GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING
GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

GEOTECHNICAL ENGINEERING

For 3rd Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)

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CHAPTER-1

INTRODUCTION

1.0 INTRODUCTION OF SOIL

The word "Soil" is derived from the Latin word solium which, according to Webster's dictionary, means the upper layer of the earth that may be dug or plowed specifically, the loose surface material of the earth in which plants grow. The above definition of soil is used in the field of agronomy where the main concern is in the use of soil for raising crops.

In geology, earth's crust is assumed to consist of unconsolidated sediments, called mantle or regolith, overlying rocks. The term 'soil' is used for the upper layer of mantle which can support plants. The material which is called soil by the agronomist or the geologist is known as top soil in geotechnical engineering or soil engineering. The top soil contains a large quantity of organic matter and is not suitable as a construction material or as a foundation for structures. The top soil is removed from the earth's surface before the construction of structures.

The term 'soil' in Soil Engineering is defined as an unconsolidated material, composed or solid particles, produced by the disintegration of rocks. The void space between the particles may contain air, water or both the solid particles may contain organic matter. The soil particles can be separated by such mechanical means as agitation in water.
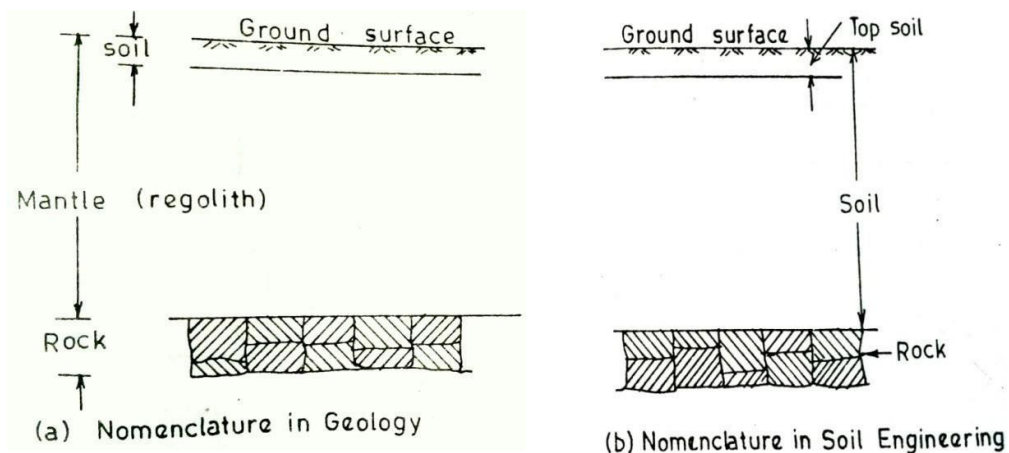

Fig. shows a cross-section through the earth's surface, indicating the nomenclature used in geology, and in Soil Engineering.

1.1 SOIL AND SOIL ENGINEERING:-

The term 'soil mechanics' was coined by Dr. Karl Terzaghi in 1925 when his book Erdbaumechanic on the subject was published in German.

According to Terzaghi, 'Soil mechanics is the application of the laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rock, regarding of whether or not they contain an admixture of organic constituents.

Soil mechanics is, a branch of mechanics which deals with the action of forces on soil and with the flow of water in soil.

The soil consists of discrete solid particles which are neither strongly bonded as in solids nor they are as free as particles of fluids. Consequently, the behavior of soil is somewhat intermediate between that of a solid and a fluid. It is not, therefore, surprising that soil mechanics draws heavily from solid mechanics and fluid mechanics. As the soil is inherently a particulate system. Soil mechanics is also called particulate mechanics.

## DEFINITION OF SOIL ENGINEERING AND GEOTECHNICAL ENGINEERING:-

Soil Engineering in an applied science dealing with the applications of principles of soil mechanics to practical problems. It has a much wider scope than soil mechanics, as it details with all engineering problems related with soils. It includes site investigations, design and construction of foundations, earth-retaining structures and earth structures.

