, d$$

$$R = 0.29\, d.$$

→ Hydraulic Efficiency vs. Economic Channels

Hydraulic efficiency is related to discharge carrying capacity (to the mass) of a given shape of channel

Different shapes of efficient channels may have same area of c/s. But considering the cost of construction economy shall be decided among various shapes. The perimeter of various shapes are as follows:-

Semi-circle < Trapezium < (Rectangle = Triangle)

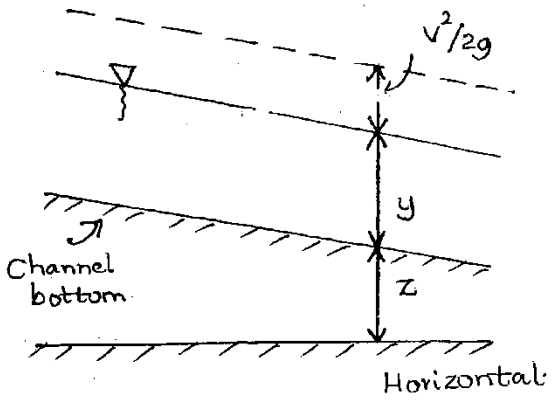
From the above, semi-circular shape is the most ~~efficient~~ economic shape. (satisfying hydraulic efficiency also)

3th Sept,
ATURDAY → Specific Energy:

The energy per unit ~~volume~~ weight of water wrt channel bottom as datum is called Specific Energy.

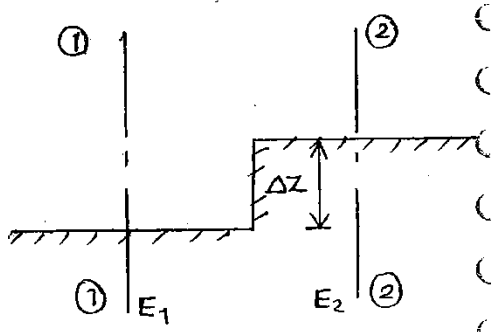
$$E = y + \frac{v^2}{2g}$$

Unit: m.



→ Specific Energy relations b/w Two sections of a Channel

Case 1: Channel bottom rise (or) hump (or) bump in the direction of flow.



Deeper the channel bottom, more the specific energy.

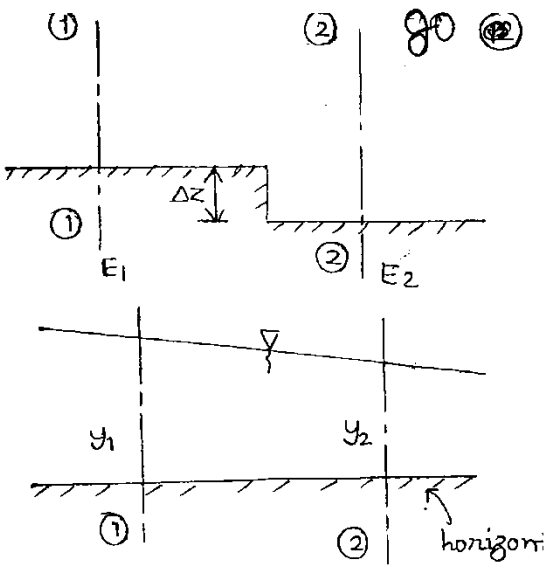
Hence, $E_1 > E_2$

$E_1 = E_2 + \Delta Z$. (neglecting h_f)

Case 2: Channel bottom drops

$E_2 > E_1$

$E_1 = E_2 - \Delta z$.



Case 3: Channel bottom horizontal.

$E_1 = E_2$ (neglecting h_f).

The two different depths possible for the same specific energy (E) are called 'Alternate Depths'.

$$E_1 = E_2 \Rightarrow y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g}$$

$$y_1\left(1 + \frac{V_1^2}{2gy_1}\right) = y_2\left(1 + \frac{V_2^2}{2gy_2}\right)$$

a) Rectangular Channel :-

$$Fr = \frac{V}{\sqrt{gy}} \quad ; \quad Fr^2 = \frac{V^2}{gy}.$$

$$\therefore \quad y_1\left(1 + \frac{Fr_1^2}{2}\right) = y_2\left(1 + \frac{Fr_2^2}{2}\right)$$

NOTE:

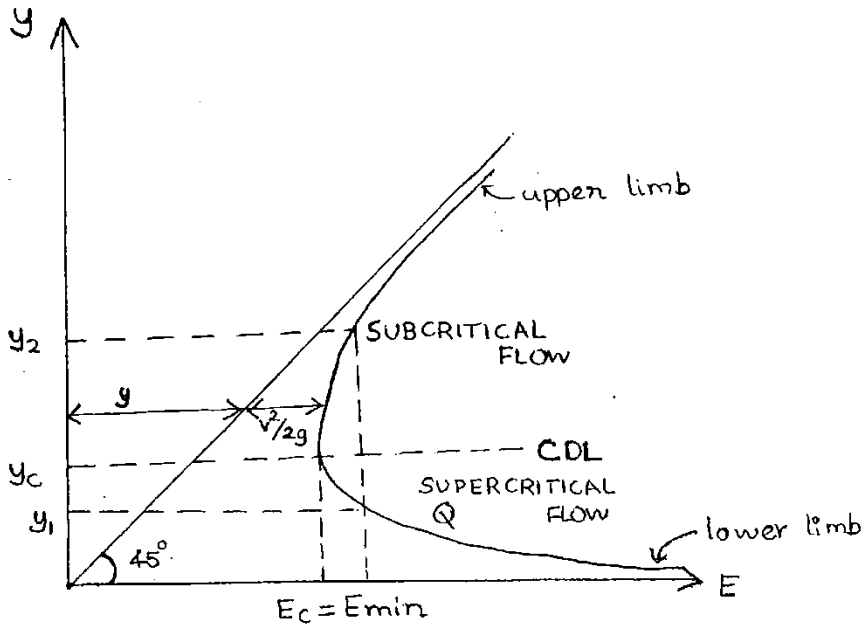
This equation shall be used if alternate depths are mentione

b) Triangular channel.

$$Fr = \frac{V\sqrt{2}}{\sqrt{gy}} \quad ; \quad Fr^2 = \frac{2V^2}{gy}.$$

$$\therefore \quad y_1\left(1 + \frac{Fr_1^2}{4}\right) = y_2\left(1 + \frac{Fr_2^2}{4}\right)$$

→ Specific Energy Curve.



- Specific energy curve is drawn for a particular discharge. If discharge changes, specific energy curve changes.

A river may have number of specific energy curves depending upon the discharge which changes from season to season.

— Critical flow is the flow corresponding to which specific energy is minimum.

— blw the CDL and the lower limb, $y < y_c$. Hence it is 'super critical region'. Similarly blw CDL and upper limb, $y > y_c$. Hence it is 'subcritical region'

— For any other E more than Emin (min specific energy), two different depths are possible (y_1 & y_2).

y_1 corresponds to supercritical flow. y_2 corresponds to subcritical flow.

— The two possible depths (y_1 & y_2) corresponding same specific energy are called Alternate depths.

— Upper limb is asymptotical to spesi '45° line' or 'potential energy line'

$$E = y + \frac{v^2}{2g}$$

$$= y\left(1 + \frac{v^2}{2gy}\right)$$

$$\frac{E}{y} = 1 + \frac{v^2}{2gy}$$

As $y \to \infty$, $\frac{E}{y} \to 1$

It means upper limb of curve is asymptotical to 45° line (potential energy line.

– Lower limb will be asymptotical to x-axis or specific energy axis.

As $y \to 0$, the specific energy curve will try to meet the x or specific energy axis asymptotically.

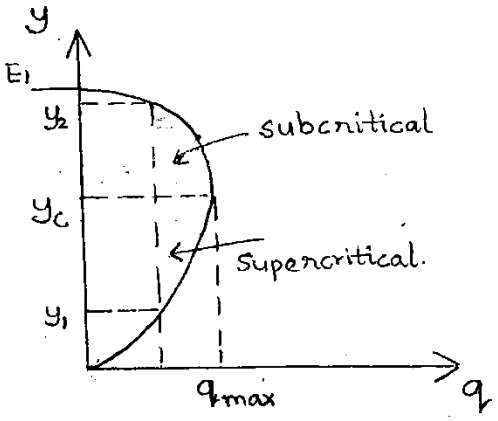
– From any point on the y-axis, the 'x offset' to the 45° line represents the corresponding potential energy line. Hence the 45° line is also called Potential energy line.

→ Discharge Diagram. (q vs y).

Discharge per 'm' width, $q = \frac{Q}{b}$

Discharge curve is drawn for a given u/s specific energy, E_1.

If u/s specific energy changes, discharge curve also changes.



Corresponding to critical flow, discharge is maximum.

For any other q less than q_{max}, two different depths are possible.

→ Specific Force (F)

The force per unit weight of water is called Specific Force, F.

NOTE: Specific force is constant if the channel is horizontal at every section neglecting losses.

$$F = \frac{Q^2}{gA} + A\bar{z}$$

$\frac{Q^2}{gA}$ → momentum of flow passing the channel section per unit time per unit weight of fluid

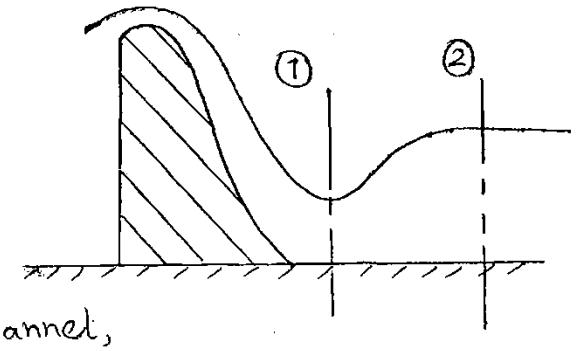
$A\bar{z}$ → hydrostatic pressure force per unit weight of fluid.

Any parameter by unit time is called corresponding flux.

Hence $\frac{Q^2}{gA}$ → momentum flux per unit weight of fluid.

$\bar{z}$ → depth of CG of c/s from free surface.

Specific force concept is used in the analysis of hydraulic jump.



$$F_1 = F_2$$

$$\left(\frac{Q^2}{gA} + A\bar{z}\right)_1 = \left(\frac{Q^2}{gA} + A\bar{z}\right)_2$$

In this eqn, Q, width of channel, either y_1 or y_2 will be known. Hence only one unknown.

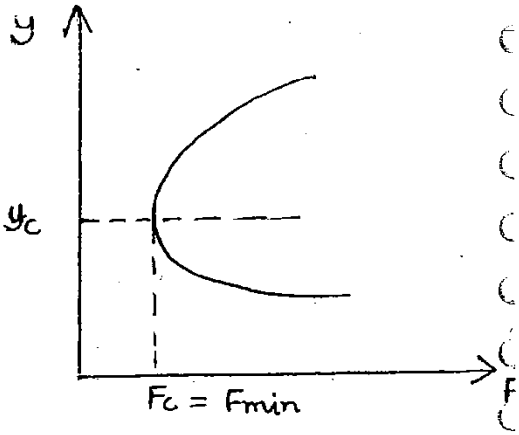
These two possible depths of same specific force are called Conjugate depths or Sequent depths

- Corresponding to critical flow, specific force is minimum.

For any other specific force,

$F > F_{min}$,

two different depths (y_1 & y_2) are possible called Conjugate or Sequent depths



→ Critical Flow

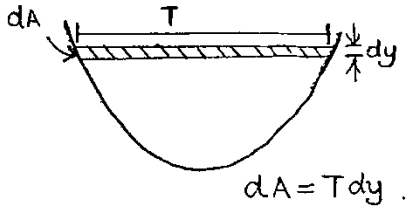
The flow corresponding to which specific energy is minimum is called Critical Flow

$$E = y + \frac{V^2}{2g}$$

$$E = y + \frac{Q^2}{2gA^2}$$

$$E_{min} \Rightarrow \frac{dE}{dy} = 0$$

$$1 + \frac{Q^2}{2g}\left(\frac{-2}{A^3}\right)\frac{dA}{dy} = 0.$$



$dA = T\,dy$.

CONDITION FOR CRITICAL FLOW $\Rightarrow$ $\boxed{\frac{Q^2}{g} = \frac{A^3}{T}}$

NOTE :

⊙ This equation is valid for any shape of channel for critical flow.

⊙ $$\frac{Q^2}{gA^2} = \frac{A}{T}$$

$$\frac{V^2}{g} = D$$

$$\frac{V}{\sqrt{gD}} = 1 = Fr$$

$\therefore$ $Fr = 1$ for critical flow.

⊙ $$\frac{V^2}{g} = D.$$

$$\frac{V^2}{2g} = \frac{D}{2}$$

For critical flow, velocity head $= \frac{1}{2}$ (hydraulic depth) for any shape of channel.

1. Rectangular Channel

$$\frac{Q^2}{g} = \frac{A^3}{T} = \frac{b^3 y_c^3}{b}$$

$$y_c^3 = \frac{Q^2}{g b^2}$$

$$\therefore \quad y_c = \left(\frac{Q^2}{g b^2}\right)^{1/3}$$

$$q = \frac{Q}{b} \Rightarrow \boxed{y_c = \left(\frac{q^2}{g}\right)^{1/3}}$$

$$E_c = y_c + \frac{V_c^2}{2g} = y_c + \frac{D}{2}$$

$$= y_c + \frac{y_c}{2}$$

$$\boxed{E_c = \frac{3}{2} y_c}$$

2. Triangular Channel

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

$$\frac{Q^2}{g} = \frac{(my^2)^3}{2my}$$

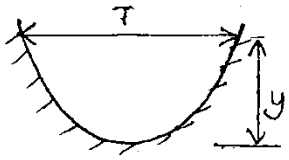
$$\frac{2Q^2}{gm^2} = y_c^5$$

$$\therefore \boxed{y_c = \left(\frac{2Q^2}{gm^2}\right)^{1/5}}$$

$$E_c = y_c + \frac{V^2}{2g} = y_c + \frac{D}{2} = y_c + \frac{y_c/2}{2}$$

$$\boxed{E_c = 1.25\, y_c}$$

3. Parabolic Channel.



$A = \frac{2}{3} Ty$

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

$$= \left(\frac{2}{3}\right)^3 \frac{(Ty)^3}{T}$$

$$y_c^3 = \left(\frac{3}{2}\right)^3 \frac{Q^2}{gT^2}$$

$$\therefore \boxed{y_c = \frac{3}{2}\left(\frac{Q^2}{gT^2}\right)^{1/3}}$$

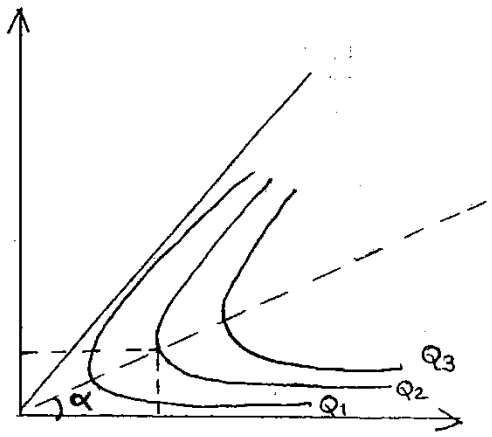
$$E_c = y_c + \frac{V_c^2}{2g} = y_c + \frac{D}{2}$$

$$= y_c + \frac{2}{3}\frac{y_c}{2}.$$

$$\boxed{E_c = \frac{4}{3} y_c}$$

th Sept,
ONDAY

The line joining the origin with minimum specific energy points for different discharges for a given shape of channel is a straight line with an angle α with the horizontal.



$Q_3 > Q_2 > Q_1$

$$\alpha = \tan^{-1}\left(\frac{y_c}{E_c}\right)$$

$= \tan^{-1}\left(\frac{2}{3}\right) = 33.69^\circ$; for rectangular channel

$= \tan^{-1}\left(\frac{3}{4}\right) = 36.86^\circ$; for parabolic channel

$= \tan^{-1}\left(\frac{4}{5}\right) = 38.65^\circ$; for triangular channel.

→ Sector Factor, K_s

$$K_s = A\sqrt{D}$$

$$\boxed{K_s = A\sqrt{\frac{A}{T}} = \frac{Q_c}{\sqrt{g}}}$$

*Physical importance of K_s.

— Based on the values of section factor, the critical discharges of different channels can be calculated straightaway.

→ Channel Transitions

Case 1 : Channel width reduction.

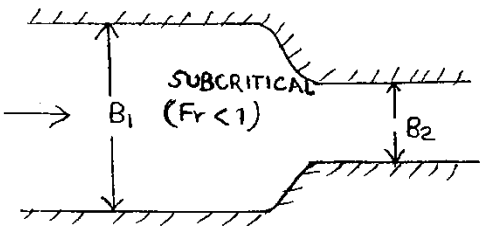
(i) Approaching flow is sub-critical.

$B_2 < B_1$

$Q_1 = Q_2 = Q$

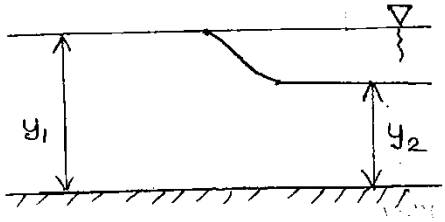
$q_1 = \frac{Q}{B_1}$ & $q_2 = \frac{Q}{B_2}$

$q_1 < q_2$



Assume channel bottom as horizontal,

$\therefore E_2 = E_1$

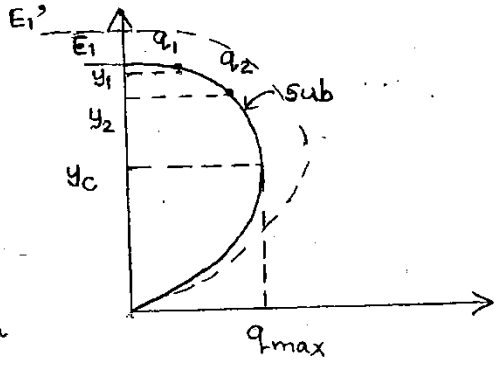


NOTE :

As $E_2 = E_1$, specific energy curve (E vs. y) is not useful in understanding the value of y_2. As 'q' changes from section 1 to section 2, discharge diagram (q vs. y) can be used

Referring discharge curve in the sub-critical region, as $q_2 > q_1$

$\Rightarrow y_2 < y_1$

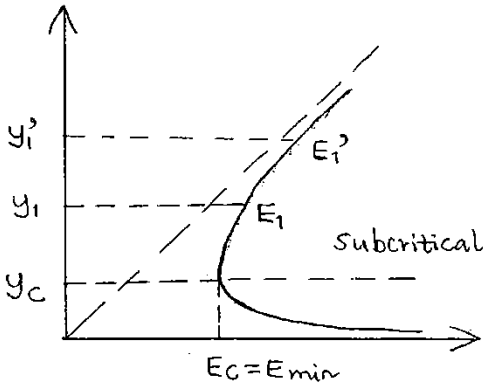


It means the level of water falls or decreases in the throat or constriction part.

Assume width of throat further decreases, then q_2 further increases and hence y_2 further decreases. If $B_2 = B_c$, ie width of throat reduces to a critical value, then $q_2 = q_{max}$ & $y_2 = y_c$.

If $B_2 < B_c$, then $q_2 > q_{max}$ which is not possible for given u/s specific energy E_1. In order to allow $B_2 < B_c$, the u/s flow gets disturbed, with increase in specific energy from E_1 to E_1'. To know the wave pattern on the upstream once it gets disturbed, specific energy curve shall be used.

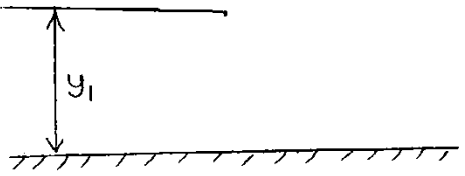
Referring specific energy curve in the sub-critical region, as E_1 increases to E_1', then y_1 increases to y_1', ie the u/s level rises from y_1 to y_1'



(ii) Approaching flow is Supercritical.

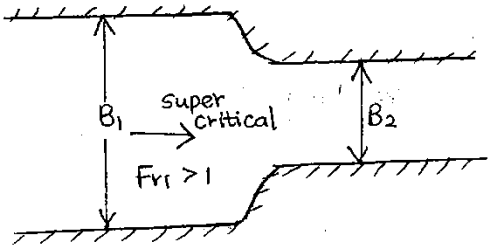
$B_2 < B_1 \Rightarrow q_2 > q_1$

Channel bottom horizontal, $E_2 = E_1$.



$q_2 > q_1$

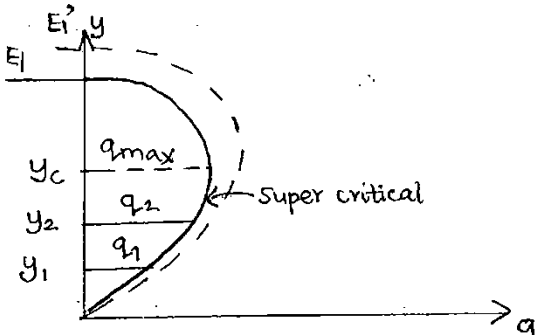
$\Rightarrow y_2 > y_1$



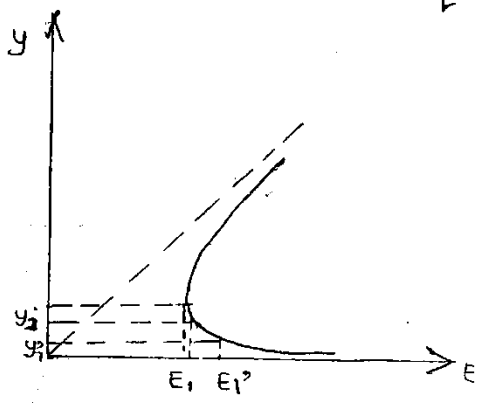
∴ depth of water rises in the contracted portion if approaching flow is supercritical.

If $B_2 = B_c$, critical flow exists in the throat portion, with.

$$q_2 = q_{max} \text{ & } y_2 = y_c$$



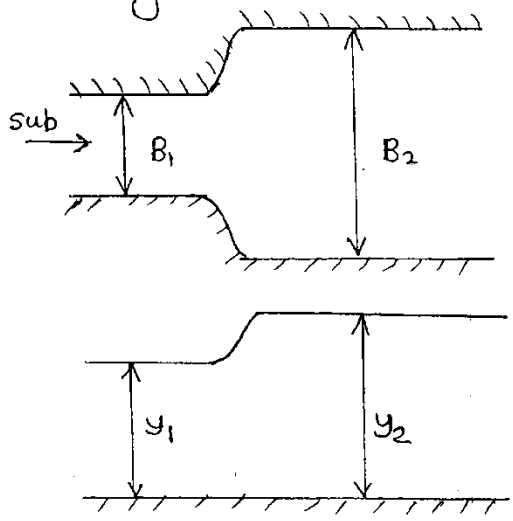
If $B_2 < B_c$, then $q_2 > q_{max}$ which is not possible for a given u/s spec. energy. In order to allow $B_2 < B_c$, the u/s flow gets disturbed with increase in spec. energy from E_1 to E_1'. It means u/s flow gets disturbed



Referring specific curve in super critical region, as E_1 increases to E_1', the u/s level decreases from y_1 to y_1'

Case 2 : Channel width increasing.

(i) Approaching flow Subcritical.



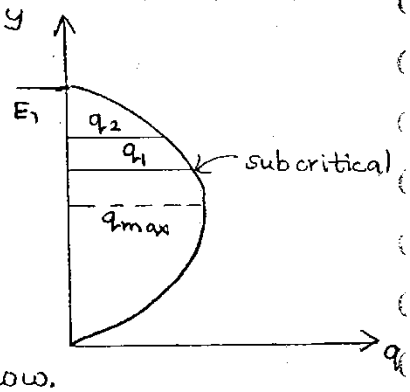
$B_2 > B_1$

$\Rightarrow q_2 < q_1$

Channel bottom horizontal,

$\therefore E_2 = E_1$

Referring discharge curve in subcritical region,



for $q_2 < q_1 \Rightarrow y_2 > y_1$

It means level of water raises in the expanded portion

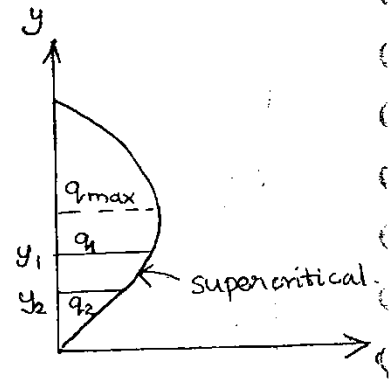
NOTE:

If $B_2 \to \infty$, $q_2 \to 0$ and hence no flow.

(ii) Approaching flow Supercritical

$B_2 > B_1 \Rightarrow q_2 < q_1$

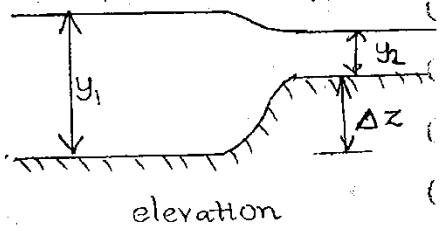
Referring discharge curve in supercritical region, $q_2 < q_1 \Rightarrow y_2 < y_1$



It means level of water falls in the expanded portion.

Case 3: Channel bottom rises (or) hump (or) bump.

a) Approaching flow subcritical.

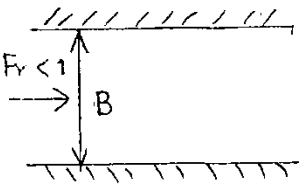


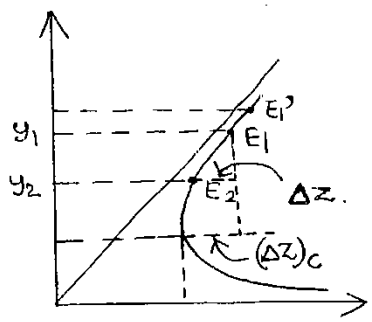
elevation

Assume width of channel is constant.

As width is assumed const, q is const. Hence discharge curve is not useful in analysing y_2. As channel bottom level rises, then $E_2 < E_1$

$$E_1 = E_2 + \Delta z.$$





Referring specific energy curve in the sub-critical region,

for $E_2 < E_1$, then $y_2 < y_1$. It means the level of water decreases in the hump portion

NOTE: 'Hump' is the friend of 'contraction' in wave pattern

If the hump height further increases, E_2 decreases for a given u/s specific energy. E_1, y_2 further decreases.

If $\Delta z = (\Delta z)_{max} = (\Delta z)_c$ (critical height of hump), then E_2 reduces to minimum or critical specific energy. It means the flow in the hump becomes critical.

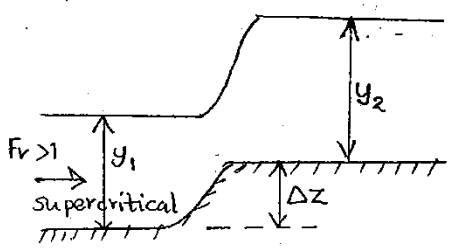
$$E_1 = E_c + (\Delta z)_c$$

If $\Delta z > (\Delta z)_{max}$, then E_2 shall be less than E_c which is not possible for a given u/s specific energy. In order to allow Δz more than $(\Delta z)_{max}$, the u/s flow gets disturbed with increase in specific energy from E_1 to E_1'. As seen in the specific energy curve, as E_1 increases to E_1', y_1 increases to y'

b) Approaching flow Supercritical.

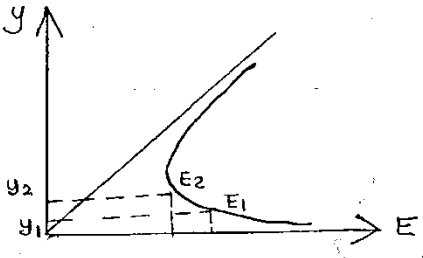
$$E_2 < E_1$$

$$\Rightarrow y_2 > y_1 \text{ (y-E diagram)}$$



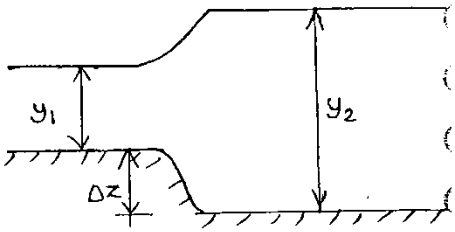
Level of water raises in the hump portion.

When $\Delta z > \Delta z_c$, E_2 shall be less than E_c which is not possible. Hence u/s flow gets disturbed with increase in sp. energy from E_1 to E_1'



Referring supercritical region of specific energy,
as E_1 increases to E_1', y_1 decreases to y_1'

Case 4: Channel bottom drops



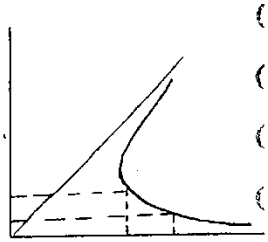
Channel width is assumed to be constant

$E_2 > E_1$

(i) Approaching flow Subcritical.

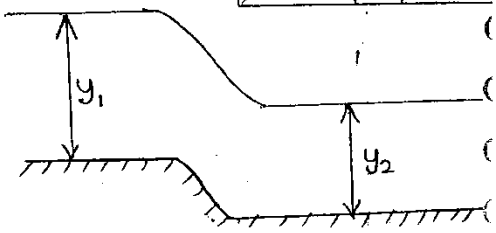
$E_2 > E_1 \Rightarrow y_2 > y_1$.

There is no limiting condition for depth of drop.



(ii) Approaching flow Supercritical.

$E_2 > E_1 \Rightarrow y_2 < y_1$

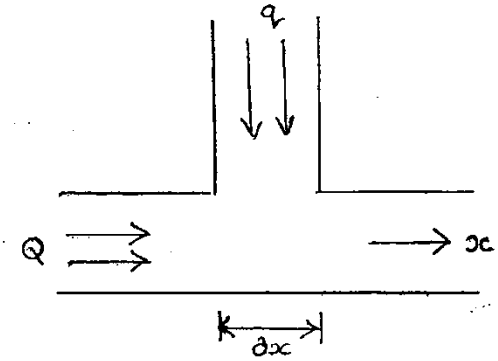


P-218

1. If there is no friction, it is called slip flow.

2. $\frac{\partial Q}{\partial x} = q$

$\Rightarrow \frac{\partial Q}{\partial x} - q = 0$



3. $Q \propto \sqrt{S}$

$\frac{15}{\sqrt{1/1440}} = \frac{Q}{\sqrt{1/1000}}$

$\Rightarrow Q = \underline{\underline{18\ m^3/s}}$

4. $Q = AV = by.\ \frac{1}{n}\ y^{2/3}\ S^{1/2}$ (wide rectangular channel, $R = y$)

$Q \propto y^{5/3}$

$\frac{Q_2}{Q_1} = \left(\frac{1.2\ y_1}{y_1}\right)^{5/3} \Rightarrow Q = \underline{\underline{1.35}}\ Q_1$ $\underline{\underline{35\%}}$

$$\frac{Q_{half}}{Q_{full}} = \left(\frac{y_{half}}{1.52}\right)^{5/3} = \frac{1}{2}$$

$$y_{half} = \underline{1 \text{ m.}}$$

06. $V = C\sqrt{RS} = C\sqrt{yS}$

$$Fr = \frac{V}{\sqrt{gy}} = \frac{C\sqrt{yS}}{\sqrt{gy}} = \underline{\frac{C\sqrt{S}}{\sqrt{g}}}$$

07. $\tau = \gamma R S_0$

$$RS_0 = \frac{6}{9810} = 6.116 \times 10^{-4}$$

$$C = \frac{V}{\sqrt{RS}} = \frac{1.5}{\sqrt{6.116 \times 10^{-4}}} = \underline{60.6}$$

08 $A = (4 + 1 \times 2)2 = 12 \text{ m}^2$

$P = 2y\sqrt{1+m^2} = 4\sqrt{2}$

$R = \frac{A}{P} = \frac{12}{4\sqrt{2}} = 2.12 \text{ m.}$

$\tau_{avg} = \gamma R S_0 = 9810 \times 2.12 \times 0.002$

$= \underline{41.6 \text{ N/m}^2}$

09. $A = my^2$

$4 = 1 \times y^2 \Rightarrow y = 2$

$$Fr = \frac{V\sqrt{2}}{\sqrt{gy}} = \frac{1.25\sqrt{2}}{\sqrt{2g}} = 0.399 \approx \underline{0.4}$$

10 $P = 2y\sqrt{1+m^2} = 2y\sqrt{2}$

$\frac{P}{y} = 2.828$

$P = 3I$ (for efficient trapezoidal channel = half of hexagon)

$= 3 y \sqrt{1+\frac{1}{3}}$

$\frac{P}{y} = \underline{3.46}$

12. $y_1 = 0.4\ m$, $y_2 = 1.6\ m$ are alternate depths.

$$y_1 + \left(\frac{q}{y_1}\right)^2 \times \frac{1}{2g} = y_2 + \left(\frac{q}{y_2}\right)^2 \times \frac{1}{2g} \qquad V = \frac{q}{y}$$

$$0.4 + \frac{q^2}{0.4^2} \times \frac{1}{2g} = 1.6 + \frac{q^2}{1.6^2} \times \frac{1}{2g}$$

$q = 2\ m^2/s$

$\therefore V_1 = \frac{q}{y_1} = \frac{2}{0.4} = 5\ m/s.$

$E_1 = y_1 + \frac{V_1^2}{2g} = 0.4 + \frac{5^2}{2g} = \underline{1.6746\ m}$

13. $y = 1.6m$, $E = 2.8m$

$E = y + \frac{v^2}{2g}$

$2.8 = 1.6 + \frac{v^2}{2g} \Rightarrow V = 4.85\ m/s$

$Fr = \frac{V}{\sqrt{gy}} = \frac{4.85}{\sqrt{g \times 1.6}} = 1.225 > 1 \Rightarrow \underline{\text{supercritical}}$

14. $Fr = 0.8$, $y = 1.5\ m$.

$Fr = \frac{V}{\sqrt{gy}} \Rightarrow 0.8 = \frac{V}{\sqrt{1.5g}} \qquad \therefore V = 3.068\ m/s$

$q = Vy = 3.06 \times 1.5 = 4.59 \ m^2/s$

$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{4.59^2}{g}\right)^{1/3} = \underline{1.29} \ m$

15. $(Fr)_n = 2.83$

$\frac{y_c}{y_n} = ?$

$Fr = \frac{V}{\sqrt{gy}} = \frac{q}{y\sqrt{gy}}$

$\Rightarrow Fr \propto \frac{1}{y^{3/2}}$; similar equation used in hydraulic jump.

$\frac{(Fr)_n}{(Fr)_c} = 1\left(\frac{y_c}{y_n}\right)^{3/2}$

$\frac{2.83}{1} = \left(\frac{y_c}{y_n}\right)^{3/2} \Rightarrow \frac{y_c}{y_n} = \underline{2}$

NOTE : $y_1\left(1 + \frac{Fr_1^2}{2}\right) = y_2\left(1 + \frac{Fr_2^2}{2}\right)$ cannot be used here, as it corresponds to same specific energy at both sections. In given problem, the critical flow and uniform flow have different specific energies. Hence, a new relation is to be used.

16. $y_1\left(1 + \frac{0.2^2}{2}\right) = y_2\left(1 + \frac{4^2}{2}\right)$.

$\frac{y_1}{y_2} = \frac{9}{1.02} = \underline{8.823}$

17. $E_c = 2.5 \ m.$

$y_c = 0.667 \ \frac{E_c}{1.25} = 2 \ m.$

$y_c = \left(\frac{2Q^2}{g m^2}\right)^{1/5}$

$2 = \left(\frac{2Q^2}{9.81 \times 1.5^2}\right)^{1/5} \Rightarrow Q = \underline{18.79} \ m^3/s$

19. $b_1 = 2m$, $y_1 = 1.2\ m$ $\quad q_1 = \frac{Q}{b_1} = \frac{3}{2} = 1.5$

$b_2 = 1.5m$, $y_2 = ?$ $\quad q_2 = \frac{Q}{b_2} = \frac{3}{1.5} = 2$

$$y_1 + \left(\frac{q}{y_1}\right)^2 \times \frac{1}{2g} = y_2 + \left(\frac{q}{y_2}\right)^2 \times \frac{1}{2g}$$ (Assume channel bottom as horizontal $\therefore E_1 = E_2$)

$$1.2 + \left(\frac{1.5}{1.2}\right)^2 \times \frac{1}{2g} = y_2 + \frac{1}{y_2^2} \times \frac{2^2}{2g}$$

$$1.28 = y_2 + \frac{0.2039}{y_2^2}$$

$$y_2 = 1.12\ m$$

Solving the above cubic equation,

$$y_2 = 1.12, -0.35, 0.52\ m.$$

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{2^2}{g}\right)^{1/3} = 0.742\ m$$

$$V_1 = \frac{Q}{b_1 y_1} = 1.25$$

$$Fr_1 = \frac{1.25}{\sqrt{g \times 1.2}} = 0.36 < 1 \Rightarrow \text{subcritical}$$

We know that, flow is subcritical. $\therefore y_2 > y_c$ at section 2. $\therefore$ $y_2 = 0.52$ & $y_2 = -0.35$ are neglected.

20. $y_1 = 1.2\ m$, $V_1 = 2.4\ m/s \Rightarrow q_1 = 2.88\ m^2/s$

$\Delta z = 0.6\ m$.

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{2.88^2}{g}\right)^{1/3} = 0.945$$

$y_1 > y_c \Rightarrow$ subcritical.

If the approaching flow is subcritical, the level of water will fall in hump portion. Hence option (b) seems to be correct. It is correct only if height of hump provided less than critical height of hump.

$$E_1 = E_2 + \Delta Z.$$

$$= E_c + (\Delta Z)_c$$

$$y_1 + \frac{V_1^2}{2g} = \frac{3}{2} y_c + (\Delta Z)_c$$

$$1.2 + \frac{2.4^2}{2g} = \frac{3}{2} \times 0.945 + (\Delta Z)_c$$

$$\therefore (\Delta Z)_c = 0.076 \text{ m}$$

Height of hump provided, 0.6 m is more than than that of $(\Delta Z)_c$. Hence u/s flow gets disturbed. with increase in its level for subcritical flow.

⊙ Moving waves are called surges.

Value of increase in u/s level, $\Delta y_1 = (y_1' - y_1) = ?$

$$E_1' = y_1' + \frac{V_1'^2}{2g}.$$

$$= y_1' + \frac{q^2}{2g\, y_1'^2}$$

$$E_1' = E_c + (\Delta Z)\text{provided}.$$

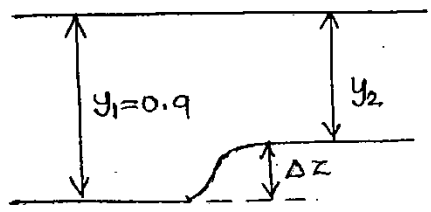
$$= 1.41 + 0.6 = 2.01 \text{ m}.$$

$$2.01 = y_1' + \frac{2.88^2}{2g \times y_1'^2}$$

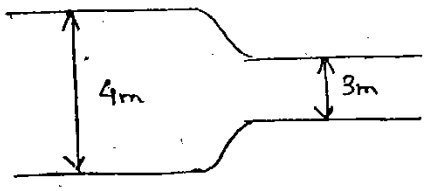
$$y_1' = 1.89, -0.42, 0.53$$

$$y_1' > y_1 \Rightarrow y_1' = 1.89$$

$$\Delta y_1 = 1.89 - 1.2 = 0.69 \text{ m}$$



21. $B_1 = 4m$



As water surface remains horizontal,

$$y_2 = y_1 - \Delta z.$$

$$E_1 = E_2 + \Delta z$$

$$y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + \Delta z$$

$$\Rightarrow V_1 = V_2 \quad \text{(flow is uniform)}.$$

Water surface remains horizontal means, the flow is uniform for which $V_1 = V_2$.

$$Q = A_1 V_1 = A_2 V_2$$

$$A_1 = A_2$$

$$b_1 y_1 = b_2 y_2.$$

$$\frac{y_2}{y_1} = \frac{b_1}{b_2}$$

$$\Rightarrow y_2 = 0.9 \times \frac{4}{3} = 1.2 \text{ m}.$$

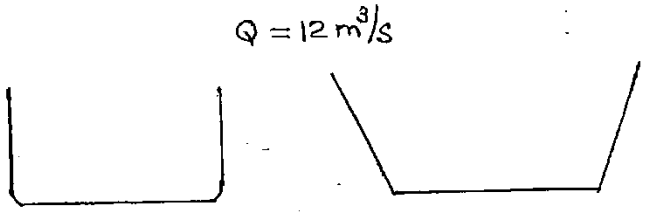
$$y_2 = y_1 - \Delta z$$

$$1.2 = 0.9 - \Delta z$$

$\therefore \Delta z = -0.3$ m (-ve indicates that assumption of hump is wrong)

Hence channel bed drops by 0.3 m

22.



$y_1 = 2$m

$b_1 = 2.4$m

$b_2 = 1.8$ m

$m = 1$

$y_2 = 1.6$ m.

$$E_1 = E_2 + \Delta z$$

$$V_1 = \frac{Q}{A_1} = \frac{12}{2.4 \times 2} = 2.5 \text{ m/s}$$

$$E_1 = 2 + \frac{2.5^2}{2\times 9.81} = 2.32 \text{ m}.$$

$$E_2 = y_2 + \frac{V_2^2}{2g}$$

$$V_2 = \frac{Q}{A_2} = \frac{12}{(1.8+1\times 1.6)1.6} = 2.21 \text{ m/s}.$$

$$E_2 = 1.6 + \frac{2.21^2}{2g} = 1.85 \text{ m}.$$

$$\Delta Z = E_1 - E_2 = 2.32 - 1.85$$

$$= \underline{0.47 \text{ m}}$$

23. $b_1 = 4$ m, $Q = 11.04$ m^3/s, $y_1 = 1.5$ m.

Assume width of channel as constant.

$$y_c = \left(\frac{Q^2}{gb^2}\right)^{1/3} = \left(\frac{(11.04)^2}{9.81\times 4^2}\right)^{1/3} = 0.92 \text{ m}.$$

$$E_c = \frac{3}{2} y_c = 1.38 \text{ m}.$$

$$E_1 = y_1 + \frac{q^2}{y_1^2 \times 2g} = 1.67 \text{ m}$$

$$1.67 = 1.38 + (\Delta Z)_c$$

$$(\Delta Z)_c = \underline{0.29 \text{ m}}$$

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24. In this problem, channel bottom is assumed as horizontal.

$$\therefore \quad E_1 = E_c$$

$$1.67 = \frac{3}{2} y_c.$$

$$\Rightarrow y_c = 1.11 \text{ m}.$$

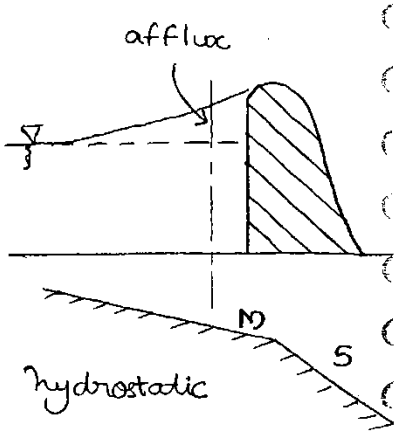
$$y_c = \left(\frac{Q^2}{g\, b_c^2}\right)^{1/3}$$

$$1.11 = \left(\frac{11.04^2}{9.81\ b_c^2}\right)^{1/3} \Rightarrow \underline{b_c = 3 \text{ m}}$$

17th Sept, WEDNESDAY

→ Gradually Varied Flow

* Reasons
  - Obstruction to flow, such as weir
  - Change of c/s and shape.
  - Change of bed slope



* Assumption
  - Pressure distribution is assumed to be hydrostatic (streamlines are straight and parallel)
  - The uniform flow formulae can be ~~eat~~ used to calcul velocity at any section.

$$V = \frac{1}{n} R^{2/3} S_0^{1/2} \quad ; \text{ uniform flow}$$

$$V = \frac{1}{n} R^{2/3} S_f^{1/2} \quad ; \text{ GVF}$$

S_0 : bed slope & S_f : energy slope

- Kinetic energy correction factor is assumed to be unity

$$\frac{p}{\gamma} + \frac{V^2}{2g} + z = \text{constant}$$; in the energy equation velocity distribution is assumed to be rectangular (uniform). But in open channel flow, it is logarithmic. So a correction factor is assumed as α, known as kinetic energy correction fact

→ Differential Equation of GVF:

$$\boxed{\frac{dy}{dx} = \frac{S_0 - S_f}{1 - \frac{Q^2 T}{g A^3}}}$$

$\frac{dy}{dx}$ → slope of water surface, wrt channel bottom

$$\frac{Q^2 T}{g A^3} = \frac{Q^2}{A^2} \cdot \frac{1}{g} \cdot \frac{1}{A/T} = \frac{V^2}{gD}$$

$$\Rightarrow \frac{Q^2 T}{g A^3} = Fr^2$$

$$\therefore \boxed{\frac{dy}{dx} = \frac{S_0 - S_f}{1 - Fr^2}}$$

Using Manning's formula, the equation of GVF is :-

$$\frac{dy}{dx} = S_0 \left[\frac{1 - \left(\frac{y_n}{y}\right)^{10/3}}{1 - \left(\frac{y_c}{y}\right)^3} \right]$$

Using Chezy's formula,

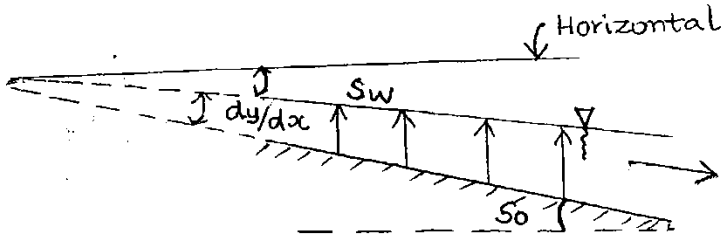
$$\frac{dy}{dx} = S_0 \left[\frac{1 - \left(\frac{y_n}{y}\right)^3}{1 - \left(\frac{y_c}{y}\right)^3} \right]$$

→ Water Surface Slope wrt to Horizontal (S_w)

Case (i) : Rising water surface. – depth is increasing in the direction of flow.

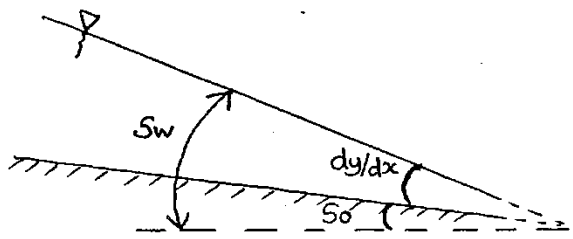
$$S_0 = S_w + \frac{dy}{dx}$$

$$\therefore S_w = S_0 - \frac{dy}{dx}.$$



Case (ii): Water surface falling – depth of water decreasing

$$S_w = S_0 + \frac{dy}{dx}$$

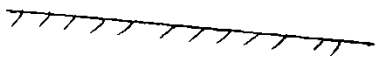


→ Differential Energy Equation of GVF

$$\frac{dy}{dx}(1 - Fr^2) = S_0 - S_f.$$

$$\boxed{\frac{\Delta E}{\Delta x} = S_0 - S_f}$$

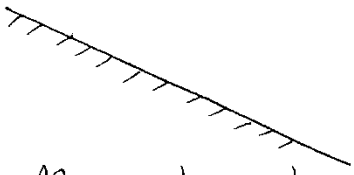
→ Classification of Slopes



Less Slope – More depth

$S_o < S_c$

y_o (or y_n) $> y_c$

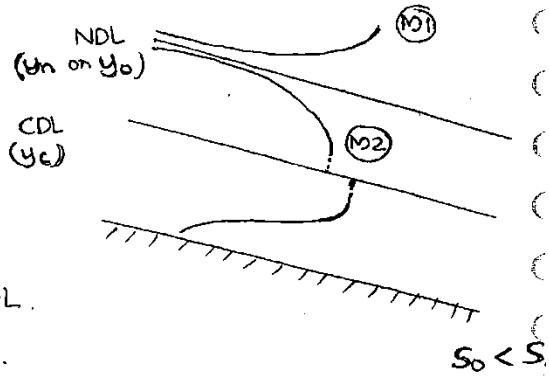


More slope – less depth

$S_o > S_c$

y_o (or y_n) $< y_c$

* Mild Slope:



First Profiles are above NDL & CDL.

Second Profiles are in blw NDL & CDL.

Third profiles are below NDL & CDL.

First & Third profiles are rising profiles.

2nd profiles are falling or drawdown curves.

$$\frac{dy}{dx} = S_o \left[\frac{1 - \left(\frac{y_n}{y}\right)^3}{1 - \left(\frac{y_c}{y}\right)^3} \right]$$

For first profile, $y > y_n > y_c$

$$\frac{dy}{dx} = S_o \left(\frac{+ve}{+ve}\right) = S_o > 0 \Rightarrow \text{rising profile.}$$

For 3rd profile (M_3), $y_n > y_c > y$.

$$\frac{dy}{dx} = S_o \left(\frac{-ve}{-ve}\right) = +ve\, S_o > 0 \Rightarrow \text{rising profile.}$$

For 2nd profile, (M_2); $y_n > y > y_c$

$$\frac{dy}{dx} = S_o \left(\frac{-ve}{+ve}\right) = -ve \Rightarrow \text{drawn down or falling curve}$$

⊙ M_1 @ u/s : $y \longrightarrow y_n$

$$\frac{dy}{dx} = S_0 \left[\frac{1 - \left(\frac{y_n}{y}\right)^3}{1 - \left(\frac{y_c}{y}\right)^3} \right] = 0.$$

$\frac{dy}{dx} = 0 \Rightarrow$ the water surface is parallel to channel bottom.

Further, we know that NDL is parallel to channd bottom.

As $y \to y_n$ (not exactly $y = y_n$), all water surface profiles meet the NDL ~~asymptotically~~ asymptotically.

⊙ M_1 @ d/s : $y \longrightarrow \infty \Rightarrow \frac{dy}{dx} = S_0$.

As $\frac{dy}{dx} \to S_0$, M_1 profile tends to become horizontal at the d/s end.

⊙ M_2 @ d/s : $y \longrightarrow y_c \Rightarrow \frac{dy}{dx} \to \infty$

As $\frac{dy}{dx} \to \infty$, all profiles water surface will meet the CDL normally on perpendicularly.

⊙ M_3 @ d/s meets the CDL normally. At u/s near the channel boundary it is not properly defined.

$M_1 \rightarrow \quad y > y_n > y_c$ (subcritical)

$M_2 \rightarrow \quad y_n > y > y_c$ (subcritical)

$M_3 \rightarrow \quad y_n > y_c > y$ (Supercritical)

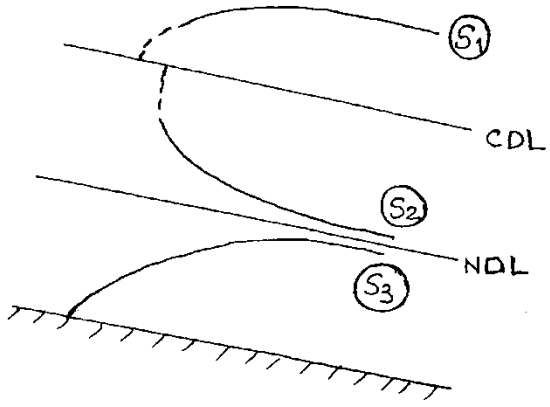
NOTE:

All the third profiles are supercritical and hence hydraulic jump is possible in all the third profiles.

* Steep Slope (S)

$S_o > S_c$

$y_n < y_c$



$S_1 \rightarrow y > y_c > y_n$ (Subcritical)

$S_2 \rightarrow y_c > y > y_n$ (Supercritical)

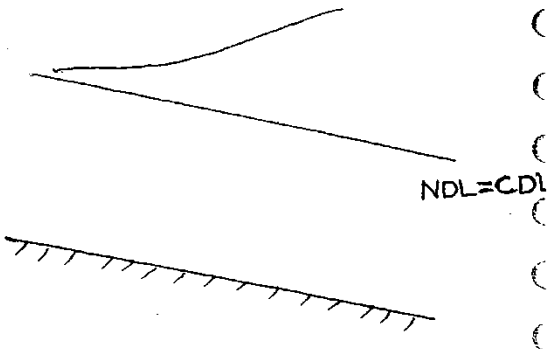
$S_3 \rightarrow y_c > y_n > y$ (Supercritical)

S_3 : Hydraulic Jump S_2 : Hydraulic drop (possible only in

* Critical Slope (C)

$S_o = S_c$

$y_n = y_c$



NOTE :

C_2 will not exist as NDL & CDL coincides.