1.2 SCOPE OF SOIL MECHANICS:-

Soil Engineering has vast application in the construction of various Civil Engineering works. Some of the important applications are as under.

(1) **Foundations:–** Every civil engineering structure, whether it is a building, a bridge, or a dam, is founded on or below the surface of the earth. Foundations are required to transmit the load of the structure to soil safely and efficiently.

A foundation is termed shallow foundation when it transmitted the load to upper strata of earth. A foundation is called deep foundation when the load is transmitted to strata at considerable depth below the ground surface. Pile foundation is a type of deep foundation. Foundation engineering is an important branch of soil engineering.

(a) Shallow foundation   (b) Pile foundation
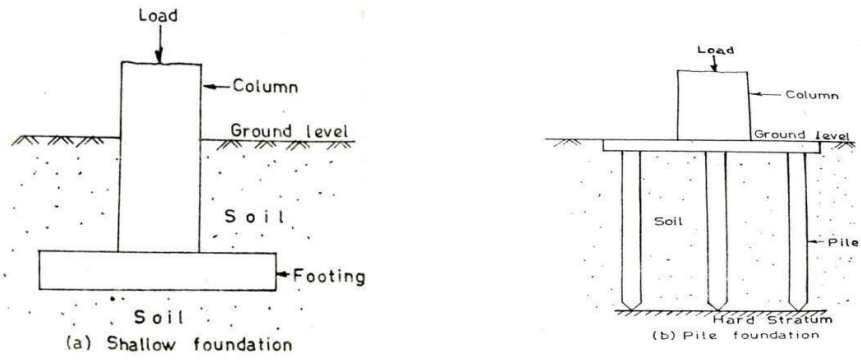
(2) **Retaining Structures:** – When sufficient space is not available for a mass of soil to spread and form a safe slope, a structure is required to retain the soil. An earth retaining structure is also required to keep the soil at different levels on its either side. The retaining structure may be rigid retaining wall or a sheet pile bulkhead which is relatively flexible (Fig. 1.3). Soil engineering gives the theories of earth pressure on retaining structures.


(a) Retaining wall   (b) Sheet pile
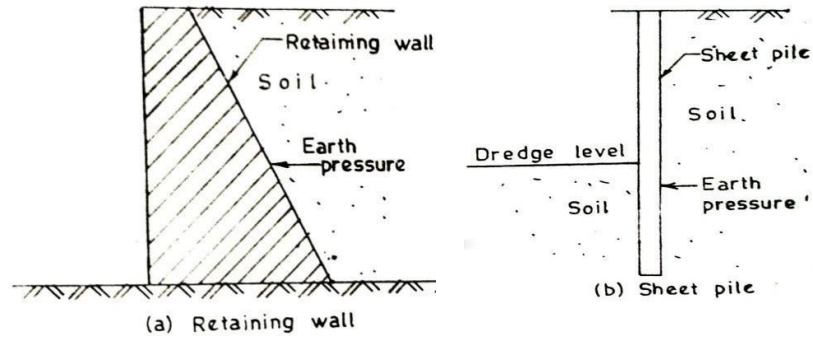
(3) **Stability of Slopes:** – If soil surface is not horizontal, there is a component of weight of the soil which tends to move it downward and thus causes instability of slope. The slopes may be natural or man-made Fig. 1.4 shows slopes in filling and cutting. Soil engineering provides the methods for checking the stability of slopes.

(a)   (b)
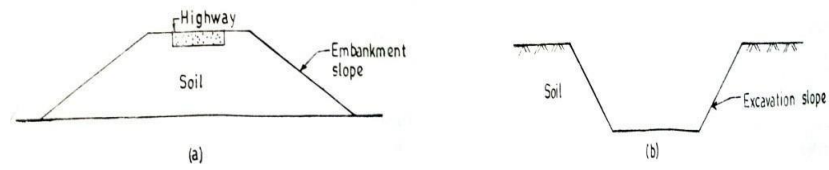
(4) **Underground structures:** – The design and construction of underground structures, such as tunnels, shafts, and conduits, require evaluation of forces exerted by the soil on these structures. These forces are discussed in soil engineering. Fig. 1.5 shows a tunnel constructed below the ground surface and a conduit laid below the ground surface.
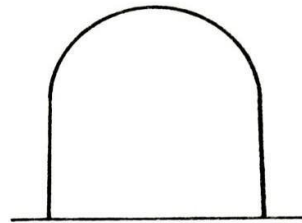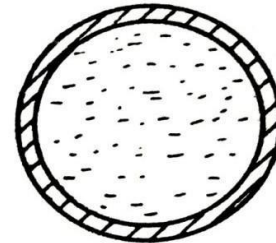
(a) Tunnel　　　　(b) Conduit

(5) **Pavement Design:** – A pavement is hard rust placed on soil (sub grade) for the purpose of proving a smooth and strong surface on which vehicles can move. The pavement consists of surfacing, such as a bitumen layer, base and subtheme (Fig. 1.6). The behavior of sub grade under vicious conditions of loading and environment change is studied in soil engineering.
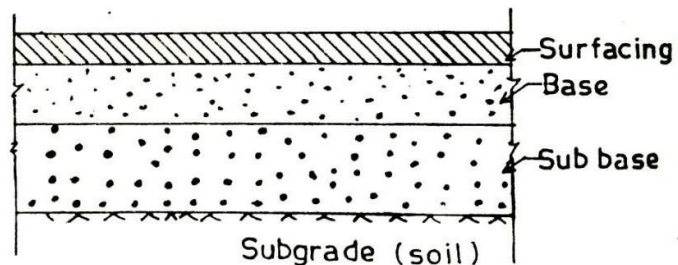
(6) **Earth Dom:** – Earth dams are the structures in which soil is used as a construction material. The earth dams are built for creating water reservoirs. Since the failure of an earth dam may cause widespread catastrophe care is taken in its design and construction. It requires thorough knowledge at soil engineering.
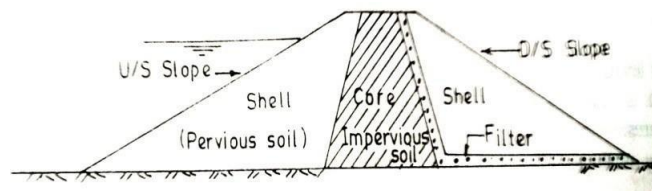
(7) **Miscellaneous soil: -** The geotechnical engineer has sometimes to tackle miscellaneous problems related with soil. Such as soil heave, soil subsidence, frost heave, shrinkage and swelling of soils.

CHAPTER-2

PRELIMINARY DEFINATION AND RELATIONSHIP

2.1 SOIL AS A THREE PHASE:

A soil mass consists of solid particles which from a porous structure. The voids in the soil mass may be filled with air, with water or partly with air and partly with water. In general, a soil mass consists of solid particles, water and air. The three constituents are blended together to form a complex material (Fig. 2.1. a) . However, for convenience, all the solid particles are segregated and placed in the lower layer of the three-phase diagram (Fig. 2.1b). Likewise, water and air particles are placed separately, as shown. The 3-phase diagram is also known as Block diagram.

It may be noted that the constituents cannot be

(a)
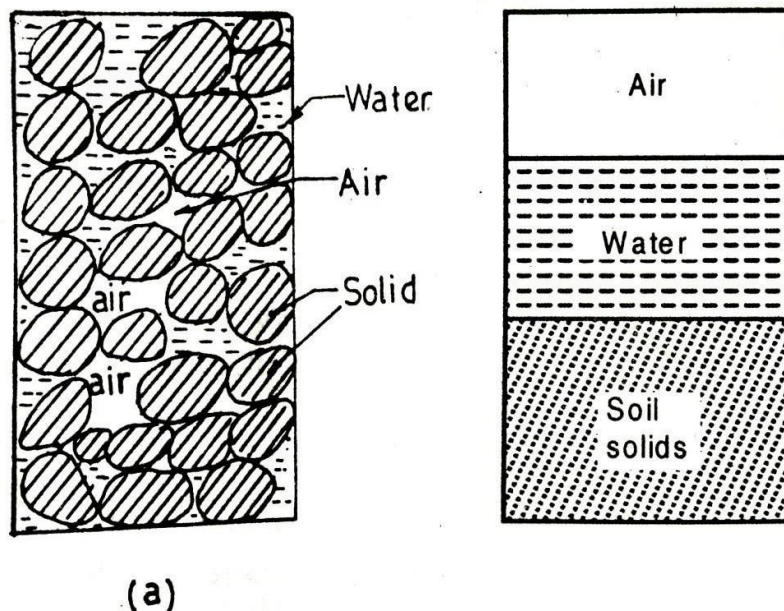
Actually segregated, as shown. A 3-phase diagram is an artifice used for easy understand and convenience in calculation.