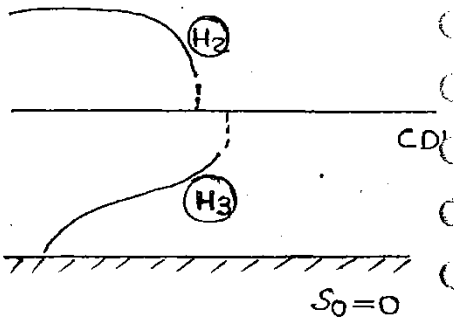
Transitional depth – Depth where NDL & CDL coincide is called transitional depth. At transitional depth, the water surface tends to become horizontal

18th SEPT, THURSDAY

* Horizontal Profile. (H)

$S_o = 0$

$y_n \rightarrow \infty$



∴ NDL do not exist, only CDL exists

$H_2 \rightarrow y > y_c$ (Subcritical)

$H_3 \rightarrow y < y_c$ (Supercritical)

H_3 : Hydraulic Jump.

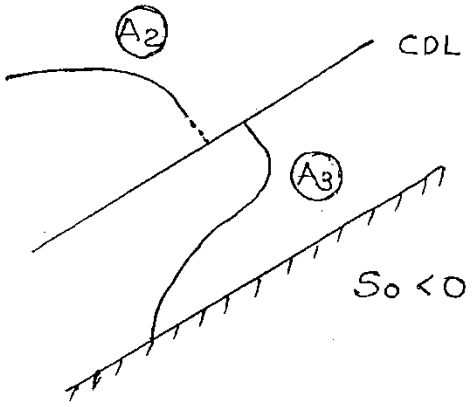
* Adverse Profiles (A)

- channel bottom rising in the direction of flow

S_0 = -ve

$y_n \rightarrow$ do not exist ; $\therefore A_1$ do not exist.

- unstable.



$A_2 \rightarrow y > y_c$; (Subcritical)

$A_3 \rightarrow y < y_c$; (Supercritical)

A_3 : Hydraulic Jump (unstable)

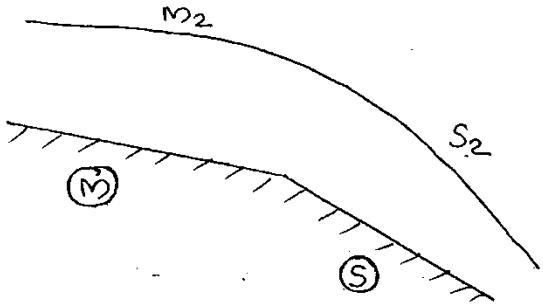
Q. Hydraulic jump is possible in which of the following profiles ...

a) M_2 b) H_2 c) M_3 d) A_3.

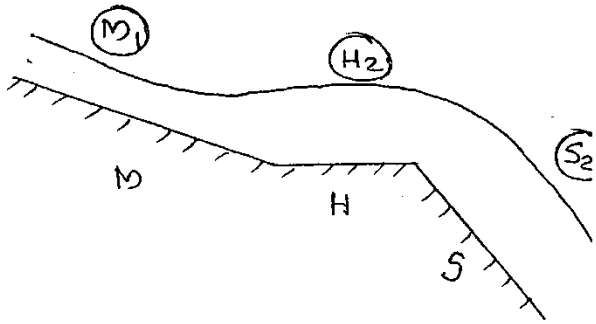
Though hydr. jump is possible in A_3, as it is unstable it is not the right answer.

→ Combinations of Water Surface Profiles :

If 2nd slope is more than 1st slope, the surface profiles are 'drawdowns'.

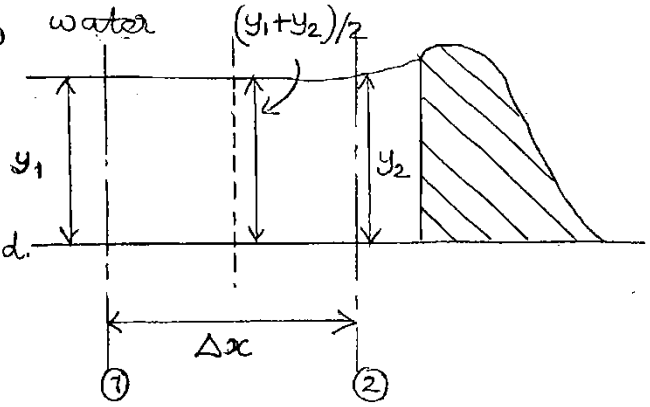


If more slope followed by less slope, profile is rising.



→ Length of Water Surface Profiles (Δx) of GVF :

Using the diff. energy equation, one can calculate the length of water surface profiles.



Method 1 : Single Step or Direct Step Method.

$$\Delta x = \frac{\Delta E}{S_0 - \overline{S_f}}$$

$\Delta E = E_1 - E_2$

$\overline{S_f}$ = average energy slope. $= \frac{S_{f1} + S_{f2}}{2}$

To save time, energy slope can be worked out for middle stream also.

Energy slope can be calculated by using either Chezy or Manning's formula.

* Demerits:
  - It is less accurate as the energy slope is not properly taken into account.
  - It is useful for prismatic (lined canals only) and not suitable for non-prismatic.

* Merits :
  - Quick solution.

Method II: Numerical or Graphical Integration Method.

$$\frac{dy}{dx} = \frac{S_0 - S_f}{1 - F_r^2}$$

$$dx = \left(\frac{1 - F_r^2}{S_0 - S_f}\right) dy.$$

$$\therefore \quad x = \int_0^1 \left(\frac{1 - F_r^2}{S_0 - S_f}\right) dy.$$

* Merits:

- It is more accurate compared to direct step method as water surface profile is more appropriately taken into account.
- It is useful for both prismatic & non-prismatic canals.

* Demerits:

- It is more time consuming.

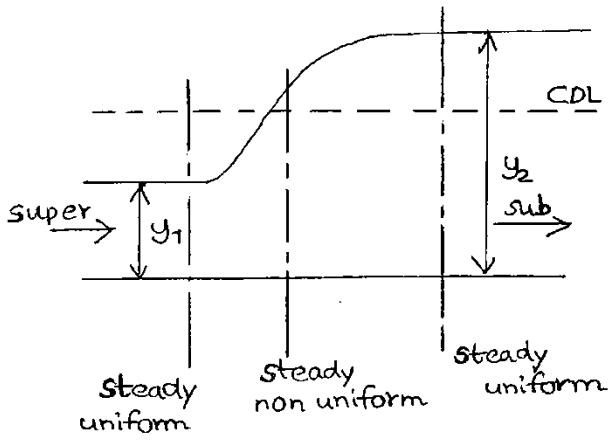
→ Hydraulic Jump

It is a steady rapidly varied flow.

y_1 : Initial / Prejump Heights

y_2 : Final / Post jump Heights

y_1 & y_2 together are called 'Conjugate or Sequent' depths.



- $\frac{y_2}{y_1}$ → Sequent depth ratio
- $(y_2 - y_1)$ → Jump height
- Length of jump $(L_j) = (5 \text{ to } 7)\, h_j$

Supercritical flow meeting subcritical flow is the condition for hydraulic jump

* Uses of Hydraulic Jump:

- energy dissipation at d/s of spillway, so that spillway is safe.
- effective mixing of chemicals in industries.

In the derivations of hydraulic jump, all the 3 fundamental equations of fluid mechanics, such as :-

(i) Energy equations

(ii) Continuity equations

(iii) Momentum equations.

are used.

NOTE:

⊙ Wherever impact, change of momentum are involved with huge loss of energy, momentum equation must be used.

⊙ In the gradually varied flow, change of momentum is negligible. Hence in its derivations only continuity and energy equations are used.

* Assumptions in the Analysis of Hydraulic Jump.

(i) Channel bottom is assumed to be horizontal. (to apply specific force concept)

(ii) Length of jump is small and hence friction is neglected

(iii) Before and after the jump, the flow is assumed to be uniform and pressure distribution is hydrostatic.

→ Relations for Hydraulic Jump of any shape of Channel.

1. Relation blw y_1 & y_2 ~~A~~ & ~~A~~

Equate $F_1 = F_2$

$$\left(\frac{Q^2}{gA} + A\bar{z}\right)_1 = \left(\frac{Q^2}{gA} + A\bar{z}\right)_2.$$

2. Energy loss in the hydraulic jump.

$$\Delta E = E_1 - E_2$$

3. Power loss (P)

$$P = \gamma Q(\Delta E)$$

* Standard Equations for Rectangular Channel

① $$\frac{2q^2}{g} = y_1 y_2 (y_1 + y_2)$$

② $$y_2 = \frac{y_1}{2}\left(-1 + \sqrt{1 + 8Fr_1^2}\right)$$

③ $$y_1 = \frac{y_2}{2}\left(-1 + \sqrt{1 + 8Fr_2^2}\right)$$

$$Fr_1^2 = \frac{q^2}{g y_1^3}; \quad Fr_2^2 = \frac{q^2}{g y_2^3}$$

④ $$\Delta E = \frac{(y_2 - y_1)^3}{4 y_1 y_2} = \frac{(V_1 - V_2)^3}{2g(V_1 + V_2)}$$

⑤ $$\frac{y_1}{y_2} = \left(\frac{Fr_2}{Fr_1}\right)^{2/3}$$

* Classification of Hydraulic Jump.

It is based on Froude number of incoming flow.

→ Surges & Wavefronts

Hydraulic Jump	Surge.
(i) Occurs because of supercritical flow meeting subcritical flow.	(i) Occur due to lifting or drop the gates across a channel.
(ii) It is stationery or fixed at a given location, hence also called stationery jump or standing wave front.	(ii) Surges are moving hydraulic jumps.
(iii) It is steady, rapidly varied flow.	(iii) Unsteady, rapidly varied flow

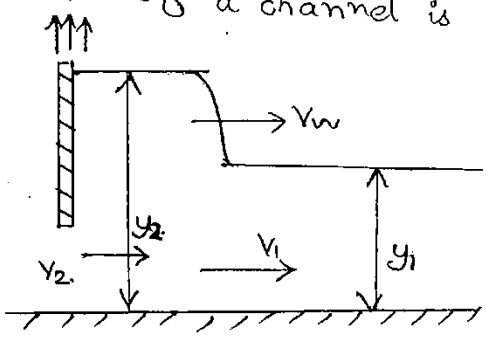
Surges:

(i) +ve surges:

The sudden rise of water by the operation of gates.

a) +ve surge moving D/s :-

- If a gate at the head or u/s of a channel is suddenly lifted.



$y_1 \rightarrow$ initial ht. of surge.

$y_2 \rightarrow$ final depth.

$y_2 - y_1 \rightarrow$ height of surge

$V_1 \rightarrow$ initial velocity

$V_w \rightarrow$ Velocity of elementary (or) surge wave (or) wave

It occurs from u/s to d/s.

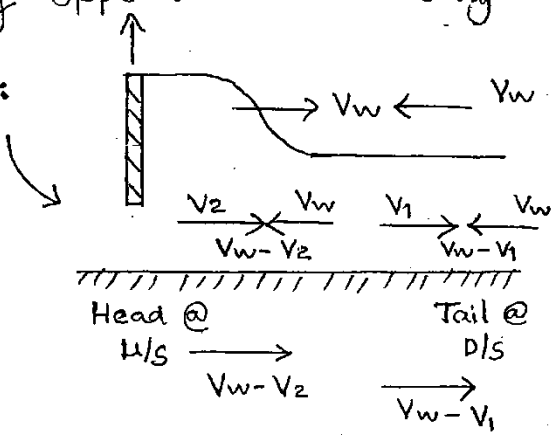
* Celerity of wave (c): It is the relative velocity of V_w & V_1.

In this problem, $c = V_w - V_1 = \sqrt{\frac{g y_2 (y_1 + y_2)}{2 y_1}}$.

* Important points in the derivation:

a) Surge is basically unsteady in nature. There is no analysis for unsteady flow directly. Unsteady flows are converted to equivalent steady state by applying opposite wave velocity.

b) Equivalent steady state is as flows:



c) Momentum equation is used to finally derive the relation.

The above equation is useful for ~~both~~ large waves. ~~and~~

Large waves $\Rightarrow (y_2 - y_1) >>> y_1$

Eg: Tidal bore, Tsunami.

Small waves $\Rightarrow$ $y_2 - y_1 <<< y_1$ (elementary waves)

Eg: Stones dropped in water.

In case of small waves,

$$C \approx \sqrt{g D_1}$$

$$C \approx \sqrt{g y_1}$$; for rectangular waves.

NOTE: '+ve surge' occurs if more flow or more water or more discharge occurs in b/w head & tail (or b/w two sections,

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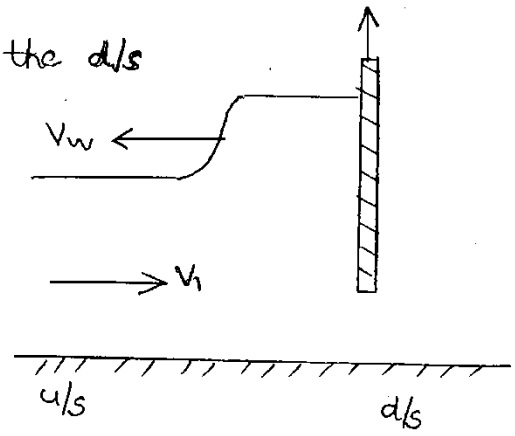
b) +ve surge moving u/s

By closing a gate at the d/s



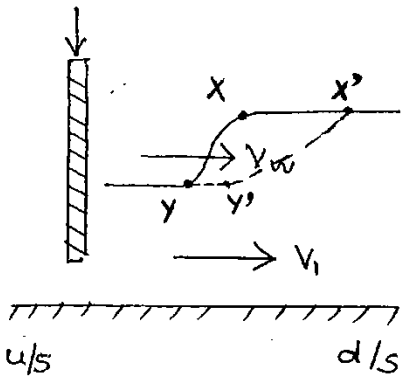
$$C = V_w + V_1$$

$$= \sqrt{\frac{g y_2 (y_1 + y_2)}{2 y_1}}$$

$$= \sqrt{g y_1}$$; for small wave.

* -ve surges

- Fall of level of water suddenly
- less flow of water b/w u/s & d/s



a) -ve surge moving d/s

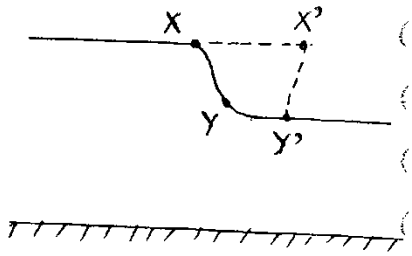
By closing a gate at the u/s

◉ -ve surge analysis is complicated as they are unstable.

In surges, top layers move faster than bottom layers As shown in fig, point x of -ve surge moves by greater distance (XX') compared to smaller distance of bottom point. Y (YY'). Due to this, the surge will lose its shape. and ∴ unsta

NOTE:

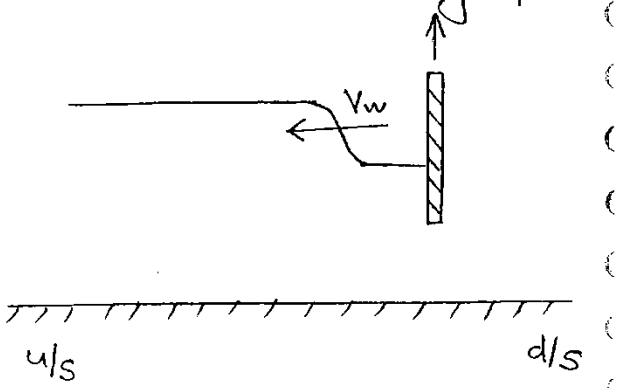
⊙ +ve surges are stable.



In +ve surges or the surge can be maintained. ∴ +ve surges are stable.

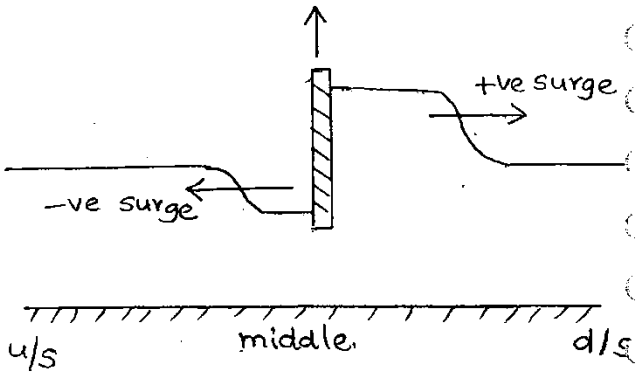
b) −ve surge moving u/s

The gate at the d/s of a channel suddenly opened

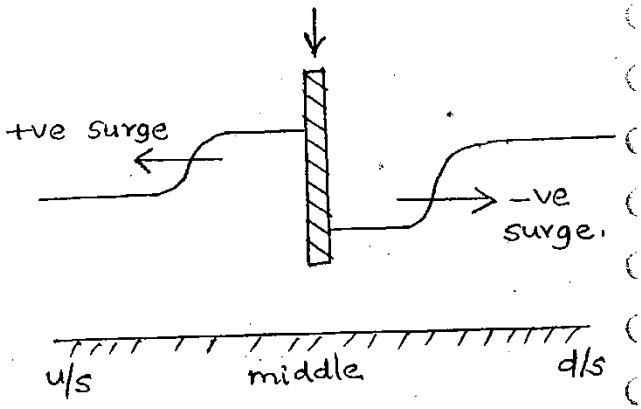


* Combination of Surges

a) Gate at the middle of a channel opened



b) Gate at the middle of a channel closed.



→ Control Section.

The sections where flow measurement is done is called Control section.

Theoretically, critical sections are better as control control section. In other flows, control sections are choosen

away from disturbances.

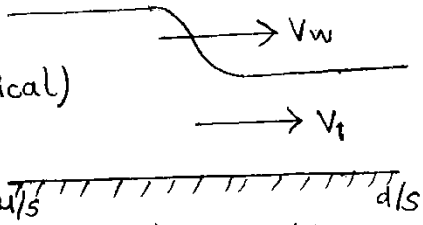
* In supercritical flow, selection of control section:

$$C = V_w - V_1 = \sqrt{gy_1}$$

For critical flow, $Fr = 1$

$$V_c = \sqrt{gy_c}$$

(supercritical)



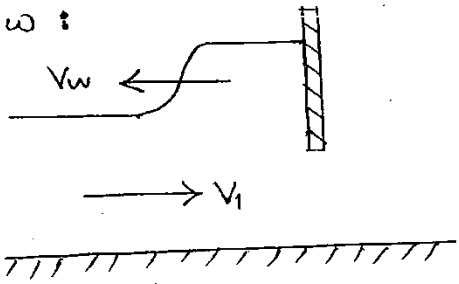
$\therefore V_w = V_1 + V_c \Rightarrow \underline{V_w > V_c}$

Wave travels towards d/s in case of supercritical flow.

If a stone is dropped in a river and if the waves travel towards d/s, it indicates supercritical flow.

In supercritical flow, control sections are selected on the u/s reaches of channel, away from disturbances

* Control sections in subcritical flow:



$$V_w + V_1 = \sqrt{gy_1}$$

$$= V_c.$$

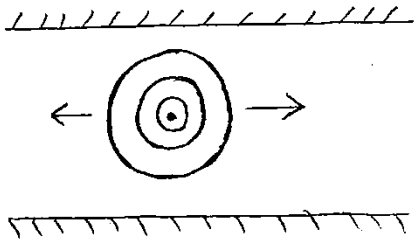
$$V_w = V_c - V_1$$

$$\Rightarrow V_w < V_c \quad \text{(Subcritical)}$$

In subcritical flow, the waves or disturbances travel towards u/s. Hence, in subcritical flow, control sections are selected on the d/s reaches of channel.

If a stone is dropped in a river and if the wave or disturbances travel towards u/s, it indicates subcritical flow.

◉ In critical flow, the waves will travel in both the directions symmetrically, if a stone is dropped.



20th sept,
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13. DIMENSIONAL ANALYSIS

→ Purpose of Dimensional Analysis

(i) Model study (or) experimental investigations.

(ii) Deriving π terms (non-dimensional terms)

(iii) Out of the various possible solution parameters, to find out the actual parameters.

→ Fourier's Theory of Dimensional Homogeneity.

- $F = ma$; rational formula valid throughout the world, hence dimensionally homogeneous.

- $F = 9.81\,m$; empirical formulae are useful in any one system only, Value of constant changes from system to system. Hence they are dimensionally non-homogeneous.

- Chezy's formula, Manning's formula - non homogeneous

→ Derivation of π terms :

$$A = f(\alpha, \beta, \gamma, \mu)$$

A ↓ dependent variable.

$\alpha, \beta, \gamma, \mu$ ↓ independent variable.

$$f(A, \alpha, \beta, \gamma, \mu) = 0.$$

Total number of variables, n = No: of independent variables + dependent variable.

So here, $n = 4 + 1 = \underline{\underline{5}}$

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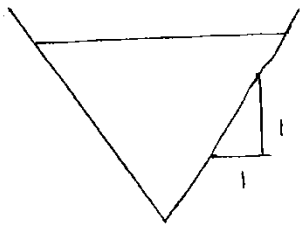
25. $S_0 = 0.001$, $Q = 0.2\ m^3/s$, $n = 0.015$



$$y_c = \left(\frac{2Q^2}{gm^2}\right)^{1/5} = \left(\frac{2\times 0.2^2}{9.81\times 1}\right)^{1/5}$$

$$= \underline{\underline{0.38\ m}}$$

$$V = \frac{Q}{A} = \frac{0.2}{my_n^2} = \frac{0.2}{1\times y_n^2} = \frac{1}{0.015}\left(\frac{y_n}{2\sqrt{2}}\right)^{2/3}\cdot (0.001)^{1/2}$$

$$y_n = 0.56\ m$$

$y_0 = y_n > y_c \Rightarrow S_0 < S_c$

$\therefore$ Mild slope.

26 $y = 0.4\ m$.

$y_c < y < y_n \Rightarrow M_2$ profile.

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$b = 15\ m$; $y = 3m$; $S_0 = \frac{1}{5000}$; $Q = 29\ m^3/s$; $C = 65$.

Depth is increasing in the direction of flow.

$$\frac{dy}{dx} = \frac{S_0 - S_f}{1 - \frac{Q^2T}{gA^3}}$$

Energy slope, S_f can be calculated using Chezy's Formula.

$$V = C\sqrt{RS_f}$$

$$\frac{29}{15\times 3} = 65\sqrt{\frac{45}{21}S_f}.$$

$$S_f = 4.587\times 10^{-5}$$

$$\frac{dy}{dx} = \frac{\frac{1}{5000} - 4.587\times 10^{-5}}{1 - \frac{29^2\times 15}{9\times 45^3}} = \underline{\underline{1.56\times 10^{-4}}}$$

Water surface slope, $S_w = S_0 - \frac{dy}{dx}$ (depth ↑ direction of flow).

$$= \frac{1}{5000} - 1.56 \times 10^{-4}$$

$$= \underline{4.366 \times 10^{-5}}$$

28. $q = 8\ m^2/s$,

$S_0 = 0.004$,

$n = 0.015$, $y_2 = 1\ m$.

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{64}{g}\right)^{1/3} = \underline{1.86\ m}$$

$$\frac{q}{y_n} = \frac{1}{n} y_n^{2/3} S_0^{1/2}.$$

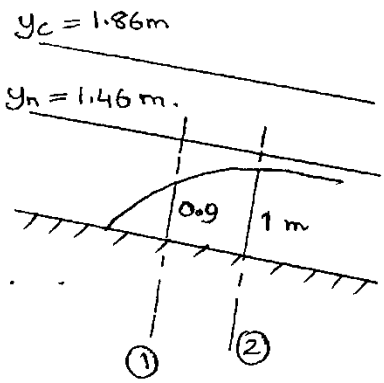
$$\frac{8}{y_n} = \frac{1}{0.015} y_n^{2/3} 0.004^{1/2}$$

$$\Rightarrow y_n = \underline{1.46\ m}$$

$y_c > y_n \Rightarrow S_c < S_0 \Rightarrow$ Steep slope.

$y_c > y_n > y \Rightarrow S_3$ profile. (raising profile).

29. As S_3 is a raising profile, 0.9m section will be on the u/s of 1m section.



$$\Delta x = \frac{\Delta E}{S_0 - \bar{S}_f} \quad \text{(direct step method)}$$

$$E_1 = y_1 + \frac{q^2}{2g y_1^2} = 0.9 + \frac{64}{2g \times 0.9^2}$$

$$= \underline{4.92\ m}$$

$$E_2 = y_2 + \frac{q^2}{2g y_2^2} = 1 + \frac{64}{2g \times 1^2} = \underline{4.26\ m}$$

$$\Delta E = 4.26 - 4.92 = \underline{-0.657\ m}$$

Calculation $\overline{S_f}$ for middle (stream section) of height $\frac{0.9+1}{2} = 0.95$

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$$V = \frac{q}{y_m} = \frac{8}{0.95} = \frac{1}{0.015} \times (0.95)^{2/3} \times (S_f)^{1/2}$$

$$\overline{S_f} = \underline{0.017}$$

$\overline{S_f} > S_0 \Rightarrow$ raising profile. (Energy slope > Bed slope)

$$\Delta x = \frac{-0.657}{(0.004-0.017)} = \underline{50.21 \text{ m}}$$

NOTE:

Energy slope > Bed slope $\Rightarrow$ Rising Profile

33 Free fall $\longrightarrow$ Draw down.

$n = 0.012$, $S_0 = 0.0004$, $q = 2 \text{ m}^3/\text{s/m}$

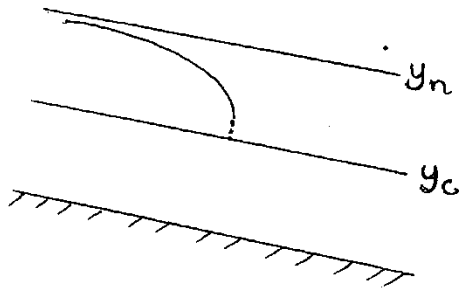
$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{4}{g}\right)^{1/3} = \underline{0.74 \text{ m}}$$

$$V = \frac{q}{y_n} = \frac{1}{n} y_n^{2/3} S_0^{1/2}.$$

$$\frac{2}{y_n} = \frac{1}{0.012} y_n^{2/3} (0.0004)^{1/2}$$

$$y_n = \underline{1.115 \text{ m}}$$

mild slope:-



$y_n > y_c \Rightarrow$ mild slope.

After several kilometres long, it will touch CDL.

As it is a falling profile, it is M_2. After several km long 2nd profile will meet CDL. Hence depth after a long distance is equal to y_c.

34 Relative loss $= \frac{\Delta E}{E_1}$

35. $q = 10\ m^2/s,\ V = 20\ m/s$

$$y_1 = \frac{q}{V} = \frac{10}{20} = 0.5\ m.$$

$$\frac{2q^2}{g} = y_1 y_2 (y_1 + y_2).$$

$$\frac{2 \times 10^2}{g} = 0.5\, y_2 (0.5 + y_2).$$

Tail water depth, $y_2 = \underline{6.14\ m}$

36. $Fr_1 = 2.5$

$$\frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1 + 8Fr_1^2}\right)$$

$$\frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1 + 8 \times 2.5^2}\right) = \underline{3.071}$$

$$\Delta E = \frac{(y_2 - y_1)^3}{4 y_1 y_2} = \frac{y_1^3 \left(\frac{y_2}{y_1} - 1\right)^3}{4 y_1 y_2}$$

$$\frac{\Delta E}{y_1} = \frac{\left(\frac{y_2}{y_1} - 1\right)^3}{4\left(\frac{y_2}{y_1}\right)} = \frac{(3.071 - 1)^3}{4 \times 3.071} = \underline{0.723}$$

37. $Q = 1\ m^3/s,\ y_1 = 0.5\ m^3/s.$

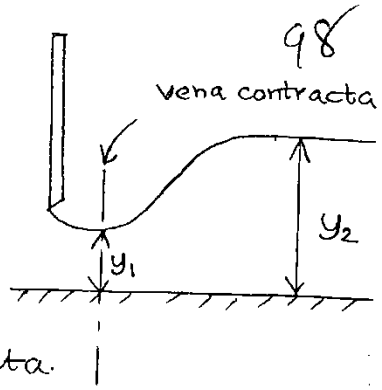
$$\left(\frac{Q^2}{gA} + A\bar{z}\right)_1 = \left(\frac{Q^2}{gA} + A\bar{z}\right)_2.$$

$$\frac{1}{g \times 0.5^2} + 0.5^2 \times \frac{0.5}{3} = \frac{1}{g \times y_2^2} + y_2^2 \times \frac{y_2}{3}$$

$$0.45 \quad 0.435 = \frac{1}{g\, y_2^2} + \frac{y_2^3}{3}$$

$$\Rightarrow y_2 = \underline{1.02\ m}$$

38. $y_2 = 3m$, $Fr_2 = \frac{1}{2\sqrt{2}}$; $C_c = 0.9$



Vena contracta is the section where c/s area of jet is least.

Say y_1 = initial depth = depth at vena contracta.

$$y_1 = \frac{y_2}{2}\left(-1 + \sqrt{1+8Fr_2^2}\right).$$

$$= \frac{3}{2}\left(-1 + \sqrt{1+8\times\frac{1}{8}}\right) = \underline{0.62\ m}$$

Coeffecient of contraction, C_c = $\frac{\text{Area at vena contracta}}{\text{Area of gap or opening}}$

$$0.9 = \frac{y_1 \times 1}{y \times 1} \quad \text{(assumed metre width)}.$$

$$\Rightarrow \quad y = \underline{0.69\ m}$$

39. $B = 4\ m$, $Q = 16\ m^3/s$

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{16}{g}\right)^{1/3} = \underline{1.177\ m}$$

$$y = \frac{q}{v} = \frac{4}{3} = \underline{1.333\ m}$$

$\Rightarrow y > y_c \Rightarrow$ subcritical.

$\therefore$ No jump occurs

41. Supercritical flow → disturbances travelling d/s.

42. $y_1 = 1.2\ m$, $V_1 = 2\ m/s$.

$$V_w + V_1 = \sqrt{g y_1} \quad \text{(wave travelling u/s)}.$$

$$V_w = \sqrt{1.2g} - 2$$

$$= \underline{1.43\ m/s}$$

In this problem, if the wave is travelling towards d/s

$$V_w = \sqrt{g y_1} + V_1 = \sqrt{1.2 g} + 2 = \underline{5.43 \text{ m/s}}$$

43. $y_2 - y_1 =$ amplitude $= 2$ cm.

$y_2 = (y_1 + 0.02)$ m.

$V_w = 1.80$ m/s.

For a pond, $V_1 = 0$. (velocity of pond is zero).

$$V_w \pm V_1 = \sqrt{\frac{g y_2}{2 y_1} (y_1 + y_2)}$$

$$V_w = \sqrt{\frac{g (y_1 + 0.02)(y_1 + y_1 + 0.02)}{2 y_1}}$$

$$1.8 = \sqrt{\frac{g (y_1 + 0.02)^2 (y_1^2 + 0.02 y_1)}{2 y_1}}$$

$$\underline{y_1 = 0.3 \text{ m}}$$

⊙ If the season changes from summer to winter, part water on surface solidifies as ice in river. The level of surface of river increases. Volume of ice is more than that of equivalent wt. of water

→ Buckingham's π theorem (or)
Method of Repeating Variables.

No: of π terms = n − m

where n → total no: of variables
m → no: of repeating variables.

⊙ Repeating variables shall contain all the fundamental or basic or primary units (such as M, L, T).

⊙ Dependent variable shall not be choosen as repeating variable.

⊙ Repeating variables consist of :-

a) a geometrical property (Area, volume, perimeter, diameter etc)

b) a kinematic property (property which describes motion of fluid)

Eg:- Velocity, acceleration, discharge.

c) a fluid or dynamic property

Eg:- mass density, viscosity.

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Q.01 $F = \phi(D^{L}, V^{T}, \omega, \mu, \rho^{M})$

$n = 6$

Mass density (ρ), velocity (V) and diameter (D) are choosen as repeating variables.

Term	Dimensions
F	MLT^{-2}
D	L
V	LT^{-1}
ω	T^{-1}
μ	$ML^{-1}T^{-1}$
ρ	ML^{-3}

No: of π terms = n − m
= 6 − 3
= 3

π_1 term :

$\pi_1 = M^0 L^0 T^0$

$= \rho^{a1} V^{b1} D^{c1} \cdot F$

$$\pi_1 = M^0 L^0 T^0 = (ML^{-3})^{a_1} (LT^{-1})^{b_1} L^{c_1} \cdot (MLT^{-2}).$$

$a_1 + 1 = 0. \Rightarrow a_1 = -1.$

$-3a_1 + b_1 + c_1 + 1 = 0. \Rightarrow 3 - 2 + c_1 + 1 = 0 \Rightarrow c_1 = -2.$

$-b_1 - 2 = 0. \Rightarrow b_1 = -2.$

$$\pi_1 = M^0 L^0 T^0 = (ML^{-3})^{-1} (LT^{-1})^{-2} L^{-2} (MLT^{-2})$$

$$\pi_1 = \rho^{-1} V^{-2} D^{-2} F. = \underline{\underline{\frac{F}{\rho V^2 D^2}}}$$

π_2 term :

$$\pi_2 = M^0 L^0 T^0 = \rho^{a_2} V^{b_2} \cdot D^{c_2} \cdot \omega$$

$$= (ML^{-3})^{a_2} (LT^{-1})^{b_2} \cdot L^{c_2} \cdot T^{-1}.$$

$a_2 = 0.; \; b_2 = -1, \; c_2 = 1.$

$$\therefore \pi_2 = \underline{\underline{\frac{\omega D}{V}}}$$

π_3 term :

$$\pi_3 = M^0 L^0 T^0 = \rho^{a_3} \cdot V^{b_3} \cdot D^{c_3} \cdot \mu$$

$$= (ML^{-3})^{a_3} (LT^{-1})^{b_3} L^{c_3} \cdot M^1 L^{-1} T^{-1}$$

$a_3 + 1 = 0 \Rightarrow a_3 = -1.$

$-b_3 - 1 = 0 \Rightarrow b_3 = -1.$

$-3a_3 + b_3 + c_3 - 1 = 0 \Rightarrow c_3 = -1.$

$$\pi_3 = \underline{\underline{\frac{\mu}{\rho V D}}}$$

* Repeating variables are called so as they are repeatedly used in the derivation of various possible π terms.

Q Time period of a pendulum is a fn of length of pendulum and g. No: of posible π terms are :

$$T = fn(l, g)$$

$n = 3, \quad m = 2.$

$n - m = 1$ possible π terms.

→ Advantages of Buckingham's Theorem

(i) It is simple even for complex problems with more no: of variables.

(ii). No: of possible π terms can be worked out initially.

→ Rayleigh's Method.

It is imposible to estimate the no: of posible π terms in this method. Hence outdated.

→ Dimensional Matrix Method

No: of π terms $= n - r$

where $n \rightarrow$ total no. of variables.

$r \rightarrow$ rank of matrix.

This method avoids the confusion of adopting MLT, FLT system.

	F	P	V	D	ω	μ
M	1	1	0	0	0	1
L	1	−3	1	1	0	−1
T	−2	0	−1	0	−1	−1

r, rank → No: of non zero row.

24th sept, WEDNESDAY

→ Model

Small scale replica of original structure.

* Scale ratio :-

$$L_r = \frac{L_m}{L_p} = \frac{B_m}{B_p} = \frac{y_m}{y_p}$$

Models

- Undistorted models
- Distorted Models

(i) Geometrically similar.

Corresponding geometrical ratios are equal.

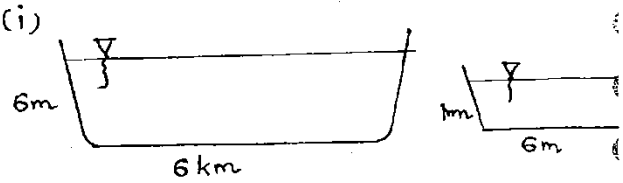
$$\frac{y_m}{y_p} = \frac{b_m}{b_p} = L_r$$

$$L_H = \frac{b_m}{b_p} = \frac{L_m}{L_p} \; ; \; L_V = \frac{y_m}{y_p}$$

$$L_H = L_V = L_r$$

(ii) Materials of model and prototype are same.

(i)



$$L_H = \frac{6}{6000} = \frac{1}{1000} \; ; \; L_V = \frac{1}{6}$$

$$L_H \neq L_V$$

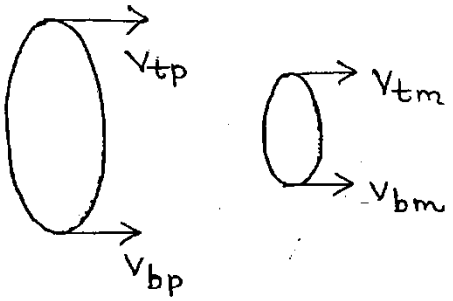
Eg: River, harbour, tidal models are distorted models.

* Similitude Criteria.

A model and prototype are completely similar, if they a

a) Geometrically Similar — similarity of shapes

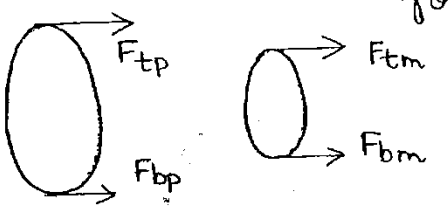
b) Kinematically Similar — similar kinematic parameters.



$$\frac{V_{tm}}{V_{tp}} = \frac{V_{bm}}{V_{bp}}$$

ie, corresponding velocity ratios are equal.

c) Dynamic Similarity — corresponding points have similar force ratios.



$$\frac{F_{tm}}{F_{tp}} = \frac{F_{bm}}{F_{bp}}$$

If all the three similarities are satisfied, then, a model and prototype are said to be completely similar

⊙ In hydraulic machine, a model and prototype are said to be completely similar if they are homologous.

Two fluid systems are said to be homologous if they have similar stream lines.

→ Froude's Law of Similarity

$$* \quad Fr = \sqrt{\frac{\text{inertia force}}{\text{gravity force}}} = \frac{V}{\sqrt{gD}}$$

$$(Fr)_m = (Fr)_p$$

$$\left(\frac{V}{\sqrt{gD}}\right)_m = \left(\frac{V}{\sqrt{gD}}\right)_p$$

$$* \text{ Velocity ratio, } V_r = \frac{V_m}{V_p} = \sqrt{\frac{g_m}{g_p}} \sqrt{\frac{D_m}{D_p}}$$

$$V_r = \sqrt{g_r} \cdot \sqrt{L_v} \text{ ; distorted model}$$

where L_v → vertical or depth scale ratio.

$$V_r = \sqrt{g_r} \cdot \sqrt{L_r} \text{ ; undistorted models.}$$

If the model and prototype are at same location,

$$g_r = \frac{g_m}{g_p} = 1$$

$$\Rightarrow \quad V_r = \sqrt{L_r}$$

$$g_{moon} = \left(\frac{1}{6}\right)^{th} g_{earth}.$$

$g_{equator} = 9.78 \text{ m/s}^2$
$g_{polar} = 9.83 \text{ m/s}.$

All the roket launching stations are located close to equator as $g_{equator} < g_{polar}$.

* Use of Froude's Law:

Froude's Law is important wherever gravitational force is important. For eg; in the analysis of open channels, spillways, notches, weirs, tides, rivers, reservoirs etc.

* Area ratio, $A_r = by = L_H \cdot L_V$; distorted model

If do $A_r = L_r^2$; undistorted model

* Volume ratio, $(Vol)_r = Lby = (L_H)^2 L_V$; distorted model

$(Vol)_r = (L_r)^3$; undistorted model

* Time ratio $(T_r) = \dfrac{L_r}{V_r}$ $\left\{V = \dfrac{L}{T}\right\}$

$$T_r = \frac{L_H}{\sqrt{L_V}} \quad ; \text{ distorted model}$$

$$T_r = \frac{L_r}{\sqrt{L_r}} = \sqrt{L_r} \quad ; \text{ undistorted model.}$$

* Acceleration ratio $(a_r) = \dfrac{V}{T}$

$$= \frac{\sqrt{L_V}}{L_H/\sqrt{L_V}}$$

$$\Rightarrow a_r = \frac{L_V}{L_H} \quad ; \text{ distorted model.}$$

$a_r = 1$; undistorted model. $(L_V = L_H = L_r)$.

$\boxed{a_m = a_p}$; as per Froude's Law of Undistorted models.

* Discharge ratio, $Q_r = A_r \cdot V_r$

$$= L_H \cdot L_V \cdot \sqrt{L_V}$$

$$\Rightarrow Q_r = L_H L_V^{3/2} \quad ; \text{ distorted model}$$

$Q_r = (L_r)^{5/2}$; undistorted model.

* Discharge per m width, $q_r = \frac{Q}{L_H} = \frac{L_H L_V^{3/2}}{L_H}$

$q_r = L_V^{3/2}$; distorted model

$q_r = L_r^{3/2}$; undistorted model.

* Mass ratio, $(\text{Mass})_r = \rho_r \times (\text{vol})_r$

$= \rho_r \times (L_r)^3$

* Momentum ratio $= M_r \cdot V_r$

$= \rho_r \cdot L_r^3 \sqrt{L_r}$

$(\text{Momentum})_r = \rho_r (L_r)^{7/2}$

In case the fluid of model and prototype are same, then $\rho_r = 1$.

$\therefore (\text{momentum})_r = (L_r)^{7/2}$

* $(\text{Work or Energy})_r = F_r \times L_r$

$= (L_r)^4$

* Force ratio, $(\text{force})_r = M_r \times a_r$

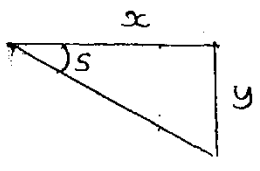
$= (L_r)^3$

* Pressure ratio, $(\text{pressure})_r = \frac{F_r}{A_r} = \frac{L_r^3}{L_r^2} = \underline{\underline{L_r}}$

* Power ratio. $= F_r \times V_r$

$= L_r^3 \cdot \sqrt{L_r} = (L_r)^{7/2}$

* Manning's n ratio :



$V = \frac{1}{n} R^{2/3} S^{1/2}$

$n = \frac{R^{2/3} S^{1/2}}{V} = \frac{L_V^{2/3} \cdot \sqrt{\frac{L_V}{L_H}}}{\sqrt{L_V}}$

$$(n)_r = \frac{L_V^{2/3}}{\sqrt{L_H}} \quad ; \text{ distorted model.}$$

$\frac{2}{3} - \frac{1}{2}$

$$(n)_r = (L_r)^{1/6} \quad ; \text{ undistorted model}$$

* Chezy's C ratio:

$$C = \frac{V}{\sqrt{RS}} = \frac{\sqrt{L_V}\ \sqrt{L_H}}{\sqrt{L_V . L_V}}$$

$$C = \sqrt{\frac{L_H}{L_V}} \quad ; \text{ distorted model}$$

$$C = 1 \quad ; \text{ undistorted model.}$$

* Hydraulic Jump ratio

$$(h_j)_r = \frac{h_{jm}}{h_{jp}} = L_V \quad ; \text{ distorted model.}$$

$$(h_j)_r = L_r \quad ; \text{ undistorted model.}$$

* Specific Energy ratio

$$(E)_r = L_V$$

→ Reynold's Law of Similarity

$$Re = \frac{\text{Inertia force}}{\text{viscous force}}$$

$$(Re)_m = (Re)_p$$

$$\left(\frac{\rho VL}{\mu}\right)_m = \left(\frac{\rho VL}{\mu}\right)_p .$$

* Velocity ratio, $V_r = \frac{V_m}{V_p} = \frac{\mu_m}{\mu_p} \cdot \frac{\rho_p}{\rho_m} \cdot \frac{L_p}{L_m}$

$$\boxed{V_r = \frac{\mu_r}{\rho_r} \cdot \frac{1}{L_r}} \qquad V_r = \frac{\gamma_r}{L_r}$$

If fluids of model & prototype are same,

$\mu_r = 1, \quad \rho_r = 1, \quad \gamma_r = 1$

$$\boxed{V_r = \frac{1}{L_r}}$$

* Use of Reynold's Law:

Wherever viscous forces are important such as in flow through pressure pipes, motion of fully submerged bodies like submarine & parachute etc.

* $(Area)_r = L_r^2$

* $(Volume)_r = L_r^3$

* $T_r = \frac{L_r}{V_r} = \frac{L_r \cdot L_r}{\gamma_r} = \frac{L_r^2}{\gamma_r} = \frac{L_r^2 \cdot \rho_r}{\mu_r}$

If fluids are same,

$$T_r = L_r^2$$

* Acceleration ratio, $a_r = \frac{1}{L_r} \cdot \frac{1}{L_r^2} = \frac{1}{L_r^3}$

* Discharge ratio, $Q_r = A_r \cdot V_r$

$= L_r^2 \times \frac{1}{L_r} = L_r$

* Mass ratio, $m_r = \rho_r L_r^3$

* Momentum ratio $= m_r \cdot V_r$

$= \rho_r L_r^3 \cdot \frac{1}{L_r} = \rho_r \cdot L_r^2$

* Force ratio $= m_r \cdot a_r$

$= \rho_r L_r^3 \cdot \frac{1}{L_r^3} = \rho_r$

* Work ratio $= F_r \cdot L_r = \rho_r L_r$

* Pressure ratio $= \frac{F_r}{A_r} = \frac{\rho_r}{L_r^2}$

Complete Class Note Solutions
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* Power ratio $= F_r \cdot V_r$

$$= \frac{P_r}{L_r}$$

* Darcy's Friction factor ratio.

$$h_f = \frac{4fLV^2}{2gD}$$

$$f = \frac{h_f \cdot 2g \cdot D}{LV^2}$$

$$(f)_r = L_r \cdot \frac{1}{L_r^3} \times L_r^2 = 1$$

$$f_r = (h_f)_r \cdot \frac{g_r}{V_r^2} \cdot \left(\frac{D}{L}\right)$$

For same location, $(g_r = 1)$.

$$f_r = \frac{L_r \cdot 1}{1/L_r^2} = L_r^3$$

→ Euler's Law of Similarity

$$\text{Euler's no., } Eu = \sqrt{\frac{\text{Inertia force}}{\text{Pressure force}}}$$

$$\frac{P}{\rho g} = \frac{V^2}{2g}$$

$$\frac{V}{\sqrt{\frac{2P}{\rho}}} = \text{constant} = \text{Euler's no.}$$

$$(Eu)_m = (Eu)_p \Rightarrow \left(\frac{V}{\sqrt{\frac{2P}{\rho}}}\right)_m = \left(\frac{V}{\sqrt{\frac{2P}{\rho}}}\right)_p$$

$$V_r = \frac{V_m}{V_p} = \sqrt{\left(\frac{\Delta P}{\rho}\right)_r}$$

Eulers law is important where pressure force is important. such as cavitation studies.

→ Mach Law of Similarity

$$M_a = \sqrt{\frac{\text{Inertia force}}{\text{Elastic force.}}}$$

Mach law is important wherever elastic forces are important such as high speed bodies like rockets, in compressible flows.

$$M_a = \frac{V}{C} = \frac{\text{velocity of body or flow}}{\text{velocity of sound.}}$$

Velocity of sound, $C = \sqrt{\frac{K}{\rho}}$

where $K \rightarrow$ bulk modulus of elasticity of fluid. system.

$\rho \rightarrow$ density of fluid

$$M_a = \frac{V}{C} = \frac{V}{\sqrt{K/\rho}}$$

$$(M_a)_m = (M_a)_p.$$

$$\left(\frac{V}{\sqrt{K/\rho}}\right)_m = \left(\frac{V}{\sqrt{\frac{K}{\rho}}}\right)_p.$$

$$\Rightarrow V_r = \sqrt{\left(\frac{K}{\rho}\right)_r}$$

⊙ If mach no., $m < 0.3$, fluids are treated as incompresible ie, compresibility is neglected.

$m_a < 1$; Subsonic flow

$m_a = 1$; Sonic flow

$1 < m_a < 3$; Supersonic

$m_a > 3$; Hypersonic flow

→ Weber's Law of Similarity

$$\text{Weber no., } W = \sqrt{\frac{\text{inertia force}}{\text{surface tension}}}$$

$$= \frac{V}{\sqrt{\frac{\sigma}{\rho l}}}$$

$$(W)_m = (W)_p$$

$$\left(\frac{V}{\sqrt{\frac{\sigma}{\rho l}}}\right)_m = \left(\frac{V}{\sqrt{\frac{\sigma}{\rho l}}}\right)_p.$$

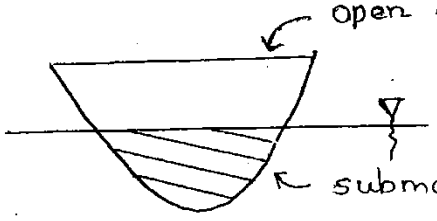
$$\Rightarrow V_r = \sqrt{\frac{\sigma_r}{\rho_r L_r}}$$

Weber's Law is important wherever surface tension is important such as in the analysis of capillarity studies and formation of spherical bubbles or droplets.

25th Sept, THURSDAY

→ Scale Effect

In certain fluid systems more than one force will be predominant. If only effect of one force is considered and others are neglected, the resulting error is called 'Scale Effect'.



For eg: in the case of a ship, the submerged part is influenced by viscous force. Hence Reynold's Law to be applied. The open part is influenced by gravitational force and hence Froude's Law to be applied. In case only one force is considered

(gravitational force), the error is scale effect.

In case, both gravity and viscous force to be considered the relation is!

$$V_r = \sqrt{g_r}\sqrt{L_r} \rightarrow \text{Froude's Law.}$$

$$= \frac{\gamma_r}{L_r} \rightarrow \text{Reynold's Law}$$

$$\frac{\gamma_r}{L_r} = \sqrt{g_r}\sqrt{L_r}$$

Say $g_r = 1$; $\therefore \boxed{\gamma_r = (L_r)^{3/2}}$

07. $V_r = \frac{1}{L_r}$

$$\frac{V_m}{V_p} = \frac{1}{1/16.}$$

$$\frac{V_m}{8} = 16. \Rightarrow V_m = \underline{128 \text{ m/s}}$$

08 $\upsilon_1 = 0.01$ stoke $\upsilon_2 = 0.03$ stoke

$V_1 = 2 \text{m/s}$ $V_2 = ?$

$d_1 = 15 \text{ cm.}$ $d_2 = 15 \text{ cm.}$

Flow through pressure pipes is governed by Reynold's Law.

$$\left(\frac{Vd}{\upsilon}\right)_1 = \left(\frac{Vd}{\upsilon}\right)_2$$

$$\frac{V_1}{V_2} = \frac{\upsilon_1}{\upsilon_2}$$

$$\Rightarrow V_2 = \frac{0.03}{0.01} \times 2 = \underline{6 \text{ m/s}}$$

09. $L_r = 1/30.$

Model	Prototype.
Tunnel	Ocean
$\upsilon_m = 1.51 \times 10^{-5}$	$V_p = 15 \text{ km/hr.}$
	$\upsilon_p = 1.02 \times 10^{-6}$

$$\left(\frac{Vd}{\nu}\right)_m = \left(\frac{Vd}{\nu}\right)_p.$$

$$L_r = \frac{d_m}{d_p} = \frac{1}{30}$$

$$\frac{V_m}{V_p} = \frac{\nu_r}{L_r} = \frac{1.51\times10^{-5}/1.02\times10^{-6}}{1/30}$$

$$V_m = \frac{1.51\times10}{1.02} \times 30 \times 15 \times \frac{5}{18} = \underline{1850.5}\ m/s$$

10 According to Froude's Laws, $a_r = 1$.

$$\therefore a_m = a_p = 1.3\ m/s^2$$

11. $L_r = 1/16$.

$$\frac{Q_m}{Q_p} = (L_r)^{5/2}.$$

$$Q_m = 1024 \times \left(\frac{1}{16}\right)^{5/2} = \underline{1\ m^3/s}$$

12. $T_r = \sqrt{L_r}$

$$\frac{T_m}{T_p} = \sqrt{\frac{1}{25}}$$

$$\frac{10}{T_p} = \frac{1}{5} \Rightarrow T_p = \underline{50 \text{ minutes}}$$

13. $L_r = \frac{1}{10}$

$F_m = 50$ N. $\quad F_p = ?$

$V_m = 5$ m/s $\quad V_p = 10$ m/s.

According to Reynold's Laws, $V_r = \frac{1}{L_r}$

According to Froude's Laws, $V_r = \sqrt{L_r}$

* The given data is not satisfying any of these laws.