Although the soil is a three-phase system, it becomes a two-phase system in the following two cases :(1) When the soil is absolutely dry, the water phase disappears (Fig. 2.2a). (2) When the soil is fully saturated three is no air phase (Fig.2.2b). It is the relative proportion of the three constituents and their interaction that governs the behavior and properties of soils. The phase diagram is a simple, diagrammatic representation of a real soil, It is extremely useful for studying the various terms used in soil engineering and their interrelationships.

In a 3-phase diagram it is conventional to write values on the left side and the mass on the right side (Fig.2.3a). The mail simple of a gives soil mass in designated as V. It is equal to the sum of the volume of solids (V1), the volume of water (F) and the volume of air (Va). The volume of voids (Vv) is equal to the sum of the volumes, the water and air
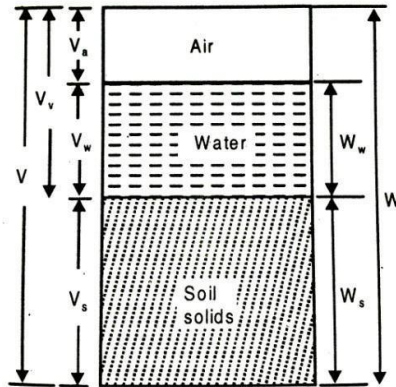
The soil mass of the soil… is represented as M. the mass of air (ma) is very small and is neglected. Therefore, the social mass of the soil is equal to the mass of solids (M2) and the mass of water (Mw). Fig.2.36 shows the 3-phase diagram in which the weights are written on the right side.
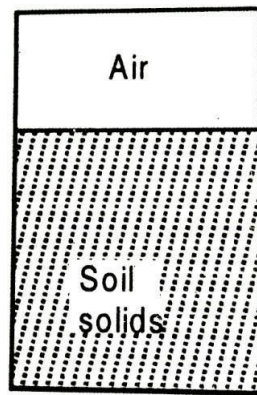
(a)

(b)

2.2 WEIGHT VOLUME RELATIONSHIPS:
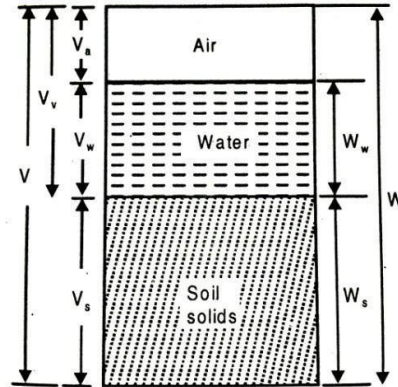
WATER CONTENT: - The water content (w) is defined as the ratio of the mass water to the mass of solids.

$$w = \frac{Ww}{Ws}$$

The water content is also known as the moisture content (m). It is expressed as a percentage, but whenever used in equation used as a decimal.

The water content of the fine-grained soils, such as silts and clays, in generally more than that of the coarse grained soils, such as gravels and sands.

The water content of some of the fine-gained soils may be even more than 100%, which indicates that more than 50% of the total mass is that of water. The water content of a soil is an important property.