Hence we have to work out from fundamentals

$$F = ma = \rho L^3 \frac{L}{T^2} = \rho L^2 \frac{L^2}{T^2} = \rho L^2 V^2$$

$$\left(\frac{F}{\rho L^2 V^2}\right)_m = \left(\frac{F}{\rho L^2 V^2}\right)_p.$$

$$F_p = F_m \cdot \frac{V_p^2}{V_m^2} \times \left(\frac{L_p}{L_m}\right)^2.$$

$$= 50 \times \left(\frac{10}{5}\right)^2 \times 10^2 = \underline{\underline{20000}}\ N.$$

14. $L_r = \frac{1}{25}$

$V_p = 10$ m/s

$$\frac{V_m}{V_p} = \sqrt{L_r}$$

$$V_m = 10\sqrt{\frac{1}{25}} = \underline{\underline{2\ m/s}}$$

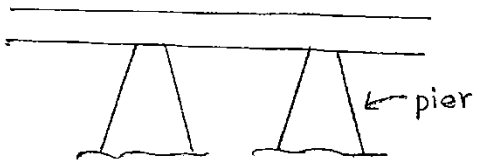
(Data is silent about viscositics. Hence ship analysis shall be done using Froude's Law)

15. $(h_j)_r = L_r$

$$\frac{(h_j)_m}{(h_j)_p} = \frac{1}{20}$$

$$\frac{0.2}{(h_j)_p} = \frac{1}{20} \Rightarrow (h_j)_p = \underline{\underline{4m}}$$

16. $F_r = L_r^3$ (Froude's Law).



$$\frac{F_m}{F_p} = \left(\frac{1}{25}\right)^3$$

$$F_p = 5 \times 25^3 = \underline{\underline{78.125}}\ kN$$

The wave force acting on bridge piers is subj. to gravitation force. Hence Froude's Law to be applied.

17. $n_r = (L_r)^{1/6}$

$$\frac{n_m}{n_p} = \left(\frac{1}{100}\right)^{1/6} \Rightarrow n_p = 0.013 \times 100^{1/6}$$

$$= \underline{\underline{0.028}}$$

18. $\frac{2q_p^2}{g} = y_1 y_2 (y_1 + y_2)$.

$$q_p = \left(\frac{0.5 \times 1.5 (0.5+1.5)}{2} \times g\right)^{1/2} = 2.712$$

$$\frac{q_m}{q_p} = L_r^{3/2}$$

$$q_m = 2.712 \times \left(\frac{1}{9}\right)^{3/2} = \underline{0.1 \ m^3/s/m}$$

21. $L_H = \frac{1}{40}$; $L_V = 1/9$.

$$\frac{Q_m}{Q_p} = L_H \cdot (L_V)^{3/2}$$

$$\frac{Q_m}{Q_p} = \frac{1}{40} \cdot \left(\frac{1}{9}\right)^{3/2}$$

$$\Rightarrow Q_p = \underline{1080} \text{ lps.}$$

23. $L_H = \frac{1}{1000}$; $L_V = \frac{1}{100}$

$$\frac{q_m}{q_p} = (L_V)^{3/2}.$$

$$\frac{0.1}{q_p} = \left(\frac{1}{100}\right)^{3/2} \Rightarrow q_p = \underline{100} \ m^2/s$$

24. $L_H = \frac{1}{150}$, $L_V = \frac{1}{60}$

The actual time interval blw two successive high tides in a sea, T_p = 12 hours 24 minutes = 744 minutes.

For distorted models, according to Froude's law,

$$T_r = \frac{L_H}{\sqrt{L_V}}$$

$$\frac{T_m}{744} = \frac{1/150}{1/\sqrt{60}} \Rightarrow T_m = \underline{38.42} \text{ minutes}$$

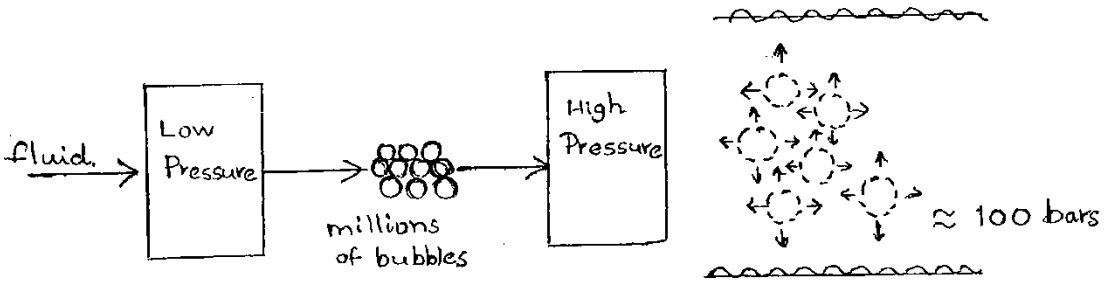
25. $V_r = 0.0894$.

$$V_r = (L_r)^{3/2}$$

$$L_r = (V_r)^{2/3} = (0.0894)^{2/3} = 0.199$$

$$\Rightarrow L_r = \text{say } 1:5$$

→ Cavitation in Hydraulic Machines.



Cavitation is due to low pressure (less than vapour pressure of flowing fluid).

Vapour pressure of water at room temp. is 2.5 m of water (absolute) or −7.8 m of water (gauge). This is also called 'Separation Head of water'.

The formation of bubbles is called Cavitation. When these bubbles reach a high pr. region, they blast or blow or condens Due to this, lot of energy is released. which acts on adjacent machine parts damaging them. This damage is called Pitting.

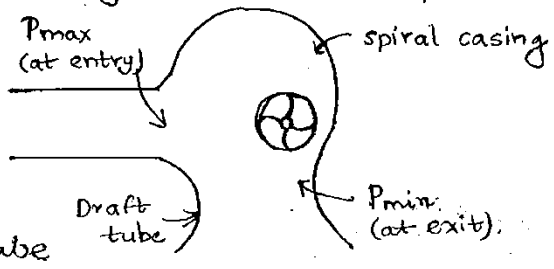
NOTE: Effeciency of machines will reduce sharply due to cavitation.

* Cavitation in Turbines

a) Pelton wheel : no cavitation

Because all its parts are subjected to atmospheric pressure.

b) Reaction Turbine



- Cavitation possible at exit of reaction turbine or entry of draft tube

as pressure is min. at this point.

- Outlet tip of blade of reaction turbine is prone for cavitation.
- Convex sides of blades are prone for cavitation.

Cavitation studies were done in hydraulic machines by Thoma. Thoma has conducted no. of experiments and gave a coeffecient called "Critical Cavitation Parameter" (σ_c) in terms of specific speed.

For any reaction turbine, actual cavitation parameter, σ

$$\sigma = \frac{H_a - H_v - H_s}{H_{net}}$$

$H_a \rightarrow$ atmospheric pressure head (in m) (absolute).

$H_v \rightarrow$ vapour pressure head (in m) (absolute)

$H_s \rightarrow$ turbine setting (height at which turbine is installed above tail race)

$H_{net} \rightarrow$ net operating head of the turbine.

For no cavitation, $\sigma \not< \sigma_c$

* Measures to control cavitation:

(i) If H_s increases, σ decreases and the turbine is prone for cavitation. Hence to avoid cavitation, if required, turbine may be submerged under water, which gives -ve value of H_s.

(ii) The last resort will be to reduce specific speed so that σ_c reduces

Q. Critical cavitation parameter, $\sigma_c = 0.1$. Net operating head, $h_{net} =$ The vapour pressure head of water $= -7.8$ m. Atmospheric pr. head $= 10.3$ (absolute). The height at which turbine to be installed to avoid cavitation is

$H_v = 10 - 7.8 = \underline{2.5\ m}$

$H_a = 10.3\ m$ (abs).

$H_{net} = 30\ m$.

$$\sigma_c = \frac{H_a - H_v - H_s}{H_{net}}$$

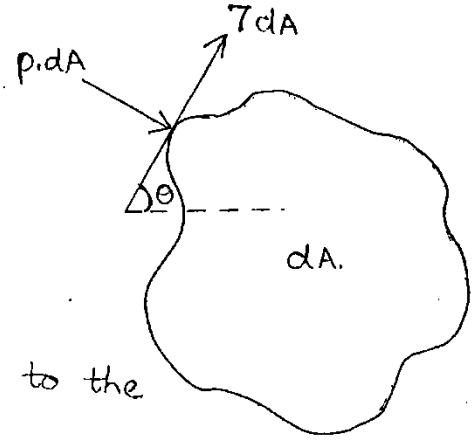
$$0.1 = \frac{10.3 - 2.5 - H_s}{30}$$

$$H_s = \underline{\underline{4.8\ m}}$$

4th Sept,
SATURDAY

11 FORCES ON SUBMERGED BODIES

Consider an infinitesmally small element of area dA. Tangent to which makes an angle of θ with horizontal.



τdA → shear force tangential to the element.

p.dA → normal force due to pressure.

* Drag Force, F_D

The component of pressure and shear forces resolved in the direction of flow.

$$F_D = \int(\tau dA \cos\theta + p dA \sin\theta)$$

- $\tau dA \cos\theta$ → shear drag (or) friction drag
- $p dA \sin\theta$ → pressure drag (or) form drag.

Pressure drag is due to differential pressures on either side of a body.

In case of ideal fluids, viscosity is zero and hence shear or skin friction drag is zero.

In case of symmetrical bodies, at high Reynold's numbers pressures will be same on both the sides. Hence they balance each other. ∴ pressure drag is zero.

∴ Total drag is zero in case of fluid flowing is ideal and the body is perfectly symmetrical.

* Lift Force, F_L

Component of pressure and shear forces in a direction ⊥r to direction of flow.

$$F_L = \int \tau dA \sin\theta - \int p dA \cos\theta.$$

NOTE:

For Ideal fluids flowing over symmetrical bodies, $F_L = 0$.

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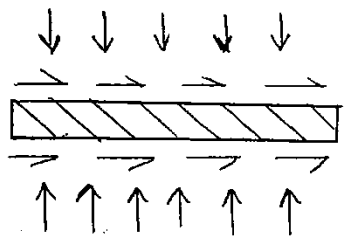
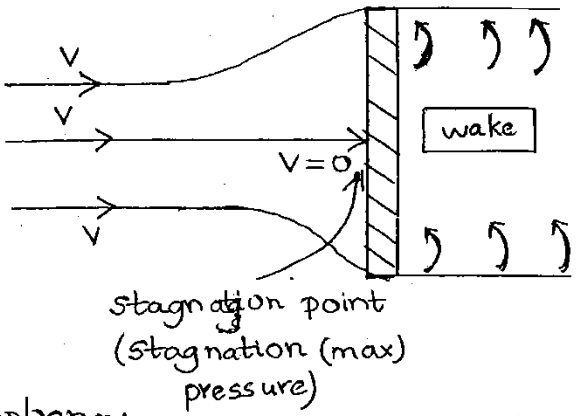
Case 1: Flow Over Horizontal Flat Plate.

In case of flow over flat plate, only shear friction / drag exists. Pressure or form drag is negligible because of perfect symmetry.



Case 2: Flow over Vertical Disc of negligible thickness

At the u/s, velocity tends to zero. called 'Stagnation point' and the pressure is maximum on u/s. The disturbed region on the d/s of an obstruction is called wake.



In order to suppress the disturbances, in the wake region lot of energy is destroyed. and pressure tends to be less. Hence there will be differential pressure on u/s & d/s. This diff. pressure causes pressure or form drag. Hence in this case, total drag primarily consist of pressure drag as shear drag is negligible due to negligible plate thickness.

Case 3: Flow over a Sphere.

(i) Approaching Flow has Laminar Boundary Layer.

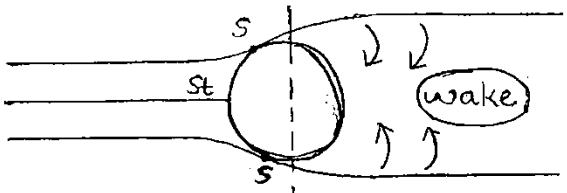
For flow over sphere,

$$Re < 2 \times 10^5$$

For flow over flat plate,

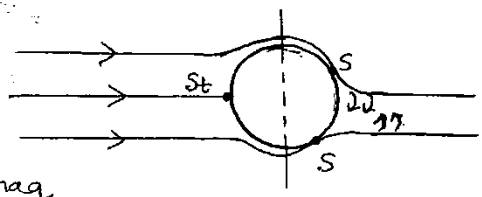
$$Re < 5 \times 10^5$$

In this case, two seperations points will form towards u/s of sphere with an angle of approximately 60°. Because of this the size of wake is very large and pressure drag is more. Even shear drag is present, but the magnitude of it is less than that of pressure drag.

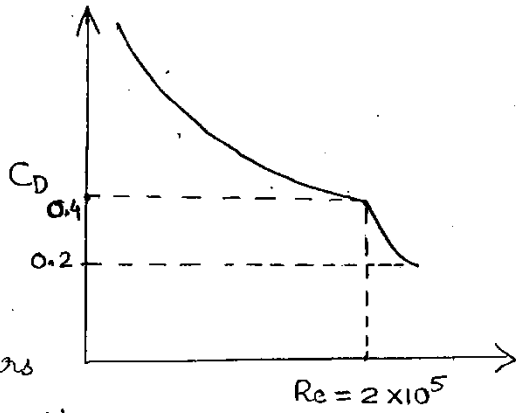


(ii) Approaching Flow has Turbulent Boundary Layer.

In this case seperation points will form on d/s. Hence the size of wake is small. Hence pressure drag is less compared to that of case (i) of LBL.

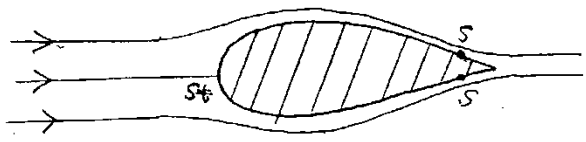


At around $Re = 2 \times 10^5$, the C_D decreases to half. It means drag resistance decreases. It has practical applications in cricket & golf. When a new ball is given, players rub it make it rough and reduce the drag resistance as the BL converts from LBL to TBL



Case 4: Aerofoil Shape

In an aerofoil, seperation points 'll form almost towards dk.



Streamlined Body

Hence the pressure drag is negligible and shear drag is predominant one.

Streamlined body: The body which suppresses the wake to the max. extend.

Bluff body: The body which has max. size of wake

For eg: vertical disc, spherical ball, cylindrical bodies.

$$F_{D\,(streamlined)} \approx \frac{1}{40} F_{D\,(vertical\ disc)} \quad \{\text{shear drag} <<< \text{pressure drag}\}$$

$$F_{D\,(sphere)} \approx \frac{1}{3} F_{D\,(vertical\ disc)}$$

→ Formula for Drag Force.

$$F_D = \frac{C_D A \rho V^2}{2}$$

where C_D → coeffecient of drag.

A → characteristic area. (= exposed area in case of plates, spheres)

A = contact surface area in case of boats, ships.

ρ → mass density of fluid.

V → free stream velocity. $(= u_\infty)$

$$F_D \propto \frac{1}{Re} \quad \& \quad C_D \propto \frac{1}{Re}$$

$$C_D = \frac{24}{Re} \quad ; \text{ for } Re < 0.1$$

$$F_D = \frac{C_D A \rho V^2}{2} = \frac{24}{Re} \frac{A\rho V^2}{2}$$

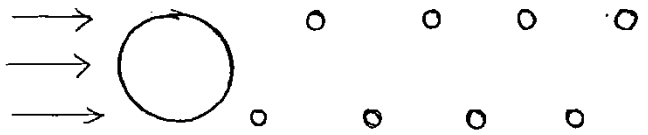
$$= \frac{24}{\frac{\rho V d}{\mu}} \cdot \frac{A\rho V^2}{2} \cdot \frac{\pi d^2}{4}$$

$$\boxed{F_D = 3\pi \mu V d} \quad ; \text{ for sphere}$$

Ratio of pressure & shear drag = 1 : 2

Case 5: Flow over cylindrical body

If $Re > 30$, alternate released of vortices called Rankine Vortex street (or) Rankine Vortex trail which causes vibration of cylindrical bodies like chimneys, cables etc.



At around $Re > 250$, the frequency of vortex shedding is equal to natural frequency of the fluid system causing Resonance. It gives music or singing of cable.

$$S = \frac{nd}{V}$$

where $S \rightarrow$ Strouhal Number

$n \rightarrow$ frequency of vortex shedding

$d \rightarrow$ diameter of cylinder.

$V \rightarrow$ free stream velocity.

Strouhal no. depends upon Reynold's number.

For $Re > 1000$, $S = 0.21$

For $Re < 1000$, $S = 0.20\left(1 - \frac{20}{Re}\right)$

$$F_L = \frac{C_L A \rho V^2}{2}$$

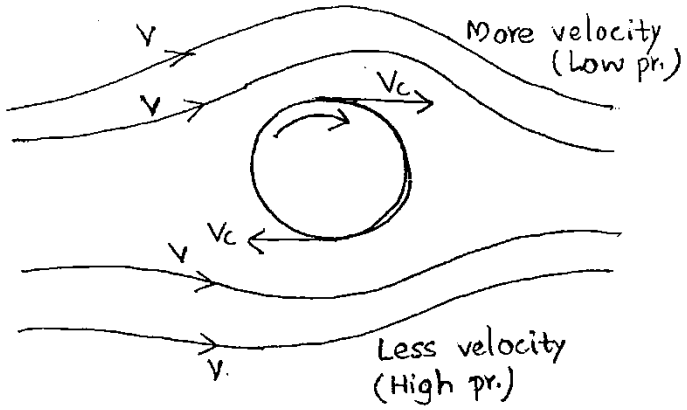
$C_L \rightarrow$ coeffecient of lift.

$A \rightarrow$ projected area (plan projection)

* Magnus Effect:

The lift force obtained by the product of spinning (circulation) with translation.

When the spin is given in downward direction, the body gets lifted in upward direction.



Q. Of the following cases, which will have more skin friction drag ?

a) Tennis ball b) Parachute c) Arrow d). Cyclist.

Parachute, arrow & cyclist will have more skin friction drag.

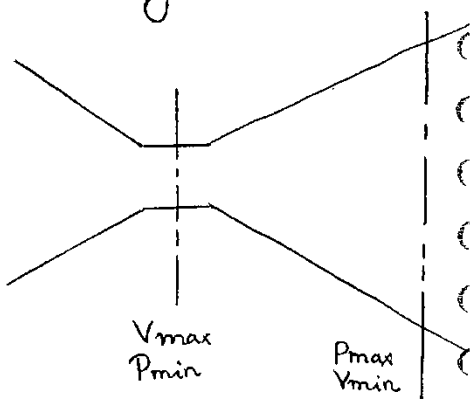
Q. Drag on cylinders and spheres reduces if the reynolds number is about 2×10^5 as flow seperation is delayed due to onset of turbulence.

Pressure drag is due to formation of disturbed region called 'wake'. Disturbance is caused due to formation of eddices and vortices. which takes place due to flow seperation.

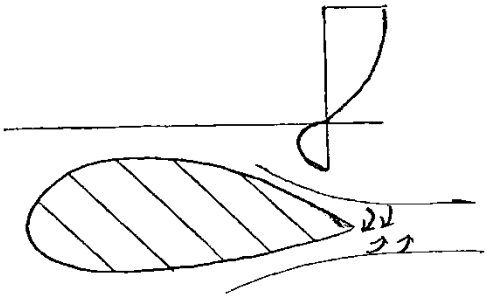
Flow seperation is because of:

(i) Adverse Pressure gradient (or) +ve pressure gradient.

When a pipe diverges from small area, pressure increases in the direction of flow. (Pressure hill); ie +ve pressure gradient. The end streamlines may not attach to the diverging boundary properly. Hence KE of flow cannot counter the +ve uphill. Hence, there will be a reverse flow in the form of eddies and vortices.

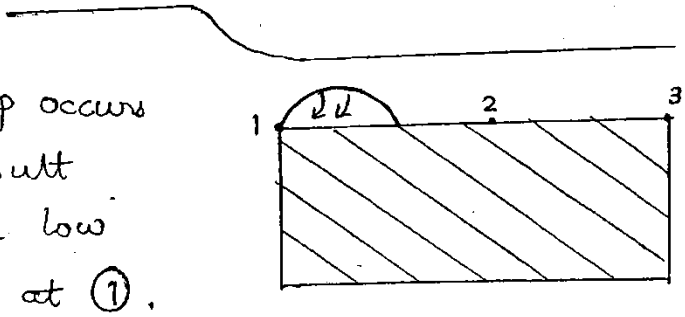


(ii) Velocity tends to zero (or) -ve velocity with an influx point.



(iii) Sharp corners over the bodies subjected to flow

When flow occurs over sharp corners, a jump occurs the sharp point. As a result eddies are formed and a low pressure region is created at ①. So cavitation occurs there. To minimise the effect, chamfered corners are provided.



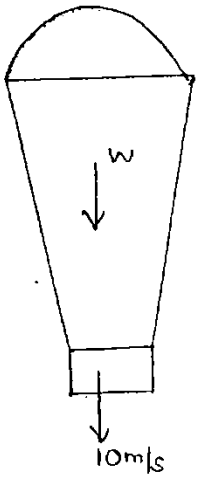
P-161

1. $P = C_D \rho A \frac{V^2}{2} \cdot V$

$\Rightarrow P \propto V^3$

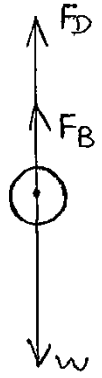
$\frac{P_2}{P_1} = \left(\frac{V_2}{V_1}\right)^3 = \left(\frac{2V}{V}\right)^3 = \underline{\underline{8}}$

2.



$w = 80\,kg$

$= 784.8\ N.$



$\rho_{air} = 1.2\ kg/m^3$

$C_D = 0.8$

Neglecting buoyancy forces,

$$w = F_D$$

$$784.8 = C_D\,\rho\,A\,\frac{V^2}{2}$$

$$= 0.8 \times 1.2 \times \frac{\pi D^2}{4} \times \frac{10^2}{2}$$

$$D = 4.56\,m$$

3 $\rho_{air} = 1.2\ kg/m^3$, $\gamma = 1.6$ stokes $= 1.6 \times 10^{-4}\ m^2/s$.

$V = 8\,m/s$, $D = 60\,mm = 0.06\ m.$

$$Re = \frac{\rho V D}{\mu} = \frac{8 \times 0.06}{1.6 \times 10^{-4}} = 3000$$

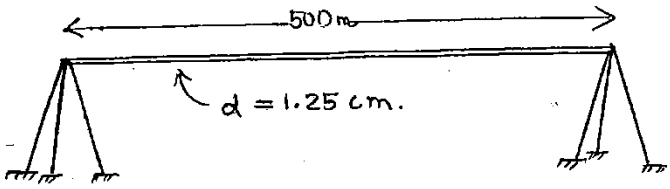
$C_D = 0.5$ for $100 < Re < 2 \times 10^5$.

Weight of ball, $w = F_D = \frac{C_D\,\rho\,A\,V^2}{2}$ (neglecting F_B)

$$= \frac{0.5 \times 1.2 \times \frac{\pi}{4} \times 0.06^2 \times 8^2}{2}$$

$$= 0.054\ N$$

4.



$V = 100\ kmph = 27.7\ m/s$

$\gamma_{air} = 13.4\ N/m^3.$

$\nu_{air} = 1.4 \times 10^{-5}\ m^2/s$

$\because Re > 10^4 \Rightarrow C_D = 1.2$

V in transverse direction

$$Re = \frac{VD}{\nu} = \frac{27.7 \times 0.0125}{1.4 \times 10^{-5}}$$

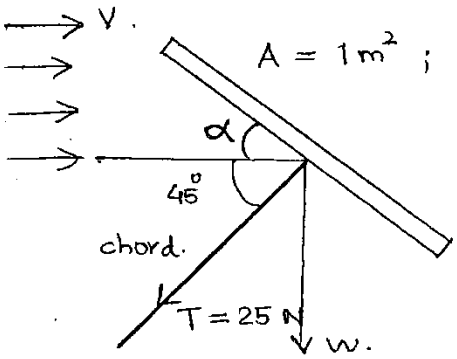
$$= 24732.14$$

Projected area is a rectangle. (L x d)

$$F_D = C_D \frac{\rho A V^2}{2} = 1.2 \times 500 \times 0.0125 \times \frac{13.4}{9.81} \times \frac{27.7^2}{2}$$

$$= 3931.651 \text{ N}$$

5.



$A = 1 m^2$; $V = 54$ kmph $= 15$ m/s

$\gamma_{air} = 12.25 \text{ N/m}^3$

$\alpha \rightarrow$ angle of attack.

Drag Force, F_D = resultant component of various forces in the direction of free stream velocity.

$$F_D = C_D \frac{\rho A V^2}{2} = 25 \cos 45.$$

$$C_D \times 1 \times \frac{12.25}{9.81} \times \frac{15^2}{2} = 25 \cos 45.$$

$$C_D = 0.126$$

Lift Force = Resultant of all forces in the direction normal to free stream velocity.

$$F_L = C_L \frac{\rho A V^2}{2} = 2.5 + 25 \sin 45.$$

$$C_L \times \frac{12.25}{9.81} \times 1 \times \frac{15^2}{2} = 20.177$$

$$\Rightarrow C_L = 0.1436$$

6. $C_{D1} = 1 \qquad C_{D2} = 0.75$

$$P_1 = P_2$$

$$C_{D1} V_1^3 = C_{D2} V_2^2$$

$$\left(\frac{V_2}{V_1}\right)^3 = \frac{C_{D1}}{C_{D2}} = \frac{1}{0.75}$$

$$V_2 = 1.1 \; V_1$$

$\Rightarrow$ 10.06 % increase in velocity.

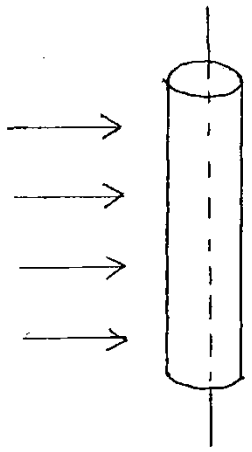
8. ~~Data is missing to~~

$$F_D = C_D \frac{\rho A V^2}{2}$$

$$30 = C_D \cdot S \cdot \rho_w \times d \times L \times \frac{V^2}{2}$$

$$= C_D \cdot 0.8 \times 1000 \times 0.08 \times 0.2 \times \frac{0.5^2}{2}$$

$$\Rightarrow \quad C_D = \underline{\underline{18.75}} \quad (?)$$

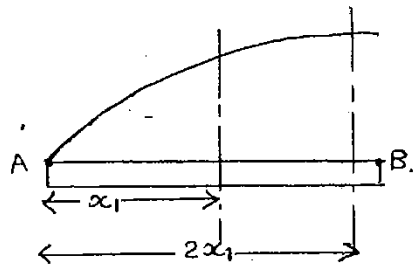


$\frac{\rho_s}{\rho_w} = S.$

Boundary layer thickness increases in the direction of flow: a) at the leading edge (or) at the front part of plate velocity gradient is steep (or) high. For a given fluid, viscosity is constant. We know,

$$\text{Shear stress, } \tau = \mu \cdot \frac{du}{dy}.$$

$$\therefore \tau_{\text{leading edge}} > \tau_{\text{trailing edge}}$$



Initial boundary layer formed will act as an obstruction for the advancing layer. and a sequence follows

$\delta \propto \sqrt{x}$; for LBL
$\delta \propto x^{4/5}$; for TBL

NOTE :

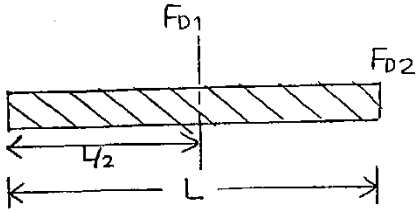
Growth of TBL is faster than growth of LBL ; other conditions remaining same as $x^{4/5} > x^{1/2}$

In the given question,

$$\frac{\delta_2}{\delta_1} = \sqrt{\frac{x_2}{x_1}} = \sqrt{\frac{2x_1}{x_1}}$$

$$\Rightarrow \quad \delta_2 = \sqrt{2}\,\delta_1$$

Q $\dfrac{F_{D1}}{F_{D2}} = ?$



Assume $C_D \propto (Re\, x)^{-1/2}$

ie $C_D \propto \dfrac{1}{\sqrt{Re \cdot x}}$

Question is assumed with LBL over a flat plate for which the relation is :

$$\frac{\delta}{x} = \frac{K}{\sqrt{Re \cdot x}}$$

$$\delta = \frac{K x}{\sqrt{\frac{V}{\nu} x}} = K \sqrt{\frac{\nu x}{V}}$$

$$\delta \propto \frac{1}{\sqrt{Re\, x}}$$

According to Blasius, $K = 5$.

$$F_D = C_D A \frac{\rho V^2}{2}$$

$F_D \propto C_D \cdot A$ (L changing from $L/2$ to L $\therefore$ A not constant)

$$\frac{F_{D1}}{F_{D2}} = \frac{C_{D1} \cdot A_1}{C_{D2} \cdot A_2} = \sqrt{\frac{Re_2}{Re_1}} \times \frac{L}{2L} = \frac{\sqrt{2}}{2} = \frac{1}{\sqrt{2}}$$

But $Re = \dfrac{VL}{\nu} \Rightarrow Re \propto L \quad \therefore \dfrac{Re_2}{Re_1} = \dfrac{2L}{L} = 2$

REFERNCES BOOKS:

Sl. NO.	Name of Authors	Titles of Book	Name of Publisher
1.	Modi & Seth	Fluid Mechanics & Hydraulic Machines	Standard Book House
2.	D.R. Biswal	Hydraulic & Fluid Mechanics	Kalyani Pbln
3.	R.K.Rajput	A text book of fluid mechanics & Hydraulic Mechanics	S.chand
4.	S.K.Garg	Irrigation Engineering & Irrigqation Structure	Khanna Publishers
5.	N.N.Basak	Irrigation Engineering	TMH Publishing
6.	S.K.Sharma	Irrigation Engineering & Hydraulic Structure	S.Chand Pbln

STRUCTURAL DESIGN – 1

SPINTRONIC TECHNOLOGY & ADVANCE RESEARCH (STAR)

DEPARTMENT OF CIVIL ENGINEERING

4TH SEMESTER

DIPLOMA

LECTURE NOTES ON

STRUCTURAL DESIGN – 1

BY

PRABHUDATTA SENAPATI

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CHAPTER-1

INTRODUCTION TO DESIGN AND DETAILING

Objectives Of Design and Detailing

Every structure must be **designed** to satisfy three basic requirements;

1) **Stability** to prevent overturning, sliding or buckling of the structure, or parts of it, under the action of loads;
2) **Strengths** to resist safely the stresses induced by the loads in the various structural members;
3) **Serviceability** to ensure satisfactory performance under service load conditions – which implies providing adequate stiffness to contain deflections , crackwidths and vibrations within acceptable limits , and also providing impermeability , durability etc.

There are two other considerations that a sensible designer ought to bear in mind, viz. **economy** and **aesthetics.**

A good structural design often involving elaborate computations is a worthwhile exercise if only it is followed by good detailing and construction practices. In normal design practices it is often seen that analysis of structures for stress resultants and design of individual members (critical sections of beams, slabs and columns) for maximum load effects(bending moments, shear, torsion and axial forces) are done regularly with insufficient attention given to supposedly lesser important aspects e.g. termination, extending and bending of bars, anchorage and development, stirrup anchorage, splices, construction details at joints or connections (slab-beam, beam-column etc.), provision of continuity and discontinuity at connection of members , construction sequencing and reinforcement placement, deflection calculations and control, crack control, cover to reinforcement ,creep and shrinkage etc.

The factors as enumerated above are very critical from the point of view of a successful structure and needs to be fairly assessed with sufficient accuracy and spelt out in detail through various drawings and specifications by the designer so that the construction of the structure can be handled by the site engineer.

Advantages Of Reinforced Concrete

The following are major advantages of reinforced cement concrete (RCC)

- Reinforced Cement Concrete has good compressive stress (because of concrete).
- RCC also has high tensile stress (because of steel).
- It has good resistance to damage by fire and weathering (because of concrete).
- RCC protects steel bars from buckling and twisting at the high temperature.

- RCC prevents steel from rusting.
- Reinforced Concrete is durable.
- The monolithic character of reinforced concrete gives it more rigidity.
- Maintenance cost of RCC is practically nil.

It is possible to produce steel whose yield strength is 3 to 4 time more that of ordinary reinforced steel and to produce concrete 4 to 5 time stronger in compression than the ordinary concrete. This may high strength material offer many advantages including smaller member cross-sections, reduce dead load and longer spans.

Different Methods of Design

Over the years, various design philosophies have evolved in different parts of the world, with regard to reinforced concrete design. A design philosophy is built upon a few fundamental assumptions and is reflective of a way of thinking.

Working Stress Method:

The earliest codified design philosophy is that of **working stress method** of design (WSM). Close to a hundred years old, this traditional method of design, based on linear elastic theory is still surviving in a number of countries. In WSM it is assumed that structural material e.g. concrete and steel behave in linearly elastic manner and adequate safety can be ensured by restricting the stresses in the material induced by working loads (service loads) on the structure. As the specified permissible (allowable) stresses are kept well below the material strength, the assumption of linear elastic behavior considered justifiable. The ratio of the strength of the material to the permissible stress is often referred to as the factor of safety. While applying WSM the stresses under applied loads are analysed by 'simple bending theory' where strain compatibility is assumed(due to bond between concrete and steel).

Ultimate Load Method:

With the growing realization of the shortcomings of WSM in reinforced concrete design, and with increased understanding of the behavior of reinforced concrete at *ultimate loads*, the ultimate load method of design (ULM) evolved in the 1950s and became an alternative to WSM. This method is sometimes also referred to as the *load factor method* or the *ultimate strength method.*

In this method, the stress condition at the state of impending collapse of the structure is analysed, and the nonlinear stress-strain curve of concrete and steel are made use of the concept of 'modular ratio' and its associated problems are avoided. The safety measure in the design is introduced by an appropriate choice of the load factor, defined as the ratio of the ultimate load(design load) to the working load. This method

generally results in more slender sections, and often more economical design of beams and columns (compared to WSM), particularly when high strength reinforcing steel and concrete are used.

Limit State Method:

The philosophy of the limit state method of design (LSM) represents a definite advancement over the traditional WSM (based on service load conditions alone) and ULM (based on ultimate load conditions alone). LSM aims for a comprehensive and rational solution to the design problem, by considering safety at ultimate loads and serviceability at working loads. The LSM uses a multiple safetyfactor format which attempts to provide adequate safety at ultimate loads as well as adequate serviceability at service loads by considering all possible 'limit states'.

CHAPTER-2

WORKING STRESS METHOD OF DESIGN

General Concept

Working stress method is based on the behavior of a section under the load expected to be encountered by it during its service period. The strength of concrete in the tension zone of the member is neglected although the concrete does have some strength for direct tension and flexural tension (tension due to bending). The material both concrete and steel, are assumed to behave perfectly elastically, i.e., stress is proportional to strain.The distribution of strain across a section is assumed to be linear. The section that are plane before bending remain plane after bending.Thus, the strain, hence stress at any point is proportional to the distance of the point from the neutral axis. With this a triangular stress distribution in concrete is obtained, ranging from zero at neutral axis to a maximum at the compressive face of the section. It is further assumed in this method that there is perfect bond between the steel and the surrounding concrete, the strains in both materials at that point are same and hence the ratio of stresses in steel and concrete will be the same as the ratio of elastic moduli of steel and concrete. This ratio being known as 'modular ratio', the method is also called 'Modular Ratio Method'.

In this method, external forces and moments are assumed to be resisted by the internal compressive forces developed in concrete and tensile resistive forces in steel and the internal resistive couple due to the above two forces, in concrete acting through the centroid of triangular distribution of the compressive stresses and in steel acting at the centroid of tensile reinforcement. The distance between the lines of action of resultant resistive forces is known as 'Lever arm'.

Moments and forces acting on the structure are computed from the service loads. The section of the component member is proportioned to resist these moments and forces such that the maximum stresses developed in materials are restricted to a fraction of their true strengths. The factors of safety used in getting maximum permissible stresses are as follows:

Material	*Factor of Safety*
For concrete	3.0
For Steel	1.78

Assumptions of WSM

The analysis and design of a RCC member are based on the following assumptions.

(i) Concrete is assumed to be homogeneous.

(ii) At any cross section, plane sections before bending remain plane after bending.

(iii) The stress-strain relationship for concrete is a straight line, under working loads.

(iv) The stress-strain relationship for steel is a straight line, under working loads.

(v) Concrete area on tension side is assumed to be ineffective.

(vi) All tensile stresses are taken up by reinforcements and none by concrete except when specially permitted.

(vii) The steel area is assumed to be concentrated at the centroid of the steel.

(viii) The modular ratio has the value $280/3\sigma_{cbc}$ where σ_{cbc} is permissible stress in compression due to bending in concrete in N/mm^2 as specified in code(IS:456-2000)

Moment of Resistance

(a) *For Balanced section*: When the maximum stresses in steel and concrete simultaneously reach their allowable values, the section is said to be a 'Balanced Section'. The moment of resistance shall be provided by the couple developed by compressive force acting at the centroid of stress diagram on the area of concrete in compression and tensile forceb acting at the centroid of reinforcement multiplied by the distance between these forces. This distance is known as 'lever arm'.

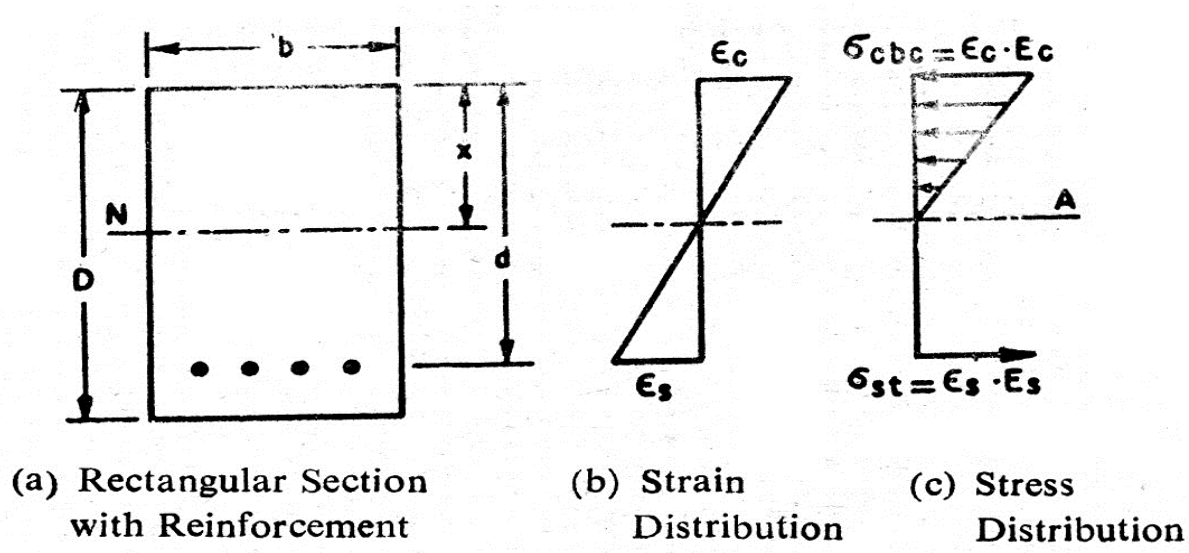


Fig.2.1 (a-c)

Let in Fig.2.1(a-c): b = width of section

D = overall depth of section

d = effective depth of section (distance from extreme compression fiber to the centroid of steel area,

A_s = area of tensile steel

ϵ_c = Maximum strain in concrete,

ϵ_s = maximum strain at the centroid of the steel,

σ_{cbc} = maximum compressive stress in concrete in bending

σ_{st} = Stress in steel

E_s/E_c = ratio of Yong's modulus of elasticity of steel to concrete

= modular ratio 'm'

Since the strains in concrete and steel are proportional to their distances from the neutral axis,

$$\frac{\varepsilon_c}{\varepsilon_s} = \frac{x}{d-x} \text{ or } \frac{d-x}{x} = \frac{\varepsilon_s}{\varepsilon_c}$$

$$\frac{d}{x} - 1 = \frac{\sigma_{st}}{E_s}\frac{E_c}{\sigma_{cbc}} = \frac{\sigma_{st}}{\sigma_{cbc}}\frac{1}{m}$$

Or $$\frac{d}{x} = 1 + \frac{\sigma_{st}}{m.\sigma_{cbc}} \text{ or } x = \frac{1}{1+\frac{\sigma_{st}}{m.\sigma_{cbc}}}.d = k.d$$

Where k= neutral axis constant= $\dfrac{1}{1+\sigma_{st}/m.\sigma_{cbc}}$

Total compressive force= $\frac{b.x.\sigma_{cbc}}{2}$

Total tensile forces = $\sigma_{st}.A_s$

Z= Lever arm= $d - \frac{x}{3} = d - \frac{k.d}{3} = d\left(1 - \frac{k}{3}\right) = j.d$

Where j is called the lever arm constant.

Moment of resistance = $MR = \frac{b.x}{2}.\sigma_{cbc}.jd = \frac{kd.j}{2}.\sigma_{cbc}.b.d = \frac{1}{2}.k.j.\sigma_{cbc}.b.d^2 = Q.b.d^2$

Where Q is called moment of resistance constant and is equal to $\frac{1}{2}.k.j.\sigma_{cbc}$

(b) Under reinforced section

When the percentage of steel in a section is less than that required for a balanced section, the section is called 'Under-reinforced section.' In this case (Fig.2.2) concrete stress does not reach its maximum allowable value while the stress in steel reaches its maximum permissible value. The position of the neutral axis will shift upwards, i.e., the neutral axis depth will be smaller than that in the balanced section as shown in Figure2.2. The moment of resistance of such a section will be governed by allowable tensile stress in steel.

Moment of resistance $= \sigma_{st}.A_s.\left(d - \frac{x}{3}\right) = \sigma_{st}A_s.j'd \qquad where \quad j' = 1 - \frac{k'}{3}$

Since $p = \frac{A_s.\times 100}{b.d}$

Moment of resistance

$$= \sigma_{st}.p.\frac{b.d}{100}.j' \times d = \frac{\sigma_{st}.p.j'}{100}.b.d^2 = Q'.Bb.d^2 \qquad where \quad Q' = \frac{\sigma_{st}.p.j'}{100}$$

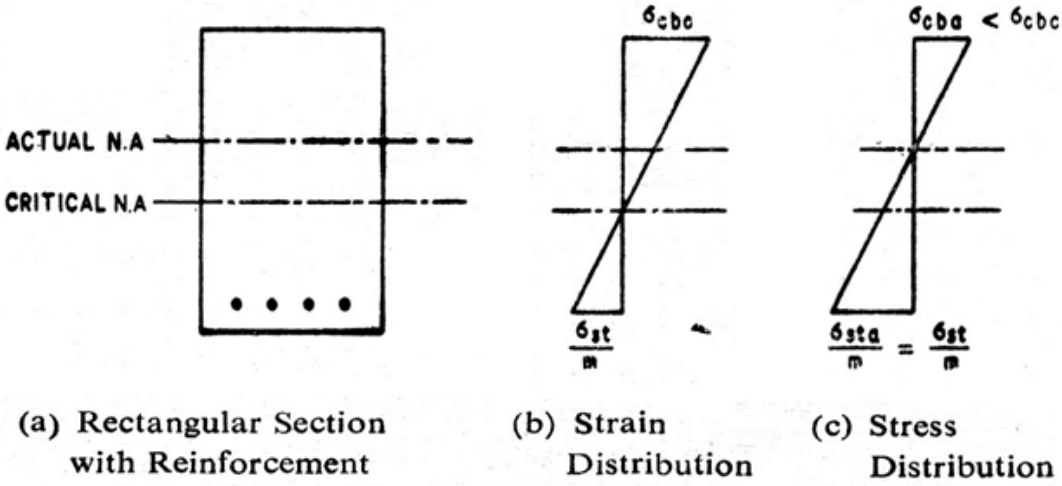
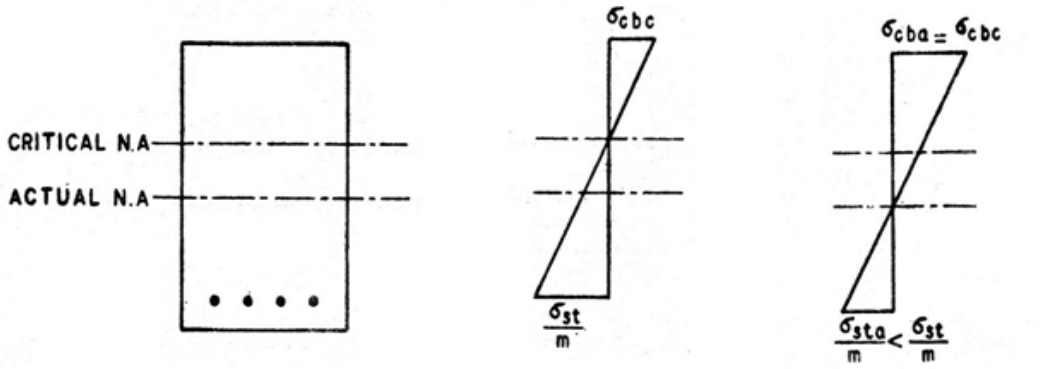


Fig.2.2 (a-c)

(c) **Over reinforced section**:

When the percentage of steel in a section is more than that required for a balanced section, the section is called 'Over-reinforced section'. In this case (Fig.2.3) the stress in concrete reaches its maximum allowable value earlier than that in steel. As the percentage steel is more, the position of the neutral axis will shift towards steel from the critical or balanced neutral axis position. Thus the neutral axis depth will be greater than that in case of balanced section.

Moment of resistance of such a section will be governed by compressive stress in concrete,



(a) Rectangular Section with Reinforcement (b) Strain Distribution (c) Stress Distribution

Fig.2.3 (a-c)

$$\text{Moment of resistance} = b.x.\sigma_{cbc}.\frac{1}{2}.\left(d-\frac{x}{3}\right)=\frac{\sigma_{cbc}}{2}.b.x.d.\left(1-\frac{k'}{3}\right)$$

$$=\frac{\sigma_{cbc}}{2}.b.x.d.j' = \frac{1}{2}.\sigma_{cbc}.k'.j'.b.d^2 = Q'.b.d^2 \qquad where\, Q' = \frac{\sigma_{cbc}}{2}.k'.j' = Constant$$

Basic concept of design of single reinforced members

The following types of problems can be encountered in the design of reinforced concrete members.

(A) Determination of Area of Tensile Reinforcement

The section, bending moment to be resisted and the maximum stresses in steel and concrete are given.

Steps to be followed:

(i) Determine *k,j.Q* (or Q') for the given stress.

(ii) Find the critical moment of resistance, $M=Q.b.d^2$ from the dimensions of the beam.

(iii) Compare the bending moment to be resisted with M, the critical moment of resistance.

(a) If B.M. is less than M, design the section as under reinforced.

$$M = \sigma_{st}.A_s.\left(d-\frac{x}{3}\right)$$

To find A_s in terms of x, take moments of areas about *N.A.*

$$b.x.\frac{x}{2} = m.A_s.(d-x)$$

$$A_s = \frac{b.x^2}{2(m)(d-x)} \therefore \qquad M = \frac{\sigma_{st}.b.x^2}{2.m.(d-x)}\left(d-\frac{x}{3}\right) = B.M. \text{ to be resisted}$$

Solve for 'x', and then A_s can be calculated.

(b) If *B.M.* is more than *M*, design the section as over-reinforced.

$M = \frac{\sigma_{cbc}}{2}.b.x.\left(d - \frac{x}{3}\right) = B.M.$ to be resisted. Determine 'x'. Then A_s can be obtained by taking moments of areas (compressive and tensile) about using the following expression.

$$A_s = \frac{b.x^2}{2.m.(d-x)}$$

(B) Design of Section for a Given loading

Design the section as balanced section for the given loading.

Steps to be followed:

(i) Find the maximum bending moment (*B.M.*) due to given loading.

(ii) Compute the constants *k,j,Q* for the balanced section for known stresses.

(iii) Fix the depth to breadth ratio of the beam section as 2 to 4.

(iv) From $M=Q.b.d^2$, find 'd' and then 'b' from depth to breadth ratio.

(v) Obtain overall depth 'D' by adding concrete cover to 'd' the effective depth.

(vi) Calculate A_s from the relation

$$A_s = \frac{B.M.}{\sigma_{st}.j.d}$$

(C) To Determine the Load carrying Capacity of a given Beam

The dimensions of the beam section, the material stresses and area of reinforcing steel are given.

Steps to be followed:

(i) Find the position of the neutral axis from section and reinforcement given.

(ii) Find the position of the critical *N.A.* from known permissible stresses of concrete and steel.

$$x = \frac{1}{1 + \frac{\sigma_{st}}{m.\sigma_{cbc}}}.d$$

(iii) Check if (i) > (ii)- the section is over-reinforced

(i) <(ii)- the section is under-reinforced

(iv) Calculate *M* from relation

$M = b.x.\frac{1}{2}.\sigma_{cbc}.\left(d - \frac{x}{3}\right)$ for over-reinforced section

and $M = \sigma_{st}.A_s.\left(d - \frac{x}{3}\right)$ for under-reinforced section.

(v) If the effective span and the support conditions of the beam are known, the load carrying capacity can be computed.

(D) To Check The Stresses Developed In Concrete And Steel

The section, reinforcement and bending moment are given.

Steps to be followed:

(i) Find the position of *N.A.*using the following relation.

$$b.\frac{x^2}{2} = m.A_s.(d - x))$$

(ii) Determine lever arm, $z = d - \frac{x}{3}$

(iii) $B.M. = \sigma_{st}.A_s.z$ is used to find out the actual stress in steel σ_{sa}.

(iv) To compute the actual stress in concrete σ_{cba}, use the following relation.

$$BM = \frac{\sigma_{cba}}{2}.b.x.z$$

Doubly Reinforced Beam Sections by Working Stress Method

Very frequently it becomes essential for a section to carry bending moment more than it can resist as a balanced section. Such a situation is encountered when the dimensions of the cross section are limited because of structural, head room or architectural reasons. Although a balanced section is the most economical section but because of limitations of size, section has to be sometimes over-reinforced by providing extra reinforcement on tension face than that required for a balanced section and also some reinforcement on compression face. Such sections reinforced both in tension and compression are also known as “Doubly Reinforced Sections”. In some loading cases reversal of stresses in the section take place (this happens when wind blows in opposite directions at different timings), the reinforcement is required on both faces.

MOMENT OF RESISTANCE OF DOUBLY REINFORCED SECTIONS

Consider a rectangular section reinforced on tension as well as compression faces as shown in Fig.2.4 (a-c)

Let b = width of section,

d = effective depth of section,

D = overall depth of section,

d'= cover to centre of compressive steel,

M = Bending moment or total moment of resistance,

M_{bal} = Moment of resistance of a balanced section with tension reinforcement,

A_{st} = Total area of tensile steel,

A_{st1} = Area of tensile steel required to develop M_{bal}

A_{st2} = Area of tensile steel required to develop M_2

A_{sc} = Area of compression steel,

σ_{st} = Stress in steel, and

σ_{sc} = Stress in compressive steel

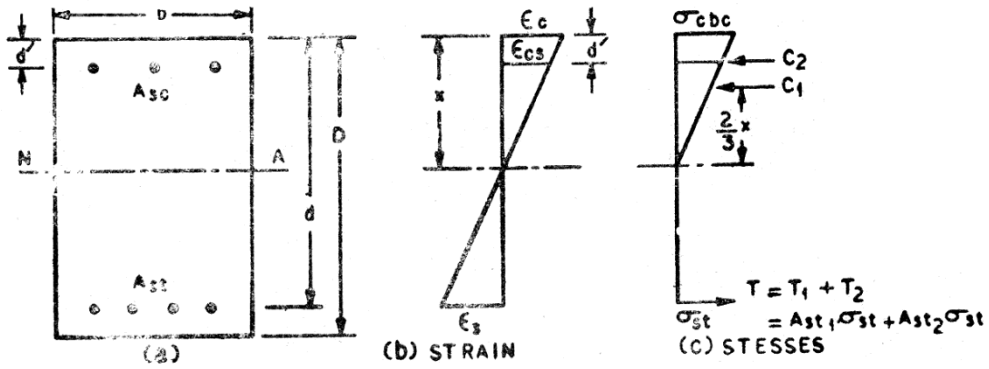


Fig.2.4 (a-c)

Since strains are proportional to the distance from *N.A.*,

$$\frac{Strain\ in\ top\ \ fibre\ \ of\ \ concrete}{Strain\ in\ Compression\ \ Steel} = \frac{x}{x-d'}$$

$$\frac{\sigma_{cbc}/E_C}{\sigma_{sc}/E_s} = \frac{x}{x-d'}$$

$$\frac{\sigma_{cbc}}{\sigma_{sc}} \cdot \frac{E_s}{E_c} = \frac{x}{x-d'}$$

$$\sigma_{sc} = \sigma_{cbc} \cdot \frac{x}{x-d'} .m$$

Since $\sigma_{cbc}.\frac{x-d'}{x}$ is the stress in concrete at the level of compression steel, it can be denoted as σ'_{cbc}

$\therefore \sigma_{sc} = m.\sigma'_{cbc}$

As per the provisions of IS:456-2000 Code , the permissible compressive stress in bars , in a beam or slab when compressive resistance of the concrete is taken into account, can be taken as 1.5m times the compressive stress in surrounding concrete (1.5m σ'_{cbc}) or permissible stress in steel in compression (σ_{sc}) whichever is less.

$\sigma_{sc} = 1.5m\ \sigma'_{cbc}$

Total equivalent concrete area resisting compression

$(x\ .\ b\text{-}A_{sc}) + 1.5\text{m}\ A_{sc} = x\ .b + (1.5\text{m-}1)A_{sc}$

Taking moment about centre of tensile steel

Moment of resistance $M = C_1.(d\text{-}x/3) + C_2(d\text{-}d')$

Where C_1 = total compressive force in concrete,

C_2= total compressive force in compression steel,

$$M = b.x.\frac{\sigma_{cbc}}{2}.(d-\frac{x}{3}) + (1.5m-1)A_{sc}.\sigma_{cbc}.\frac{x-d'}{x}.(d-d') = Q.b.d^2 + (1.5m-1)A_{sc}.\sigma_{cbc}.\frac{x-d'}{x}(d-d')$$

$$= M_1 + M_2$$

$Where\ \ M_1 = Moment\ of\ resistance\ of\ the\ balanced\ section = M_{bal}$

$M_2 = Moment\ of\ resistance\ of\ the\ compression\ steel$

$$Area\ of\ tension\ steel = A_{st1} = \frac{M_1}{\sigma_{st}.j.d}.$$

$$Area\ of\ tension\ steel\ \ equivalent\ to\ compression\ steel = A_{st2} = \frac{M_2}{\sigma_{st}(d-d')}$$

Thus the total tensile steel A_{st} shall be:

$\therefore \quad A_{st} = A_{st1} + A_{st2}$

The area of compression steel can be obtained as

$(1.5m-1)A_{sc}(x-d') = mA_{st2}.(d-x)$

Design Concept of T-Beam

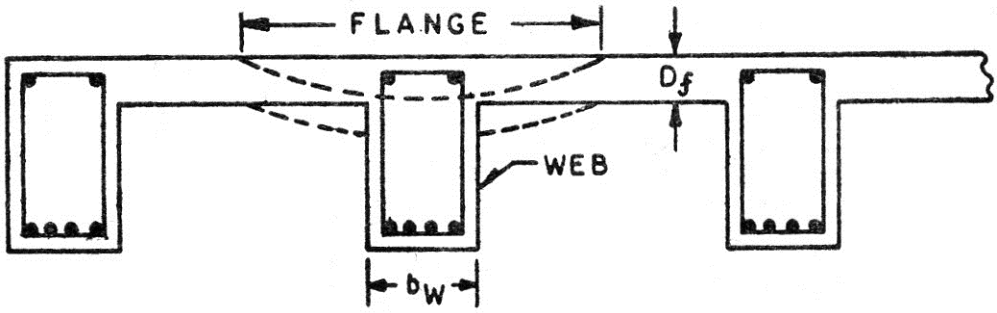


Fig.2.5

Flanged beam sections comprise T-beams and L-beams where the slabs and beams are cast monolithically having no distinction between beams and slabs. Consequently the beams and slabs are so closely tied that when the beam deflects under applied loads it drags along with it a portion of the slab also as shown in Fig.2.5 .this portion of the slab assists in resisting the effects of the loads and is called the 'flange' of the T-beams. For design of such beams, the profile is similar to a T-section for intermediate beams. The portion of the beam below the slab is called 'web' or 'Rib'. A slab which is assumed to act as flange of a T-beam shall satisfy the following conditions:

(a) The slab shall be cast integrally with the web or the the web and the slab shall be effectively bonded together in any other manner; and

(b) If the main reinforcement of the slab is parallel to the beam, transverse reinforcement shall be provided as shown in Fig.2.6, such reinforcement shall not be less than 60% of the main reinforcement at mid-span of the slab.

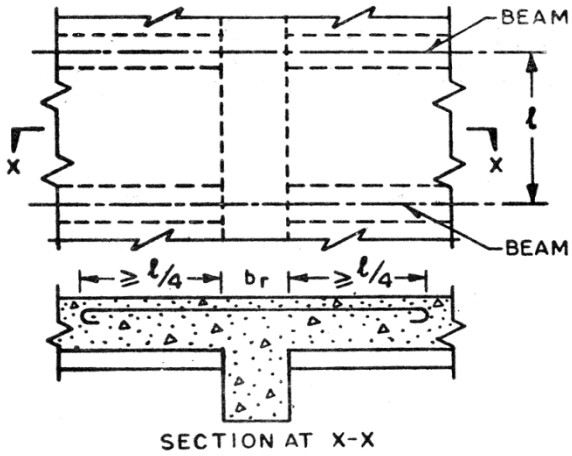


Fig.2.6

CHAPTER- 3

LIMIT STATE METHOD

SAFETY AND SERVICEABILITY REQUIREMENTS

In the method of design based on limit state concept, the structure shall be designed to withstand safely all loads liable to act on it throughout its life; it shall also satisfy the serviceability requirements, such as limitations on deflection and cracking. The acceptable limit for the safety and serviceability requirements before failure occurs is called a 'limit state'. The aim of design is to achieve acceptable probabilities that the structure will not become unfit for the use for which it is intended that it will not reach a limit state.

All relevant limit states shall be considered in design to ensure an adequate degree of safety and serviceability. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

For ensuring the above objective, the design should be based on characteristic values for material strengths and applied loads, which take into account the variations in the material strengths and in the loads to be supported. The characteristic values should be based on statistical data if available; where such data are not available they should be based on experience. The 'design values' are derived from the characteristic values through the use of partial safety factors, one for material strengths and the other for loads. In the absence of special considerations these factors should have the values given in 36 according to the material, the type of loading and the limit state being considered.

Limit State of Collapse

The limit state of collapse of the structure or part of the structure could be assessed from rupture of one or more critical sections and from buckling due to elastic or plastic instability (including the effects of sway where appropriate) or overturning. The resistance to bending, shear, torsion and axial loads at every section shall not be less than the appropriate value at that section produced by the probable most un favourable combination of loads on the structure using the appropriate partial safety factors.

Limit State Design

For ensuring the design objectives, the design should be based on characteristic values for material strengths and applied loads (actions), which take into account the probability of variations in the material strengths and in the loads to be supported. The characteristic values should be based on statistical data, if available. Where such data is not available, they should be based on experience. The design values are derived from the characteristic values through the use of partial safety factors, both for material strengths and for loads. In the absence of special considerations, these factors should have the values given in this section according to the material, the type of load and the limit state being considered. The reliability of design is ensured by requiring that

$$\text{Design Action} \leq \text{Design Strength}.$$

Limit states are the states beyond which the structure no longer satisfies the performance requirements specified. The limit states are classified as

a) Limit state of strength
b) Limit state of serviceability

a) The limit state of strength are those associated with failures (or imminent failure), under the action of probable and most unfavorable combination of loads on the structure using the appropriate partial safety factors, which may endanger the safety of life and property. The limit state of strength includes:

a) Loss of equilibrium of the structure as a whole or any of its parts or components.
b) Loss of stability of the structure (including the effect of sway where appropriate and overturning) or any of its parts including supports and foundations.
c) Failure by excessive deformation, rupture of the structure or any of its parts or components.
d) Fracture due to fatigue.
e) Brittle fracture.

b) The limit state of serviceability include

a) Deformation and deflections, which may adversely affect the appearance or, effective, use of the structure or may cause improper functioning of equipment or services or may cause damages to finishes and non-structural members.
b) Vibrations in the structure or any of its components causing discomfort to people, damages to the structure, its contents or which may limit its functional effectiveness. Special consideration shall be given to floor vibration systems susceptible to vibration, such as large open floor areas free of partitions to ensure that such vibrations is acceptable for the intended use and occupancy.
c) Repairable damage due to fatigue.
d) Corrosion and durability.

Limit States of Serviceability

To satisfy the limit state of serviceability the deflection and cracking in the structure shall not be excessive. This limit state corresponds to deflection and cracking.

Deflection

The deflection of a structure or part shall not adversely affect the appearance or efficiency of the structure or finishes or partitions.

Cracking

Cracking of concrete should not adversely affect the appearance or durability of the structure; the acceptable limits of cracking would vary with the type of structure and environment. The actual width of cracks will vary between the wide limits and predictions of absolute maximum width are not possible. The surface width of cracks should not exceed 0.3mm.

In members where cracking in the tensile zone is harmful either because they are exposed to the effects of the weather or continuously exposed to moisture or in contact soil or ground water, an upper limit of 0.2 mm is suggested for the maximum width of cracks. For particularly aggressive environment, such as the 'severe' category, the assessed surface width of cracks should not in general, exceed 0.1 mm.

CHARACTERISTIC AND DESIGN VALUES AND PARTIAL SAFETY FACTORS

1. Characteristic Strength of Materials

Characteristic strength means that value of the strength of the material below which not more than 5 percent of the test results are expected to fall and is denoted by f. The characteristic strength of concrete (f_{ck}) is as per the mix of concrete. The characteristic strength of steel (f_y) is the minimum stress or 0.2 percent of proof stress.

2. Characteristic Loads

Characteristic load means that value of load which has a 95 percent probability of not being exceeded during the life of the structure. Since data are not available to express loads in statistical terms, for the purpose of this standard, dead loads given in IS 875 (Part l), imposed loads given in IS 875 (Part 2), wind loads given in IS 875 (Part 3), snow load as given in IS 875 (Part **4)** and seismic forces given in IS 1893-2002(part-I) shall be assumed as the characteristic loads.

Design Values

Materials

The design strength of the materials f_d is given by

$$f_d = \frac{f}{\gamma_m}$$

where

f = characteristic strength of the material

γ_m =partial safety factor appropriate to the material and the limit state being considered.

Load

The design load, F, is given by

$$f_d = \frac{F}{\gamma_f}$$

Where, F=characteristic load

and γ_f = partial safety factor appropriate to the nature of loading and the limit state being considered.

Consequences of Attaining Limit State

Where the consequences of a structure attaining a limit state are of a serious nature such as huge loss of life and disruption of the economy, higher values for γ_f and γ_m than those given under 36.4.1 and 36.4.2 may be applied.

Partial Safety Factors:

1. Partial Safety Factor γ_f for Loads

Sr. No.	Load Combination	Ultimate Limit State	Serviceability Limit State
1	DL + LL	1.5 (DL + LL)	DL + LL
2	DL + WL i) DL contribute to stability ii) DL assists overturning	0.9 DL + 1.5 WL 1.5 (DL + WL)	DL + WL DL + WL
3	DL + LL + WL	1.2 (DL + LL + WL)	DL + 0.8 LL + 0.8 WL

2. Partial Safety Factor γ_m for Material Strength

Sr. No.	Material	Ultimate Limit State	Serviceability Limit State
1	Concrete	1.50	$E_c = 5000\sqrt{f_{ck}}$ MPa
2	Steel	1.15	$E_s = 2 \times 10^5$ MPa

When assessing the strength of a structure or structural member for the limit state of collapse, the values of partial safety factor, should be taken as 1.5 for concrete and 1.15 for steel.

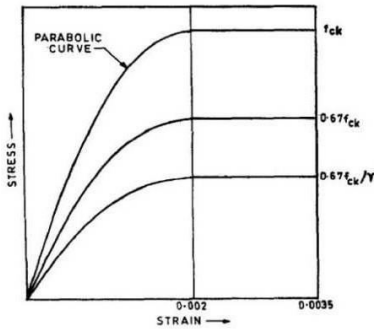
LIMIT STATE OF COLLAPSE: FLEXURE

Assumptions for Limit State of Collapse (Flexure):

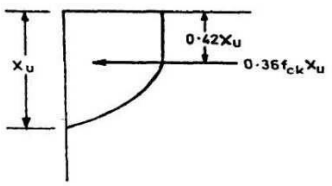
1) Plane section normal to the axis remains plane even after bending. i.e. strain at any point on the cross section is directly proportional to the distance from the N.A.

2) Maximum strain in concrete at the outer most compression fibre is taken as 0.0035 in bending.

3) The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoid, parabola or any other shape which results in prediction of strength in substantial agreement with the results of test. An acceptable stress strain curve is as shown below.



STRESS-STRAIN CURVE FOR CONCRETE



STRESS BLOCK PARAMETERS

For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor $\gamma_m = 1.5$ shall be applied in addition to this.

NOTE - For the above stress-strain curve the design stress block parameters are as follows:

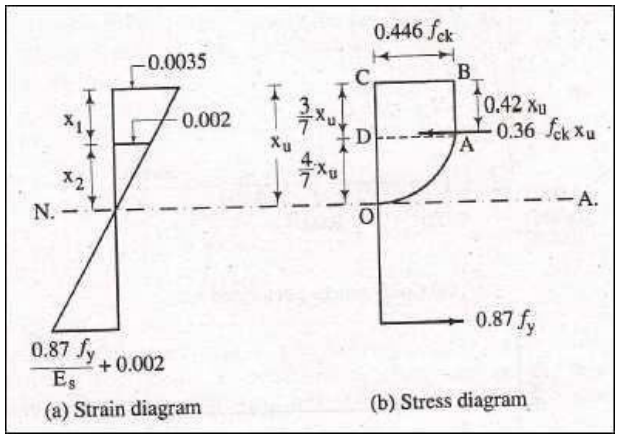
Area of stress block = $0.36.f_{ck}.x_u$

Depth of centre of compressive force = $0.42x_u$ from the extreme fibre in compression

Where

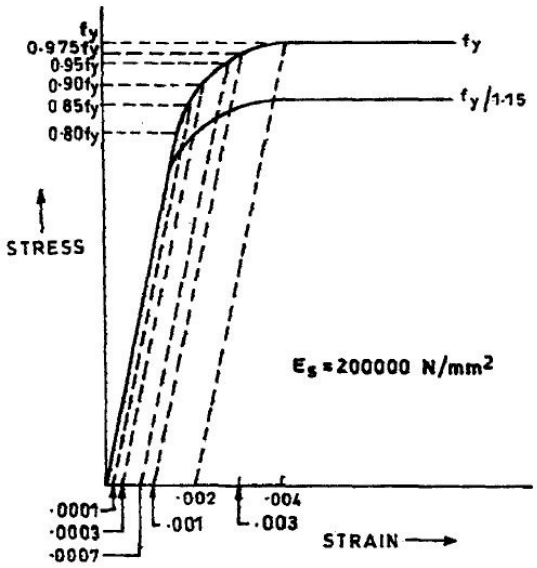
f_{ck} = characteristic compressive strength of concrete, and

x_u = depth of neutral axis.

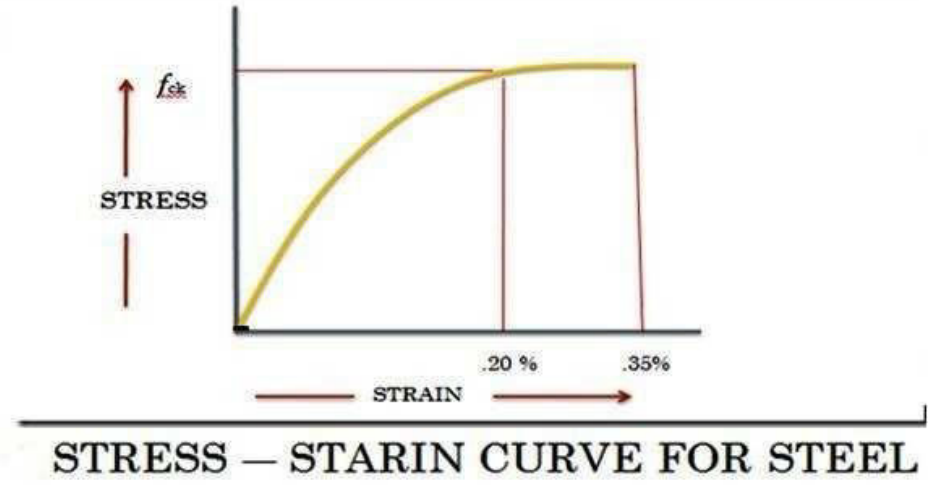


4) the tensile strength of the concrete is ignored.

5) the stresses in the reinforcement are derived from representative stress – strain curve for the type of steel used.



Cold Worked Deformed Bar



STRESS — STARIN CURVE FOR STEEL

6) the maximum strain in tension reinforcement in the section at failure shall not be less than

$$\frac{f_y}{1.15E_s} + 0.002 = \frac{0.87f_y}{E_s} + 0.002$$

CHAPTER 4

LIMIT STATES OF COLLAPSE OF SINGLE REINFORCED MEMBERS IN BENDING

Limit state method of design

- The object of the design based on the limit state concept is to achieve an acceptable probability, that a structure will not become unsuitable in it's lifetime for the use for which it is intended, i.e. It will not reach a limit state
- A structure with appropriate degree of reliability should be able to withstand safely.
- All loads, that are reliable to act on it throughout it's life and it should also satisfy the subs ability requirements, such as limitation son deflection and cracking.
- It should also be able to maintain the required structural integrity, during and after accident, such as fires, explosion & local failure.i.e. limit sate must be consider in design to ensure an adequate degree of safety and serviceability
- The most important of these limit states, which must be examine in design are as follows Limit state of collapse

- Flexure
- Compression
- Shear
- Torsion

This state corresponds to the maximum load carrying capacity.

Types of reinforced concrete beams

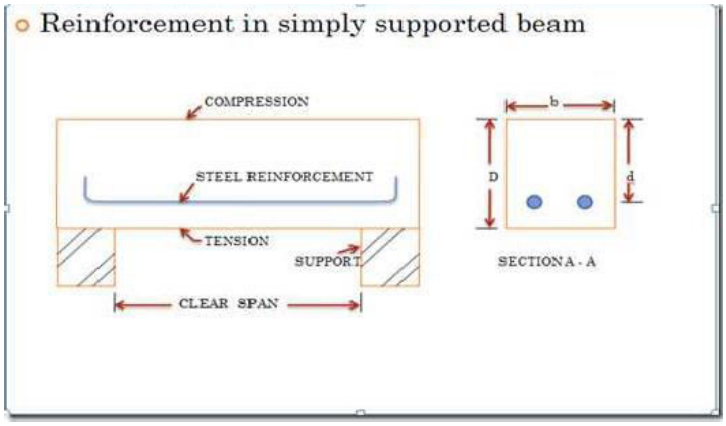
a) Singly reinforced beam

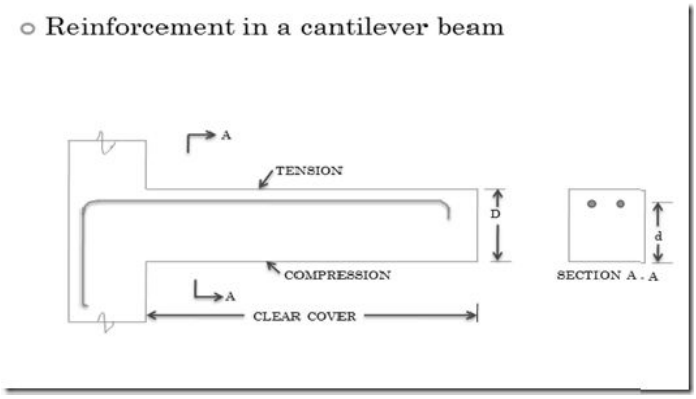
b) Doubly reinforced beam

c) Singly or Doubly reinforced flanged beams

Singly reinforced beam

In singly reinforced simply supported beams or slabs reinforcing steel bars are placed near the bottom of the beam or slabs where they are most effective in resisting the tensile stresses.





TYPES OF BEAM SECTIONS

Section in which, tension steel also reaches yield strain simultaneously as the concrete reaches the failure strain in bending are called, **'Balanced Section'.**

Section in which, tension steel also reaches yield strain at loads lower than the load at which concrete reaches the failure strain in bending are called, **'Under Reinforced Section'.**

Section in which, tension steel also reaches yield strain at loads higher than the load at which concrete reaches the failure strain in bending are called, **'Over Reinforced Section'.**

Sr. No.	**Types of Problems**	**Data Given**	**Data Determine**
1.	Identify the type of section, balance, under reinforced or over reinforced	Grade of Concrete & Steel, Size of beam & Reinforcement provided	If $\frac{X_u}{d} = \frac{X_{u\max}}{d} \Rightarrow$ Balanced If $\frac{X_u}{d} < \frac{X_{u\max}}{d} \Rightarrow$ Under Reinforced If $\frac{X_u}{d} > \frac{X_{u\max}}{d} \Rightarrow$ Over Reinforced $\frac{X_u}{d} = \frac{0.87 f_y . A_{st}}{0.36 b . d f_{ck}}$ f_y — $\frac{X_{u\max}}{d}$ 250 — 0.53 415 — 0.48 500 — 0.46

2	Calculate Moment of Resistance	Grade of Concrete & Steel, Size of beam & Reinforcement Provided	1) If $\frac{x_u}{d} = \frac{x_{u,\max}}{d}, balanced$ $M.R = M_u = 0.36.\frac{x_{u,\max}}{d}(1-0.42\frac{x_{u,\max}}{d})b.d^2.f_{ck}$
			2) If $\frac{X_u}{d} < \frac{X_{u\max}}{d}$ Under Reinforced M.R=M_u = $0.87 f_y.A_{st}.d(1-\frac{A_{st}.f_y}{b.d.f_{ck}}) or M.R = 0.87 f_y.A_{st}.d(1-0.42\frac{x_u}{d})$
			3) If $\frac{X_u}{d} > \frac{X_{u\max}}{d} \Rightarrow$ over reinforced, Revise the depth
3	Design the beam. Find out the depth of Beam D & Reinforcement required $A_{st.}$	Grade of Concrete & Steel, width of beam & Bending Moment or loading on the beam with the span of the beam Reinforcement Provided	We have to design the beam as a 'Balanced Design'. For finding 'd' effective depth use the equation; $M.R = M_u = 0.36.\frac{x_{u,\max}}{d}(1-0.42\frac{x_{u,\max}}{d})b.d^2.f_{ck}$ For finding A_{st} use the equation $0.87 f_y.A_{st}.d(1-\frac{A_{st}.f_y}{b.d.f_{ck}}) or M.R = 0.87 f_y.A_{st}.d(1-0.42\frac{x_u}{d})$

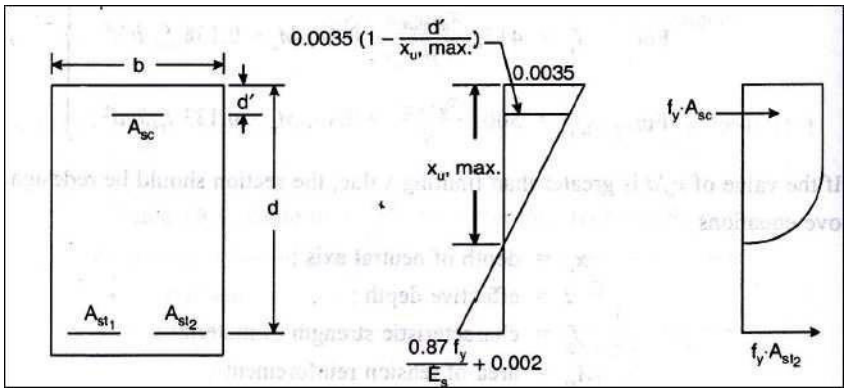
Where

d = effective depth of beam in mm.
b = width of beam in mm
x_u = depth of actual neutral axis im mm from extreme compression fibre.
$x_{u,\max}$ = depth of critical neutral axis in mm from extreme compression fibre.
A_{st} = area of tensile reinforcement
f_{ck} = characteristic strength of concrete in MPa.
f_y = characteristic strength of steel in MPa.
$M_{u,lim}$ = Limiting Moment of Resistance of a section without compression reinforcement

Doubly Reinforced Section or sections with Compression Reinforcement

Doubly Reinforced Section sections are adopted when the dimensions of the beam have been predetermined from other considerations and the design moment exceeds the moment of resistance of a singly reinforced section. The additional moment of resistance is carried by providing compression reinforcement and additional reinforcement in tension zone. The moment of resistance of a doubly reinforced section is the sum of the limiting moment of resistance $M_{u,lim}$ of a single reinforced section and the additional moment of resistance M_{u2}.

$M_{u2} = M_u - M_{u,lim}$



The lever arm for the additional moment of resistance is equal to the distance between the centroids of tension and compression reinforcement, (d – d').

$M_{u2} = 0.87\ f_y.A_{st2}(d - d') = A_{sc}.(f_{sc} - f_{cc})(d - d')$

Where : A_{st2} = Area of additional tensile reinforcement

A_{sc} = Area of compression reinforcement

f_{sc} = Stress in compression reinforcement

f_{cc} = Compressive stress in concrete at the level of compression reinforcement

Since the additiona reinforcement is balanced by the additional compressive force.

$$A_{sc}.(f_{sc} - f_{cc}) = 0.87\ f_y.A_{st2}$$

The strain at level of compression reinforcement is $0.0035\left(1 - \frac{d'}{x_{u,max.}}\right)$

Total area of reinforcement shall be obtained by

$$A_{st} = A_{st1} + A_{st2}$$

A_{st1} = Area of reinforcement for a singly reinforced section for $M_{u,lim}$

$$A_{st2} = \frac{A_{sc}\ (f_{sc} - f_{cc})}{0.87 f_y}$$

EXAMPLE 4.1

Calculate the area of steel of grade Fe 415 required for section of 250mm wide and overall depth 500mm with effective cover 40mm in M20, if the limit state of moment be carried by the section is

a) 100 KN b) 146 KN c) 200KN

SOLUTION:

For f_y =415N/mm^2, $\frac{X_{u\,max}}{d} = 0.48$

$$M_{u,\lim} = 0.36.\frac{x_{u,\max}}{d}(1-0.42\frac{x_{u,\max}}{d})b.d^2.f_{ck}$$

$$= 0.36 \text{ X } .48(1\text{-}0.42 \text{ X } 0.48) \text{ X } 250 \text{ X } 460^2 \text{ X } 20$$

$$= 146 \text{ X } 10^6 \text{N.mm}$$

a) For M_u = 100 KN.m < 146 KN.m

Area of steel required is obtained from , $M_u = 0.87 f_y.A_{st}.d(1-\frac{A_{st}.f_y}{b.d.f_{ck}})$

$100 \text{ X } 10^6 = 0.87 \text{ X } 415 \text{ X } A_{st} \text{ X} 460 (1- \frac{A_{st} X 415}{250 X 460 X 20})$

A_{st} =686 or 4850 mm^2, taking minimum steel 686mm^2

b) M_u = 146 KN.m =$M_{u,lim}$= 146 KN.m

$x_u = x_{u,max}$

Area of tension reinforcement required

$$\frac{X_{u,\max}}{d} = \frac{0.87 f_y.A_{st}}{0.36b.df_{ck}}$$

$$A_{st} = \frac{0.48X\,0.36X\,20X\,250X\,460}{0.87X\,415} = 1100mm^2$$

c) M_u = 200 KN.m >$M_{u,lim}$= 146 KN.m

Reinforcement is to be provided in the compression zone also along with the reinforcement in tension zone.

M_u=$M_{u,lim}$=f_{sc}.A_{sc}(d – d')

f_{sc} is stress corresponding to strain of $\frac{0.0035(x_{u,\max} - d')}{x_{u,\lim}} = \frac{0.0035(0.48X\,460 - 40)}{0.48X\,460} = 0.002866$

f_{sc}=360.8N/$_{mm}{}^2$

(200-146) X 10^6 = 360.8. A_{sc}(460-40)

A_{sc}= 356mm^2

A_{st1}= Area of tension reinforcement corresponding to $M_{u,lim}$

146 X 10^6 = 0.87 X 460 X 415A_{st1}(1 - $\frac{A_{st}X\,415}{250X\,460X\,20}$)

A_{st1} = 1094mm^2

$A_{st2} = A_{sc}.\ f_{sc} / 0.87$ X415 =356mm^2

$A_{st} = A_{st1} + A_{st2}$ = 1094 + 356 = 1450mm^2

EXAMPLE: 4.2

Design a rectangular beam which carries a maximum limiting bending moment of 65 KN.m. Use M20 and Fe 415 as reinforcement.

At balanced failure condition

$M_u = M_{u,lim}$

$$M_{u,\lim} = 0.36.\frac{x_{u,\max}}{d}(1 - 0.42\frac{x_{u,\max}}{d})b.d^2.f_{ck}$$

$M_{u,lim}$ = 0.36 X 0.48 X 20(1-0.42 X 0.48) bd^2

= 2.759b d^2

Assuming width of beam as 250 mm

$$d = \sqrt{\frac{65X10^6}{2.759X\,250}} = 307\text{mm}$$

Area of reinforcement

$$\frac{X_{u,\max}}{d} = \frac{0.87 f_y.A_{st}}{0.36b.df_{ck}}$$

$$0.48 = \frac{0.87\,X\,415XA_{st}}{0.36X\,20X\,250X\,307}$$

A_{st} =734.66 mm^2

EXAMPLE: 4.3

Find out the factored moment of resistance of a beam section 300mm wide X 450mm effective depth reinforced with 2 X 20mm diameter bars as compression reinforcement at an effective cover of 50mm and 4 X 25mm diameter bars as tension reinforcement. The materials are M20 grade concrete and Fe 415 HYSD bars.

Solution:

Given;

Width= b =300mm

Effective depth = d = 450mm

Cover to compression reinforcement = d' = 50mm

$\frac{d}{d'} = \frac{50}{450} = 0.11$, next higher value 0.15 may be adopted.

A_{sc} =area compression reinforcement = 2 $\pi 16^2 = 628mm^2$

A_{st} = area of reinforcement in tension = 4 x $\pi 25^2 = 1964mm^2$

f_{sc} = stress in compression steel=342 N/mm^2

Equating total force

$0.36 f_{ck}.b.x_u + f_{sc}.A_{sc} = 0.87 f_y.A_{st}$

0.36 X 20 X 300 x_u + 628 X 342 = 0.87 X 415 X 1964

x_u = 228.85mm

But $x_{u,max}$ = 0.48d for Fe415

$x_{u,max}$ = 0.48 X 450 = 216mm

So $x_u > x_{u,max}, \Rightarrow$ over reinforced

The moment of resistance can be found out by takin moments of compressive forces about centroid of tensile reinforcement.

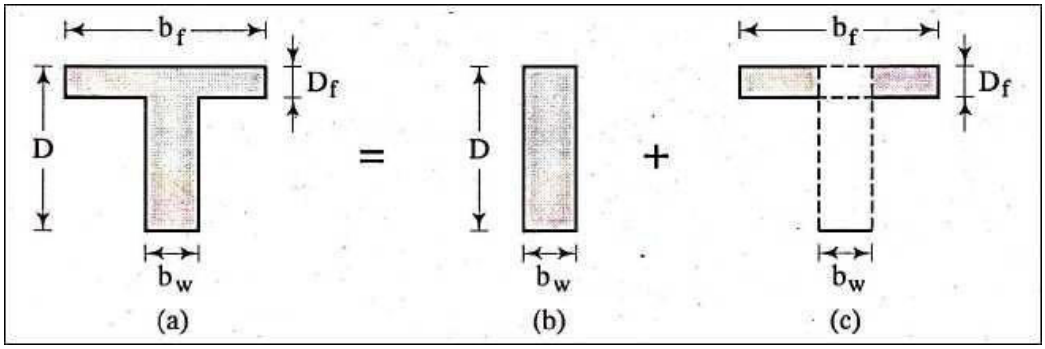
$M_u = 2160x_u(450-0.42x_u) + 214776 (450-50) X 10^{-6}$

Putting x_u = 216mm

M_u = 253.54 KN.m

BEHAVIORS OF 'T' AND 'L' BEAMS (FLANGED BEAM)

A 'T' beam or 'L' beam can be considered as a rectangular beam with dimensions b_w. D plus a flange of size $(b_f - b_w)$ X D_f. It is shown in the figure beam (a) is equivalent to beam (b) + beam (c).



The flanged beam analysis and design are analogous to doubly reinforced rectangular beam. In doubly reinforced beams additional compressive is provided by adding reinforcement in compression zone, whereas in flanged beams, this is provided by the slab concrete, where the spanning of the slab is perpendicular to that of beam and slab is in compression zone.

If the spanning of the slab is parallel to that of the beam, some portion of slab can be made to span in the direction perpendicular to that of the beam by adding some reinforcement in the slab.

A flanged beam can be also doubly reinforced.

The moment of resistance of a T beam is sum of the moment of resistance of beam (a) is the sum moment of resistance of beam (b) and moment of resistance of beam (c)

CHAPTER-5

LIMIT STATE OF COLLAPSE IN SHEAR (Design of Shear by LSM)

5.1. SHEAR STRESS IN REINFORCED CONCRETE BEAMS:-

When a beam is loaded with transverse loads the Bending Moment(BM) varies from section to section. Shearing stresses in beams are caused by this variation of BM in the beam span. Due to the variation of BM at two sections distance dx apart, there are unequal bending stresses at the same fibre. This inequality of bending stresses produces a tendency in each horizontal fibre to slide over adjacent horizontal fibre causing horizontal shear stress, which is accompanied by complimentary shear stress in vertical direction.

SHEAR CRACKS IN BEAMS:-

Under the transverse loading , at any section of the beam, there exists both Bending Moment(BM) and Shear Force (V).Depending upon the ratio of Bending Moment(BM) to Shear Force(V) at different sections, there may be three regions of shear cracks in the beam as follows.

(a) Region I : Region of flexure Cracks.
(b) Region II : Region of flexure shear Cracks.
(c) Region II : Region of web shear Cracks or diagonal tension cracks.

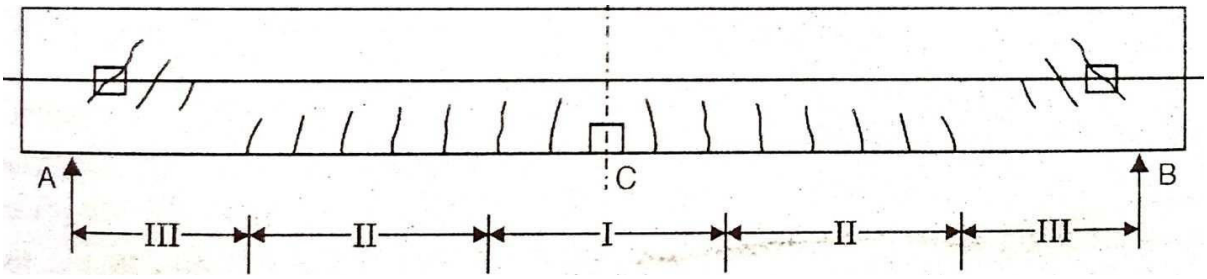


Fig-5.2.1 DIFFERENT REGION OF CRACKS IN BEAMS

(a) Region I : Region of flexure Cracks.

This region normally occurs adjacent to mid-span where BM is large and shear force is either zero or very small.The principal planes are perpendicular to beam axis. When the principal tensile stress reaches the tensile strength of the concrete (which is quite low) tensile cracks develop vertically. The cracks are known as flexural cracks resulting primarily due to flexture.

(b) Region II : Region of flexure shear Cracks. This regions are near the quarter span, to both the sides, where BM is considerable and at the same time Shear force is significant. The cracks in this region are initiated at the tension face, travel vertically (due to flexture) and gradually tend to develop in the inclined direction towards the Nutral Axis(N.A.), as the shear stress goes on increasing towards the N.A. Since the cracks develop under the combined action of BM and Shear, these cracks are known as flexure- shear cracks.

(c) Region II : Region of web shear Cracks or diagonal tension cracks.

This regions are adjacent to each support of the beam where S.F is predominant. Since Shear stress is maximum at the N.A., inclined cracks starts developing at the N.A. along the diagonal of an element subject to the action of pure shear.Hence these cracks known as diagonal tension cracks or web-shear cracks.

MECHANISM OF SHEAR TRANSFER IN REINFORCE CONCRETE BEAM WITHOUT SHEAR.

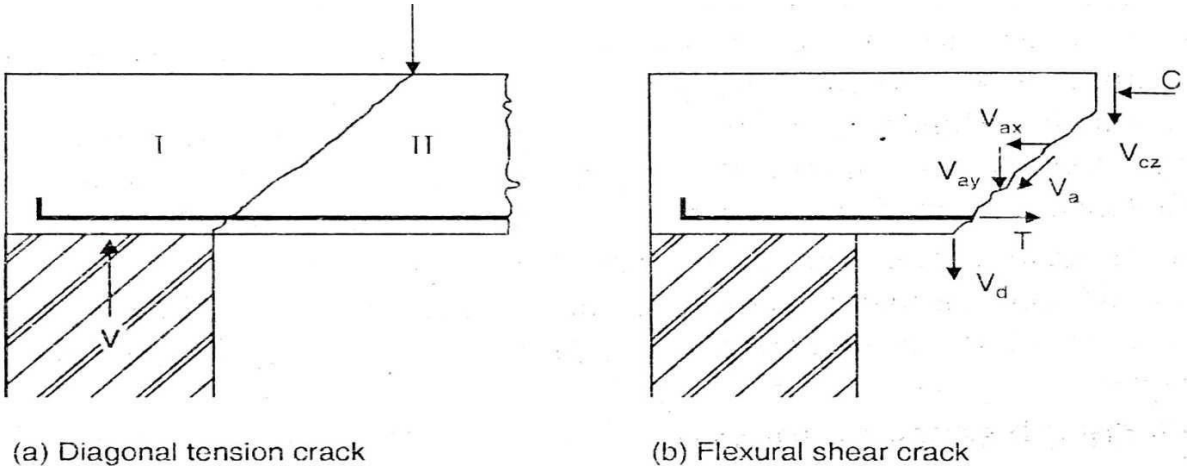


Fig- 5.3.1

Shear is transferred between two adjacent planes in a R C beam by the following mechanism.

(a) Shear resistance V_{cz} of the uncracked portion of concrete.

(b) Vertical Component V_{ay} of the interface shear or aggregate interlock force V_a. and

(c) Dowel force V_d in the tension reinforcement, due to dowel action.

Thus $V = V_{cz} + V_{ay} + V_d$.

The relative contribution of each of the above three mechanism depend upon the stage of loading and extent of cracking. In the initial stage before the flexural cracking starts, the entire shear is resisted by the shear resistance of the concrete (i.e $V = V_{cz}$).

As the flexural cracking starts, the interface shear comes into action resulting in the redistribution of stresses. Further extension of flexural cracks results in sharing the shear by the dowel force V_d of the tension reinforcement. Thus at the final stage of collapse , the shear is transferred by the shear is born by all the three mechanism expressed by the equation above.

MODES OF SHEAR FAILURE

The shear Failure of a R C beam, without shear reinforcement is governed by a_v / d, ratio. A beam may experience following types of shear failure.

1. Case I : $a_v / d < 1$: Splitting or compression failure.
2. Case II : $1 < a_v / d < 2.8$: Shear compression or shear tension failure.
3. Case III : $2.8 < a_v / d < 6$: Diagonal tension failure.
4. Case-IV : $a_v / d > 6$: Flexure failure

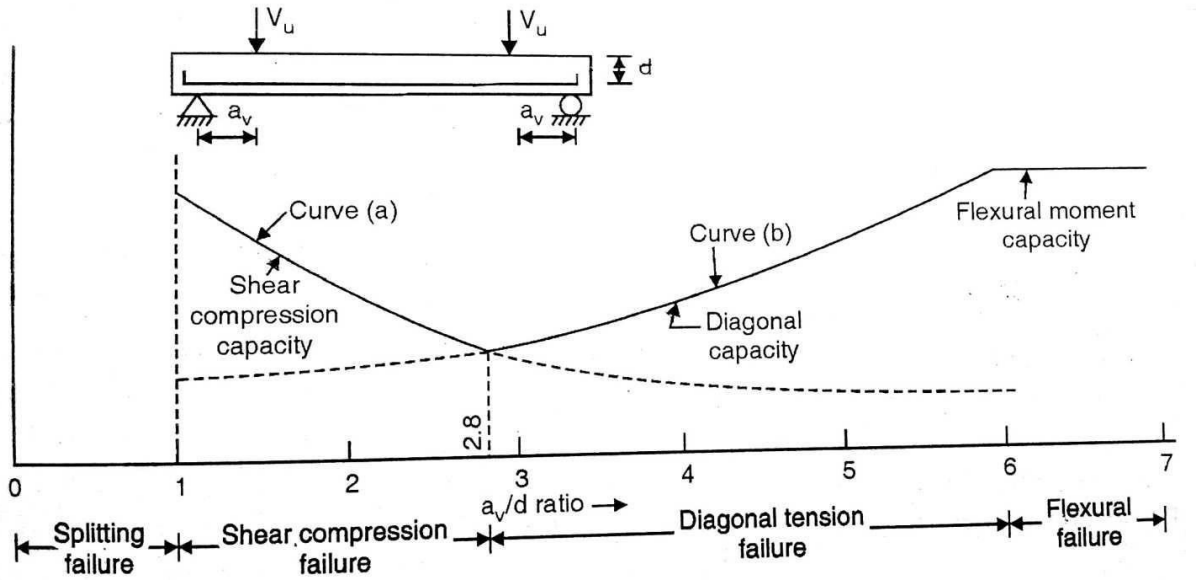


Fig – 5.4.1. EFFECT OF a_v / d ON SHEAR STRENGTH OF R C BEAM

CASE I : a_v / d < 1 (Deep Beams) : Splitting or compression failure:

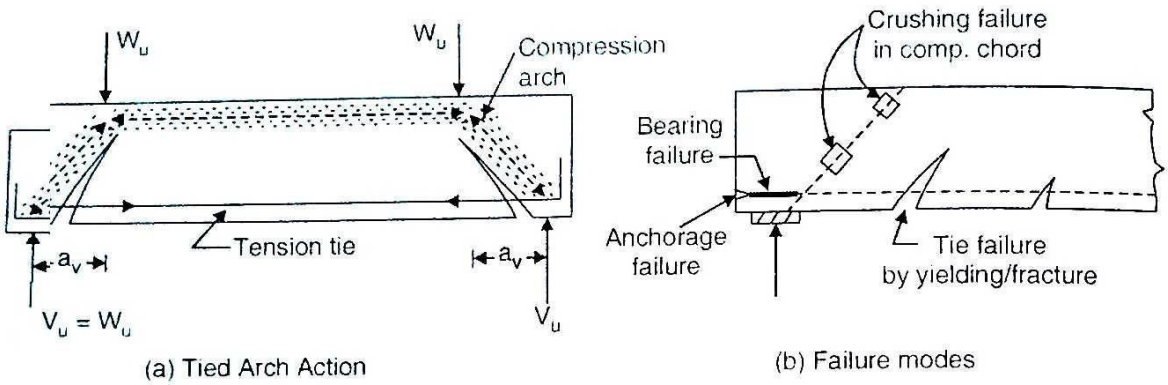


Fig – 5.4.2. CASE I : a_v / d < 1 (DEEP BEAMS)

This case correspond to a deep beam without shear reinforcements where the inclined cracking transforms the beam into a tied arch (Fig-a). The load is carried by (i) direct compression in the concrete between the load and reaction point by crossing of concrete and by (ii) tension in the longitudinal steel by yielding or fracture or anchorage failure or bearing failure .

CASE II : 1< a_v / d < 2.8 : Shear compression or shear tension failure.

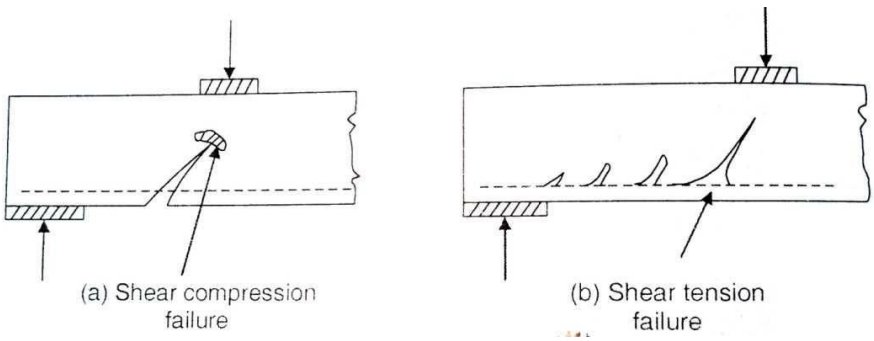


Fig- 5.4.3 CASE II : 1< a_v / d < 2.8

This case is common in short beams with a_v / d ratio between 1 to 2.8, where failure is initiated by an inclined crack – more commonly a flexural shear crack. Fig-a shows the shear compression failure due to vertical compressive stresses developed in the vicinity of the load. Similarly the vertical compressive stress over the reaction limits the bond splitting and diagonal cracking along the steel. The crack extends towards the tension reinforcement and then propagates along the reinforcements (Fig-b) resulting in the failure of the beam by anchorage failure.

CASE III : 2.8 < a_v / d < 6 : Diagonal tension failure.

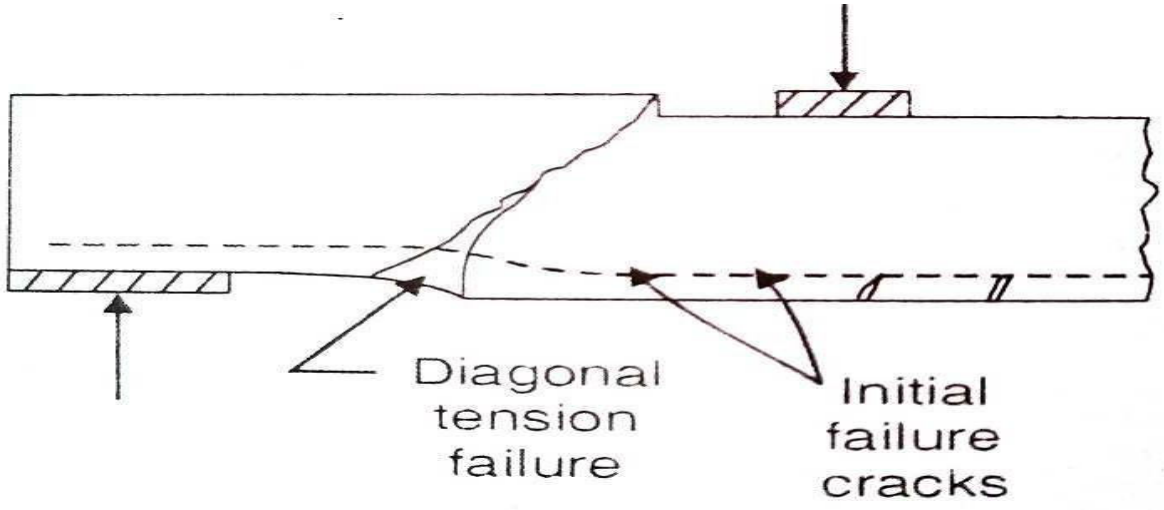


Fig- 5.4.4 CASE III : 2.8< a_v / d < 6

Diagonal tension failure occurs when the shear span to the effective depth ratio is in the range of 2.8 to 6 . Normal beams have a_v / d ratio in excess of 2.8. Such beams may fail either in shear or in flexure.

CASE - IV : a_v / d > 6 : Flexure failure

Flexural failure is encountered when a_v / d ratio > 6. Two cases may be encountered; (i) under reinforced beam and (ii) over reinforced beam. In the case of under reinforced beam, tension reinforcement is less than the limiting one, due to which failure is initiated by yielding of tension reinforcement, leading to the ultimate failure due to crushing of concrete in compression zone. Such a ductile failure is known as flexural tension failure, which is quite slow giving enough warning. In the over reinforced sections failure occurs due to crushing of concrete in compression zone before yielding of tension reinforcement. Such a failure, known as flexural compression failure is quite sudden.

FACTORS AFFECTING THE SHEAR RESISTANCE OF A R C MEMBER.

The shear resistance of rectangular beams, without shear renforcements depends on the following factors.

1.Grade of concrete : Higher grade of concrete has higher characteristic strength which in turn results in (i) higher tensile strength (ii0 greater dowel shear resistance (iii) greater aggregate interlock capacity, and (iv) greater concrete strength in compression zone. Hence shear resistance increases with the increase in the grade of concrete.

2. Percentage and grade of longitudinal tensile reinforcement : The increase in percentage (p_t) of longitudinal tensile reinforcement results in the increase in dowel shear (V_d). Due to this reason, the design Codes make the shear strength (τ_c) of concrete a function of p_t and grade of concrete (see Table 5.1). However, higher grade of steel results in lesser shear resistance of R.C. beam because the percentage of steel (p_t) corresponding to a higher grade steel is less than that required for a low grade steel, say mild steel.

3. Ratio of shear span to effective depth (i.e. a_v/d ratio) : As discussed in the previous article, for a_v/d ratio between 6 and 2.8, the shear capacity, being governed by inclined crack resistance, decrease with decrease in a_v/d ratio (curve *b* of Fig.5.4.1). However, for a value of a_v/d less than 2.8, the shear capacity, being dependent on *shear-compression* or *shear-bond capacity*, increases rapidly. The minimum shear capacity is at a_v/d ratio around 2.8.

4. Compressive force : Presence of axial compressive force result in increase of shear capacity. The effect of axial compression on the design shear strength has been taken into account by I.S. Code by increasing the design shear strength by a modification factor ð.

5. Compressive reinforcement : The shear resistance is found to increase with the increase in the percentage of compressive steel (p_c).

6. Axial tensile force : Axial tensile force reduces marginally the shear resistance of concrete as per the equation $ð = 1 - \frac{P_w}{3.45Æ_g}$.

7. Shear reinforcement: The shear resistance of a R C Beam increases with the increase in shear reinforcement ratio. This is due to two reasons (i) concrete gets conformed between stirrup spacing and (ii) the shear/web reinforcement itself provides shear resistance.

5.6 . DESIGN SHEAR STRENGTH OF CONCRETE WITHOUT SHEAR RENFORCEMENT (IS 456: 2000)

The magnitude of design shear strength (r_c) depends basically on the grade of concrete (f_{ck}) and the percentage of tension steel(P_t). As per IS 456 : 2000 the design shear strength of concrete in beams without shear reinforcement shall be given in table 5.1.

TABLE 5.1 DESIGN SHEAR STRENGTH (τ_c) OF CONCRETE, (N/mm^2)

$100\frac{A_{st}}{bd}$	Grade of concrete					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.0 and above	0.71	0.82	0.92	0.96	0.99	1.01

Analytical expression for design shear strength :

The Values of r_c given in the above table by the code are based on the following semi empirical expression (SP 24, 1983).

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\,(\sqrt{1 + 5\beta} - 1)}{6\beta} \qquad \text{..... 5.6.1}$$

where $\beta = \dfrac{0.8 f_{ck}}{6.89 p_t}$, but not less than 1

$p_t = \dfrac{100 A_{st}}{bd}$ (percentage steel in rib width only)

$0.8 f_{ck}$ = cylinder strength in terms of cube strength

0.85 = reduction factor similar to $1/\gamma_m$

The formula in BS 8110 for design shear strength of concrete is slightly different, and is given by the expression

$$\tau_c = 0.79\,(p_t)^{1/3}\left(\frac{400}{d}\right)^{1/4}\left(\frac{1}{\gamma_m}\right)\left(\frac{f_{ck}}{25}\right)^{1/3} \qquad \ldots 5.6.2$$

where $\left(\frac{400}{d}\right)$ = the correction factor for depth and should not be less than 1

$\left(\frac{f_{ck}}{25}\right)$ = the correction factor for the strength of concrete and should not be greater than 40

$\gamma_m = 1.25$

p_t = percentage steel, the value of which should not exceed 3

Design shear strength for solid slabs

For solid slabs, the design shear strength for concrete shall be $\tau_c \cdot k$, where k has the values given in Table 5.2

TABLE 5.2 VALUES OF k (IS 456 : 2000)

Overall depth of slab (mm)	300 or more	275	250	225	200	175	150 or less
k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

Note : The above provision shall not apply to flat slabs.

Shear strength of members under axial compression (IS 456 : 2000)

For members subjected to axial compression P_{uc}, the design shear strength of concrete, given in Table 7.1, shall be multiplied by the following factor :

$$\delta = 1 + \frac{3\,P_{uc}}{A_g \cdot f_{ck}}, \text{ but not exceeding } 1.5 \qquad \ldots. 5.6.3$$

where P_{uc} = factored axial compressive force in Newtons

A_g = gross area of concrete section in mm^2, and

f_{ck} = characteristic compressive strength of concrete, in N/mm^2

Shear strength of members under axial tension (ACI Code, 1989):

Though it is evident that there is some reduction in design shear strength of a member under axial tension, IS Code (IS 456 : 2000) does not explicitly mention this case. However, the following simplified expression for δ, based on ACI Code (1989) may be used :

$$\delta = 1 - \frac{P_{ut}}{3.45\,A_g} \qquad \ldots\ldots 5.6.4$$

where P_{ut} = factored axial tensile force in Newtons.

Maximum shear stress in concrete with shear reinforcement (IS 456 : 2000)

(*a*) ***Maximum shear stress in beams*** :

Under no circumstances, even with shear reinforcement, shall the nominal shear stress (τ_v) in beams exceed $\tau_{c,max}$ given is Table 5.3

TABLE 5.3. MAXIMUM SHEAR STRESS r_c , max (N/mm^2)

Grade of concrete	M15	M20	M25	M30	M35	M40 & above
r_c , max (N/mm^2)	2.5	2.8	3.1	3.5	3.7	4.0

(b) Maximum shear stress in solid slabs

For solid slabs the nominal shear stress shall not exceed half the appropriate values given in table 5.3.

WEB REINFORCEMENT FOR DIAGONAL TENSION:

As stated earlier, proper reinforcement must be provided to resist the diagonal tension. The shear resisted by shear reinforcement can be worked out by considering the equilibrium of forces across a potential diagonal crack, which is assumed to be inclined at an angle of 45° with axis of the beam. Fig. 7.11 shows a diagonal crack *AB*. Let the web reinforcement be inclined at angle α with the axis of the beam, and be spaced at distance s_v apart. Let the diagonal crack *AB* intersect n number of *web reinforcing bars*.

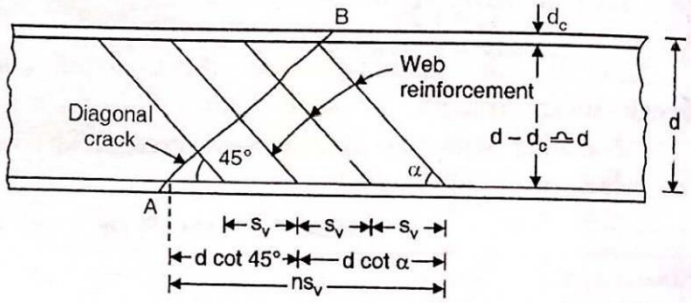


FIG. 5.7.1 SHEAR RESISTED BY WEB STEEL

Let
V_{us} = Ultimate shear carried by shear (or web) reinforcement
f_{vd} = design yield stress in web steel = 0.87 f_y
n = number of bars/links crossing the crack
α = inclination of web steel
A_{sv} = total cross-section area of each set of bar or link.

The web reinforcement is anchored to the main tensile steel at the bottom, and to the holding bars (at a cover d_c) at the top. Hence the vertical component of length of inclined bar = $(d - d_c)$. Since d_c in normally quite small in comparison to d, we can take $(d - d_c \triangleq d)$, as marked in Fig. 5.7.1 Now, for equilibrium

Shear carried by shear reinforcement = Sum of vertical components of tensile forces developed in shear reinforcement

$$\therefore \quad V_{us} = n\, A_{sv}\, f_{vd} \sin \alpha \qquad \text{..... 5.7.1 (a)}$$

In order to get the value of n, we have from geometry,

$$n\, s_v = d \cot 45° + d \cot \alpha \quad \text{or} \quad n = \frac{d \cot 45° + d \cot \alpha}{s_v} = \frac{d\,(1 + \cot \alpha)}{s_v} \qquad \text{... 5.7.1 (b)}$$

Substituting the value of n and f_{vd} (= 0.87 f_y) in Eq. 7.13 (a) we get

$$V_{us} = \frac{d\,(1 + \cot \alpha)}{s_v} \cdot A_{sv}\,(0.87\, f_y) \sin \alpha = \frac{0.87\, f_y\, A_{sv}\, d}{s_v} (\sin \alpha + \cos \alpha) \qquad \text{.... 5.7.2}$$

Rearranging the above, we get

$$s_v = \frac{0.87\, f_y\, A_{sv}\, d}{V_{us}} (\sin \alpha + \cos \alpha) \qquad \text{..... 5.7.2 (a)}$$

The above equation gives the spacing of the bars inclined at α with horizontal.

Here A_{sv} = Area of C/S of bars X No of legs = A_{ϕ} X No of legs.

Special Cases :

(i) Bars inclined at 45^0. (i.e. $\alpha = 45^0$)

If $\alpha = 45°$, $$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v} (\sqrt{2})$$ 5.7.3

or $$s_v = \frac{0.87 f_y A_{sv} d}{V_{us}} \sqrt{2}$$ ..... 5.7.3 (a)

(ii) Bars inclined at 90° (i.e. vertical stirrups)

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$ 5.7.4

or $$s_v = \frac{0.87 f_y A_{sv} d}{V_{us}}$$ ..... 5.7.4 (a)

(iii) Single bar or single group of bars

For a single bar, or single group of bars, all bent up at the same cross-section, we get from Eq. 5.7.1 (a) taking $n = 1$

$$V_{us} = 0.87 f_y \, . \, A_{sv} \sin \alpha$$ 5.7.5

TYPES OF SHEAR REINFORCEMENT.

Shear reinforcement is necessary if the nominal shear stress (r_v) exceeds the design shear stress r_c . In general shear reinforcement is provided in any one of the following three forms.

(a) Vertical stirrups
(b) Bend up bars along with the stirrups.
(c) Inclined stirrups.

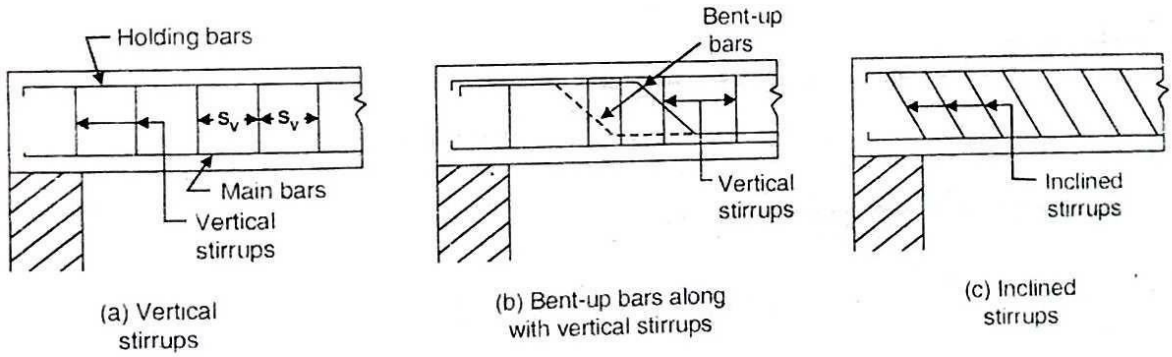


FIG. 5.8 TYPES OF SHEAR REINFORCEMENT

Where bent-up bars are provided , their contribution towards shear resistance shall not be more than half that of total shear reinforcement.

The total external shear V_u is jointly resisted by concrete as well as shear reinforcement and is represented by the expression

$V_u = V_{uc} + V_{us}$

Where V_{uc} = Shear strength of concrete and

V_{us} = Shear reinforcement.

VERTICAL STIRRUPS:

Shear reinforcement in the form of vertical stirrups consists of 5 mm to 15 mm dia steel bars bend round the tensile reinforcement where it is anchored to 6 to 12 mm dia. Anchor bars or holding bars. Depending upon the magnitude of the shear stress to be resisted , a stirrup may be one legged, two legged, four legged or multi legged, as shown in Figure.

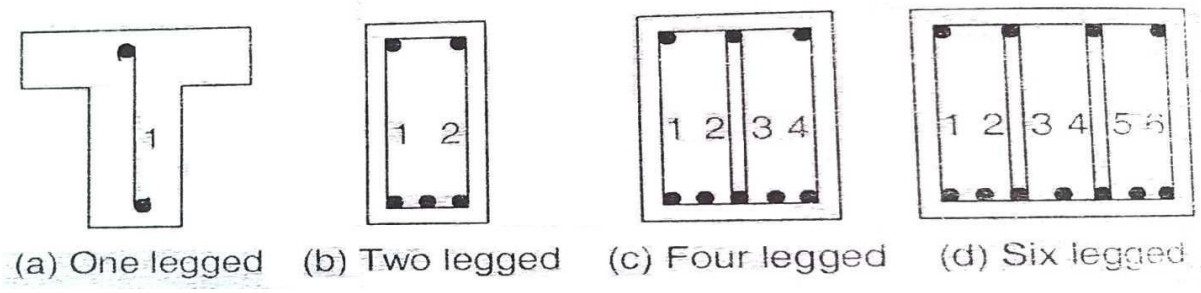


FIG-5.9. FORMS OF VERTICAL STIRRUPS

The strength of shear reinforcement in the form of vertical stirrups is given by

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v} \qquad \ldots\ldots\ldots \quad 5.9.1$$

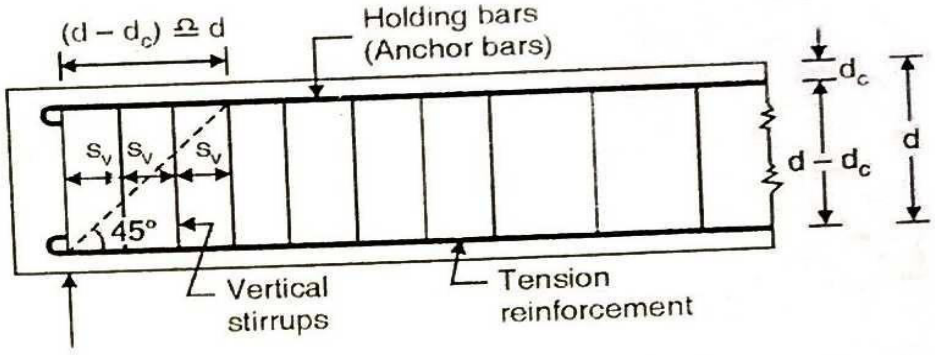


FIG. 5.9.1 SPACING OF VERTICAL STIRRUPS

Here A_{sv} =m $A_{\emptyset}$, where m = no of legs in the stirrups and $A_{\emptyset}$ = Area of c/s of the bar stirrups.

Let us assume that in absense of shear reinforcement, the beam fails in diagonal tension, the inclination of the tenson crack being at 45^0 to the axis of the beam and extended up to a horizontal distance equal to $(d- d_c) = d$

Hence No of stirrups resisting shear force $= d/S_v$, Or,

$$V_{us} = 0.87\, f_y \,.\, A_{sv} \times \frac{d}{s_v}$$

$$s_v = \frac{0.87\, f_y \,.\, A_{sv} \,.\, d}{V_{us}} = \frac{0.87\, f_y \,.\, A_{sv} \,.\, d}{V_u - V_{uc}} \qquad \ldots. \ 5.9.4$$

Spacing diagram for vertical stirrups. The spacing of the stirrups at any section along the length of the beam can be determined from Eq. 5.9.1 since the net shear V_{us} at that section can be easily determined. This will give variable spacings all along the length of the beam. These variable spacings can be *averaged* out by the construction of *spacing diagram.*

Procedure. (1) Determine net shear force at various sections distant x_1 , x_2 , x_3 etc. from the suppor· and calculate the spacings at these sections·

(2) Plot the spacıng diagram with the calculated spacing as ordinate (Fig. 5.9.2)

(3) Calculate the spacing of the first stirrup, on the basis of net S.F. at the support. Set the tırst stirrup at half the distance of this spacing, from the support.

(4) Where this ordinate at (1) intersects the spacing diagram, set a line at 45°, meeting the span line in (2). This will give the position of the second stirrup. Repeat this procedure to get the position of other stirrups.

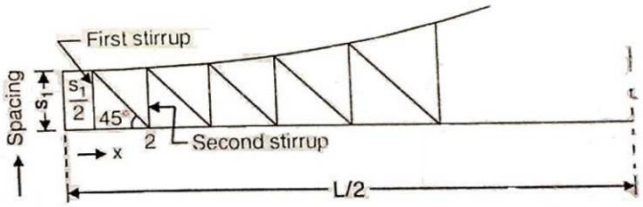


FIG. 5.9.2 SPACING DIAGRAM.

MINIMUM SHEAR REINFORCEMENT (IS 456 : 2000)

The shear reinforcement in the form of stirrups remain unstressed till the diagonal crack occurs at the critical location. However, the instant a diagonal crack occurs. The web reinforcement receives sudden increase in stress. If web reinforcement is not provided. Shear failure may occur without giving any warning. The code therefore, specifies that all the beams should be provided with atleast some minimum reinforcement called nominal shear reinforcement even if nominal shear stress is less than the design shear stress of concrete.

Reasons for providing minimum shear reinforcement:

1. It prevents *sudden* shear failure with the formation of diagonal tension crack, and imparts *ductility* to provide sufficient warning of impending failure. Thus *brittle shear failure* is prevented.

2. It guards against any sudden failure of a beam if concrete cover bursts and bond to tension steel is lost.

3. It holds the main reinforcements in place while pouring the concrete. Thus minimum requirement of cover and clear distance between longitudinal bars are maintained.

4. It acts as necessary ties for the compression steel (if any) and makes it effective.

5. It prevents pressing down of the longitudinal reinforcement, thereby maintaining the *dowel capacity*.

6. It confines the concrete, thereby increasing its strength and rotation capacity.

7. It prevents failure that can be caused by tension due to shrinkage and thermal stresses and internal cracking in the beam.

As per IS 456 : 2000, minimum shear reinforcement in the form of stirrups shall be provided such that

$$\frac{A_{sv}}{b\, s_v} \geq \frac{0.4}{0.87\, f_y} \qquad \text{.... 5.10.1}$$

where A_{sv} = total cross-sectional area of stirrup legs effective in shear.

s_v = stirrup spacing along the length of the member

b = breadth of beam or breadth of the web of flanged beam.

f_y = characteristic strength of stirrup reinforcement in N/mm^2, which *shall not be taken greater than 415 N/mm^2*.

Hence spacing based on minimum shear reinforcement is given by

$$s_v \leq \frac{0.87\, f_y \,.\, A_{sv}}{0.4\, b} \leq \frac{2.175\, f_y\, A_{sv}}{b} \qquad \text{.... 5.10.2}$$

However, where the maximum shear stress calculated is less than half the permissible value, and in members of minor structural importance such as lintels, this provision need not be complied with.

Shear resistance of minimum shear reinforcement

The shear resistance of minimum reinforcement envisaged in Eq. 5.10.1 is found by substituting the value of $\frac{0.87\, f_y \,.\, A_{sv}}{s_v} = 0.4\, b$

$$V_{us,\,min} = \left(0.87 \frac{f_y\, A_{sv}}{s_v} \right) d = (0.4\, b)\, d = 0.4\, bd \qquad \text{..... 5.10.3}$$

Thus, shear carried by concrete and that carried by minimum stirrups is given by

$$V_{u,\,min} = \tau_c \,.\, bd + 0.4\, bd \qquad \text{..... 5.10.4}$$

MAXIMUM SPACING OF SHEAR REINFORCEMENT:-

The maximum spacing of shear reinforcement measured along the axis of the member shall not exceed 0.75d for vertical stirrups and d for inclined stirrups at 45^0, where d is the effective depth of the section under consideration. In no case shall the spacing exceed 300 mm.

Example-5.1.

A reinforced concrete beam 250 mm wide and 400 mm effective depth is subjected to ultimate design shear force of 150 KN at the critical section near supports. The tensile reinforcement at the section near supports is 0.5 percent. Design the shear stirrups near the supports also design the minimum shear reinforcement at the mid span. Assume M20 concrete and F_e 250 mild steel.

Solution : Given : $b = 250$ mm ; $d = 400$ mm; $A_{st}/bd = 0.5\% = 0.005$

$$\tau_v = \frac{V_u}{bd} = \frac{150 \times 10^3}{250 \times 400} = 1.5 \text{ N/mm}^2$$

From Table 5.1 $\tau_c = 0.48$ N/mm^2 for M 20 concrete and $100\,A_{st}/bd = 0.5$

Also, from Table 5.3 $\tau_{c,max} = 2.8$ N/mm^2 for M 20 concrete.

Thus, τ_v is less than $\tau_{c,max}$, but greater than τ_c. Hence shear reinforcement is necessary.

$$V_{uc} = \tau_c\, bd = 0.48 \times 250 \times 400 = 48000 \text{ N}$$

Hence $$V_{us} = V_u - V_{uc} = 150000 - 48000 = 102000 \text{ N}$$

The shear resistance of nominal stirrups is given by

$$V_{us,min} = 0.4\, bd = 0.4 \times 250 \times 400 = 40000 \text{ N} < V_{us}$$

Hence nominal stirrups are *not* sufficient t the section near supports.

We Know that $$s_v = \frac{0.87\, f_y\, A_{sv}}{V_{us}} \cdot d$$

Using two legged stirrups of 10 mm dia. bars, $A_{sv} = 2\,\frac{\pi}{4}(10)^2 = 157.08$ mm^2

$$\therefore \quad s_v = \frac{0.87 \times 250 \times 157.08}{102000} \times 400 \approx 134 \text{ mm}$$

Again we know $$s_v = \frac{0.87\, f_y\, A_{sv}}{(\tau_v - \tau_c)\, b} = \frac{0.87 \times 250 \times 157.08}{(1.5 - 0.48)\, 250} \approx 134 \text{ mm}$$

Maximum spacing = 0.75 d or 300 mm, which ever is less.

Hence provide 10 mm dia. two legged stirrups @ 130 mm c/c at the section near supports.

At mid-span, the spacing of minimum shear reinforcement for 10 mm φ – 2 lgd stirrups is given by Eqn 5.10.2

$$s_v = 0.87\,\frac{f_y\, A_{sv}}{0.4\, b} = \frac{0.87 \times 250 \times 157.08}{0.4 \times 250} = 341.6 \text{ mm}$$

However, maximum spacing is limited to 0.75 d or 300 mm which ever is less.

Hence $s_v = 300$ mm.

Hence provide 10 mm dia. two legged stirrups @ 300 mm c/c at the mid-span.

Example- 5.2 -

A simply supported beam, 300 mm wide and 500 mm effective depth carries a uniformly distributed load of 50 KN/m, including its own weight over an effective span of 6 m. Design the shear reinforcement in the form of vertical stirrups. Assume that the beam contains 0.75% of reinforcement throughout the length. The concrete is of M 20 grade and steel for stirrups is of Fe 250 grade. Take width of support as 400 mm.

Solution:- $W_u = 1.5 \times 50 = 75$ KN/m.

$V_{u\,max} = W_u L/2 = (75 \times 6)/2 = 225$ KN

The critical section lies at a distance of d = 500 mm from the face of support or at a distance of 500 + 400/2 = 700 mm from the centre of the support.

$V_{uD} = 225\text{-}75 \times 0.7 = 172.5$ kN.

And $\tau_v = (172.5 \times 10^3) / (300 \times 500) = 1.15$ N/ mm^2.

From Table-5.1 for 100 $A_s/bd = 0.75\%$, we get $\tau_c = 0.56$ N/ mm^2 for M20 Concrete.

$V_{uc} + 0.56 \times 300 \times 500 = 84000$ N = 84 KN.

Also, $\tau_{v,\,max} = 2.8$ N/mm^2 for M 20 concrete. Since $\tau_v < \tau_{v,\,max}$ it is OK.

However, $\tau_v > \tau_c$; hence shear reinforcement is necessary.

$$V_{us} = V_{uD} - V_{uc} = 172500 - 84000 = 88500 \text{ N}$$

Using 10 mm φ 2-lgd vertical stirrups, $A_{sv} = 2\,\frac{\pi}{4}(10)^2 = 157.1$ mm^2

$$\therefore \text{ Spacing } s_v = \frac{0.87\, f_y \,.\, A_{sv} \,.\, d}{V_{us}} = \frac{0.87 \times 250 \times 157.1 \times 500}{88500} = 193 \text{ mm} \triangleq 190 \text{ mm (say)}$$

Spacing corresponding to minimum shear reinforcements is

$$s_v = \frac{0.87\, f_y\, A_{sv}}{0.4\, b} = \frac{0.87 \times 250 \times 157.1}{0.4 \times 300} = 284.7 \text{ m} \triangleq 280 \text{ mm (say)}$$

However in no case should the spacing exceed $0.75\, d = 0.75 \times 500 = 375$ mm, or 300 m whichever is less. Hence the spacing is to vary from 190 mm at the end section @ 280 mm at a section distant x m (say) from the mid-span. Let us locate this section where the S.F. is V_{ux}.

$$\therefore \quad V_{ux} = \frac{V_{u,\,max}}{3}\, x = \frac{225000}{3}\, x = 75000\, x$$

$$\therefore \quad V_{us} = V_{ux} - V_{uc} = 75000\, x - 84000$$

$$s_v = 280 = \frac{0.87 \times 250 \times 157.1 \times 500}{75000\, x - 84000}$$

from which, we get $x = 1.93$ m from mid-span or 1.07 m from supports. Hence provide 8 mm φ 2 lgd stirrups at a spacing of 190 mm c/c from supports to a section distant 1.07 m from the centre of either supports. For the remaining length, provide the stirrups @ 280 mm c/c.

CHAPTER-6

BOND, ANCHORAGE, DEVELOPMENT LENGTHS, AND SPLICING

5. BOND:

One of the most important assumption in the behavior of reinforced concrete structure is that there is proper 'bond' between concrete and reinforcing bars. The force which prevents the slippage between the two constituent materials is known as bond. In fact , bond is responsible for providing ' strain compatibility ' and composite action of concrete and steel. It is through the action of bond resistance that the axial stress (tensile or compressive) in a reinforcing bar can undergo variation from point to point along its length. This is required to accommodate the variation in bending moment along the length of the flexural member.

When steel bars are embedded in concrete, the concrete, after setting, adheres to the surface of the bar and thus resists any force that tends to pull or push this rod. The intensity of this adhesive force bond stress. The bond stresses are the longitudinal shearing stress acting on the surface between the steel and concrete, along its length. Hence bond stress is also known as interfacial shear. Hence bond stress is the shear stress acting parallel to the reinforcing bar on the interface between the bar and the concrete.

TYPES OF BOND:-

Bond stress along the length of a reinforcing bar may be induced under two loading situations, and accordingly bond stresses are two types :

1. Flexural bond or Local bond
2. Anchorage bond or development bond

Flexural bond (τ_{bf}) is one which arises from the change in tensile force carried by the bar, along its length, due to change in bending moment along the length of the member. Evidently, flexural bond is critical at points where the shear (V=dM/dx) is significant. Since this occurs at a particular section, flexural bond stress is known as local bond stress [Fig- 5.1(b)].

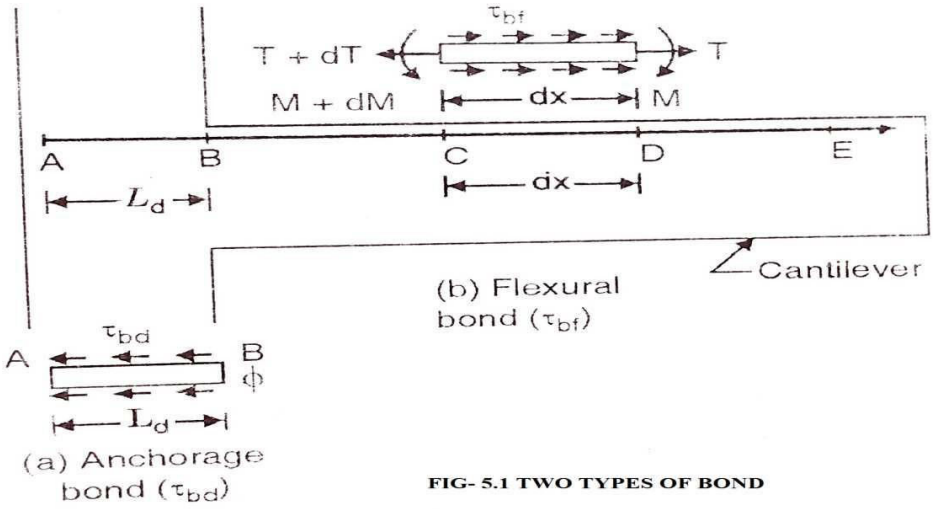


FIG- 5.1 TWO TYPES OF BOND

Anchorage bond (τ_{bd}) is that which arises over the length of anchorage provided for a bar. It also arises near the end or cutoff point of reinforcing bar. The anchorage bond resists the 'pulling out' of the bar if it is in tension or 'pushing in' of the bar if it is in compression. Fig.[8.1 (a)] shows the situation of anchorage bond over a length AB(=L_d). Since bond stresses are developed over specified length L_d, anchorage bond stress is also known as developed over a specified length L_d, anchorage bond stress is also known as development bond stress.

Anchoring of reinforcing bars is necessary when the development length of the reinforcement is larger than the structure. Anchorage is used so that the steel's intended tension load can be reached and pop-outs will not occur. Anchorage shapes can take the form of 180 or 90 degree hooks.

5.2. ANCHORAGE BOND STRESS:

Fig- 5.2 shows a steel bar embedded in concrete And subjected to a tensile force T. Due to this force There will be a tendency of bar to slip out and this tendency is resisted by the bond stress developed over the perimeter of the bar, along its length of embedment .

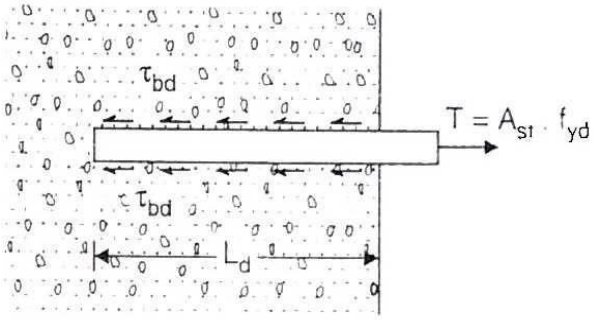


FIG- 5.2

Let us assume that average uniform bond stress is developed along the length. The required length necessary to develop full resisting force is called **Anchorage length** in case of axial tension or compression and **development length** in case of flexural tension and is denoted by L_d.

DESIGN BOND STRESS:-

The design bond stress in limit state method for plain bars in tension shall be as given below (Table 6.1)

Table- 6.1

Grade of concrete	M 20	M 25	M 30	M 35	M 40 and above
Design bond stress τ_{bd} (N/mm^2)	1.2	1.4	1.5	1.7	1.9

Design bond stresses for deformed bars in tension : For deformed bars conforming to IS 1786. These values shall be increased by 60%.

Design bond stress for bars in compression : For bars in compression, the values of bond stress for in tension shall be increased by 25%.

DEVELOPMENT LENGTH OF BARS (IS 456 : 2000)

The development length is defined as the length of the bar required on either side of the section under consideration, to develop the required stress in steel at that section through bond. The development length L_d given by

$$L_d=\phi\sigma_s/4\ \tau_{bd}=k_d\phi \quad \text{..... 5.4.1}$$

Where ϕ= nominal diameter of the bar

σ_s= stress in bar at the section considered at design load

k_d= development length factor = $\sigma_s/4\ \tau_{bd}$

Note : The development length includes the anchorage values of hooks in tension reinforcement

Taking σ_s= 0.87 f_y at the collapse stage, k_d= 0.87 $f_y/4\ \tau_{bd}$ 5.4.2

For bars in compression, the value of τ_{bd}given in table 1.1 are to be increased by 25%. Hence developed length (L_{dc}) for bars in copressio is given by

$L_{ds}=\phi\ \sigma_{sc}/5\ \tau_{bd}$ 5.4.3

Hence the values of k_d for bars In compression will be $=0.87 f_y/5 \tau_{bd}$

Table 6.2 gives the values of development length factor for various grades of concrete and the various grades of steel, both in tension as well as compression. The values have been rounded-off to the higher side.

TABLE 6.2- VALUES OF DEVELOPMENT LENGTH FACTOR

Grade of concrete	M 20			M 25		
Grade of steel	Fe 250	Fe 415	Fe 500	Fe 250	Fe 415	Fe 500
Bars in tension	46	47	57	39	41	49
Bars in comp.	37	38	46	31	33	39

Grade of concrete	M30			M35			M40		
Grade of steel	Fe 250	Fe 415	Fe 500	Fe 250	Fe 415	Fe 500	Fe 250	Fe 415	Fe 500
Bars in tension	37	38	46	32	34	40	29	30	36
Bars in comp.	29	31	37	26	27	32	23	24	29

Note : When the actual reinforcement provided is more than that theoretically required, so that the actual stress (σ_s) in steel is less than the full deign stress ($0.87 f_y$), the development length required may be reduced by the following relation :

Reduced development length $L_{dr} = L_d$ (A_{st} required $\div A_{st}$ provided)

This principle is used in the design of footing and other short bending members where bond is critical. By providing more steel, the bond requirements are satisfied.

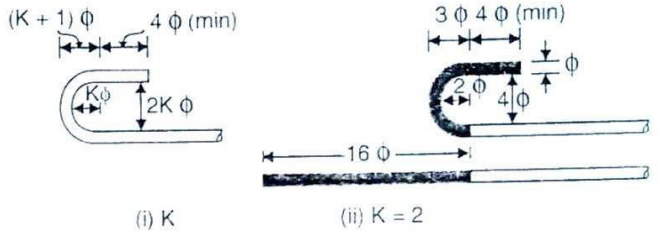
Bars bundled in contact : The development length of each bar bundled bars shall be that for the individual by 10% for two bars in contact,20% for three bars in contact and 33% for four bars in contact.

STANDARD HOOKS & BENDS FOR END ANCHORAGEANCHORAGE LENGTH

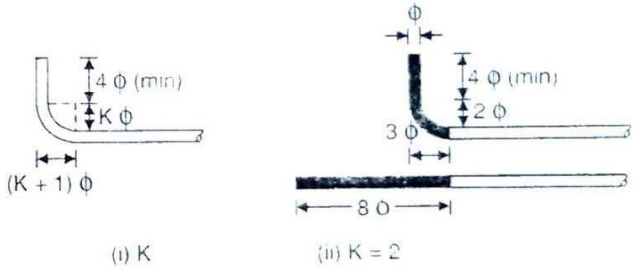
The development length required at the end of a bar is known as *anchorage length.* However, in the case of development length, the force in the bar is developped by transfer. of force from concrete to steel, while in the case of anchorage length, there is dissipation of force from steel to concrete.

Quite often, space available at the end of beam is limited to accommodate the full development length L_d . In that case, hooks or bends are provided. The anchorage value (L_v) of hooks or bend is accounted as contribution to the development length L_d.

Fig. 5.5 (*ai*) shows a semi-circular hook, fully dimensioned, with respect to a factor K. The value of K is taken as 2 in the case of mild steel conforming to IS : 432–1966, (specifications for Mild-Steel and Medium Tensile Steel bars and Hard-Drawn steel wires for concrete reinforcement) or IS : 1139–1959. (specifications for 'Hot rolled mild steel and medium tensile steel deformed bars for concrete reinforcement'). The hook with K = 2 is shown in Fig. 5.5 (*aii*) with equivalent horizontal length of the hook. For the case of Medium Tensile Steel conforming to IS : 432–1966 or IS : 1139–1959, K is taken as 3. In the case of cold worked steel conforming to IS : 1986–1961, (specifications for cold twisted steel bars for concrete reinforcement), K is taken as 4. In the case of bars above 25 mm, however, it is desirable to increase the value of K to 3, 4 and 6 respectively.



(a) Semi circular Hook



(b) Right Angle Bend

FIG- 5.5

Fig- 5.5 shows a right angled bend, with dimensions in terms of K, the value of which may be taken as 2 for ordinary mild steel for diameters below 25 mm and 3 for diameters above 25 mm.

In the case of deformed bars , the value of bond stress for various grades of concrete is greater by 60% than the plane bars. Hence deformed bars may be used without hooks, provided anchorage requirements are adequately met with.

CODE REQUIREMENTS FOR ANCHORING REINFORCING BARS (IS 456 : 2000)

(i) **Anchoring Bars in Tension :-** Deformed bars may be used without end anchorages provided development length required is satisfied. Hooks should normally be provided for plain bars in tension. The anchorage value of bend shall be taken as 4 times the diameter of the bar for each 45° bend subject to a maximum of 16 times the diameter of the bar. The anchorage value of a standard U-type hook shall be equal to 16 times the diameter of the bar.

(ii) **Anchoring Bars in Compression :-** The anchorage length of straight bar in compression shall be equal to the development length of bars in compression. The projected length of

hooks, bends and straight lengths beyond bends if provided for a bar in compression, shall be considered for development length.

(iii) Anchoring Shear Reinforcement :-

Inclined bars :- The developments length shall be as for bars in tension ; this length shall be measured as under : (1) in tension zone from the end of the sloping or inclined portion of the bar and (2) in the compression zone, from mid depth of the beam.

Stirrups :- Notwithstanding any of the provisions of this standard, in case of secondary reinforcement , such as stirrups and traverse ties, complete development lengths and anchorage shall be deemed to have been provided when the bar is bent through an angle of atleast 90° round a bar of atleast its own diameter and is continued beyond the end of the curve for a length of atleast eight diameters, or when the bar is bent through an angle of 135° and is continued beyond the end of curve for a length of atleast six bar diameters or when the bar is bent through an angle of 180° and is continued beyond the end of the curve for a length atleast four bar diameters.

CHECKING DEVELOPMENTS LENGTH OF TENSION BARS :-

As stated earlier, the computed stress (σ_s) in a reinforcing bar, at every section must be developments on both the sides of section. This is done by providing development length L_d to both sides of the section. Such a developments length is usually available at mid-span location where positive (or sagging) B.M. is maximum for simply supported beams. Similarly, such a developments length is usually available at the intermediate support of a continuous beam where negative (or hogging) B.M. is maximum. Hence no special checking may be necessary in such locations. However special checking for developments length is essential at the following locations :

1. At simple supports
2. At cantilever supports
3. In flexural members that have relatively short spans
4. At points of contraflexure
5. At lap splices
6. At points of bar cutoff
7. For stirrups and transverse lies.

DEVELOPMENTS LENGTH REQUIREMENTS AT SIMPLE SUPPORTS : DIAMETER OF REINFORCING BARS :-

The code stipulates that at the simple supports (and at the point of inflection), the positive moment tension reinforcement shall be limited to a diameter such that

$$L_d \leq M_1/V + L_o \quad \text{..... 5.8.1}$$

Where L_d = developments length computed for design stress f_{yd} (=0.87 f_y) from Eq^n

M_1= Moments resistance of the section assuming all reinforcement at the section to be stressed to f_{yd}(= 0.87 f_y)

V= Shear force at the section due to design loads

L_o= sum of anchorage beyond the centre of supports and the equivalent anchorage value of any hook or mechanical anchorage at the simple support (At the point of inflexion, L_o is limited to *d* or 12ϕ which ever is greater).

The code further recommends that the value of M_1/V in eq^n - 5.8.1 may be increased by 30% when the ends of the reinforcement are confined by a compressive reaction. This condition of confinement of reinforcing bars may not be available at all the types of simple supports.

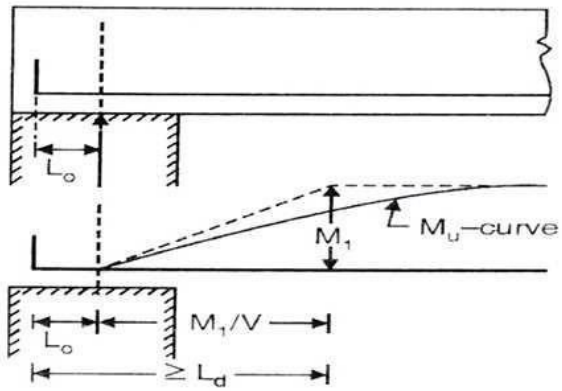


FIG-5.8.1

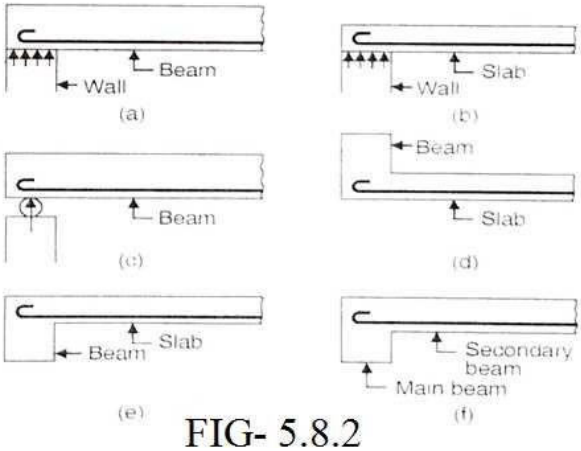


FIG- 5.8.2

Four type of simple supports are shown in fig-5.8.2. In fig- 5.8.2 (a) , the beam is simply supported on a wall which offers a compressive reaction which confines the ends of reinforcement. Hence a factor 1.3 will be applicable. However in fig-5.8.1 (c) and (d) though a simple support is available , the reaction does not confine the ends of the reinforcement, hence the factor 1.3 will not be applicable with M_1/V term.Simillarly for the case of a slab connected to a beam Fig- 5.8.2€ or for the case of secondary beam connected to a main beam [Fig-5.8.2(f)]

Tensile reaction is induced and hence a factor 1.3 will not be available.

Thus at simple supports where the compressive reaction confines the ends of reinforcing bars we have

$L_d \leq 1.3\, M_1/V + L_0$ 5.8.2

Computation of the Moment of Resistance M_1 of bars available at supports:

In eqn 5.8.1 , M_1 = Moment of Resistance of the section corresponding to the area of steel (A_{st}) continued into the support and stressed to design stress equal to design stress equal to $0.87f_y$.

$M_1 = 0.87f_y . A_{st} (d- 0.416 X_u)$.....5.8.3

Where $X_u = 0.87f_y . A_{st} / 0.36f_{ck} b$.....5.83(a)

Computation of Length (L_0) :

For the computation of L_0 , the support width should be known.Fig- 5.8.3 (a) and (b) show abeam with end support with a standard hook and 90^0 bend respectively.

Let X be the side cover to the hook (Or bend) and X_0 be the distance of the beginning of the hook (Or Bend) from the center line of the support.

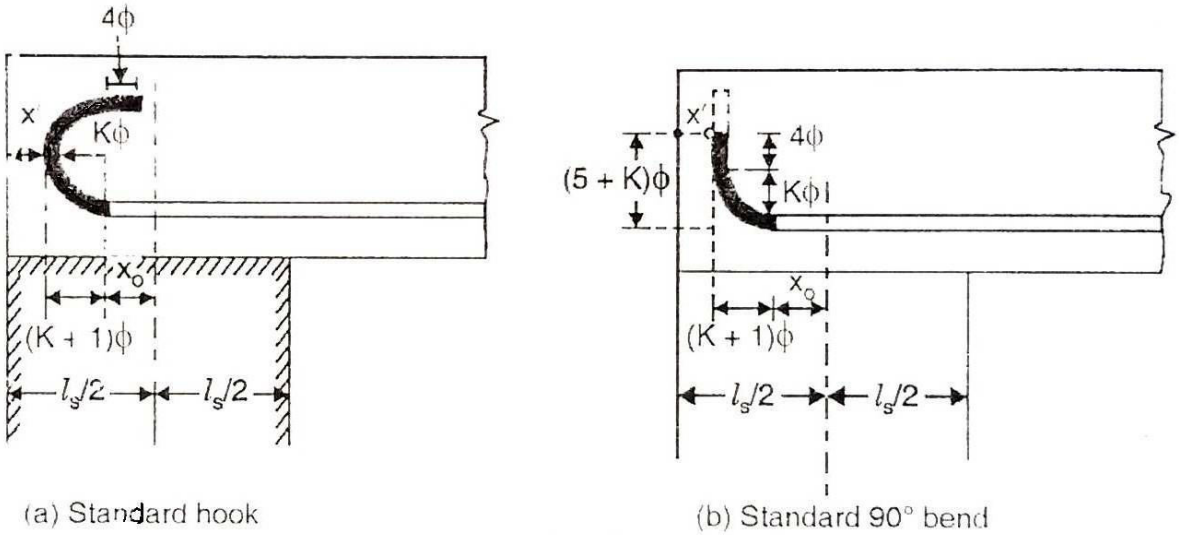


FIG. 5.8.3 COMPUTATION OF L_0

(a) **Case-I : Standard Hook at the end [Fig-5.8.3(a)]:-** The dark portion shows the hook which has an anchorage value of 16φ (IS 456: 2000) for all types of steel. The distance of the beginning of the hook from its apex of the semi circle is equal to (K+1)φ . For mild steel bars K=2 and for HYSD bars, K=4, Hence the distance φ for mild steel and 5φ for HYSD bars. Let *l* be the width of the support.

Then $L_0 = x_0 + 16\ \varphi$ where $x_0 = \frac{l_s}{2} - x' - (K+1)\ \varphi$

$\therefore$ $L_0 = \left(\frac{l_s}{2} - x' - (K+1)\ \varphi \right) + 16\ \varphi = \frac{l_s}{2} - x' + (15 - K)\ \varphi$ 5.84

Taking $K = 2$ for mild steel bars, $L_0 = \frac{l_s}{2} - x' + 13\ \varphi$ ..... 5.84(a)

Taking $K = 4$ for HYSD bars, $L_0 = \frac{l_s}{2} - x' + 11\ \varphi$ ..... 5.84 (b)

(*b*) **Case 2 : 90° standard bend** (Fig. 8.7 *b*) : The dark portion shows the 90° bend which has an anchorage value of 8 φ (IS 456 : 2000) for all types of steel. Here also, the distance of beginning of the hook from its apex of the semi-circle is equal to $(K + 1)\ \varphi$.

Then $L_0 = x_0 + 8\ \varphi$ where $x_0 = \frac{l_s}{2} - x' - (K+1)\ \varphi$

$\therefore$ $L_0 = \left(\frac{l_s}{2} - x' - (K+1)\ \varphi \right) + 8\ \varphi = \frac{l_s}{2} - x' + (7 - K)\ \varphi$ 5.8.5

Taking $K = 2$ for mild steel bars, $L_0 = \frac{l_s}{2} - x' + 5\ \varphi$ .... 5.8.5(a)

Taking $K = 4$ for HYSD bars $L_0 = \frac{l_s}{2} - x' + 3\ \varphi$ ..... 5.8.5(b)

Remedies to get development length : If the check for the satisfaction of Eq. 5.8.1 is not obtained, following remedial measures may be adopted to satisfy the check .

1. Reduce the diameter φ of the bar, thereby reducing the value of L_d, keeping the area of steel at the section unchanged. This is the standard procedure envisaged in the Code, i.e. reducing the value of L_d by *limiting* the diameter of the bar to such a value that Eq. 8.6 is satisfied.

2. Increasing the value of L_0 by providing *extra length* of the bend over and above the standard value $(5 + K)\ \varphi$ shown by dotted lines in Fig. 5.8.2 (b).

3. By increasing the number of bars (there by increasing A_{st}) to be taken into the support. This method is uneconomical.

4. By providing adequate mechanical anchorage.

We shall discuss the first remedial method in the following section.

CONDITIONS FOR CURTAILMENT OF REINFORCEMENT

In most of the cases, the B.M. varies appreciably along the span of the beam. From the point of view of economy, the moment of resistance of the beam should be reduced along the span according to the variation of B.M. This is effectively achieved by reducing the area of reinforcement, i.e. by curtailing the reinforcement provided for maximum B.M. In general, all steel, whether in tension or in compression, should extend d or $12\,\varphi$ (which ever is greater) beyond the theoretical point of cut off (TPC).

Conditions for termination of tension reinforcement in flexural members:

Curtailment of Flexural tension reinforcement results in the loss of shear strength in the region of cutoff and hence it is necessary to make provision to guard against such loss. Flexural reinforcement shall not be terminated in a tension zone unless any one of the following condition is satisfied.

(*a*) The shear at the cutoff point does not exceed two thirds that permitted, including the shear strength of web reinforcement. In other words, the total *shear capacity* shall be atleast 1.5 times the applied shear at the point of curtailment, thus

$$V_u \not> \frac{2}{3}(V_{uc} + V_{us}) \quad \text{or} \quad V_{uc} + V_{us} \geq 1.5\ V_u$$

Where V_{uc} = shear capacity of concrete, based on continuing reinforcement only.
V_{us} = shear capacity of shear reinforcement
V_u = applied shear at the point of curtailment.

(*b*) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar over a distance from cutoff point equal to three fourth the effective depth of the member. Excess area of shear reinforcement is given by :

Excess
$$A_{sv} \geq \frac{0.4\ b\ s_v}{f_y}$$

where
$$s_v \not< \frac{d}{8\ \beta_b} \not> \frac{0.87\ f_y\ A_{sv}}{0.4\ b}$$

$$\beta_b = \frac{\text{area of bars cutoff at the section}}{\text{total area of bars at the section}}$$

(*c*) For 36 mm or smaller bars, the continuing bars provide double the area required for flexure at the cutoff point and the shear does not exceed three fourth that permitted.

Thus,
$$M_{ur} \geq 2\ M_u$$

and
$$V_{uc} + V_{us} \geq 1.33\ V_u$$

where M_{ur} = moment of resistance of remaining (or continued) bars
M_u = B.M. at cutoff point ; V_u = S.F. at cutoff point

5.9 DEVELOPMENT LENGTH AT POINT OF INFLEXION

Fig. 8.8 shows the conditions at a point of inflection (P.I.) As already indicated in § 8.11, the Code states that the following condition be satisfied

$$\left(\frac{M_1}{V} + L_0\right) \geq L_d \qquad \ldots\ldots 5.9.1$$

where L_0 should not be greater than d or 12 φ whichever is greater, and V is the shear force at the point of inflexion.

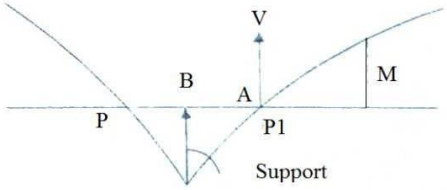


FIG. 5.9 DEVELOPMENT LENGTH AT A POINT OF INFLEXION

5.10 SPLICING:

(a) The purpose of splicing is to transfer effectively the axial force from the terminating bar to the connecting bar with the same line of action at the junction. [Fig-5.10 (a)].

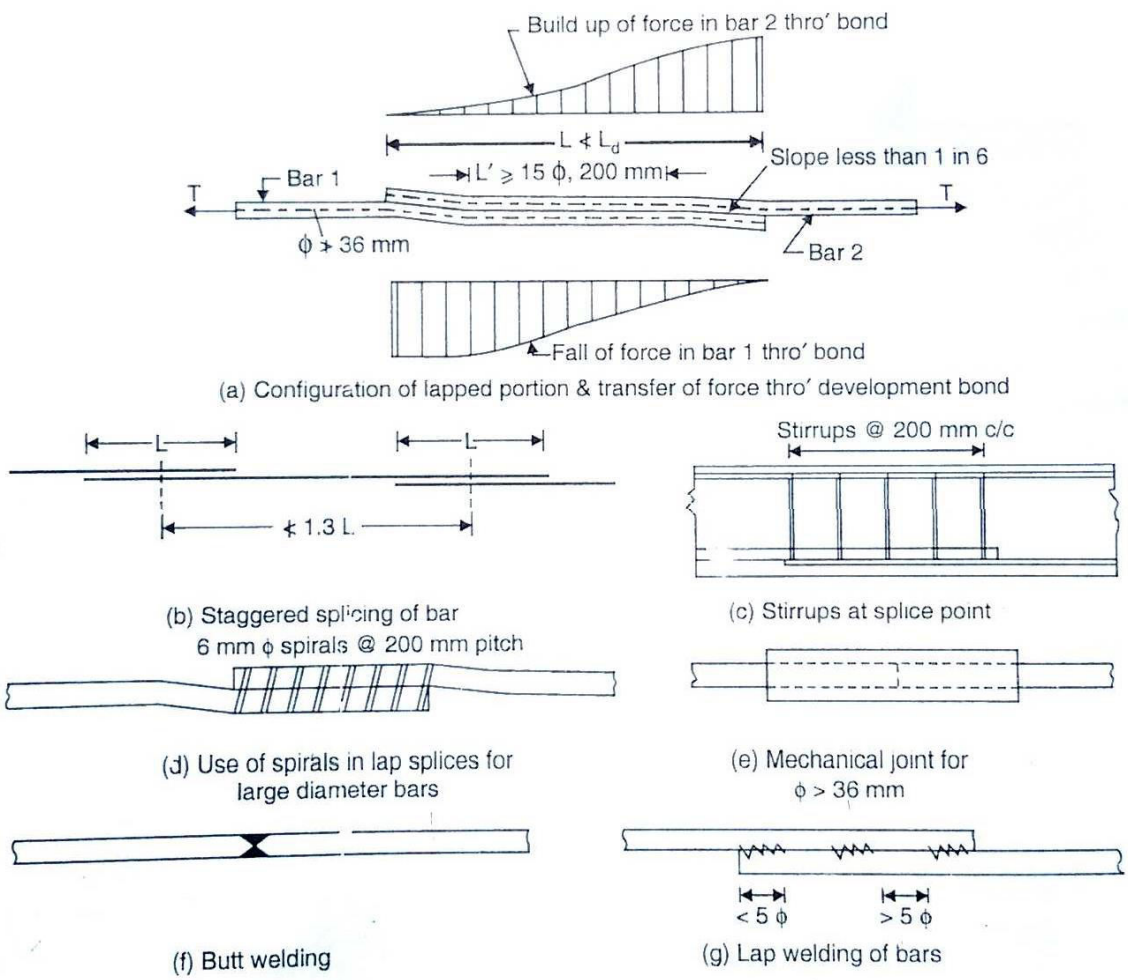


FIG- 5.10 REINFORCEMENT SPLICING

Slicing of a bar is essential in the field due to either the requirements of construction or non availability of bars of desired length. The Figures given are as per the recommendation of the IS 456 : 2000.

(a) Lap slices shall not be used for bars larger than 36 mm. For larger diameters bars may be weld. In case where welding is not practicable , lapping of bars larger than 36mm may be permitted, in which case additional spiral should be provided around the lapped bars [Fig-5.10(d)].

(*b*) Lap splices shall be considered as staggered if the centre to centre distance of the splices is not less than 1.3 times the lap length calculated as described in (*c*).

(*c*) The lap length including anchorage value of hooks for bars in *flexural tension shall be* L_d *or* 30 φ whichever is greater and for *direct tension* shall be 2 L_d or 30 φ whichever is greater. The *straight length* (*L'*) of the lap shall not be less than 15 φ or 200 mm (Fig. 5.10 [a]) The following provisions shall also apply :

(1) Top of a section as cast and the minimum cover is less than twice the diameter of the lapper bar, the lapped length shall be increased by a factor of 1.4.

(2) Corner of a section and minimum cover to either face is less than twice the diameter of the lapped bar or where the clear distance between adjacent laps is less than 75 mm or 6 times the diameter of lapper bar, whichever is greater, the lap length should be increased by a factor of 1.4.

Where both conditions (1) and (2) apply , the lap length should be increased by a factor of 2.0.

Note : Splices in tension members shall be enclosed in spirals made of bars not less than 6 mm diameter with pitch not more than 100 mm.

(*d*) The lap length in compression shall be equal to the development length in compression, but not less than 24 φ.

(*e*) When bars of two different diameter are to be spliced, the lap length shall be calculated on the basis of diameter of the smaller bar.

(*f*) When splicing of welded wire fabric is to carried out, lap splices of wires shall be made so that overlap measured between the extreme cross wires shall be not less than spacing of cross wires plus 100 mm.

(*g*) In case of bundled bars, lapped splices of bundled bars shall be made by splicing one bar at a time : such individual splices within a bundle shall be staggered.

Strength of Welds :

The following values may be used where the strength of weld has been proved by tests to be at least as great as that of the parent bars.

(a) Splices in compression:

For welded splices and mechanical connection, 100 percent of the design strength of joined bars.

(b) Splices in tension:

(1) 80% of the design strength of welded bars (100% if welding is strictly supervised and if at any c/s of the member not more than 20% of the tensile reinforcement is welded)

(2) 100% of the design strength of mechanical connection.

End Bearing Splices: End bearing splices should be used only for bars in compression. These are of square cut and concentric bearing ensured by suitable devices.

EXAMPLE-6.1

A SIMPLY SUPPORTED IS 25 cm X50cm and has 2 – 20 mm TOR bars going into the support. If the shear force at the center of the support is 110 KN at working loads, determine the anchorage length.assume M20 mix and Fe 415 grade TOR steel .

Solution:-

For a load factor equal to 1.5,the factored SF= 1.5 x 110=165 kN.

Assuming 25 mm clear cover to the longitudinal bars

Effective depth = 5000- 25 – 20/2 = 465 mm.

Characteristic strength of TOR steel $\sigma_y = 415\ N/mm^2$

Moment of resistance $M_1 = 0.87\,\sigma_y\,A_t\,(d - 0.42\,x)$

$$x = \frac{0.87\sigma_y A_t}{0.36\sigma_{ck}\,b} = \frac{0.87\times415\times628}{0.36\times20\times250} = 126\ mm < x_m \qquad \text{OK}$$

or $M_1 = 0.87 \times 415 \times 2 \times \pi/4 \times 20^2\,(465 - 0.42 \times 126) = 93.45 \times 10^6\ Nmm$

Bond stress $\tau_{bd} = 1.2\ N/mm^2$ for M20 mix. It can be increased by 60% in case of TOR bars.

$$\text{Development length } L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{0.87\times415\phi}{4\times(1.6\times1.2)} = 47\,\phi$$

If the bar is given a 90° bend at the centre of support, its anchorage value

$$L_o = 8\,\phi = 8 \times 20 = 160\ mm$$

$$L_d \le 1.3\,M_1/V + L_o$$

$$47\,\phi \le \left[\frac{1.3\times93.45\times10^6}{165\times1000}\right] + 160$$

or, $$\phi \le 19\ mm$$

Since actual bar diameter of 20 mm is greater than 19 mm, there is a need to increase the anchorage length. Let us increase the anchorage length L_o to 240 mm. It gives

$$\phi \leq 20.8 \text{ mm} \qquad \text{OK}$$

The arrangement of 90° bend is shown in Fig. 8.19a.

Alternatively

Provide a U bend at the centre of support, its anchorage value,

$$L_o = 16\,\phi = 320 \text{ mm}$$

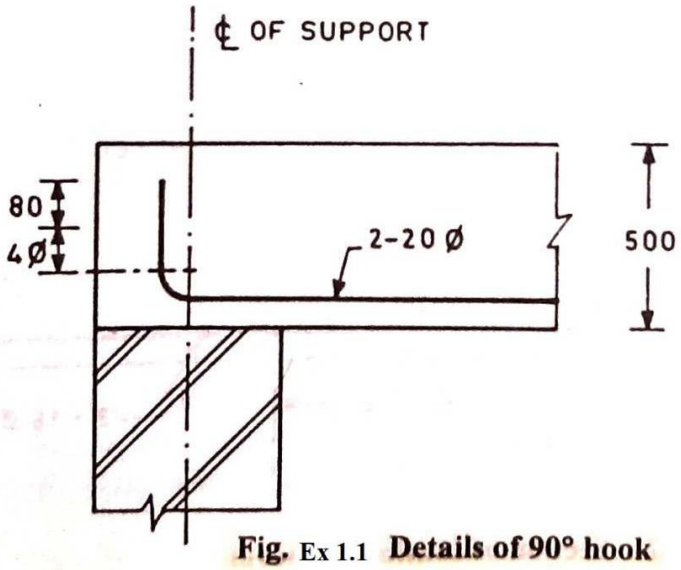


Fig. Ex 1.1 Details of 90° hook

$L_d \leq 1.3\ M_1/V + L_0$.

$$47\emptyset \leq \left[\frac{1.3\ X\ 93.45\ X\ 10^6}{165\ X\ 1000}\right] + 320$$

Or. $\emptyset \leq 22.47$ mm

Actual bar diameter provided is 20 mm < 22.47 mm.

The arrange ment of U- Bend is shown in Fig-Ex 1.2.

In High strength reinforced bars U- Bend should be avoided as far as possible since they may be brittle and may fracture with bending.

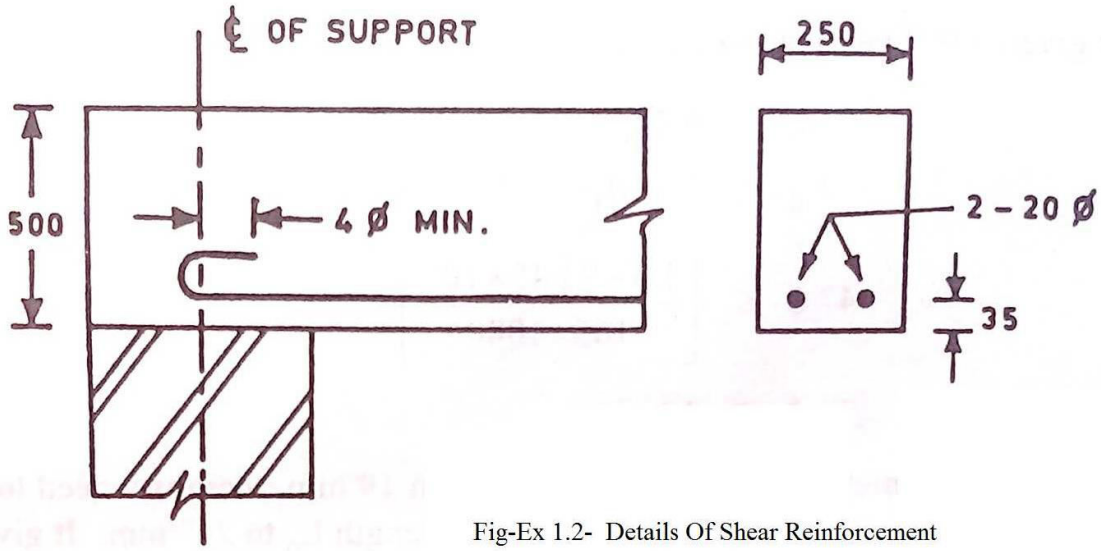


Fig-Ex 1.2- Details Of Shear Reinforcement

Example 5.2:

A continuous beam 25 cm X 40 cm carries 3-16 mm longitudinal bars beyond the point of inflection in the sagging moment region as shown in Fig.Ex 1.3,.If the factored SF at the point of inflection is 150 KN, σ_{ck} = 20 N/mm^2 and σ_y = 415 N/mm^2, check if the beam is safe in bond ?

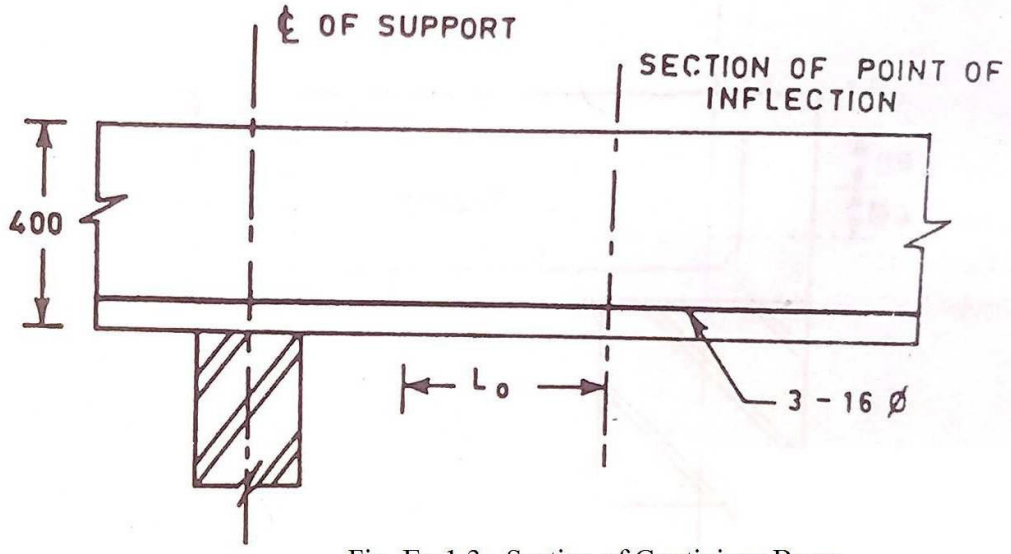


Fig- Ex 1.3 - Section of Continious Beam

Depth of neutral axis $x = \dfrac{0.87\sigma_y A_t}{0.36\sigma_{ck} b} = \dfrac{0.87\times415\times3\times\pi/4\times16^2}{0.36\times20\times250}$

$= 120 \text{ mm} < x_m (= 0.48\ d)$ OK

Moment of resistance $M_1 = 0.87\ \sigma_y A_t (d - 0.42\ x)$

$= 0.87\times 415 \times 603\ (367 - 0.42 \times 120) = 68.90 \times 10^6$ Nmm

Development length $L_d = \dfrac{\sigma_s \phi}{4\tau_{bd}}$

Bond stress $\tau_{bd} = 1.6 \times 1.2$ N/mm^2 for M20 mix and HSD steel

or $L_d = \dfrac{0.87\times415\phi}{4\times1.6\times1.2} = 47\ \phi$

Anchorage length $L_o =$ greater of d or 12 ϕ

$=$ greater of 367 mm, or $12 \times 16 = 192$ mm

$= 367$ mm

$L_d \leq \dfrac{M_1}{V} + L_o$

or $47\ \phi \leq \dfrac{68.9\times10^6}{150\times1000} + 367$ or, $\phi \leq 17.6$ mm

Thus, 16 mm bars are safe in bond at the point of inflection.

CHAPTER 07

INTRODUCTION TO BEAM IN LIMIT STATE METHOD

Beams (LSM)

Control of deflection and ensuring lateral stability in beams as per Clause 23.2 & 23.3 of IS-456.

Design of singly reinforced rectangular beams, Design of doubly reinforced beams as per IS 456/SP-16 for bending and shear

Design of T beams as per IS 456 for bending and shear

Course Objectives:

Control of deflection and ensuring lateral stability in beams as per Clause 23.2 & 23.3 of IS-456.

Introduction

Structures designed by limit state of collapse are of comparatively smaller sections than those designed employing working stress method. They, therefore, must be checked for deflection and width of cracks. Excessive deflection of a structure or part thereof adversely affects the appearance and efficiency of the structure, finishes or partitions. Excessive cracking of concrete also seriously affects the appearance and durability of the structure. Accordingly, cl. 35.1.1 of IS 456 stipulates that the designer should consider all relevant limit states to ensure an adequate degree of safety and serviceability. Clause 35.3 of IS 456 refers to the limit state of serviceability comprising deflection in cl. 35.3.1 and cracking in cl. 35.3.2. Concrete is said to be durable when it performs satisfactorily in the working environment during its anticipated exposure conditions during service. This lesson discusses about the different aspects of deflection of beams and the requirements as per IS 456. In addition, lateral stability of beams is also taken up while selecting the preliminary dimensions of beams.

Short- and Long-term Deflections

Short-term deflection refers to the immediate deflection after casting and application of partial or full service loads, while the long-term deflection occurs over a long period of time largely due to shrinkage and creep of the materials The following factors influence the short-term deflection of structures:

(a) magnitude and distribution of live loads,

(b) span and type of end supports,

(c) cross-sectional area of the members,

(d) amount of steel reinforcement and the stress developed in the reinforcement,

(e) characteristic strengths of concrete and steel, and

(f) amount and extent of cracking

.

The long-term deflection is almost two to three times of the short-term deflection. The following are the major factors influencing the long-term deflection of the structures.

(a) Humidity and temperature ranges during curing,

(b) age of concrete at the time of loading, and

(c) type and size of aggregates, water-cement ratio, amount of compression reinforcement, size of members etc., which influence the creep and shrinkage of concrete.

Control of deflection and ensuring lateral stability in beams as per Clause 23.2 & 23.3 of IS-456.

Clause 23.2 of IS 456 stipulates the limiting deflections under two heads as given below:

(a) The maximum final deflection should not normally exceed span/250 due to all loads including the effects of temperatures, creep and shrinkage and measured from the as-cast level of the supports of floors, roof and all other horizontal members.

(b) The maximum deflection should not normally exceed the lesser of span/350 or 20 mm including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes.

It is essential that both the requirements are to be fulfilled for every structure.

Selection of Preliminary Dimensions :

The two requirements of the deflection are checked after designing the members. However, the structural design has to be revised if it fails to satisfy any one of the two or both the requirements. In order to avoid this, IS 456 recommends the guidelines to assume the initial dimensions of the members which will generally satisfy the deflection limits. Clause 23.2.1 stipulates different span to effective depth ratios and cl. 23.3 recommends limiting slenderness of beams, a relation of *b* and *d* of the members, to ensure lateral stability. They are given below:

(A) For the deflection requirements

Different basic values of span to effective depth ratios for three different support conditions are prescribed for spans up to 10 m, which should be modified under any or all of the four different situations:

(i) for spans above 10 m,

(ii) depending on the amount and the stress of tension steel reinforcement,

(iii) depending on the amount of compression reinforcement,

(iv) for flanged beams.

(B) For lateral stability as per clause 23.3 of IS-456

The lateral stability of beams depends upon the slenderness ratio and the support conditions. Accordingly cl. 23.3 of IS code stipulates the following:

(i) For simply supported and continuous beams, the clear distance between the lateral restraints shall not exceed the lesser of $60b$ or $250b^2/d$, where d is the effective depth and b is the breadth of the compression face midway between the lateral restraints.

(ii) For cantilever beams, the clear distance from the free end of the cantilever to the lateral restraint shall not exceed the lesser of $25b$ or $100b^2/d$.

Span/depth ratios and modification factors

Sl. No.	Items	Cantilever	Simply supported	Continuous
1	Basic values of span to effective depth ratio for spans up to 10 m	7	20	26
2	Modification factors for spans > 10 m	Not applicable as deflection calculations are to be done.	Multiply values of row 1 by 10/span in metres.	
3	Modification factors depending on area and stress of steel	Multiply values of row 1 or 2 with the modification factor from Fig.4 of IS 456.		
4	Modification factors depending as area of compression steel	Further multiply the earlier respective value with that obtained from Fig.5 of IS 456.		
5	Modification factors for flanged beams	(i) Modify values of row 1 or 2 as per Fig.6 of IS 456. Further modify as per row 3 and/or 4 where reinforcement percentage to be used on area of section equal to $b_f d$.		

Design of singly reinforced rectangular beams, Design of doubly reinforced beams as per IS 456/SP-16 for bending and shear

Design of singly reinforced rectangular beams as per IS 456/SP-16 for bending and shear

Types of Problems

Two types of problems are possible: (i) design type and (ii) analysis type.

In the design type of problems, the designer has to determine the dimensions b, d, D, A_{st} and other detailing of reinforcement, grades of concrete and steel from the given design moment of the beam. In the analysis type of the problems, all the above data will be known and the designer has to find out the moment of resistance of the beam

Design of the singly reinforced beam for bending

Type 1 :To design a singly reinforced rectangular section for a given width and applied factored moment

$d_{bal} = \sqrt{\frac{M_u}{Q_{lim} X b}}$

1. Assume 5% larger effective depth for d ≤ 500 more &10% larger depth for 1000 mm ≥ 500 mm
2. Assume one layer of 20mm diameter for (case 1) & two layers of 20mm diameter, bars (case 2)
3. If the clear cover to main reinf is 30mm Effective cover = 30+10 (Assume 1 – Layer 20ø) = 40 mm =40+20 (Assume 2 Layers 20ø)
4. Over all depth D over all = d+40 D over all = d+60
5. Now d = D overall – 40mm (case1)
   D overall – 60 mm (case-2)
6. Determine $\frac{Mu}{bd^2}$, p_t, Ast, Ast_{lim}
7. Select the bar size and number such that Ast > Ast required & also Ast < Ast_{lim}

Type 2 :- To find the steel area for a given factored moment

We know $d_{bal} = \sqrt{\frac{M_u}{Q_{lim} X b}}$ (Assume b)

For a given factored moment, width & depth of Section,final

$mu_{lim} = Q_{lim} x b d^2$

Mu < M_{ulim}, design as under reinforced Section

Mu= M_{ulim}, design as balanced Section

Mu> M_{ulim}, design as doubly reinforced Section

Design of the singly reinforced beam for shear

Shear reinforced In beams

The shear reinforcement shall be provided by the reinforcement which cross the cracks. These shear reinforcement. minimize the size of diagonal tension crack & carry diagonal tension stress from one side of crack to the other. The provision of shear reinforcement is made by the following forms.

(i) Vertical Stirrups

(ii) Inclined Stirrups

(iii) Bent up bars along with Stirrups

Design for Shear

1. Find the Maximum S.F
2. Find the factored S.F (Vu)
3. Find the nominal shear stress $r_v = \frac{V_u}{bd}$
4. Obtain the design shear strength of concrete corresponding to percentage of tensile reinforcement provided for flexure from table 19,pg-73 IS 456.

   Design shear strength (r_c) :- It is the capacity of concrete along with tensile reinforcement. To take the amount of shear force without providing any reinforcement for shear.
5. Find the excess shear force for which shear reinforcement. is required to be provide i.e. V_{us} . V_{us} = Applied shear – shear force to be resisted by concrete without any shear reinforcement.

   $V_{us} = V_u - r_c\, bd$
6. To determine the quantity of shear reinforcement in terms of stirrup spacing as under.

(i) For vertical stirrups

(ii) $V_{us} = \dfrac{0.87 f_y \text{Æ}_{sv} \times d}{S_v}$

Vus =Exces S.F to be resisted

Asv=Area of shear reinforcement.

SV = Spacing of strirrups

$$\text{Sv} = \frac{0.87 f_y A_{sv}}{\text{d}}\ \text{V}_{\text{us}}$$

(iii) For inclined stirrups

$$V_{us} = \frac{0.87\, f_y\, \text{Æ}_{sv} \times d\ (sin\alpha + \cos\alpha\ )}{S_v}$$

Or

$$Sv\,\frac{0.87\, fy\ \text{Æ}sv\ d(Sin\alpha + Cos\alpha)}{Vus}$$

(iv) For Bent up bars

$$V_{us} = 0.87\, f_y\, A_{sv}\, .\, sin\alpha$$

IS 456 says the contribution of bent up bars towards shear resistance shall not exceed half of the shear resistance.

α =angle between bent up the inclined stirrups or bent up bar and total axis of member not less than 45^o

Minimum shear reinforcement .

Spacing of shear stirrups should not exceed the following.

$$\frac{Asv}{bSv} \geq \frac{0.4}{0.87\,fy}$$

(i) $S_v = \frac{0.87\,fy\,Æsv}{0.4\,b}$

(ii) 0.75 d (d = Effective depth)

(iii) 300 mm c/c distance

Example 1

A simply supported rectangular beam of 4 m span carries a uniformly distributed characteristic load including self-weight of 20 kN/m. The beam section is 230 mm x 450 mm overall. Design the beam for bending and shear. The materials are grade M20 concrete and HYSD reinforcement of grade Fe 415. The beam is suspended from the upper floor level.

PG 356 fig 9.5

Solution:

Pu = 1.5 x 20 = 30 kN/m

Mu,max = 30 x $\frac{4^2}{8}$ = 60 kNm and $V_{u,max}$ = 30x$\frac{4}{2}$ = 60 kN.

(a) Moment steel

The section is 230 mm x 450 mm overall. Assuming one layer of 16 mm diameter bars, effective depth shall be

d = 450 – 30(cover)-8 (centre of reinforcement) = 412 mm.

Depth required for singly reinforced section

$$=\sqrt{\frac{60x10^6}{2.76 \times 230}}$$

= 308 mm < 412 mm

∴ Design as singly reinforced section.

$$\frac{Mu}{bd^2} = \frac{60\,x\,10^6}{230\,x\,412\,x412} = 1.54$$

From SP 16 table no 2 P_t = 0.474

$$A_{st} = \frac{0.474 \times 230 \times 412}{100} = 449\text{mm}^2$$

Minimum steel required

$$A_s = \frac{0.205}{100} \times 230 \times 412 = 194\ mm^2$$

$$A_{sbal} = \frac{0.96}{100} = 230 \times 412 = 910\ mm^2$$

∴ 194 mm^2< $A_{st,\ provided}$< 910 mm^2.

Provide 4 no. 12mm # =4x113 =452 mm^2

Let 2 bar be bent at 1.25 D

= 1.25 X 450 =562.5 mm, say 600 mm, from the face of the support

Check for shear :

As support V_u =60 k N

$$\tau_v = \frac{60\ x\ 10^3}{230\ x\ 412} = 0.633\ N/mm^2.$$

$$\frac{100\ Æ_s}{bd} = \frac{2\ x\ 113\ x\ 100}{230\ x\ 412} = 0.238$$

$.r_c$ =0.332 N/mm^2 <0.633 N/mm^2.

∴ Shear design is necessary

Note that the critical section for checking the shear stress in this case is the face of the support (and not at distance d from the face of the support) because the reaction at support induces tension in end region.

As support 2 bent bras can be used to carry shear stress. These give a shear resistance of

sin 45° x 2 x 113 x 0.87 x 415 x 10^{-3} = 57.69 kN.

t_c bd = 0.332 x 230 x 412 x 10^{-3} = 31.46 kN.

$V_{us} = V_u - \tau_c$ bd = 60 – 31.46 = 28.54 kN.

Bent bars share 50% = 14.27 kN.

Stirrups provide 50% = 14.27 kN.

Using 6 mm ø two legged M.S. stirrups, spacing can be given by $\frac{0.87fy.Æsv.d}{Vus}$

Where A_{sv} = 2 x 28 = 56 mm^2

∴ $S_v = \frac{0.87 \times 250 \times 56 \times 412}{14.27\ 10^3}$ = 351 mm .....(1)

At distance of 550 mm from support, where two bars are bent

V_u = 60 –(0.55 x 30)= 43.5 kN

V_{us} = 43.5 – 31.46 = 12.04 kN

This will give larger spacing than above.

Minimum shear reinforcement .

Spacing of shear stirrups should not exceed the following.

$$\frac{Asv}{bSv} \geq \frac{0.4}{0.87\,fy}$$

(i) $S_v = \frac{0.87\,fy\,Æsv}{0.4\ b}$

(ii) 0.75 d (d = Effective depth)

(iii) 300 mm c/c distance

Using 2legged 6 mm ø mild steel stirrup $S_v = \frac{0.87\times 250\times 56}{0.4\times 232}$ = 132.4 mm

For 230 mm wide beam minimum shear reinforcement is 6 mm ø about 130 mm c/c..... (2)

From (1) and (2) minimum shear reinforcement shall be provided, i.e., 6 mm ø about 130 mm c/c.

Design of doubly reinforced beams as per IS 456/SP-16 for bending and shear

DOUBLY REINFORCEMENT BEAMS

When the applied moment is greater than M.R of a singly reinforced section, then steel reinforcement is added in compression zone to increase the M.R of the section, then this is known as doubly reinforced section

There are three alternatives

i. Increase the dimensions of the section i.e. depth
ii. Higher grades of concrete can be increased to increase the M.R of the section.
iii. Steel reinforcement. may be added in compression zone to increase the M.R of the section.

Design of doubly reinforced beams

Type -1: To find out reinforcement for flexure for a given section & factored moment.

(i) Find out M_{ulim} & reinforcement Ast_{lim} for a given section by using the equation

$M_{ulim} = Q_{lim} \times bd^2$

$= 0.36 f_{ck}\ b.x_{umax}\ (d - 0.42x_{umax})$

& $A_{stlim} = \frac{M_{ulim}}{0.81\, f_y\ (d - 0.42 xumax)}$

087xA.stx fy (d-0.42 Xumax)

(ii) Obtain moment $M_{u2} = M_u - M_{ulim}$

(iii) Find out compression steel from equation $M_{u2} = Asc\ (f_{sc} . f_c)(d - d')$

Neglecting fcc, $A_{sc} = \frac{Mu2}{f_{sc}(d - d')}$

(iv) Corresponding Ast Tensile stepA_{st2} can be found out from

$$A_{st2} = \frac{Æ_{sc} f_{sc}}{0.87\, f_y}$$

(v) $A_{st} = A_{stlim} + A_{st2}$

(vi)Provide the reinforcement

Example 2

Design a simply supported rectangular beam of size 230 mm x 600 mm overall for a super-imposed load of 46 kN/m. Span of the beam is 5 m. The materials are M 20 grade concrete and HYSD reinforcement of grade Fe 415.

Solution:

Self weight of beam =0.3 x 0.60 x 25 = 3.45 kN/m

Super-imposed load = $\frac{46.00\ kN/m}{Total\ 49.45 \frac{kN}{m},}\ say\ 50\ kN/m$

Factor load = 1.5 x 50 = 75 kN/m

$M_u = \frac{5^2}{8} \times 75 = 243.4$ kNm

$V_u = \frac{5}{2} \times 75 = 187.5$ kN

Moment steel:

Assuming 2 layers of 20 mm diameter bars

D = 600 – 30 – 20 -10 = 540 mm

$\frac{Mu}{bd2} = \frac{243.4.x\ 10^6}{230\ x\ 540^2} = 3.63 > 2.76$

∴ Doubly reinforced section

$M_{u,lim} = 2.76 \times 230 \times 540^2 \times 10^{-6} = 185.10$ kNm

M_{u2} = 243.4 – 185.10 = 58.3 kNm.

Let the compression reinforcement be provided at an effective cover of 40 mm

$\frac{d'}{d} = \frac{40}{545} = 0.07$, consider $\frac{d'}{d} = 0.1$

Stress in compression steel, fsc = 353 N/mm^2 (refer to table f of S.P-16)

$$A_{sc} = \frac{Mu2}{f_{sc}(d-d')} = \frac{58.3 \times 10^6}{353(540-40)} = 330 \text{ mm}^2$$

Corresponding tension steel

$$A_{st2} = \frac{Æ_{sc} f_{sc}}{0.87 f_y} = \frac{330 \times 353}{0.87 \times 415} = 323 \text{ mm}^2$$

$$A_{st1} = A_{st,lim} = P_{t,lim} \times \frac{bd}{100}$$

$$= 0.96 \times \frac{230 \times 540}{100} = 1192 \text{ mm}^2$$

Ast = Ast1 + Ast2 = 1192 + 323 mm^2 = 1515 mm^2

Asc = 2-16 # = 402 mm^2

Ast = 5-20# = 1570 mm^2 (all straight). Provide 3 bars in first layers and 2 bars in second layer.

For designed section, equating total compression and total tension

0.36 f_{ck} bx_u + Asc fsc = 0.87 fyAst

0.36 x 20 x 230 x_u + 402 x 353 = 0.87 x 415 x 1570

x_u = 256.6 mm

$x_{u,max}$ = 0.48d = 0.48 x 540 = 259.2 mm

$x_u < x_{u,max}$

Hence the beam is under reinforced section.

(b) Check for shear:

$$r_v = \frac{187.5 \times 10^3}{230 \times 540} = 1.51 \text{ N/mm}^2 < 2.8 \text{ N/mm}^2$$

$$\frac{100 Æ_s}{bd} = \frac{100 \times 5 \times 314}{230 \times 540} = 1.26$$

$$r_c = 0.672 \text{ N/mm}^2$$

As the ends are confined with compressive reaction, shear at distance d will be used for checking shear at support. At 540 mm, shear is equal to

Using 8 mm # two legged stirrups, spacing can be given by $\frac{0.87 f_y . Æ_{sv} . d}{V_{us}}$

Where $A_{sv} = 2 \times 50 = 100 \text{ mm}^2$

$\therefore S_v = \frac{0.87\times 415 \times 100 \times 540}{63.5\times 10^3} = 307$ mm ..... (1)

At distance of 550 mm from support, where two bars are bent

$V_u = 187.5 - (0.540 \times 75) = 147$ kN

$V_{us} = 147 - 0.672 \times 230 \times 540 \times 10^{-3} = 63.5$ kN

This will give larger spacing than above.

Minimum shear reinforcement .

Spacing of shear stirrups should not exceed the following.

$$\frac{Asv}{bSv} \geq \frac{0.4}{0.87\,fy}$$

(i) $S_v = \frac{0.87\,fy\,Æsv}{0.4\ b}$

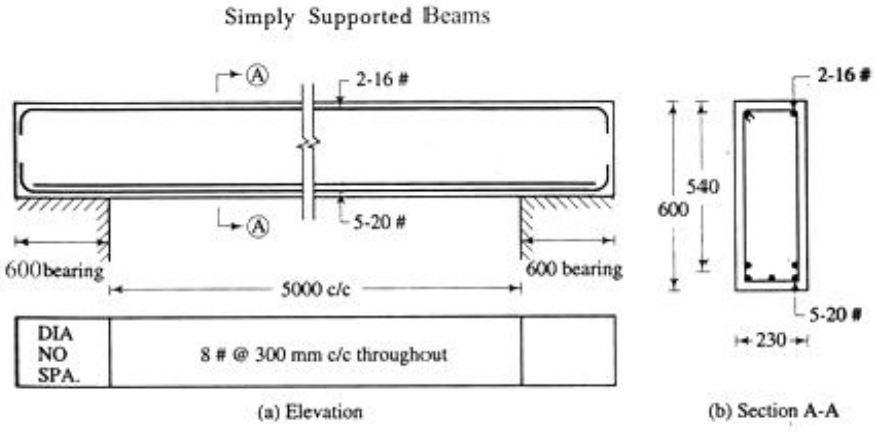
(ii) 0.75 d (d = Effective depth)= 0.75 x 540 =405 mm

(iii) 300 mm c/c distance

Using 2legged 8 mm # stirrup $S_v = \frac{0.87\times 415\times 100}{0.4\times 230} = 392.4$ mm

For 230 mm wide beam minimum shear reinforcement is 8 mm # about 300 mm c/c ..... (2)

Hence provide mm # about 300 mm c/c through out the beam.



7.3:Design of T beams as per IS 456 for bending and shear

DESIGN OF T BEAM

Case-1:To design the beam for a given loading condition

(i) Fixe the width of the beam using architectural consideration. Also the width shall be sufficient to accommodate the reinforcement satisfying the requirement of spacing of bars

(ii) Effective width of flange for T beam $= b_f = \frac{l_0}{6} + bw + 6Df$

(iii) Assume overall depth D equal to 1/12 th to 1/10 of the span and subtracting effective concrete cover from overall depth, find out the effective depth d

(iv) Assume lever arm Z=d-D_f/2

(v) Find out the reinforcement $A_{st} = \frac{Mu}{0.87 \times f_y \times Z}$

(vi) Provide the reinforcement as per requirement.

(vii) Then find out $\frac{b_f}{b_w}$ and $\frac{D_f}{d}$ then from table 58 of S.P16 find out the value of M_{ulim}.T .And check whether under reinforced or over reinforcement assuming the position of neutral axis

If the moment of resistance is less than the applied moment,revise the section.

When $D_f \leq \frac{3}{7} x_u$

$Mu = 0.36 f_{ck} b.x_u (d - 0.42x_u) + 0.446 f_{ck} (b_f - b_w) \times D_f (d - \frac{D_f}{2})$

When $D_f \geq \frac{3}{7} x_u$

$Mu = 0.36 f_{ck} b.x_u (d - 0.42x_u) + 0.446 f_{ck} (b_f - b_w) \times D_f (d - \frac{Y_f}{2})$

Where $Y_f = 0.15\, x_u + 0.65 D_f$

Design for Shear

7. Find the Maximum S.F
8. Find the factored S.F (Vu)
9. Find the nominal shear stress $r_v = \frac{V_u}{bd}$
10. Obtain the design shear strength of concrete corresponding to percentage of tensile reinforcement provided for flexure from table 19,pg-73 IS 456.

    Design shear strength (r_c) :- It is the capacity of concrete along with tensile reinforcement. To take the amount of shear force without providing any reinforcement for shear.
11. Find the excess shear force for which shear reinforcement. is required to be provide i.e. V_{us} . V_{us} = Applied shear – shear force to be resisted by concrete without any shear reinforcement.

    $V_{us} = V_u - r_c\, bd$
12. To determine the quantity of shear reinforcement in terms of stirrup spacing as under.

(v) For vertical stirrups

(vi) $V_{us} = \frac{0.87 f_y Æ_{sv} \times d}{S_v}$

Vus =Exces S.F to be resisted

Asv=Area of shear reinforcement.

SV = Spacing of strirrups

$$Sv = \frac{0.87 f_y A_{sv}}{d} V_{us}$$

(vii) For inclined stirrups

$$V_{us} = \frac{0.87\, f_y Æ_{sv} \times d\, (sin\alpha + cos\,\alpha)}{S_v}$$

Or

$$Sv \frac{0.87\, fy\, Æsv\, d(Sin\alpha + Cos\alpha)}{Vus}$$

(viii) For Bent up bars

$$V_{us} = 0.87\, f_y\, A_{sv}\, .sin\alpha$$

IS 456 says the contribution of bent up bars towards shear resistance shall not exceed half of the shear resistance.

α =angle between bent up the inclined stirrups or bent up bar and total axis of member not less than 45^o

e Minimum shear reinforcement .

Spacing of shear stirrups should not exceed the following.

$$\frac{Asv}{bSv} \geq \frac{0.4}{0.87\, fy}$$

(i) $S_v = \frac{0.87\, fy\, Æsv}{0.4\ b}$

(ii) 0.75 d (d = Effective depth)

(iii) 300 mm c/c distance

Design a simply supported tee beam of span 7 m and spaced at 3 m centres. The thickness of slab is 100 mm and total characteristic load including self-weight of the beam is 30 kN/m. The overall size of the beam is 230 mm × 600 mm. The materials are M 20 grade concrete and HYSD reinforcement of grade Fe 415.

Solution:

$$\text{Factored load} = 1.5 \times 30 = 45 \text{ kN/m}$$

$$\text{Factored moment } M_u = 45 \times \frac{7^2}{8} = 275.6 \text{ kNm}$$

$$\text{Factored shear } V_u = 45 \times \frac{7}{2} = 157.5 \text{ kN.}$$

(a) *Moment steel:*

Assuming 2 layers of 20 mm # bars

$$d = 600 - 30 - 20 - 10 = 540 \text{ mm.}$$

As a preliminary design assume lever arm

$$z = d - \frac{D_f}{2} = 540 - \frac{100}{2} = 490 \text{ mm.}$$

$$A_{st} = \frac{M_u}{0.87 f_y z} = \frac{275.6 \times 10^6}{0.87 \times 415 \times 490} = 1558 \text{ mm}^2.$$

Provide 5-20 mm # $= 5 \times 314 = 1570 \text{ mm}^2$.

The section is now checked for moment of resistance.

$$b_f = \frac{l_0}{6} + b_w + 6 D_f \not> 3000$$

$$= \frac{7000}{6} + 230 + 6 \times 100 = 1996 \text{ mm.}$$

Use $b_f = 1990$ mm

$$F_{tc} = 0.36 f_{ck} b_f D_f$$

$$= 0.36 \times 20 \times 1990 \times 100 \times 10^{-3} = 1432.8 \text{ kN}$$

$$F_{ts} = 0.87 f_y A_{st}$$

$$= 0.87 \times 415 \times 1570 \times 10^{-3} = 566.8 \text{ kN}$$

$$F_{tc} > F_{ts}$$

$\therefore$ Neutral axis lies in flange.

Equating the forces

Total compression = total tension

$$0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$$

$$036 \times 20 \times 1990 x_u = 0.87 \times 415 \times 1570$$

$$\therefore \quad x_u = 39.56 \text{ mm}$$

$$x_{u,max} = 0.48 d = 0.48 \times 540 = 259.2 \text{ mm}$$

$$x_u < x_{u,max}.$$

$\therefore$ Under-reinforced section.

$$M_u = 0.87\ f_y\ A_{st}\ (d - 0.42\ x_u)$$
$$= 0.87 \times 415 \times 1570 \times (540 - 0.42 \times 39.56) \times 10^{-6}$$
$$= 296.7 \text{ kNm} > 275.6 \text{ kNm} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(O.K.}$$

Let 2 bars bent up at $1.25 \times 600 = 750$ mm from the face of the support.

(b) *Check for devel*

At support, $A_{st} = 3 \times 314 = 942$ mm^2.

$$0.36\ f_{ck}\ b_f\ x_u = 0.87\ f_y\ A_{st}$$
$$\therefore\ 0.36 \times 20 \times 1990\ x_u = 0.87 \times 415 \times 942$$
$$x_u = 23.74 \text{ mm.}$$
$$M_{u1} = 0.87\ f_y\ A_{st}\ (d - 0.42\ x_u)$$
$$= 0.87 \times 415 \times 942 \times (540 - 0.42 \times 23.74) \times 10^{-6}$$
$$= 180.2 \text{ kNm}$$
$$V_u = 157.5 \text{ kN.}$$

As the ends of reinforcement are confined with compressive reaction, we have

$$1.3\ \frac{M_{u1}}{V_u} + L_0 \geq L_d.$$

Assume $L_0 = 12\ \#$

$$\therefore\ 1.3 \times \frac{180.2 \times 10^6}{157.5 \times 10^3} + 12\ \# \geq 47\ \#$$
$$\therefore\ 42.5 \geq\ \#$$
$$\#_{\text{provided}} = 20 \text{ mm} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(Safe)}$$

(c) *Check for shear:*

As the ends are confined by compressive reaction, shear at distance d will be used at the support.

$$\therefore\ V_u = 157.5 - 0.540 \times 45 = 133.2 \text{ kN}$$
$$\frac{100\ A_s}{b\ d} = \frac{100 \times 942}{230 \times 540} = 0.758$$
$$\tau_c = 0.562 \text{ N/mm}^2$$
$$\tau_v = \frac{133.2 \times 10^3}{230 \times 540} = 1.06 \text{ N/mm}^2 > \tau_c$$

$\therefore$ Shear design is necessary.

2-20 # bars can resist a shear of

$0.87 \times 415 \times 2 \times 314 \times \sin 45^\circ \times 10^{-3} = 160.32$ kN.

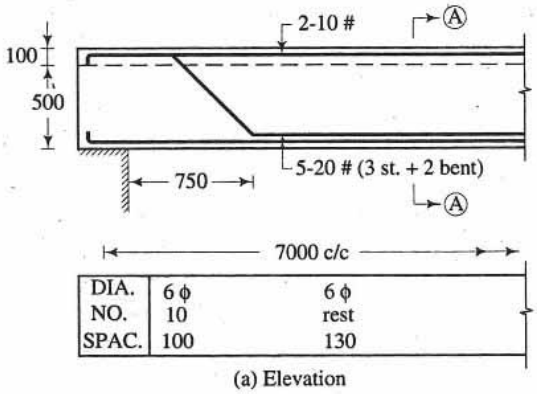
$$V_{us} = V_u - \tau_c\ b\ d$$
$$= 133.2 - 0.562 \times 230 \times 540 \times 10^{-3} = 63.40 \text{ kN.}$$

Bent bars share 50 % = 31.7 kN

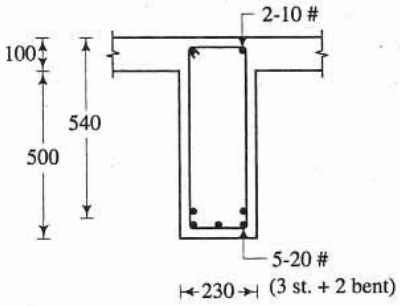
Stirrups share 50 % = 31.7 kN.

Using 6 mm ϕ M.S. two-legged stirrups, $A_{sv} = 56$ mm^2

Use 2-10 # as anchor bars. The designed section is shown in fig.



(a) Elevation



(b) Section A-A

CHAPTER O8

INTRODUCTION TO SLABS BY LIMIT STATE METHOD

SLABS

: One way slab & two way slab

: One way & cantilever slabs as per IS-456 for bending & shear.

8.3: Explain the action of two way slabs with defected shape.

: Provision for two way slab as per IS-

456-20008.4.1: Middle strip & edge strip

: B.M Co-Efficient

: Torsion reinforcement

: Design of two way slab as per IS-456-2000 & SF-16 hand hook

: Check for **deflection**, development length & reinforcement. Requirement & spacing as per Sp-16 & IS-456

Slabs: Slabs are plate elements & carry loads primarily by flexure. They usually carry Vertical loads

Classification of Slab:

1. One way spanning slab
2. Two-way spanning slab
3. Flat slab
4. Grid slab
5. Circular and other shapes
6. Ribbed slab

One way spanning slab: The slab supported on two opposite supports is a one way spanning slab.

On the other way a slab which transfer is load on one of the set of two opposite edge supports is called one-way slab. In this case ly/lx is greater than two.

Two way spanning Slab: The Slab which is supported on all four edges and if ly $\leq$ 2lx slabs bend in both directions. Such slabs are called two way spanning slab.

$$\frac{ly}{lx} < 2$$

One way & cantilever slabs as per IS: 456 for bending & shear.One way spanning slab

1. Effective Span = Clear Span + Effective depth

or

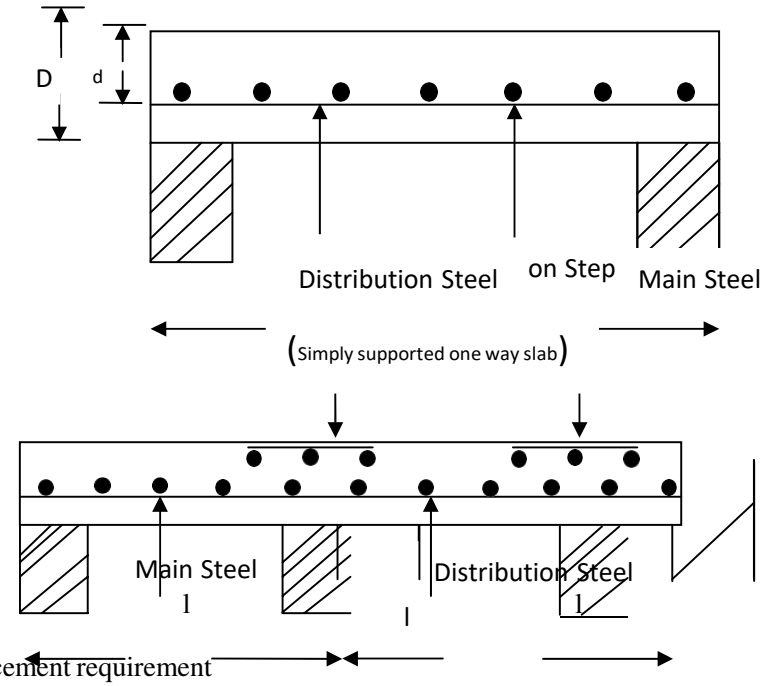
Centre to centre of Support whichever is less

Moment Steel: The reinforcement In the direct of span is known as moment steel. The moment steel is known as main reinforcement.

Distribution Steel: The reinforcement perpendicular to the main reinforcement. Is known as distribution steel & placed in second layer. This reinforcement resists temp & shrinkage stresses, keeps the main reinforcement in position and distributes the concentrated or non uniform loads throughout the slab.

For a continuous slab at support, top reinforcement is provided as main steel to resist negative B.M.

Spacing of slab bar = $\frac{area\ of\ one\ bar\ x\ 1000}{required\ area\ in\ mm^2\ per\ meter}$



2. Reinforcement requirement

(i) Minimum reinforcement:- The reinforcement in either direction in slabs shall not be less than 0.15% of the total c/s area. For HYSD bars, it shall not be less than 0.12%. thus in slabs, minimum reinforcement less than 0.85/fy is permissible .

(ii) Maximum diameter: The diameter of reinforcing bar shall not exceed 1/8th the total thickness of slab.

(iii) Minimum diameter: For main bars, minimum diameter shall be 10mm for plain bars & 8 mm for deformed bar for distribution bars, the minimum diameter shall be 6mm

Shear Stress:

Design shear strength = K $r_c c_{max}$

Nominal shear stress 0.5 $\tau_{c\ max}$

This shall be checked

Deflection: It shall be checked as per beam

Cracking:

(i) The horizontal distance between parallel main reinforcement. shall not be more than 3 times the effective depth of a solid slab or 300mm whichever is smaller.

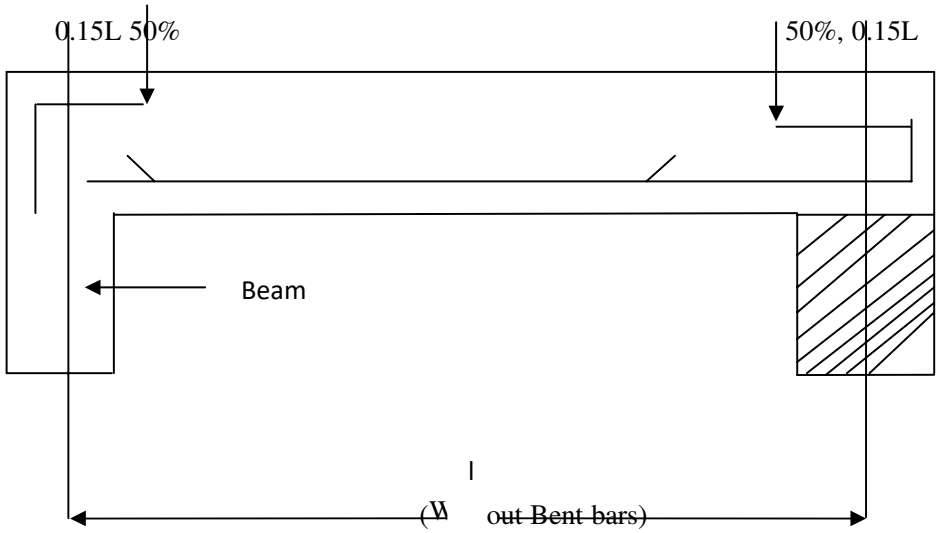
(ii) The horizontal distance between parallel reinforcement bars against shrinkage & temperature shall not be more than 5 times the effective depth of a solid slab or 450 mm whichever is smaller.

Cover: For mild exposure, clear over = 20mm

When ø ~~12~~ mm clear cover = 15mm

Ø> 12mm, clear cover = 20mm

Development Length:



(Typical details of simply supports slab)

The bars can be bent up or curtailed only if continuing bars provide minimum reinforcement. & check for development length is satisfied.

Example

A simply supported one-way slab of clear span 3.0 m is supported on masonry walls of thickness 350 mm. Slab is used for residential loads. Design the slab. The materials are grade M 20 concrete and HYSD reinforcement of grade Fe 415. Live load shall be 2 kN/m^2.

Solution:

Depth of slab: The first trial of depth of slab can be arrived at by considering deflection criterion. Assuming percentage of steel reinforcement, find out modification factor as explained in art. 8-1. Percentage of steel depends on the loading on slab. A designer, after some practice will be able to find out his own thumb rules for the trial depth.

Assume 0.30 per cent steel as a first trial with service stress of $0.58\ f_y = 0.58 \times 415 = 240$ N/mm^2. Basic $\frac{\text{span}}{d}$ ratio = 20. Also modification factor from fig. 8-1 is 1.45. Then permissible $\frac{\text{span}}{d}$ ratio $= 20 \times 1.45 = 29$. The depth $d = \frac{3100}{29} = 106.9$ mm. Considering mild exposure and maximum diameter of reinforcement be 12 mm, clear cover = 15 mm. Therefore $D = 106.9 + 6 + 15 = 127.9$ mm.

Assume 130 mm overall depth of slab.

$DL = 0.13 \times 25 = 3.25$ kN/m^2

Floor finish = 1.00 kN/m^2

Live load = 2.00 kN/m^2

Total 6.25 kN/m^2

Factored load $= 1.5 \times 6.25 = 9.4$ kN/mm^2.

Effective span (1) 3000 + 350 = 3350 mm c/c supports

(2) 3000 + 110 (effective depth) = 3110 mm.

Use 3.11 m effective span.

Moment and shear:

Consider 1 m length of slab

$\therefore$ load = 9.4 kN/m.

Maximum moment $= 9.4 \times \frac{3.11^2}{8} = 11.36$ kNm.

Maximum shear $= 9.4 \times \frac{3}{2} = 14.1$ kN (based on clear span).

Effective depth required for flexure L_x

$$= \sqrt{\frac{11.36 \times 10^6}{1000 \times 2.76}} = 64.15 \text{ mm.}$$

($Q = 2.76$ for M 20 mix and Fe 415 steel)

$d_{provided}$ = 130 − 15 (cover) − 5 (assume 10 ϕ bar)

= 110 mm ..... (O.K)

Design for flexure:

$$\frac{M_u}{b d^2} = \frac{11.36 \times 10^6}{1000 \times 110^2} = 0.939$$

$$p_t = \frac{100 A_{st}}{b d} = 0.281$$

$$\therefore \quad A_{st} = \frac{0.281 \times 1000 \times 110}{100} = 309 \text{ mm}^2.$$

Provide 8 mm # about 150 mm c/c = 335 mm^2.

Note that use of design tables give correct answer for steel required.

Half the bars are bent at $0.1\,l = 0.1 \times 3100 = 310$ mm.

Remaining bars provide 167.5 mm^2 area.

$$\frac{100 A_s}{b D} = \frac{100 \times 167.5}{1000 \times 130} = 0.129 > 0.12$$

i.e., remaining bars provide minimum steel. Thus half the bars may be bent up

Distribution steel = $\frac{0.15}{100} \times 1000 \times 130 = 195$ mm^2, using mild steel.

Maximum spacing 5 × 100 = 500 or 450 mm, i.e., 450 mm.

Provide 6 mm ϕ about 140 mm c/c = 202 mm^2.

Check for shear:

For bars at support

correct d = 130 − 15 − 4

= 111 mm

$$\frac{100 A_s}{b D} = \frac{100 \times 167.5}{1000 \times 111} = 0.15.$$

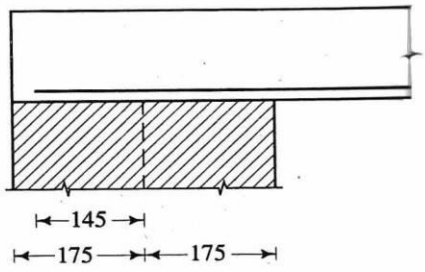
For slab upto 150 mm thickness, $k = 1.3$

τ_c from table 7-1 = 0.28 N/mm^2.

Design shear strength

$= k\,\tau_c = 1.3 \times 0.28$

= 0.364 N/mm^2.



Details at support for bottom bars
FIG. 10-5

Actual shear stress = $\frac{13.5 \times 10^3}{1000 \times 111} = 0.122$ N/mm^2 < 0.364 N/mm^2 ..... (safe)

Check for development length:

Refer to fig. 10-5.

L_0 = 145 mm. L_0 is limited to 12 ϕ = 96 mm or
d = 100 mm, whichever is greater.

L_0 = 100 mm.

For continuing bars, A_s = 167.5 mm^2. Also, the ends of the reinforcement are confined by compressive reaction.

$$M_{u1} = 0.87\ f_y\ A_{st}\ d \left(1 - \frac{f_y\ A_{st}}{b\ d\ f_{ck}}\right)$$

$$M_{u1} = 0.87 \times 415 \times 167.5 \times 111 \left(1 - \frac{415 \times 167.5}{1000 \times 111 \times 20}\right) \times 10^{-6}$$

$$= 6.5 \text{ kNm.}$$

$$V_u = 13.5 \text{ kN.}$$

$$1.3\ \frac{M_{u1}}{V_u} + L_0 \geq L_d \quad \text{where} \quad L_d = 47\ \#$$

$$1.3 \times \frac{6.5 \times 10^6}{13.5 \times 10^3} + 100 \geq 47\ \#$$

or 15.4 mm $\geq$ # ..... (O.K.)

Check for deflection :

Basic $\frac{\text{span}}{d}$ ratio = 20

$$p_t = \frac{100 \times 335}{1000 \times 111} = 0.3$$

service stress = $0.58 \times 415 \times \frac{303}{335}$ = 218 N/mm^2.

modification factor = 1.62

permissible $\frac{\text{span}}{d}$ ratio = $20 \times 1.62 = 32.4$

actual $\frac{\text{span}}{d}$ ratio = $\frac{3100}{111}$

= 28 < 32.4 ..... (O.K.)

Note: The depth could be slightly reduced. Try with D = 125 mm from beginning and rework the problem

Check *for cracking:*

Maximum spacing permitted for main reinforcement

= 3 × 100 = 300 mm.

Actual spacing = 150 mm < 300 mm ..... (O.K.)

For distribution steel, maximum spacing permitted

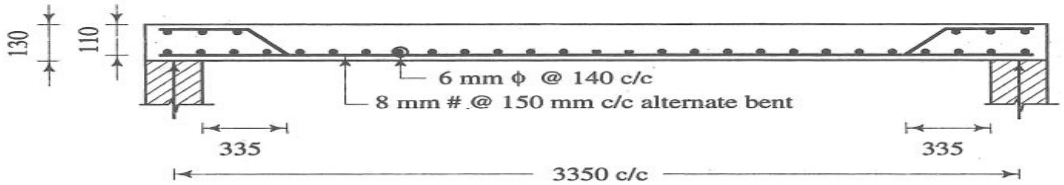
= 5 × 100

= 500 or 450 mm, i.e., 450 mm.

Spacing provided = 150 m ..... (O.K.)

For tying the bent bars at top, 6 mm ϕ about 150 mm c/c distribution steel shall be provided.

Sketch: The cross-section of the slab



Design the slab S2 – S1 of above figure , if it is to be used for residential purpose at the free end of slab S1 there is a concrete parapet of 75mm thick and 1 m high. The materials are M20 grade concrete and HYSD reinforcement of grade Fe 415 . Use IS:875 for live loads.

Solution

Estimation of loads:

For slab S2 live load shall be 2 kN/m^2. For slab s1 which is a balcony slab live load shall be 3 kN/m^2. Assume 120 mm thick slab.

Slab S_2 : Self load =0.12X25 = 3 + 0kN/m^2

Floor finish = 1 + 0 kN/m^2

Live load = 0 + 2 kN/m^2

Total = 4 + 2 kN/m^2

P_u = 1.5(4+2)

= (6+3) kN/m^2

$Slab_1$: Self load = 3 + 0kN/m^2

Floor finish = 1 + 0 kN/m^2

Live load = 0 + 3 kN/m^2

Total = 4 + 3 kN/m^2

P_u = 1.5(4+3)

= 6+4.5 kN/m^2

Weight of parapet

$$0.075 \times 25 \times 1 = 1.875 \text{ kN/m.}$$

$$p_u = 1.5 \times 1.875 = 2.8 \text{ kN/m.}$$

Analysis :

Consider 1m long strip.

1 To get maximum positive moment in slab s2 only dead load on slab s1 and total load on slab s2 shall be considered. The parapet load is a dead load but will not be considered as sometimes the owner of the building or architect may change his mind and would provide simply a railing.

Considering above figure

Cantilever moment $= \frac{1.2^2}{2} \times 6 = 4.32$ kNm.

Reaction at A $= \frac{9\times3}{2} - \frac{4.32}{3} = 12.06$ kN

Point of zero shear from A $= \frac{12.06}{9} = 1.34$ m.

Maximum positive moment = $12.06 \times 1.34 - \frac{1.34^2}{2} \times 9$

=16.16 – 8.08 kNm.

To check shear and devlopement length at A, shear may be considered as 12.06 kN. Note that for the cantilever, clear span is considered

2 To get maximum negative moment and maximum shear at B , the slab is loaded with full loads

Maximum negative moment $= \frac{1.2^2}{2} \times 10.5 + 1.2 \times 2.8$

= 7.56 + 3.36=10.92kNm.

Maximum shear at B, $V_{u,BA} = \frac{9\times3}{2} + \frac{10.92}{3} = 13.5 + 3.64 = 17.14\ kN$

$V_{u,BC} = 10.15 \times 1.2 + 2.8 = 15.4\ kN$.

C) Moment steel:

Maximum moment = 10.92 kN.

$d_{required} = \sqrt{\frac{10.92\times10^6}{1000\times2.76}} = 62.9\ mm$

$d_{provided}$ = 120 – 15 – 5 (assume 10# bar)

= 100 mm..... (O.K)

$\frac{M_u}{bd^2}\ (+) = \frac{8.08\times10^6}{1000\times100\times100} = 0.81$

Pt =0.236

$$\text{Ast } (+) = \frac{0.236\times1000\times100}{100} = 2.36\ mm^2$$

$$\frac{M_u}{bd^2}\ (-) = \frac{10.92 \times 10^6}{1000\times100\times100} = 1.09$$

Pt =0.324

$$\text{Ast } (-) = \frac{0.324\times1000\times100}{100} = 324\ mm^2$$

For positive moment provide 8 mm # about 170 mm c/c giving 294 mm^2 alternate bent up and for negative moment provide 8 mm # about 340 mm c/c (bent bar extended) + 10 mm # 340 mm c/c giving 378 mm^2 area . The arrangement of reinforcement is shown in the below figure.

Note that at simple support, the bars are bent at o.1 L whereas at continuity of slab it is bent at 0.2 L.

Minimum steel $= \frac{0.12}{100} \times 1000 \times 120 = 144\ mm^2$.

Remaining positive moment bars give As = $\frac{294}{2} = 147$ mm^2 Thus bar can be bent up.

Distribution steel $= \frac{0.15}{100} \times 1000 \times 120 = 180\ mm^2$.

Provide 6 mm Ø about 150 mm c/c =187 mm^2 .

For negative moment reinforcement

Ld = 47 #

Ld = $47 \times \frac{(8+10)}{2} = 423$ mm.

The bars must be anchored upto 423 mm. also they should be extended upto 12 # beyond the poin of contraflexure, which may be found out . Alternatively as a thumb rule, a bar shall be given an anchorage equal to the length of the cantilever. Adopting this, carry the top bars upto 1200 mm in the internal span. This is shown in above figure.

d) Check for Development length :

At A, Mul =$0.87 f_y A_{st}\ (d - \frac{f_y Æ_{st}}{bf_{ck}})$

$$= 0.87 \times 415 \times 147\ (100 - \frac{415 \times 417}{1000 \times 20}) \times 10^{-6}$$

= 5.15 kNm.

Vu = 12.06 kN.

Consider L0 = 8 #

Then $1.3\frac{M_{ul}}{V_u} + L_o \geq L_d$

$$1.3 \times \frac{5.15 \times 10^6}{12.06 \times 10^3} + 8 \ \# \ \geq 47 \ \#$$

At B, M_{ul} = 5.15 kNm.

Near point of contraflexure, i.e. 0.15 L from B

Vu = 17.14 – 0.45 x 9 = 13. 09 kN.

$1.3x(5.15x10^6/13.09x10^3)+8\#\geq 47\#$

#≤13.11 mm

Check for Shear

Span AB:At A,$V_{u,AB}$=12.06 KN

At B,shear at point of contraflexure=13.09 kN

Hence $V_{u=13.09}$ KN

Shear stress τ_v=13.09x10^3/1000x100=0.131N/mm^2

$\frac{100 Æ_s}{bd}$ = *100x147/1000x100= 0.147*

τc=0.28

kx τc=0.28x1.3=0.364N/mm^2>τv

Span BC

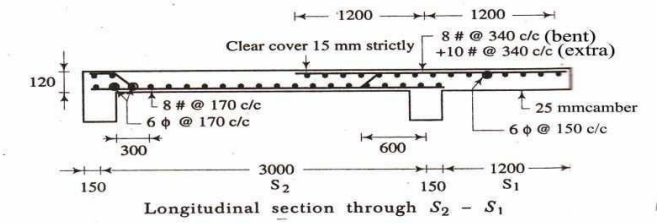
:,$V_{u,}$ =17.14 KN

Shear stress τ_v=17.14 x10^3/1000x100=0.171N/mm^2

$\frac{100 Æ_s}{bd}$ = *100x378/1000x100= 0.378*

τc=0.28

kx τc=0.421x1.3=0.547N/mm^2>τv



Longitudinal section through S_2 – S_1

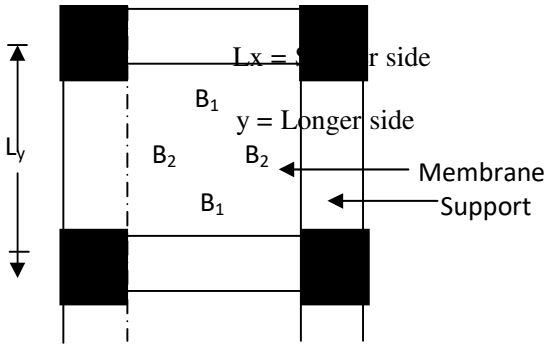
Two way Slabs:

The two way spanning slab

Occurs when the slab is supported

On all four edges

When $\frac{ly}{lx}$ <2, it is called two way slab.



Design

Mx = ∂x.w.Lx^2

My = ∂y.w.ly^2

Mx, My = Moments on strips of unit width

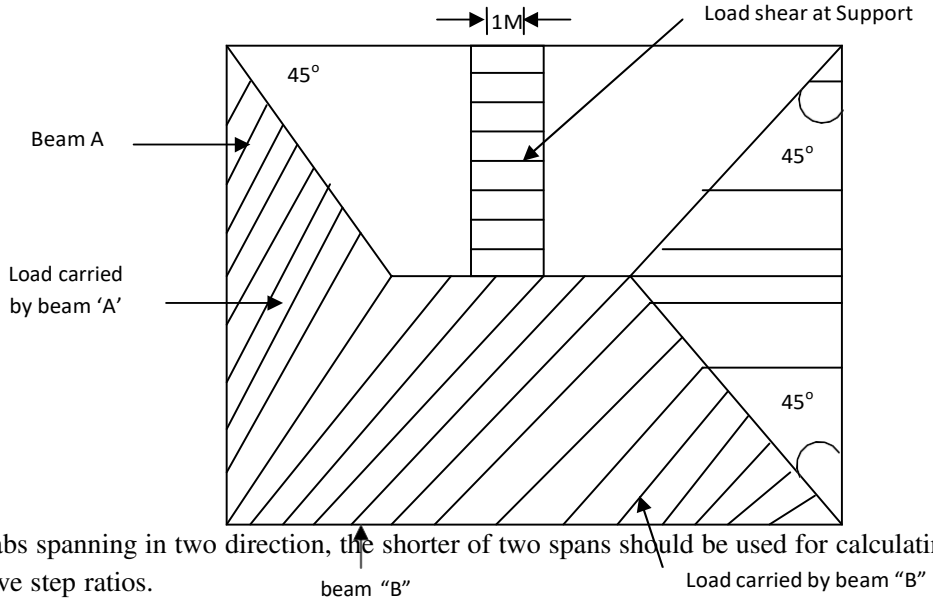
spanning Lx & Ly respectively.

= ∂x, dy = Co- efficient

= Lx, Ly = Lengths of shorter span & Longer span

Respectively

W = Total design load per unit area.IS -456 also states that at least 50% of the tension reinforcement. Provided at mid span intend to within 0.1 Lx or 0.1 Ly of the support.



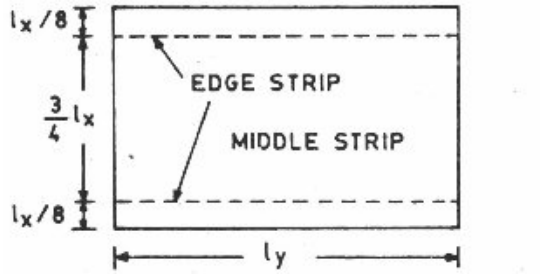
For slabs spanning in two direction, the shorter of two spans should be used for calculating the span to effective step ratios.

For two way slabs for span up to 3.5 with mild steel reinforcement. $\frac{span}{d}$ = 35 for simply supported slabs

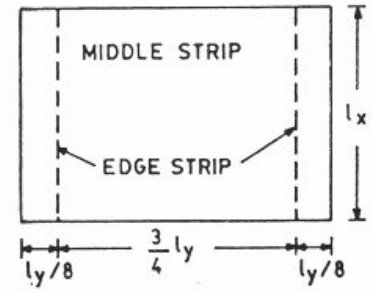
$\frac{span}{d}$ = 40 for continuous slab for HYSD of grade Fe 415, there values are multiplied by 0.8.

Provison two way slab as per IS 456:2000

Middle Strip and edge strip



Middle and edge strips along the long span



Middle and edge strips along the short span

Bending moment Coefficients with torsion reinforcement

Table 8.6 Maximum bending moments of Problem 8.2 For	Short span			Long span
	a_x	M_x (kNm/m)	α_y	M_y (kNm/m)
Negative moment at continuous edge	0.075	18.6	0.047	11.66
Positive moment at mid-span	0.056	13.89	0.035	8.68

Table 26 Bending Moment Coefficients for Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners

(*Clauses* D-1.1 *and* 24.4.1)

Case No.	Type of Panel and Moments Considered	Short Span Coefficients α_x (Values of l_y/l_x)								Long Span Coefficients α_y for All Values of
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	l_y/l_x
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	*Interior Panels:*									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2	*One Short Edge Continuous:*									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3	*One Long Edge Discontinuous:*									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4	*Two Adjacent Edges Discontinuous:*									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5	*Two Short Edges Discontinuous:*									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	—
	Positive moment at mid-span	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6	*Two Long Edges Discontinuous:*									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.045
	Positive moment at mid-span	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7	*Three Edges Discontinuous (One Long Edge Continuous):*									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	—
	Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8	*Three Edges Discontinuous (One Short Edge Continuous):*									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.057
	Positive moment at mid-span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9	*Four Edges Discontinuous:*									
	Positive moment at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

moments per unit width are given by the following equation:

$$M_x = \alpha_x \, w \, l_x^2$$

$$M_y = \alpha_y \, w \, l_x^2$$

where

M_x, M_y, w, l_x, l_y are same as those in **D-1.1**, and α_x and α_y are moment coefficients given in Table 27

D-2.1.1 At least 50 percent of the tension reinforcement provided at mid-span should extend to the supports. The remaining 50 percent should extend to within 0.1 l_x or 0.1 l_y of the support, as appropriate.

Table 27 Bending Moment Coefficients for Slabs Spanning in Two Directions at Right Angles, Simply Supported on Four Sides

(*Clause* D-2.1)

l_y/l_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
α_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
α_y	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

Problem 1:

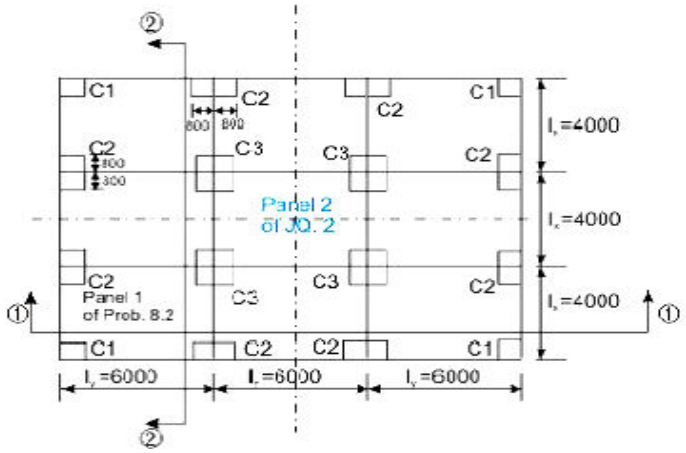


Fig. 8.19.7: Problem 8.2 (panel 1) and TQ 2 (panel 2)

Step 1: Selection of preliminary depth of slab

The span to depth ratio with Fe 415 is taken from cl. 24.1, Note 2 of IS 456 as 0.8 (35 + 40) / 2 = 30. This gives the minimum effective depth d = 4000/30 = 133.33 mm, say 135 mm. The total depth D is thus 160 mm.

Step 2: Design loads, bending moments and shear forces

Dead load of slab (1 m width) = 0.16(25) = 4.0 kN/m^2

Dead load of floor finish (given) = 1.0 kN/m^2

Factored dead load = 1.5(5) = 7.5 kN/m^2

Factored live load (given) = 8.0 kN/m^2

Total factored load = 15.5 kN/m^2

The coefficients of bending moments and the bending moments M_x and M_y per unit width (positive and negative) are determined as per cl. D-1.1 and Table 26 of IS 456 for the case 4, "Two adjacent edges discontinuous" and presented in Table 8.6. The l_y / l_x for this problem is 6/4 = 1.5.

Table 8.6 Maximum bending moments of Problem 8.2

For	Short span			Long span
	$x\alpha$	M_x (kNm/m)	$y\alpha$	M_y (kNm/m)
Negative moment at continuous edge	0.075	18.6	0.047	11.66
Positive moment at mid-span	0.056	13.89	0.035	8.68

Maximum shear force in either direction is determined from Eq.8.1 (Fig.8.19.1) as

$V_u = w(l_x/2)$ = 15.5 (4/2) = 31 kN/m

Step 3: Determination/checking of the effective depth and total depth of slab

Using the higher value of the maximum bending moments in x and y directions from Table 8.6, we get from Eq.3.25 of Lesson 5 (sec. 3.5.5):

$M_{u,lim} = Q_{lim} bd^2$

or $d = [(18.6)(10^6)/\{2.76(10^3)\}]^{1/2}$ = 82.09 mm,

where 2.76 N/mm^2 is the value of $Q_{,lim}$ Since, this effective depth is less than 135 mm assumed in Step 1, we retain d = 135 mm and D = 160 mm.

Step 4: Depth of slab for shear force

Table 19 of IS 456 gives the value of τc = 0.28 N/mm^2 when the lowest percentage of steel is provided in the slab. However, this value needs to be modified by multiplying with k of cl. 40.2.1.1 of IS 456. The value of k for the total depth of slab as 160 mm is 1.28. So, the value of τc is 1.28(0.28) = 0.3584 N/mm^2.

Table 20 of IS 456 gives τcmax = 2.8 N/mm^2. The computed shear stress $\tau v = V_u /bd$ = 31/135 = 0.229 N/mm^2.

Since, $\tau_v < \tau_c < \tau_{cmax}$, the effective depth of the slab as 135 mm and the total depth as 160 mm are safe.

Step 5: Determination of areas of steel

The respective areas of steel in middle and edge strips are to be determined. It has been shown that the areas of steel computed from Eq.3.23 and those obtained from the tables of SP-16 are in good agreement. Accordingly, the areas of steel for this problem are computed from the respective Tables 40 and 41 of SP-16 and presented in Table 8.7. Table 40 of SP-16 is for the effective depth of 150 mm, while Table 41 of SP-16 is for the effective depth of 175 mm. The following results are, therefore, interpolated values obtained from the two tables of SP-16.

Table 8.7 Reinforcing bars of Problem 8.2

Particulars	Short span l_x					Long span l_y
	Table No.	M_x (kNm/m)	Dia. & spacing	Table No.	M_y (kNm/m)	Dia. & spacing
Top steel for negative moment	40,41	18.68 > 18.6	10 mm @ 200 mm c/c	40,41	12.314 > 11.66	8 mm @ 200 mm c/c
Bottom steel for positive moment	40,41	14.388 > 13.89	8 mm @ 170 mm c/c	40,41	9.20 > 8.68	8 mm @ 250 mm c/c

The minimum steel is determined from the stipulation of cl. 26.5.2.1 of IS 456 and is $A_s = (0.12/100)(1000)(160) = 192$ mm^2 and 8 mm bars @ 250 mm c/c (= 201 mm^2) is acceptable. It is worth mentioning that the areas of steel as shown in Table 8.7 are more than the minimum amount of steel.

Step 6: Selection of diameters and spacing of reinforcing bars

The advantages of using the tables of SP-16 are that the obtained values satisfy the requirements of diameters of bars and spacing. However, they are checked as ready reference here. Needless to mention that this steel may be omitted in such a situation.

Maximum diameter allowed, as given in cl. 26.5.2.2 of IS 456, is 160/8 = 20 mm, which is more that the diameters used here.

The maximum spacing of main bars, as given in cl. 26.3.3(1) of IS 456, is the lesser of 3(135) and 300 mm. This is also satisfied for all the bars.

Step 7: Determination of torsional reinforcement

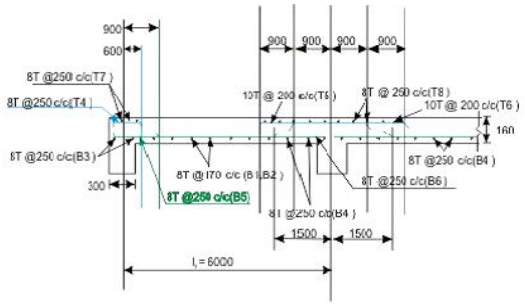


Fig. 8.19.8: Problem 8.2, Sec 1-1 of Panel 1 of Fig. 8.19.7

Torsional reinforcing bars are determined for the three different types of corners as explained in sec. 8.19.6 (Fig.8.19.4). The length of torsional strip is 4000/5 = 800 mm and the bars are to be provided in four layers. Each layer will have 0.75 times the steel used for the maximum positive moment. The C1 type of corners will have the full amount of torsional steel while C2 type of corners will have half of the amount provided in C1 type. The C3 type of corners do not need any torsional steel. The results are presented in Table 8.8 and Figs.8.19.10 a, b and c.

Table 8.8 Torsional reinforcement bars of Problem 8.2

Type	Dimensions along		Bar diameter & spacing	No. of bars along	Cl. no. of IS 456	
	x (mm)	*y* (mm)			*x*	*y*
C1	800	800	8 mm @ 200 mm c/c	5	5	D-1.8
C2	800	1600	8 mm @ 250 mm c/c	5	8	D-1.9
C2	1600	800	8 mm @ 250 mm c/c	8	5	D-1.9

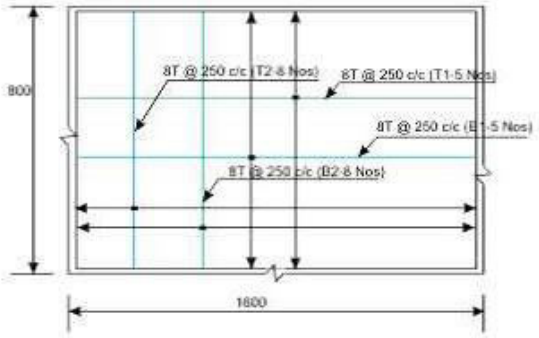


Fig. 8.19.10(c): Corners C2

Fig. 8.19.10: Torsion reinforcement bars of Problem 8.2

Problem No 2

A drawing room of a residential building measures 4.3 m × 6.55 m. It is supported on 350 mm thick walls on all four sides. The slab is simply supported at edges with no provision to resist torsion at corners. Design the slab using grade M 20 concrete and HYSD reinforcement of grade Fe 415.

Solution:

Consider 1 m wide strip. Assume 180 mm thick slab, with 160 mm effective depth.

$$l_x = 4.3 + 0.16 = 4.46 \text{ say } 4.5 \text{ m.}$$

$$l_y = 6.55 + 0.16 = 6.71 \text{ say } 6.75 \text{ m.}$$

Dead load: self $0.18 \times 25 = 4.5$ kN/m^2

floor finish $= 1.0$ kN/m^2

Live load (residence) $= 2.0$ kN/m^2

Total 7.5 kN/m^2

For 1 m wide strip

$$P_u = 1.5 \times 7.5 = 11.25 \text{ kN/m.}$$

$$\frac{l_y}{l_x} = \frac{6.75}{4.5} = 1.5$$

$$M_{ux} = 0.104 \times 11.25 \times 4.5^2 = 23.7 \text{ kNm}$$

$$M_{uy} = 0.046 \times 11.25 \times 4.5^2 = 10.48 \text{ kNm}$$

$$d_{required} = \sqrt{\frac{23.7 \times 10^6}{1000 \times 2.76}} = 92.7 \text{ mm}$$

$$d_{short} = 180 - 15 \text{ (cover)} - 5$$

$$= 160 \text{ mm} > 92.7 \text{ mm} \quad \text{..... (O.K.)}$$

$$d_{long} = 160 - 10 = 150 \text{ mm}$$

Larger depth is provided to satisfy deflection check.

$$\frac{M_u}{bd^2} \text{ (short)} = \frac{23.7 \times 10^6}{1000 \times 160 \times 160} = 0.926$$

$$p_t = 0.273$$

$$A_{st} \text{ (short)} = \frac{0.273 \times 1000 \times 160}{100} = 437 \text{ mm}^2.$$

$$\frac{M_u}{bd^2} \text{ (long)} = \frac{10.48 \times 10^6}{1000 \times 150 \times 150} = 0.466$$

$$p_t = 0.129$$

$$A_{st} = \frac{0.129 \times 1000 \times 150}{100} = 194 \text{ mm}^2.$$

$$\text{Minimum steel} = \frac{0.12}{100} \times 1000 \times 180 = 216 \text{ mm}^2.$$

Provide 10 mm ϕ about 180 mm c/c = 436 mm² in short span and 8 mm ϕ about 230 mm c/c = 217 mm² in long span.

The bars cannot be bent or curtailed because if 50% of long span bars are curtailed, the remaining bars will be less than minimum.

At top on support, provide 50% of bars of respective span to take into account any possible negative moment created due to monolithic nature of slab.

Check for development length:

Long span $V_u = 11.25 \times 2.25 = 25.31$ kN.

$$M_{u1} = 0.87 \times 415 \times 217 \left(150 - \frac{415 \times 217}{1000 \times 20}\right) \times 10^{-6}$$

$$= 11.40 \text{ kNm.}$$

Assuming $L_0 = 8$ ϕ

$$1.3 \times \frac{11.40 \times 10^6}{25.31 \times 10^3} + 8\phi \geq 47\phi$$

which gives $\phi \leq 15.01$ mm ..... (O.K.)

short span $V_u = 11.25 \times 2.25 = 25.31$ kN.

$$M_{u1} = 0.87 \times 415 \times 436 \left(160 - \frac{415 \times 436}{1000 \times 20}\right) \times 10^{-6}$$

$$= 23.76 \text{ kNm.}$$

Assuming $L_0 = 8$ ϕ

$$1.3 \times \frac{23.76 \times 10^6}{25.31 \times 10^3} + 8\phi \geq 47\phi$$

which gives $\phi \leq 31.3$ mm ..... (O.K.)

Note that the bond is usually critical along long direction.

Check for shear:

This is critical along long span

Shear stress $\tau_v = \frac{25.31 \times 10^3}{1000 \times 150} = 0.169$ N/mm²

$$\frac{100 A_s}{bd} = \frac{100 \times 217}{1000 \times 150} = 0.145$$

$$\tau_c = 0.28 \text{ N/mm}^2$$

$$k\tau_c = 0.28 \times 1.2 = 0.336 \text{ N/mm}^2. \text{ ..... (O.K.)}$$

Check for deflection:

The deflection shall be checked along short span.

Basic $\frac{\text{span}}{d}$ ratio = 20

$$\frac{100\,A_s}{b\,d} = \frac{448 \times 100}{1000 \times 160} = 0.28.$$

$$\text{service stress} = 0.58 \times 415 \times \frac{435}{448} = 234 \text{ N/mm}^2$$

Note that $A_{st,req}$ is used here.

Modification factor = 1.5

Permissible $\frac{\text{span}}{d}$ ratio = $20 \times 1.5 = 30$

actual $\frac{\text{span}}{d}$ ratio = $\frac{4480}{160} = 28$ ..... (O.K)

Check for cracking:

Maximum spacing permitted for short span steel

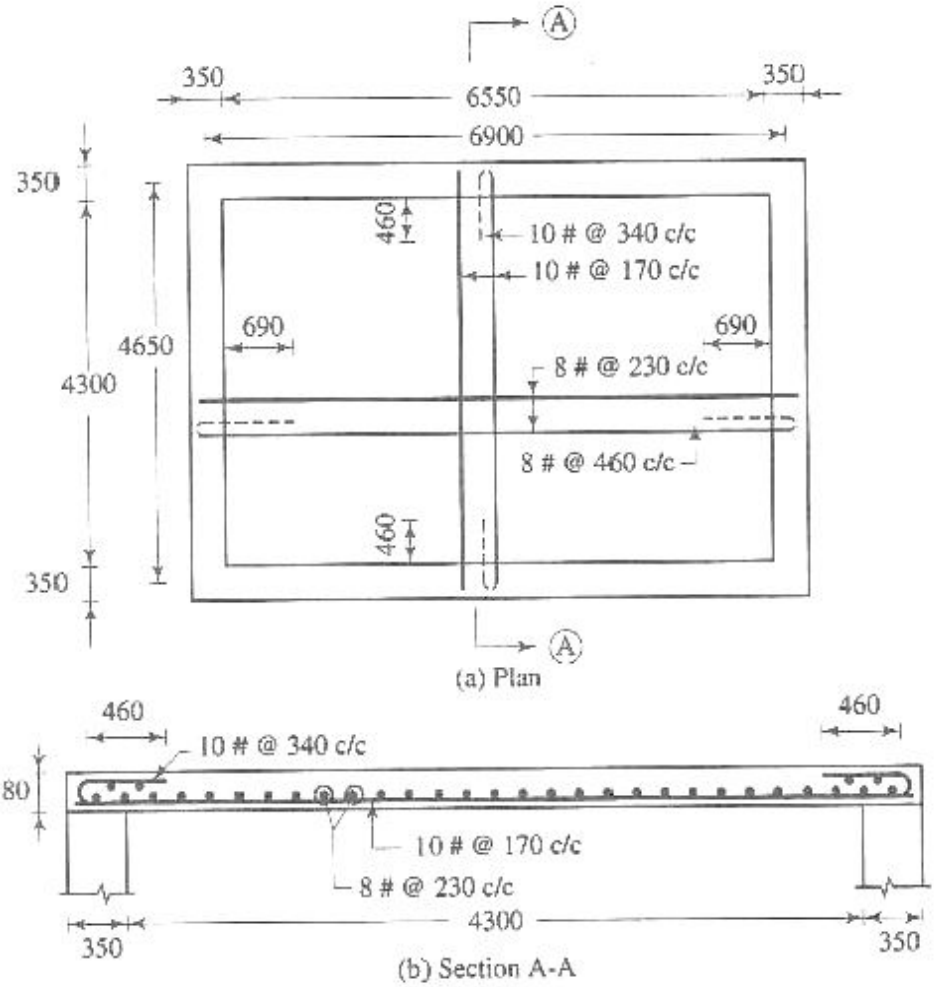
$= 3 \times 160 = 480$ or 300 mm, i.e., 300 mm

Spacing provided = 180 mm ..... (O.K)

Maximum spacing permitted for long span steel = $3 \times 150 = 450$ mm.

Spacing provided = 230 mm ..... (O.K)

Sketch: The designed reinforcements of slab are shown in fig. 10-26.



(a) Plan

(b) Section A-A

Note: It is possible to bend short span reinforcement alternately. Rework the problem

CHAPTER – 9

AXIALLY LOADED SHORT COLUMN

Introduction –

A column may be defined as an element used primarily to support axial compressive loads and with a height of at least three times its least lateral dimension. All columns are subjected to some moment which may be due to accidental eccentricity or due to end restraint imposed by monolithically placed beams or slabs. The strength of a column depends on the strength of the materials, shape and size of the cross-section, length and the degree of the positional and directional restraint at its end.

A column may be classified as short or long column depending on its effective slenderness ratio. The ratio of effective column length to least lateral dimension is referred to as effective slenderness ratio. A short column has a maximum slenderness ratio of 12. Its design is based on the strength of the materials and the applied loads. A long column has a slenderness ratio greater than 12. Its design is based on the strength of the materials and the applied loads. A long column has a slenderness ratio greater than 12. However , maximum slenderness ratio of a column should not exceed 0. A long column is designed to resist the applied loads plus additional bending moments induced due to its tendency to buckle.

ASSUMPTIONS -

The following assumptions are made for the limit state of collapse in compression.

1. Plane sections normal to the axis remain plane after bending.
2. The relationship between stress-strain distribution in concrete is assumed to be parabolic. The maximum compressive stress is equal to 0.67 δ_{ck} / 1.5 or 0.446 δ_{ck}.
3. The tensile strength of concrete is ignored.
4. The stresses in reinforcement are derived from the representative stress-strain curve for the type of steel used.
5. The maximum compressive strain in concrete in axial compression is taken as 0.002.
6. The maximum compression strain at the highly compressed extreme fibre in concrete subjected to axial compression and bending, but when there is no tension on the section, is taken as 0.0035 minus 0.75 times the strain at the least compressed extreme fibre.
7. The maximum compressive strain at the highly compressed extreme fibre in concrete subjected to axial compression and bending, when part of the section is in tension, is taken as 0.0035. in the limiting case when the neutral axis lies along one edge of the section, the strain varies from 0.0035 at the highly compressed edge to zero at the opposite edge.

The types of failures to the columns –

Columns, when centrally loaded , fails in one of the three following modes, depending on the slenderness ratio. Fig.

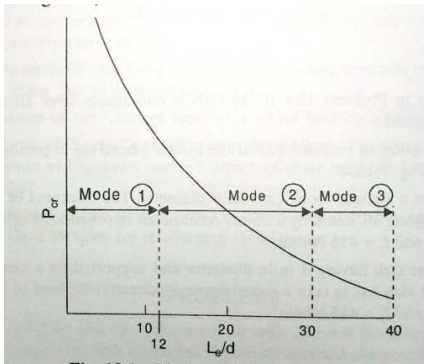


Fig. 13.1 Modes of failure of columns.

Mode :1 Pure compression failure

The columns fails under axial loads without undergoing any lateral deformation. Steel and concrete reach the yield stress values at failure. The collapse is due to material failure.

Mode :2 Combined compression and bending failure

Short columns can be subjected to direct load (P) and moment (M). slender columns even when loaded axially undergo deflection along their length as beam columns, and these deflection produce additional moments in the columns. When material failure is reached under the combined action of these direct loads and bending moment. It is called combined compression and bending failure.

Mode : 3 Failure by elastic instability

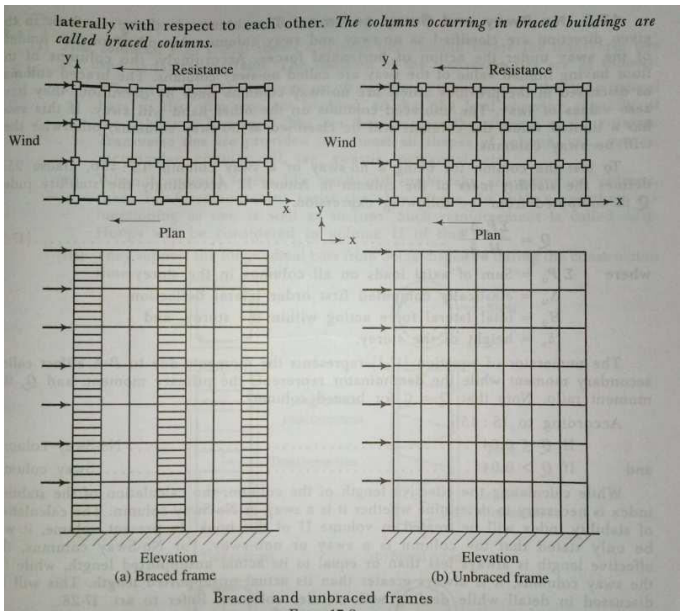
Very long columns can become unstable even under small loads well before the material reaches yield stresses. Under such cases the member fails by lateral ‘ elastic bulking’ .

failure by third mode is unacceptable in practical construction. R.C.C members that may fail by this type of failure is prevented by the codal provision that columns beyond a specified slenderness should not be allowed in structure.

Braced and unbraced columns :

The columns in a building are classified as braced or unbraced according to the method applied to provide the lateral stability of the building.

(1) **Braced column :** In braced frames, the lateral loads like wind , earthquake etc. are resisted by some special arrangements like shear walls , bracings or special supports. In other words, the sidesway or joint translation is not possible in such columns. Sidesway or joint translation means that one or both the ends of a column can move laterally with respect to each other. The columns occurring in braced buildings are called braced columns.



(2) Unbraced columns : A unbraced frames no special bracing systems are provided to resist horizontal forces. In other words the sidesway or joint translation do occur in such columns. The columns shall have to be designed to resist the lateral loads. The column those occur in the buildings where the resistance to lateral loads is provided by the bending in the columns and beams in that plane are called unbraced columns.

Effective length of columns :

The unsupported length or height of a column (Lo) is generally taken as the clear height of the columns. It is defined in IS 456, clause 25.1.3 for various cases of constructions. The effective length of column is different from unsupported length. Effective length (L_e) is dependent on the bracing and end conditions. It should be noted that for braced columns the effective column height is less than the clear height between restraints , whereas for unbraced and partially braced columns the effective height is greater than the clear height.

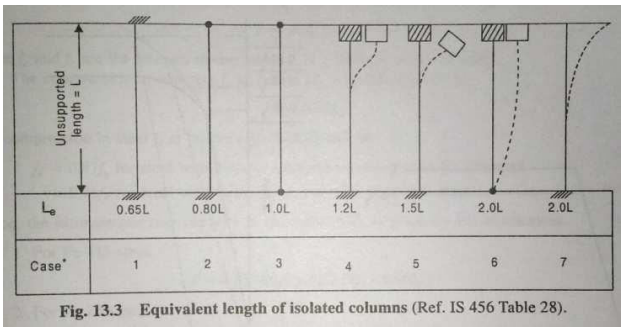
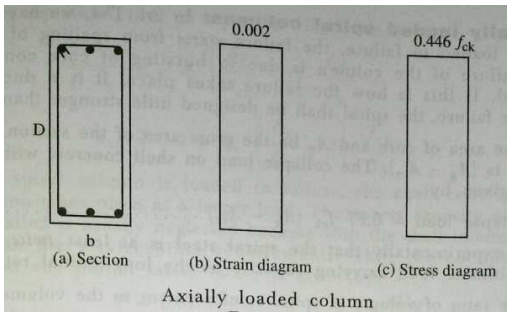


Fig. 13.3 Equivalent length of isolated columns (Ref. IS 456 Table 28).

Design formula for Short column –

A rectangular column section bearing pure axial load. The design stress in mild steel at strain of 0.002 is 0.87 f_y, however for , HYSD bars it is not so. The stresses corresponding to 0.002 strain in HYSD bars are as follows.

F_e 415	0.79 f_y
F_e 500	0.75 f_y



Axially loaded column

The code adopts the critical value of 0.75 f_y for all grades of steel for finding out the pure axial load carrying capacity of the column. Accordingly

$$P_{uz} = 0.446 f_{ck} A_c + 0.75 f_y A_{sc}$$

This is approximated as

$P_{uz} = 0.446 f_{ck} A_c + 0.75 f_y A_{sc}$

P_{uz} = Pure ultimate axial load carrying capacity of column.

f_{ck} = Characteristic compressive strength of concrete.

f_y = Characteristic strength of reinforcement.

A_c = Area of concrete in column section.

A_{sc} = Area of reinforcement in column section.

Axially loaded practical columns are subjected to moments due to minimum eccentricity . thus all the columns , even if the design load is axial, shall be designed for moments also.

The code simplifies the work for the columns in which minimum eccentricity $e_{min} \leq 0.05$ D. Thus , when $e_{min} \leq 0.05$ D , the above equation is midified as

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \ldots\ldots\ldots$$

It can be seen that , the load carrying capacity of the column is reduced by about 10 percent when $e_{min} \leq 0.05$ D. however if $e_{min} \leq 0.05$ D , the column shall be designed for moment also.

The above equation can written as –

$$P_u = 0.4 f_{ck} \left(A_g - \frac{P\text{Æ}_g}{100} \right) + 0.67 f_y \frac{P\text{Æ}_g}{100}$$

Where A_g = Gross area of cross section

P = Percentage of reinforcement.

Dividing both sides by A_g

$$\frac{P_u}{A_g} = 0.4 f_{ck} \left(1 - \frac{PA_g}{100}\right) + 0.67 f_y \frac{P}{100}$$

$$= 0.4 f_{ck} \frac{P}{100} \left(0.67 f_y - 0.4 f_{ck}\right) \ldots.$$

The compression in steel f_s at failure ($\epsilon_c = 0.002$) will be

$f_s = 0.87 f_y$ for steel with bilinear stress-strain curve as in Fe 250 steel.

$= 0.75 f_y$ for steel with stress-strain curve as in Fe 415 steel.

Hence , the ultimate carrying capacity of the column p_u is given by the expression

1. For Fe 415 steel, $P = A_c (0.45 f_{ck}) + A_s (0.75 f_y - 0.45 f_{ck})$
2. For Fe 250 steel , $P = A_c (0.45 f_{ck}) + A_s (0.87 f_y - 0.45 f_{ck})$

However it is never possible to apply the load centrally on a column. Accidental eccentricities are bound to happen. Indian and British codes allow an accidental eccentricity of 5 percent of the lateral dimension of the column in the plane of bending (0.05 D) in the strength formula itself. For this purpose the ultimate load p_u for Fe 415 steel reduces to

$P_u = 0.9 (0.45 f_{ck} A_c + 0.75 f_y A_s)$

$P_w = 0.4 f_{ck} A_c + 0.67 f_y A_s$ as given in IS 456, clause 38.3

With Fe 250 steel the corresponding expression will be

$P_u = 0.4 f_{ck} A_c + 0.75 f_y A_s)$

It should be clearly noted theses formula already take into account a maximum accidental eccentricity of (0.05 D) or (0. 05b) in theses columns.

Check for Minimum Eccentricity

Minimum eccentricities are caused by imperfections in construction, inaccuracy in loading etc. The BS code 8110 (1985) Cluase 3.8.2.4 assumes that its value will be equal to 0.05 D, but not more than 20mm.

IS 456, Cluase 25.4 gives an expression for the possible minimum eccentricity as $e_{min} = \frac{L0}{500} + \frac{D}{30}$ but not less than 20mm.

Where L0 = the unsupported length

D = lateral dimensions in the plane of bending.

For sections other than rectangular, the Explanatory handbook SP 24 recommends a value of L_e / 300 , where L_e is the effective length of the column.

Thus for example , for a column 600 x 450 of unsupported height 3 m, considering the long direction according to IS formula,

$e_{min} = \frac{L0}{500} + \frac{D}{30} = 3000/500 + 600/30 = 26$mm.

As 26mm is greater than the minimum specified 20mm, use e_{min} = 26mm. then $\frac{emin}{D}$ = 26/600 = 0.043

Considering the short direction, we have

e_{min} = 3000/500 + 450/30 = 21mm > 20mm.

Hence $\frac{e}{b}$ = 21/450 = 0.047.

Both these values are less than the specified ratio of 0.05, and hence the simple column formula is applicable to the above column. If the eccentricities are more, then the column has to be designed as subjected to direct load P and moment Pe.

Minimum Longitudinal and Transverse reinforcement –

The reinforcement requirements are set out in clause 26.5.3 of IS 456.

1. **Longitudinal Reinforcement:**

(i) The cross sectional area of longitudinal reinforcement shall be not less than 0.8 percent of gross cross sectional area of the column.

The minimum area of the reinforcement is specified to avoid the sudden non-ductile failure of the column, to resist creep and shrinkage and to provide some bending strength to the column.

(ii) In any column that has a larger cross sectional area than that required to support the load, the minimum percentage of steel should be based upon the area of concrete required to resist the direct stress and not upon the actual area.

Because of the architectural or the other reasons, sometimes the columns are made larger in section than that required to resist the load. In such a case , according to this criteria , the minimum percentage of steel is based on concrete area required to resist the direct load. A concrete pedestal used to transfer the load from steel stanchion to the foundation in an industrial building is a typical example for this case. In this case , the size of the pedestal is governed by the size of the base plate under the steel column.

(iii) The cross sectional area of longitudinal reinforcement shall be not more than 6 percent of the gross cross sectional area of the column.

The use of 6percent reinforcement may involve practical difficulties in placing and compacting of concrete, hence lower percentage is recommended where bars from the columns below have to be lapped with those in the column under consideration, the percentage of steel usually should not exceed 4 percent.for the column with more than 4 percent steel, the laps may be staggered.

(iv) The minimum number of longitudinal bars provided in a column shall be four in rectangular columns and six in circular columns.

(v) The bar shall not be less than 12 mm in diameter.

(vi) A reinforced concrete column having helical reinforcement shall have at least six bars of longitudinal reinforcement within the helical reinforcement.

(vii) In a helical reinforced column , the longitudinal bars shall be in contact with the helical reinforcement and equidistant around its inner circumference.

(viii) Spacing of longitudinal bars measured along the periphery of the column shall not exceed 300 mm. this is a cracking requirement.

(ix) In case of the pedestals in which the longitudinal reinforcement is not taken into account in strength calculation, nominal reinforcement not less than 0.15 percent of the cross sectional area shall be provided.

(2) Transverse Reinforcement:

(i) General : a reinforcement concrete compression member shall have transverse or helical reinforcement so disposed that every longitudinal bar nearest to thecompression face has effective lateral support against bulking subject to provisions in (b). The effective lateral support is given by transverse reinforcement either in the form of circular rings capable of taking up circumferential tension or by polygonal links (lateral ties) with internal angles not exceeding 135 degree. The ends of the transverse reinforcement shall be properly anchored.

(ii) Arrangement of transverse reinforcement :

(a) If the longitudinal bars are not spaced more than 75mm on either side, transverse reinforcement need only to go round the corner and alternate bars for the purpose of providing effective lateral supports.

(b) If the longitudinal bars spaced at a distance of not exceeding 48 times the diameter of the tie are effectively tied in two directions, additional longitudinal bars in between theses bars need to be tied in one direction by open ties.

(c) Where the longitudinal reinforcing bars in a compression member are placed in more than one row, effective lateral support to the longitudinal bars in the inner rows may be assumed to have been provided if-

(1) Transverse reinforcement is provided for outer most row in accordance with (b) and

(2) No bar of the inner row is closer to the nearest compression face than three times the diameter of the largest bar in the inner row.

(d) Where the longitudinal bars ina compression member are grouped and each group adequetly tied transverse reinforcement in accordance with the above requirements, the transverse reinforcement for the compression member as a whole may be provided on the assumption that each group is a single longitudinal bar for purpose of determining the pitch and diameter of the transverse reinforcement in accordance with above requirements. The diameter of such transverse reinforcement need not, however exceed 20mm.

(iii) Pitch and diameter of lateral tie :

(a) Pitch : the pitch of transverse reinforcement shall be not more than the least of the following distances:

(1) The least lateral dimension of the compression member.

(2) Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied.

(3) 300mm.

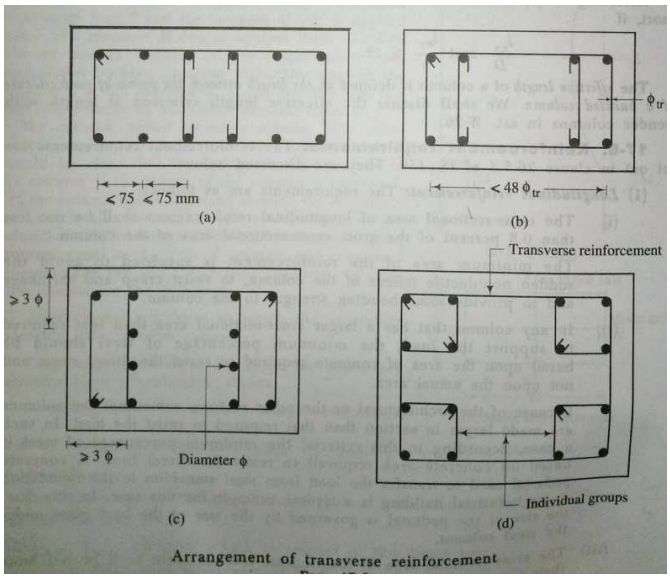
(b) Diameter : the diameter of the polygonal links or ties shall be not less than one fourth of the diameter of the largest longitudinal bar and in no case less than 6 mm.

(iv) Helical reinforcement :

(a) Pitch : Helical reinforcement shall be of regular formation with the turns of the helix spaced evenly and its ends shall be anchored properly by providing one and a half extra turns of the spiral bar. Where an increase load on the column on the strength of the helical reinforcement is allowed for, the pitch of helical turns shall be not more than 75 mm, nor more than one-sixth of the core diameter of the column, nor less than 25mm, nor less than there times the diameter of the steel bar forming the helix.

(b) Fiameter : the diameter of the helical reinforcement shall be as per lateral ties.

(v) Cover : the longitudinal reinforcing bar in a column shall have concrete cover, not less than 40mm, nor les than the diameter of such bar. This requirements gives a fire protection of 0.5 h to 4h (h = hour) and is suitable for moderate exposure assuming a maximum of 10 mm diameter tie. However the cover to the ties may be governed by exposure conditions. In the case of columns the minimum dimensions of 200 mm or under, whose reinforcing bars do not exceed 12mm, a cover of 25mm may be used.



Arrangement of transverse reinforcement

Design of short column by IS 456 and SP 16 –

Charts 24 to 26 of the IS publication design aids SP 16 can be used for routine office design of short columns. These charts are made from the column formula

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

A_g = area of cross section

$P = percentage \text{ of } steel = 100 \frac{Æs}{Æg}$

$The\ area \text{ of } steel\ and\ concrete\ are\ given\ by$

$A_s = \frac{P Æg}{100}$

$A_c = A_g - A_s = A_g (1 - \frac{P}{100})$

$Rewritting\ the\ equation\ with\ above\ quantities$, we $obtain$

$\frac{P_u}{Æg} = 0.4 f_{ck} (1 - \frac{P}{100}) + 0.67 f_y \frac{P}{100}$

$P_u = [0.4 f_{ck} \frac{P}{100} (0.67 f_y - 0.4 f_{ck}] A_g$

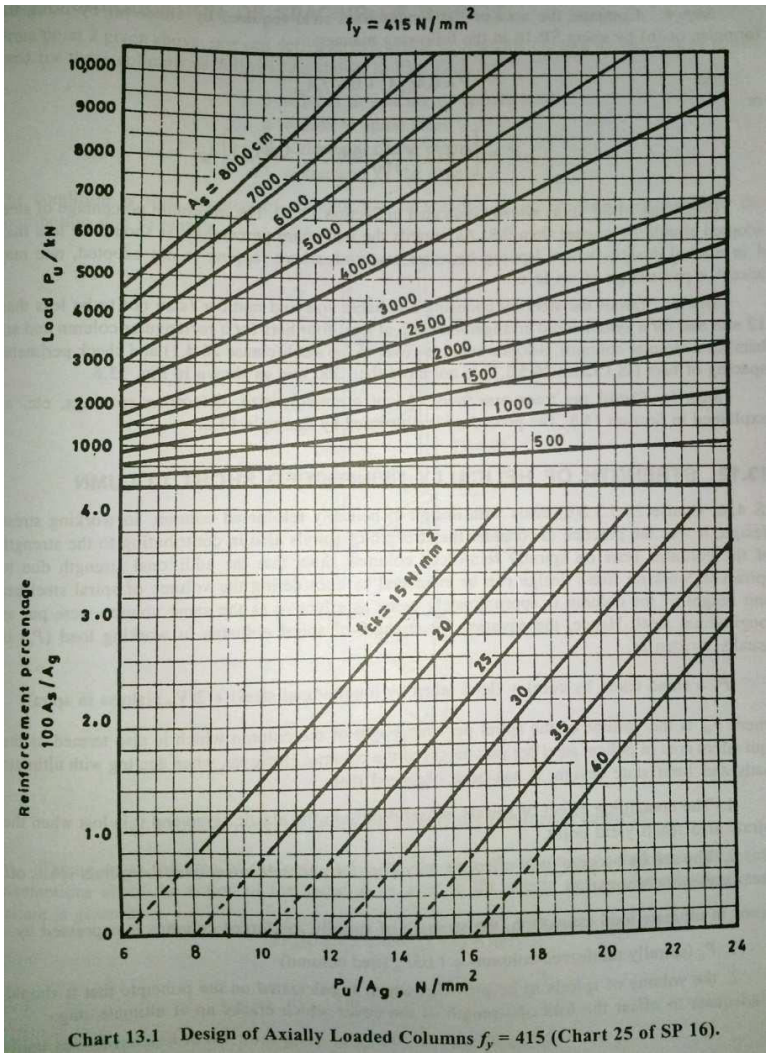


Chart 13.1 Design of Axially Loaded Columns f_y = 415 (Chart 25 of SP 16).

Chart 24 to 26 of SP 16 have been prepared from these formula for Fe 250 , Fe 415 and Fe 500 and f_{ck} = 15, 20, 30, 35, and 40.

To use design chart , choose the value of the factored design load P_u , and proceed horizontally till the A_g corresponding to the size of the column selected is reached. The value of percentage of steel required for the adopted value of f_{ck} is read off from the lower half of chart 13.1.

Procedure for design of centrally loaded short column:

The step-by-step procedure for design of a centrally loaded column can be arranged as follows:

Step 1 : compute the factored load on the column.

Step 2 : Choose a suitable size for the column, depending on the size of the beam that has to be placed on it and the architectuerral requirements. Usually the beams are accommodated inside the column. Check also the minimum eccentricity.

Step 3: Determine the effective length length and slenderness of the column about the principal axes. If it is less than 12, it can be can be considered as a short column. If it is 12 or more, it is to be designed as a long column.

Step 4 : compute the area of the longitudinal steel required by either (a) by using the formula or (b) by using SP 16 in the following manner.

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

or $$P_u = [0.4 f_{ck} \frac{P}{100} (0.67 f_y - 0.4 f_{ck}] A_g$$

The minimum percentage of steel adopted should be greater than 0.8. As regards the maximum percentage , it should be less than 4 in normal designs where lapping becomes essentials. Where lapping is not adopted, one may accept a percentage of up to 6.

Step 5: Detain the steel by choosing a suitable size and number (size not to be less than 12mm and for a symmetrical arrangement with at least four bars for a rectangular column and six bars for a circular ciolumn). Adopt a suitable cover to teh steel (clause 26.4.1) and check perimeter spacing of bars (IS clause 26.5.3.1) is not more than 300mm.

Step 6: Detail the transverse steel. Adopt a suitable size, determine spacings, etc.

Strength of helically reinforced short column:

IS 456, clause 26.5.3.2(d) deals with design of helically reinforced column. In working stress design, it was practice to consider the strength of spirals also in contributing to the strength of the column. Tests on spirally reinforced reinforced columns show that the additional strength due to spirals in working stress design can be estimated by considering the volume of spiral steel per unit height of the column is approximately twice as effective as the same volume were put as longitudinal steel. Hence the equation for strength of spiral columns in working load ($_{Pc}$) is usually written as

Pc = (Load taken by core) + (load taken by longitudinal steel) + 2 (V_{sh}) (stress in spiral)

Where V_{sh} is the volume of the spiral per unit length of the column which is also termed as the equivalent area of helical steel per unit height of the column. However when dealing with ultimate loads and limit state design, it has been observed that

1. The containing effect of spirals is useful only in the elastic stage and it is lost when the spirals also reach yield point.
2. The spirals become fully effective only after the concrete cover over the spirals spalls off after excessive deformation.

Hence in ultimate load estimation , the strength of spirally reinforced columns is expressed by

1. P_u (spirally reinforced column) = 1.05 P_u (tied column)
2. The volume of spirals to be provided which is calculated on the principle that it should be adequate to offset the loss of strength of the cover which cracks up at ultimate stage.

Calculation of spacing of spirals:

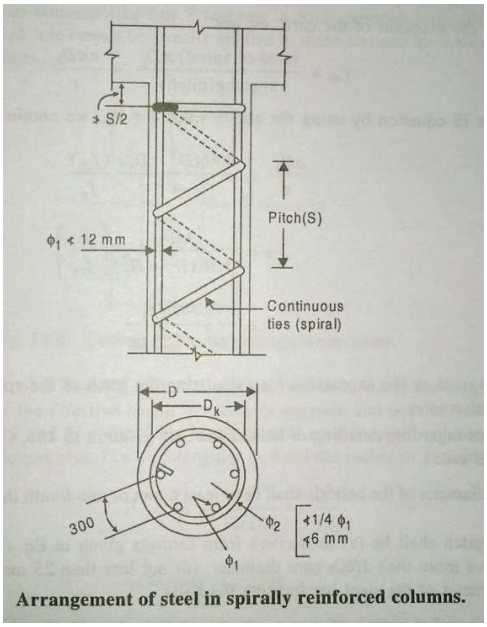
S = Pitch or spacing of spirals used.

a = area of spiral steel .

D = diameter of the column.

D_k = diameter of the core.

The condition is that the loss of strength due to spalling of cover should be equal to the contribution due to spirals.



Arrangement of steel in spirally reinforced columns.

Taking A_k as the area of the core and A_g as the area of cross section and using the same assumption about the action of the spiral as is used in the elastic design, the relationship at failure is given by

$2V_{sh}(0.87f_y) = 0.63\, f_{ck}\,(A_g - A_k)$

$V_{sh} = 0.36\,(A_g - A_k)\left(\frac{f_{ck}}{f_y}\right)$

Which can be reduced to the form given in IS 456, clause 38.4.1 as

$\frac{V_{sh}}{A_k} = 0.36\left(\frac{A_g}{A_k} - 1\right)\left(\frac{f_{ck}}{f_y}\right)$

Where A_g = gross area of section

A_k = area of core.

This expression gives the ratio of the volume of the helical reinforcement required for the volume of the core per unit height of the column.

Simplifying this expression further, one can write

V_{sh} = (Volume of the spiral in one ring) x (No. Of rings per unit length)

Taking D_k as the diameter of the core , we get

$V_{sh} = \frac{(area\ of\ sprial)\ \pi\ D_k}{Spacing\ (pitch)} = \frac{a\pi D_k}{s}$

Rewriting the IS equation by using the above value for V_{sh}, we get

$$\frac{aD_k}{s} = \frac{0.36\,(D^2 - D^2_k)}{4}\,\frac{f_{ck}}{f_y}$$

$$S = \frac{4aD_k)}{0.36\,(D^2 - D_k^2)}\,\frac{f_y}{f_{ck}}$$

$$S = \frac{11.1aD_k f_y)}{f_{ck}(D^2 - D_k^2)} \quad \text{.....(1)}$$

Which can be used as the expression for calculating the pitch of the spirals for a given steel of cross section area a.

The rules regarding detailing of helical steel are given IS 456, clause 26.5.3.2. the main considerations are:

1. The diameter of the helicals shall be at least 6mm or one fourth the diameter of longitudinal steel.
2. The pitch shall be (a) as derived from formula equation (1) (b) not more than 75mm, (c) not more than 1/6th core diameter , (d) not less than 25mm, (e) not less than three times the diameter of the steel bar forming the helix.

   If the diameter and the pitch of the spirals do not comply with the above rules, the strength is to be taken as only that of a tied column of similar dimension.

Problem 1:

A column of 400 x400mm has an unsupported length of 7m and effective length of 4.5 m. Can it be designed as a short column under axial compression , if the load placed centrally on it?

Solution:

Step 1: Slenderness ratio consideration

$\frac{L_e}{D}$ = 4500 / 400 = 11.25 (IS456 Cl.25.1.2)

As columns with slenderness less than 12 can be considered as short , the column is short.

Step 2: Eccentricity considerations

$e_{min} = \frac{L0}{500} + \frac{D}{30}$ not less than 20mm. (IS 456 Cl . 25.4)

= 7000/ 500 + 400/30

= 14 + 13.3

27.3 mm is greater than 20mm. Adopt 27.3mm

The eccentricity for which short column formula is applicable id D/20

D/20 = 400/20 = 20mm (IS 456 Cl.39.3)

e_{min} > D/20

Hence formula for axial load is not applicable.

Column should be designed as subject to axial load and moment due to e_{min}.

(M = P e_{min})

Problem – 2

Design an axially loaded tied column 400 x 400 mm pinned at both ends with an unsupported length of 3m for carrying a factored load of 2300 KN. Use grade 20 concrete and Fe 415 steel.

Solution:

Step 1: Factored load on column

P_u = 2300KN

$Step\ 2$: Size of column and check e_{min} (IS 456 Cl. 25.4)

$Size\ of\ column = 400\ x400$, D/20 = 20mm

$e_{min} = \frac{L0}{500} + \frac{D}{30}$ = 3000/500 + 400/30 = 19.33 < 20mm

e_{min} less than D/20 is assumed in the formula. Hence short column formula for axial load can be used.

$Step\ 3$: Calculation of slenderness (IS456 Cl. 25.1.2)

L_e = 1.0L = 3000mm

$\frac{L_e}{b}$ = 3000/400 = 7.5 < 12

$Step\ 4$: Find area of steel and check percentage

(a) By formula, $P_u = 0.4\, f_{ck}\, A_c + 0.67\, f_y\, A_{sc}$ (IS 456 Cl. 39.3)

2300 x 10^3 = 0.4 x 20 x (400^2 - A_s) + 0.67 x 415 x A_s

A_s = 3777 mm^2, P = 3777/ 400^2 x 100 = 2.36%

$This\ is\ more\ than\ 0.8\%\ and\ less\ than\ 6\%.\ hence$ o. k

(b) By SP 16 , A_g = 1600 cm^2, P = 2300 KN. P = 2.4% (SP 16 chart 25)

A_s = (2.4x400x400) / 100 = 3840mm^2 (Use 8T -25 / 3927mm^2)

$Step\ 5$: Detail the longitudinal steel

Use cover = 40mm (Cl. 26.4.2.1)

Steel spacing = (400 – 40 – 40 -25) / 2 = 147.5

Clear spacing between bars = 147.5 – 25 = 122.5 < 300 (Cl.26.5.3.1 g)

Step 6: Design transverse steel

Diameter of links : not less than 25/4 or 6mm.

Use 10mm

Spacing least of [Cl. 26.5.3.2 (c)]

(a) Dimension of column = 400mm

(b) 16 times Ø of long steel = 16 x25 = 400mm

(c) 300 mm adopt 300mm.

Use Fe250 steel for ties.

(a) S = $\frac{11.1 a D_k f_y)}{f_{ck}(\ D^2 - D_k^2)}$ =(11.1 x 28 x 400 x 145) / (57600 x 30)

= 30mm Cl. 39.4.1

(b) Spacing not more than 75mm

(c) Spacing not more than 320 / 6 = 53.3mm

(d) Spacing not less than 25

(e) Spacing not less than 6 x 3 = 18mm

Choose 30mm spacing.

(Design helically reinforced columns)

Problem 1 :

Design a circular pin ended column 400 mm dia and helically reinforced , with an unsupported length of 4.5 m to carry a factored load of 900 KN. Assume M30 concrete and Fe 415 steel.

Solution:

Step 1: Factored load, P_u = 900 Kn.

Step 2: Size of column D = 400mm, cover = 40mm

D_{core} = 320mm

D/20 = 400/20 = 20mm.

$e_{min} = \frac{L_0}{500} + \frac{400}{30} = \frac{4500}{500} + \frac{400}{30}$ = 22.3mm > 20mm.

As e_{min} > D/20, theoretically short column formula for centrally loaded column is not applicable. However the column is designed as centrally loaded, as the moment to be considered is small.

Step 3: Slenderness of column

$\frac{L_e}{D}$ = 4500/40 = 11.25 < 12. (centrally loaded Short Column)

Step 4: Area of longitudinal steel

$P_u = 1.05\ (0.4\ f_{ck}\ A_c + 0.67\ f_y\ A_s$ (IS456 Cl. 39.4)

$A_c = \pi$ x 400² / 4 = 125.6 x 10 ³ mm², $\frac{P}{1.05} = \frac{900}{1.05}$ = 857 KN.

857 x 10³ = [.4 x 30 x (125600 - A_s) + 0.67 x 415 A_s]

= 1507 x 10³ + A_s (278 – 12)

Concrete itself can carry more than the required load.

Hence provide minimum steel.

$A_{s(min)}$ = 0.8% (of area required to carry P) IS 456 Cl 26.5.3.1 (a) (b)

A_c to resist given P = $\frac{900x\ 10^3}{1.05\ x\ 0.4\ x\ 30}$ = 71428mm²

$A_{s(min)} = \frac{0.8}{100}$ x (71428) = 571.4 mm².

Provide 6 nos. Of 12mm bars giving area 678 mm² as minimum number of bars allowed is 6 nos.

Step 5: Design Spirals CL.26.5.3.1(c)

Choose 6mm, a = 28 mm² (area) , s = pitch.

Detailing at junctions with reinforcement footing –

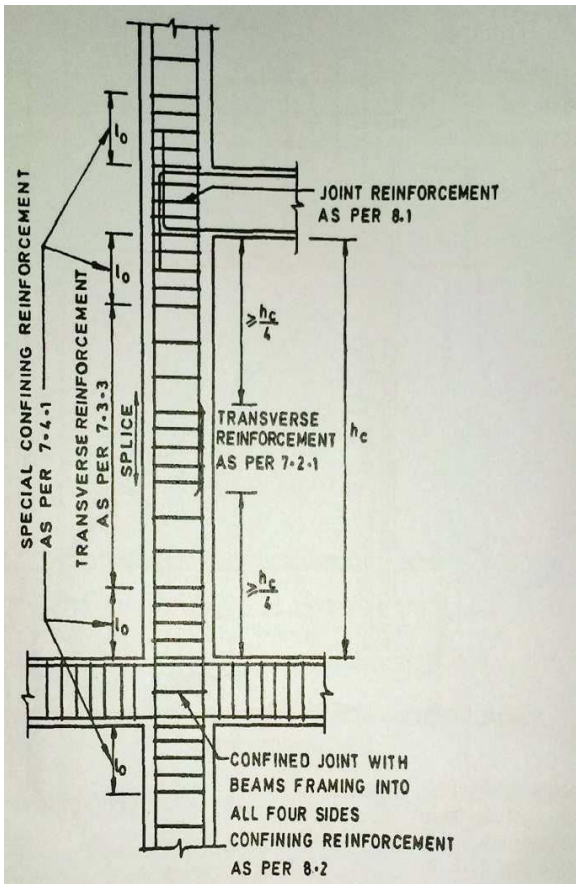
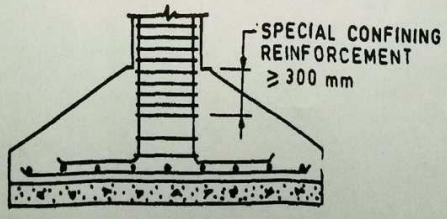


FIG. 9 COLUMN AND JOINT DETAILING



0 PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTINGS

CHAPTER – 10

Ductility

Introduction –

Member or structural ductility is defined as the ratio of absolute maximum deformation to the corresponding yield deformation. However ductility has no precise meaning until the method of measuring the deformation has been defined. This can be defined with respect to strains, rotations, curvatures or deflections. Strain based ductility definition depends almost exclusively on the materials, while rotation or curvature based ductility definition also includes the effects of shape and size of the cross section. When the definition is applied to deflections, the entire configuration of structure and loading is also taken into account.

Necessity of ductility -

When a ductile structure subjected to overloading it will tend to deform in elastically and in doing so, will redistribute the excess load to elastic parts of the structure. This concept can be utilized in several ways.

1. If a structure is ductile, it can be expected to adapt to unexpected overloads, load reversals, impact and structural movements due to foundation settlements and volume changes. These items are generally ignored in the analysis and design but are assumed to have been taken care of by the presence of some ductility in the structure.
2. If a structure is ductile, its occupants will have sufficient warning of the impending failure thus reducing the probability of loss of life in the event of collapse.
3. The limit state design procedure assumes that all the critical sections in the structure will reach their maximum capacities at design load for the structure. For this to occur, all joints and splices must be able to withstand forces and deformations corresponding to yielding of the reinforcement.

Methods of improving ductility in R.C structure –

Selection of cross –section that will have adequate strength is rather easy. But it is much more difficult to achieve the desired strength as well as ductility. To ensure sufficient ductility , the designer should pat attention to detailing of reinforcement, bar cut-offs, splicing and joint details. Sufficient ductility can be ensured by following certain simple design details such as .

1. The structural layout should be simple and regular avoiding offsets of beams to columns. or offsets of columns from floor to floor. Changes in stiffness should be gradual from floor to floor.
2. The amount of tensile reinforcement in beams should be restricted and more compression reinforcement should be provided. The later should be enclosed by stirrups to prevent it from buckling.
3. Beams and columns in a reinforced concrete frame should be designed in such a manner that inelasticity is confined to beams only and the columns should remain elastic. To ensure this, sum of the moment capacities of the columns for the design axial loads at a beam column joint should be greater than the moment capacities of the beams along each principal plane.

$$\sum M_{column} > 1.2 \sum M_{beam}$$

4. The shear reinforcement should be adequate to ensure that the strength in shear exceeds the strength in flexture and thus, prevent a non-ductile shear failure before the fully reversible flexural strength of a member has been developed.

   Cluase 6.3.3 of IS : 13920 – 1993 requires that the shear resistance shall be the maximum of the :

   (a) Calculated factored shear forces as per analysis and

   (b) Shear force due to formation of plastic hinges at both ends plus the factored gravity loads on the span.

5. Closed stirrups or spiral should be used to confine the concrete at sections of maximum moment to increase the ductility of members. Such sections includes upper and lower end of columns, and within beam- column joints which do not have beams on all sides. If axial load exceeds 0.4 times the balanced axial load, a spiral column is preffered.
6. Splices and bar anchorages must be adequate to prevent bond failures.
7. The reversal of stresses in beams and columns due to reversal of direction of earthquake force must be taken into account in the design by appropriate reinforcement.
8. Beam-column connections should be made monolithic.

Detailing of beam as per I.A – 13920 Clause no . 6.1,6.2,6.3

5.2 For all buildings which are more than 3 storeys in height, the minimum grade of concrete shall preferably be M20 (f_{ck} = 20 MPa).

5.3 Steel reinforcements of grade Fe 415 (*see* IS 1786 : 1985) or less only shall be used.

6 FLEXURAL MEMBERS

6.1 General

These requirements apply to frame members resisting earthquake induced forces and designed to resist flexure. These members shall satisfy the following requirements.

6.1.1 The factored axial stress on the member under earthquake loading shall not exceed $0{\cdot}1\,f_{ck}$.

6.1.2 The member shall preferably have a width-to-depth ratio of more than 0·3.

6.1.3 The width of the member shall not be less than 200 mm.

6.1.4 The depth D of the member shall preferably be not more than 1/4 of the clear span.

6.2 Longitudinal Reinforcement

6.2.1 a) The top as well as bottom reinforcement shall consist of at least two bars throughout the member length.

b) The tension steel ratio on any face, at any section, shall not be less than $\rho_{min} = 0{\cdot}24\sqrt{f_{ck}}/f_y$; where f_{ck} and f_y are in MPa.

6.2.2 The maximum steel ratio on any face at any section, shall not exceed $\rho_{max} = 0{\cdot}025$.

6.2.3 The positive steel at a joint face must be at least equal to half the negative steel at that face.

6.2.4 The steel provided at each of the top and bottom face of the member at any section along its length shall be at least equal to one-fourth of the maximum negative moment steel provided at the face of either joint. It may be clarified that redistribution of moments permitted in IS 456 :1978 (clause 36.1) will be used only for vertical load moments and not for lateral load moments.

6.2.5 In an external joint, both the top and the bottom bars of the beam shall be provided with anchorage length, beyond the inner face of the column, equal to the development length in tension plus 10 times the bar diameter minus the allowance for 90 degree bend(s) (*see* Fig. 1). In an internal joint, both face bars of the beam shall be taken continuously through the column.

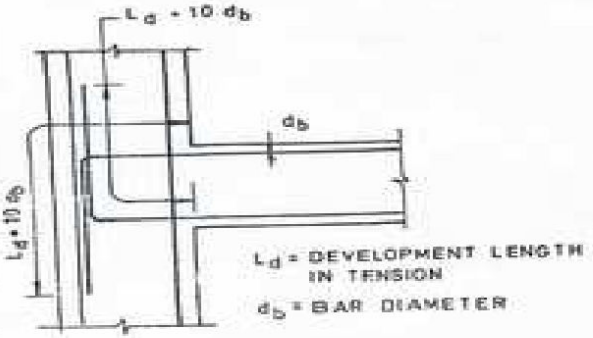


FIG. 1 ANCHORAGE OF BEAM BARS IN AN EXTERNAL JOINT

6.2.6 The longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at a spacing not exceeding 150 mm (*see* Fig. 2). The lap length shall not be less than the bar development length in tension. Lap splices shall not be provided (a) within a joint, (b) within a distance of 2*d* from joint face, and (c) within a quarter length of the member where flexural yielding may generally occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.

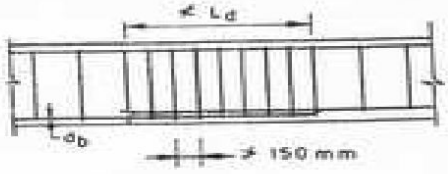


FIG. 2 LAP SPLICE IN BEAM

6.2.7 Use of welded splices and mechanical connections may also be made, as per 25.2.5.2 of IS 456 : 1978. However, not more than half the reinforcement shall be spliced at a section where flexural yielding may take place. The location of splices shall be governed by 6.2.6.

6.3 Web Reinforcement

6.3.1 Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end that is embedded in the confined core (*see* Fig. 3a). In compelling circumstances, it may also be made up of two pieces of reinforcement; a U-stirrup with a 135° hook and a 10 diameter extension (but not < 75 mm) at each end, embedded in the confined core and a crosstie (*see* Fig. 3b). A crosstie is a bar having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

6.3.2 The minimum diameter of the bar forming a hoop shall be 6 mm. However, in beams with clear span exceeding 5 m, the minimum bar diameter shall be 8 mm.

6.3.3 The shear force to be resisted by the vertical hoops shall be the maximum of :

a) calculated factored shear force as per analysis, and

b) shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span. This is given by (*see* Fig. 4):

i) for sway to right:

$$V_{u,a} = V_a^{D+L} - 1{\cdot}4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$

$$\text{and } V_{u,b} = V_b^{D+L} + 1{\cdot}4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right], \text{ and}$$

ii) for sway to left:

$$V_{u,a} = V_a^{D+L} + 1{\cdot}4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

$$\text{and } V_{u,b} = V_b^{D+L} - 1{\cdot}4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right],$$

where $M_{u,lim}^{As}$, $M_{u,lim}^{Ah}$ and $M_{u,lim}^{Bs}$, $M_{u,lim}^{Bh}$ are the sagging and hogging moments of resistance of the beam section at ends A and B, respectively. These are to be calculated as per IS 456 : 1978. L_{AB} is clear span of beam. V_a^{D+L} and V_b^{D+L} are the shears at ends A and B, respectively, due to vertical loads with a partial safety factor of 1·2 on loads. The design shear at end A shall be the larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear at end B shall be the larger of the two values of $V_{u,b}$ computed above.

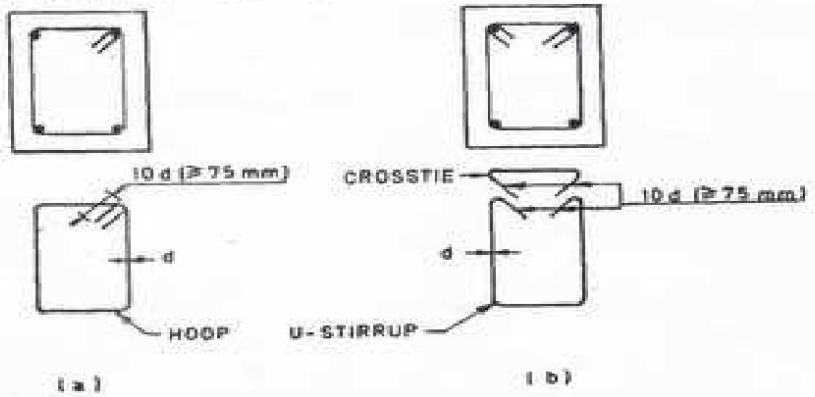


FIG. 3 BEAM WEB REINFORCEMENT

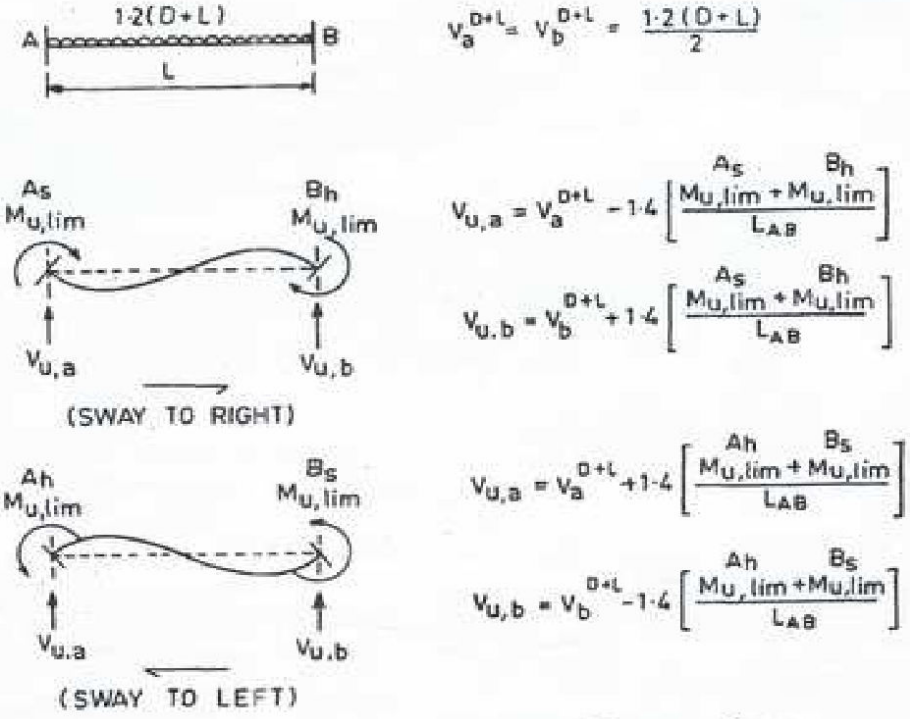


FIG. 4 CALCULATION OF DESIGN SHEAR FORCE FOR BEAM

6.3.4 The contribution of bent up bars and inclined hoops to shear resistance of the section shall not be considered.

6.3.5 The spacing of hoops over a length of 2d at either end of a beam shall not exceed (a) d/4, and (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm (*see* Fig. 5). The first hoop shall be at a distance not exceeding 50 mm from the joint face. Vertical hoops at the same spacing as above, shall also be provided over a length equal to 2d on either side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding d/2.

7.1.2 The minimum dimension of the membe shall not be less than 200 mm. However, i frames which have beams with centre to centr span exceeding 5 m or columns of unsupporte length exceeding 4 m, the shortest dimension the column shall not be less than 300 mm.

7.1.3 The ratio of the shortest cross section dimension to the perpendicular dimension sha preferably not be less than 0·4.

7.2 Longitudinal Reinforcement

7.2.1 Lap splices shall be provided only in t central half of the member length. It should proportioned as a tension splice. Hoops shall provided over the entire splice length at spaci not exceeding 150 mm centre to centre. N more than 50 percent of the bars shall

Detailing of beam as per I.A – 13920 Clause no . 7.1,7.2,7.3 , 7.4

6.3.4 The contribution of bent up bars and inclined hoops to shear resistance of the section shall not be considered.

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7 COLUMNS AND FRAME MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

7.1 General

7.1.1 These requirements apply to frame members which have a factored axial stress in excess of $0{\cdot}1 f_{ck}$ under the effect of earthquake forces.

7.1.2 The minimum dimension of the member shall not be less than 200 mm. However, in frames which have beams with centre to centre span exceeding 5 m or columns of unsupported length exceeding 4 m, the shortest dimension of the column shall not be less than 300 mm.

7.1.3 The ratio of the shortest cross sectional dimension to the perpendicular dimension shall preferably not be less than 0·4.

7.2 Longitudinal Reinforcement

7.2.1 Lap splices shall be provided only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be provided over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50 percent of the bars shall be spliced at one section.

7.2.2 Any area of a column that extends more than 100 mm beyond the confined core due to architectural requirements, shall be detailed in the following manner. In case the contribution of this area to strength has been considered, then it will have the minimum longitudinal and transverse reinforcement as per this code.

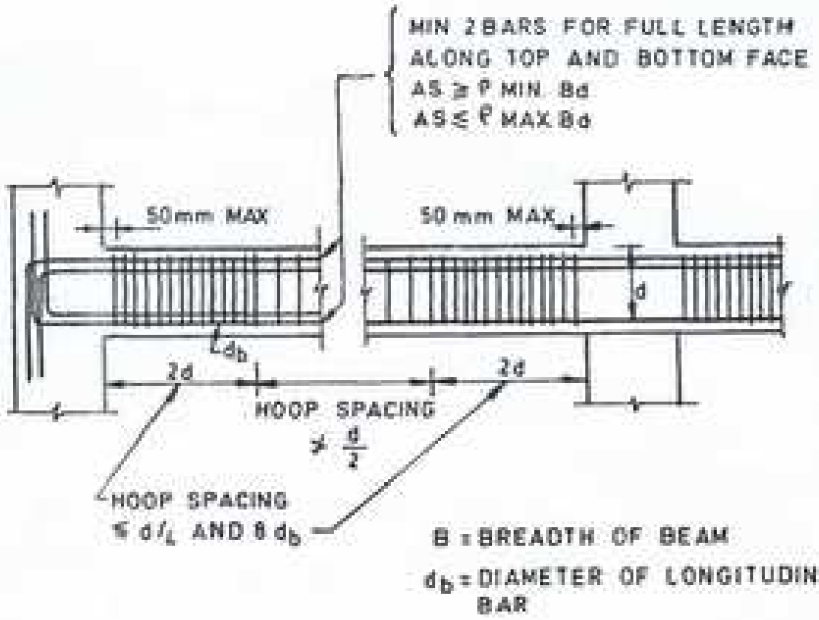


FIG. 5 BEAM REINFORCEMENT

However, if this area has been treated as non-structural, the minimum reinforcement requirements shall be governed by IS 456 : 1978 provisions minimum longitudinal and transverse reinforcement, as per IS 456 : 1978 (*see* Fig. 6).

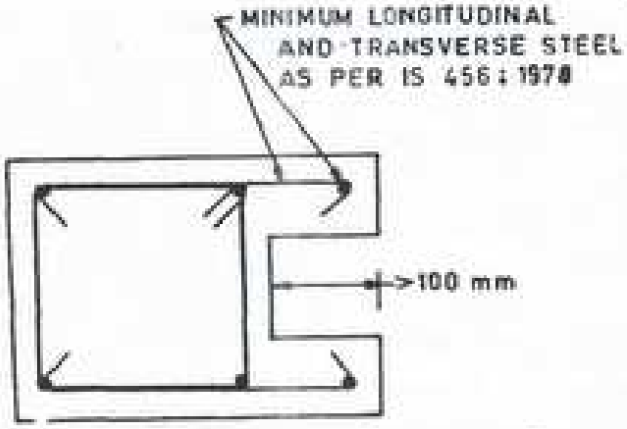


FIG. 6 REINFORCEMENT REQUIREMENT FOR COLUMN WITH MORE THAN 100 mm PROJECTION BEYOND CORE

7.3 Transverse Reinforcement

7.3.1 Transverse reinforcement for circular columns shall consist of spiral or circular hoops. In rectangular columns, rectangular hoops may be used. A rectangular hoop is a closed stirrup, having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end, that is embedded in the confined core (*see* Fig 7A).

7.3.2 The parallel legs of rectangular hoops shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, a crosstie shall be provided (Fig. 7B). Alternatively, a pair of overlapping hoops may be provided within the column (*see* Fig. 7C). The hooks shall engage peripheral longitudinal bars.

7.3.3 The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided, as per **7.4**.

7.3.4 The design shear force for columns shall be the maximum of:

a) calculated factored shear force as per analysis, and

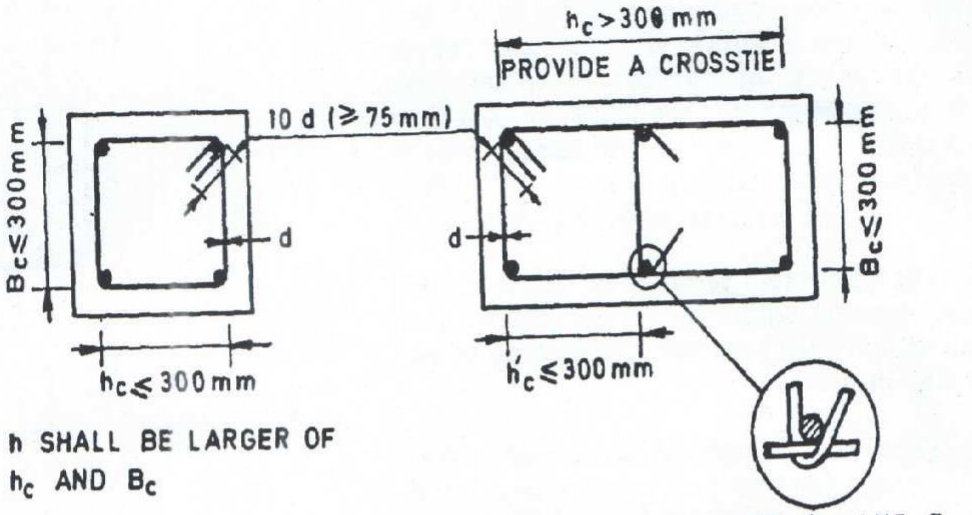
b) a factored shear force given by

$$V_u = 1.4\left[\frac{M^{bL}_{u,lim} + M^{bR}_{u,lim}}{h_{st}}\right]$$

where $M^{bL}_{u,lim}$ and $M^{bR}_{u,lim}$ are moment of resistance, of opposite sign, of beams framing into the column from opposite faces (*see* Fig. 8); and h_{st} is the storey height. The beam moment capacity is to be calculated as per IS 456 : 1978.

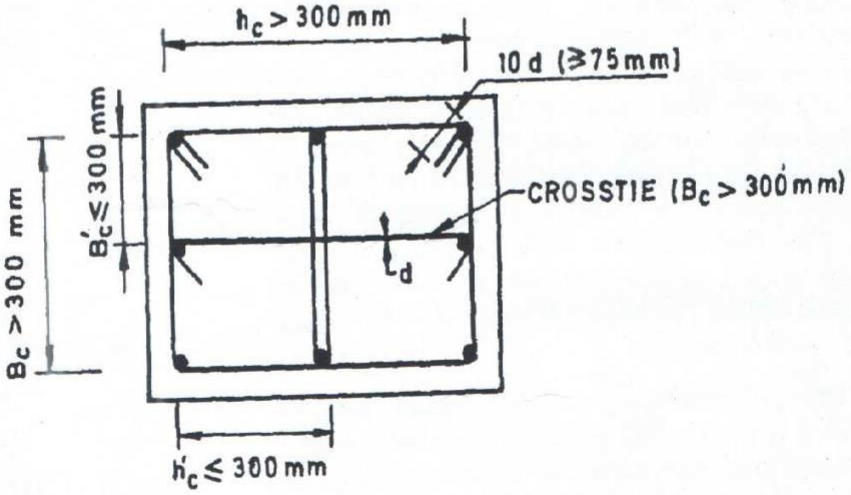
7.4 Special Confining Reinforcement

This requirement shall be met with, unless a larger amount of transverse reinforcement is required from shear strength considerations.



7A SINGLE HOOP

7B SINGLE HOOP WITH A CROSSTIE



7C OVERLAPPING HOOPS WITH A CROSSTIE

FIG. 7 TRANSVERSE REINFORCEMENT IN COLUMN

7.4.1 Special confining reinforcement shall be provided over a length l_0 from each joint face, towards midspan, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces (*see* Fig. 9.). The length 'l_0' shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) 1/6 of clear span of the member, and (c) 450 mm.

7.4.2 When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat (*see* Fig. 10).

7.4.3 When the calculated point of contra-flexure, under the effect of gravity and earthquake loads, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.

7.4.4 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (*see* Fig. 11). This reinforcement shall also be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length.

7.4.5 Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result

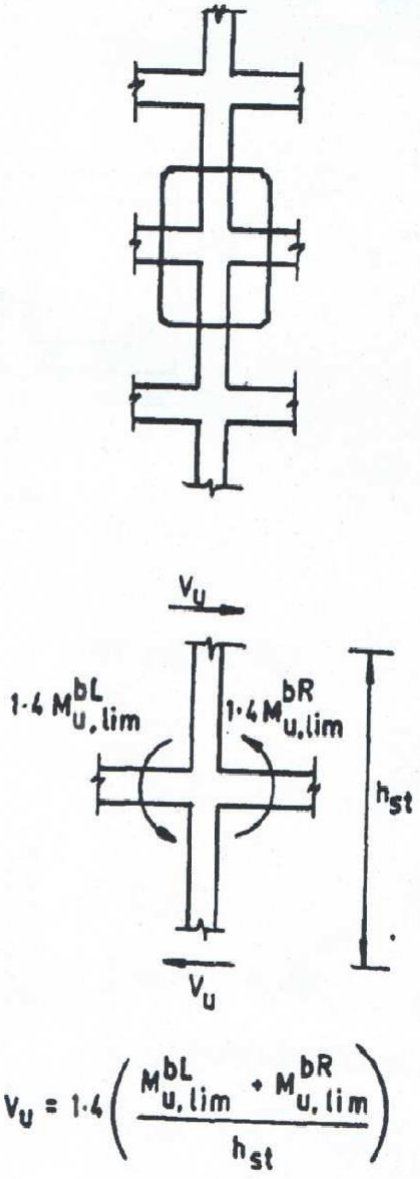


FIG. 8 CALCULATION OF DESIGN SHEAR FORCE FOR COLUMN

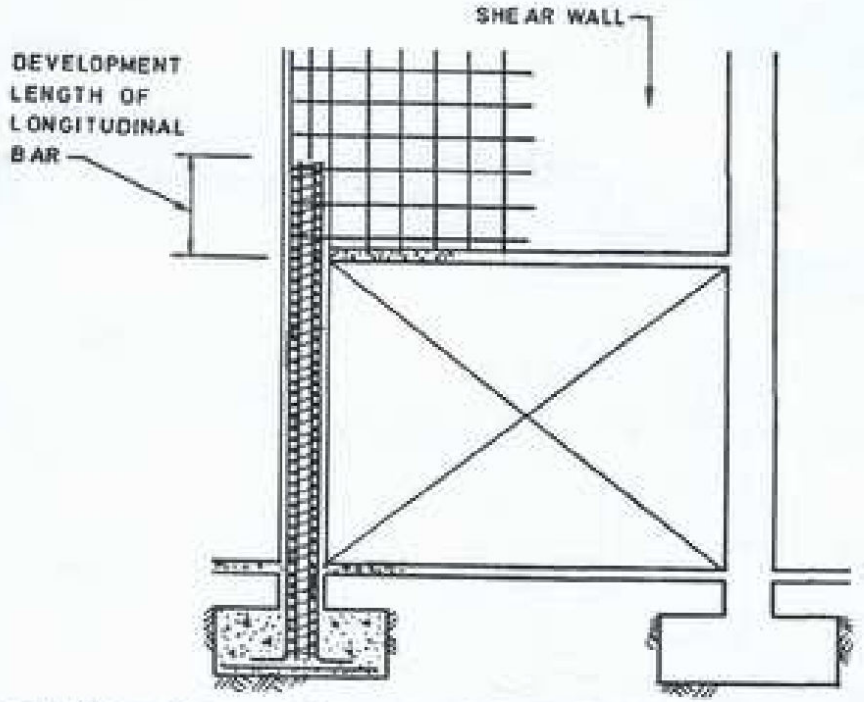


FIG. 11 SPECIAL CONFINING REINFORCEMENT REQUIREMENT FOR COLUMNS UNDER DISCONTINUED WALLS

due to the presence of bracing, a mezzanine floor or a R.C.C. wall on either side of the column that extends only over a part of the column height (*see* Fig. 12).

7.4.6 The spacing of hoops used as special confining reinforcement shall not exceed 1/4 of minimum member dimension but need not be less than 75 mm nor more than 100 mm.

7.4.7 The area of cross section, A_{sh}, of the bar forming circular hoops or spiral, to be used as special confining reinforcement, shall not be less than

$$A_{sh} = 0{\cdot}09\, SD_k \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1{\cdot}0\right]$$

where

A_{sh} = area of the bar cross section,

S = pitch of spiral or spacing of hoops,

D_k = diameter of core measured to the outside of the spiral or hoop,

f_{ck} = characteristic compressive strength of concrete cube,

f_y = yield stress of steel (of circular hoop or spiral),

A_g = gross area of the column cross section, and

A_k = area of the concrete core $= \frac{\pi}{4} D_k^2$

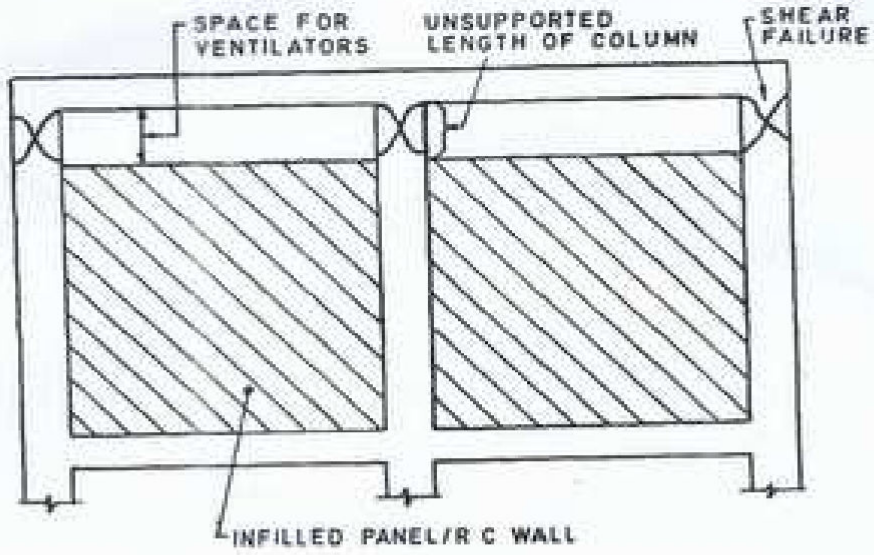
Example : Consider a column of diameter 300 mm. Let the grade of concrete be M20, and that of steel Fe 415, for longitudinal and confining reinforcement. The spacing of circular hoops, S, shall not exceed the smaller of (a) 1/4 of minimum member dimension = 1/4 × 300 = 75 mm, and (b) 100 mm. Therefore, S = 75 mm. Assuming 40 mm clear cover to the longitudinal reinforcement and circular hoops of diameter 8 mm, $D_k = 300 - 2 \times 40 + 2 \times 8 = 236$ mm. Thus, the area of cross section of the bar forming circular hoop works out to be 47·28 mm². This is less than the cross sectional area of 8 mm bar (50·27 mm²). Thus, circular hoops of diameter 8 mm at a spacing of 75 mm centre to centre will be adequate.

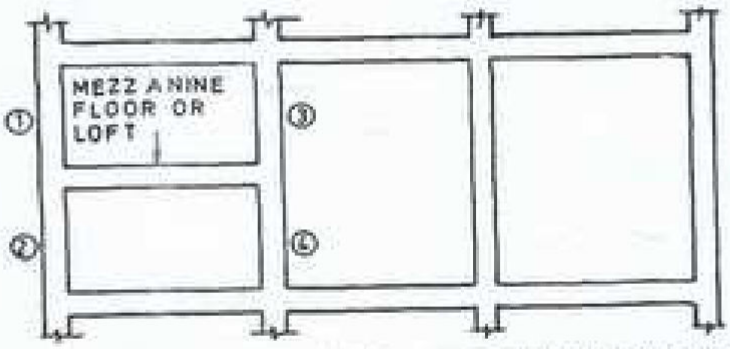
7.4.8 The area of cross section, A_{sh}, of the bar forming rectangular hoop, to be used as special confining reinforcement shall not be less than

$$A_{sh} = 0{\cdot}18\, Sh \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1{\cdot}0\right]$$

where

h = longer dimension of the rectangular confining hoop measured to its outer





(1), 2), (3) and (4) relatively stiff columns — They attract large seismic shear force.

FIG. 12 COLUMNS WITH VARYING STIFFNESS

face. It shall not exceed 300 mm (*see* Fig. 7), and

A_k = area of confined concrete core in the rectangular hoop measured to its outside dimensions.

NOTE : The dimension '*h*' of the hoop could be reduced by introducing crossties, as shown in Fig. 7B. In this case, A_k shall be measured as the overall core area, regardless of the hoop arrangement. The hooks of crossties shall engage peripheral longitudinal bars.

Example : Consider a column of 650 mm × 500 mm. Let the grade of concrete be M20 and that of steel Fe 415, for the longitudinal and confining reinforcement. Assuming clear cover of 40 mm to the longitudinal reinforcement and rectangular hoops of diameter 10 mm, the size of the core is 590 mm × 440 mm. As both these dimensions are greater than 300 mm, either a pair of overlapping hoops or a single hoop with crossties, in both directions, will have to be provided. Thus, the dimension '*h*' will be the larger of (i) 590/2 = 295 mm, and (ii) 440/2 = 220 mm. The spacing of hoops, *S*, shall not exceed the smaller of (a) 1/4 of minimum member dimensions = 1/4 × 500 = 125 mm, and (b) 100 mm. Thus, *S* = 100 mm. The area of cross section of the bar forming rectangular hoop works out to be 64·47 mm². This is less than the area of cross section of 10 mm bar (78·54 mm²). Thus, 10 mm diameter rectangular hoops at 100 mm c/c will be adequate. Similar calculations indicate that, as an alternative, one could also provide 8 mm diameter rectangular hoops at 70 mm c/c.

8 JOINTS OF FRAMES

8.1 The special confining reinforcement as required at the end of column shall be provided

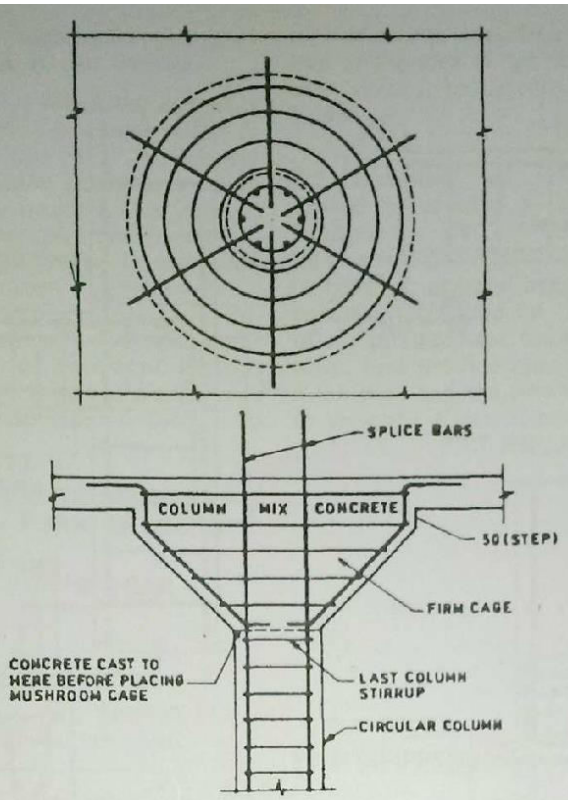


FIG. 7.12 REINFORCEMENT OF MUSHROOM HEADS

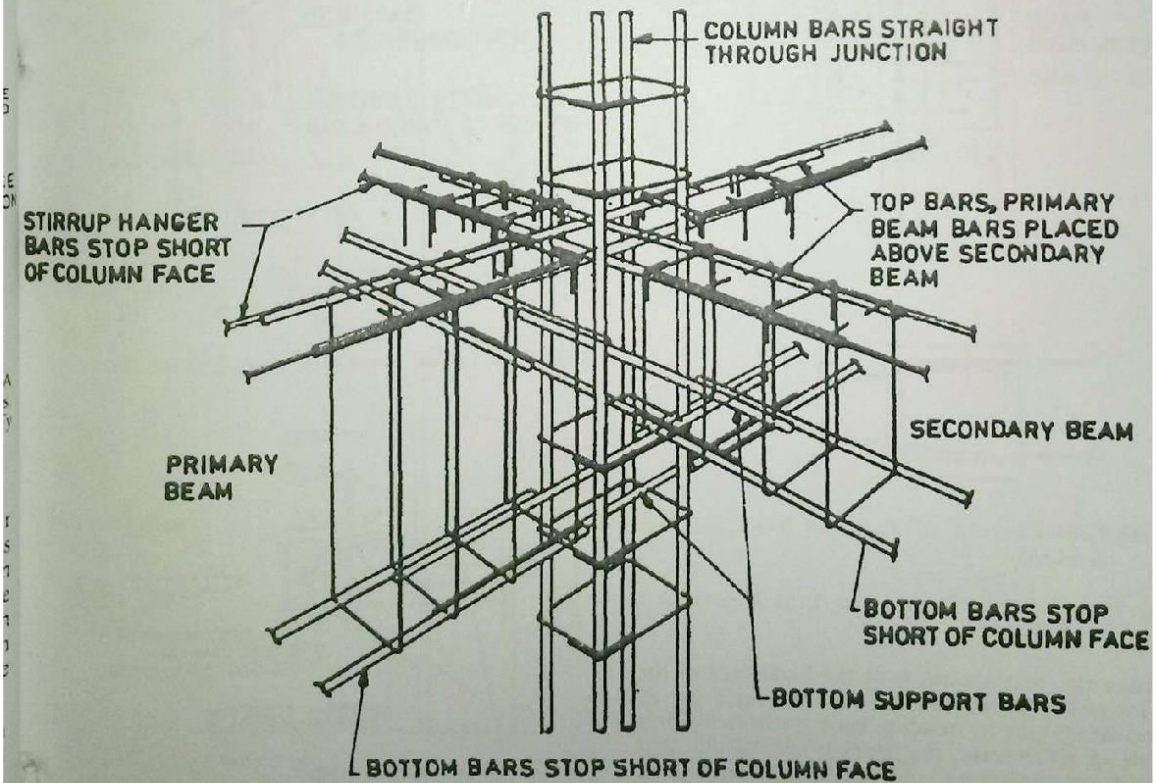
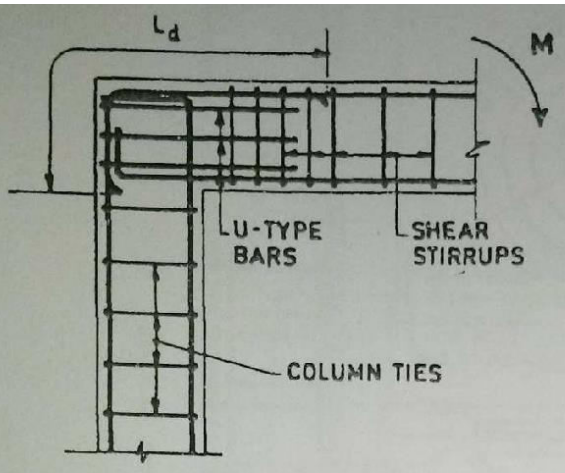
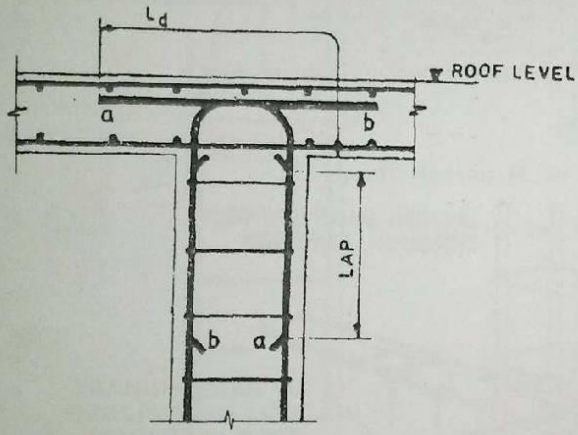
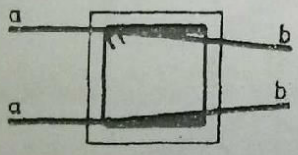


FIG. 7.13 BEAM-COLUMN INTERSECTION



7.14A FIXED END JOINT IN A COLUMN





7.14B TERMINATION OF COLUMN BARS INSIDE A SLAB

Fig. 7.14 TYPICAL DETAILS OF END JOINTS IN A COLUMN

Anchor the reinforcement at the front face of the corbel either by welding it to a transverse bar of equal stength or by bending back the bars to form loops; in either case, the bearing area of the load should not project beyond the straight portion of

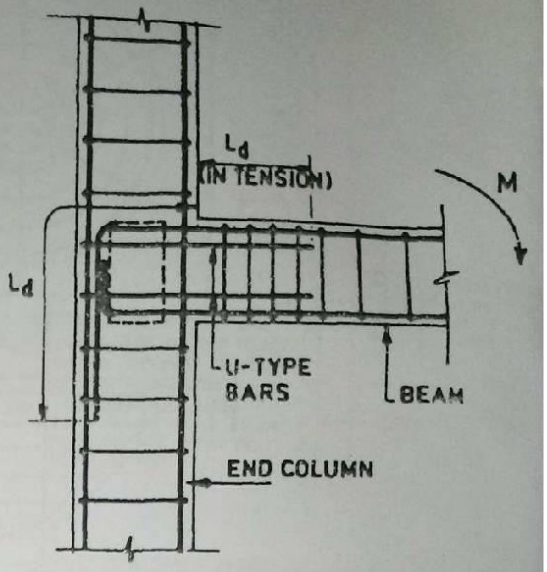


FIG. 7.15 TYPICAL DETAILS OF A BEAM-COLUMN JUNCTION AT EXTERIOR COLUMN

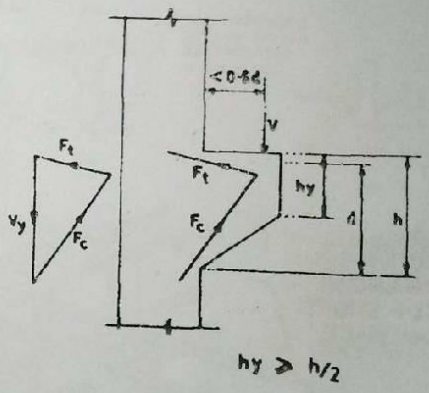
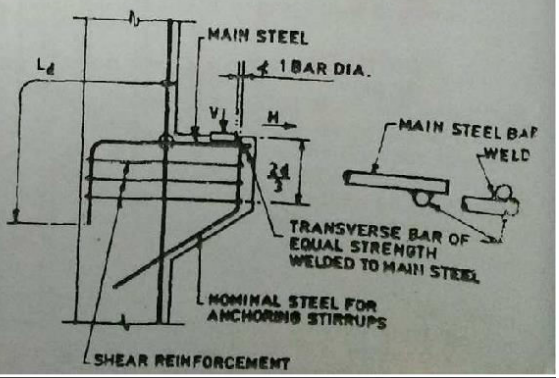


FIG. 7.16 TRUSS ANOLOGY FOR CORBEL DESIGN



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6.	IS:456-2000		BIS Publication
7.	SP_16		BIS Publication