SPECIFIC GRAVITY (G):-

The specific gravity of solid particles (G) is defined as the ratio of the mass of a given volume of solids to the mass of an equal volume of water at 4^0C. Thus, the specific gravity is given by $G = \dfrac{\rho s}{\rho w}$

*The mass density of water ρw at 4^0C is one gm/ml, 1000 kg/m3 or 1 Mg/m^3.

The specific gravity of solids for most natural soils falls in the general range of 2.65 to 2.80, the smaller values are for the coarse-grained soils. Table gives the average values of specific gravity for different soils. It may be mentioned that the specific gravity of different particles in a soil mass may not be the same. Whenever

the specific gravity of a soil mass is indicated, it is the average value of all the solid particles present in the soil mass. Specific gravity of solids is an important parameter. It is used for determination of void ratio and particle size.

Table: Typical Values of G

Sl.No.	Soil Type	Specific Gravity
1	Grevel	2.65-2.68
2	Sand	2.65-2.68
3	Sands	2.66-2.70
4	Slit	2.66-2.70
5	Inartistic Clays	2.68-2.80
6	Organic Soils	Variable, may fall below 2.00

Besides the following two terms related with the specific gravity are also used.

(1) **Mass Specific Gravity (G_m):-** It is defined as the ratio of the mass density of the soil to the mass density of water.

The value of the mass specific gravity of a soil is much smaller than the value of the specific gravity of solids.

The mass specific gravity is also known as the apparent specific gravity or the bulk specific gravity.

(2) **Absolute Specific Gravity (G_a):-** The soil solids are not perfect solids but contain voids. Some of these voids are permeable through which water can enter, whereas others are impermeable. Since the permeable voids get filled when the soil is wet, these are in reality a part of void space in the total mass and not apart of soil solids. If both the permeable and impermeable voids are excluded from the volume of solids ,the remaining volume is the true or absolute volume of the solids.

The mass density of the absolute solids is used for the determination of the absolute specific gravity of solids.

VOIDS RATIO: - It is defined as the ratio of the volume of voids to the volume of solids.

$$e = \frac{V_v}{V_s}$$

The void ratio is expressed as a decimal, such as 0.4,0.5, etc. For coarse-grained soils, the void ratio is generally smaller than that for fine-grained soils. For some soils, void ratio may have a value even greater than unity.

POROSITY:- It is defined as the ratio of the volume of voids to the total volume.

$$n = \frac{Vv}{V}$$

The Porosity is generally expressed as percentage. However, in equations, it is used as a ratio. For example; a porosity of 50% will be used as 0.5 in equations. The porosity of a soil cannot exceed 100% as it would mean Vv is greater than V, which is absurd. Porosity is also known as percentage voids.

Both porosity and void ratio are measures of the denseness (or looseness) of soils. As the soil becomes more and more dense, their values decrease. The term porosity is more commonly used in other disciplines such as agricultural engineering. In soil engineering, the term void ratio is more popular. It is more convenient to use void ratio than porosity. When the volume of a soil mass changes, only the numerator (i.e.Vv) in the void ratio changes and the denominator (i.e.Vs) remains constant. However, if the term porosity is used, both the numeration and the denominator change and it become inconvenient.

Relationship between the void ratio and the porosity as under.

$$\frac{1}{n} = \frac{V}{Vv} = \frac{Vv+Vs}{Vv}$$

$$\frac{1}{n} = 1 + \frac{1}{e} = \frac{1+e}{e}$$

$$n = \frac{e}{1+e} \text{----------(1)}$$

$$\frac{1}{e} = \frac{1}{n} - 1 = \frac{1-n}{n}$$

$$e = \frac{n}{n} \text{---------(2)}$$

The porosity should be expressed as a ratio (and not percentage).

PERCENTAGE OF AIR VOIDS (n_a):-

It is the ratio of the volume of air to the total volume.

$$n_a = \frac{Va}{V}$$

It is represented as a percentage.

AIR CONTENT (a_c) : - Air content is defined as the ratio of the volume of air to the volume of voids.

$$a_c = \frac{Va}{Vv}$$

The Air content is usually expressed as a percentage. Both air content and the percentage air voids are zero when the soil is saturated ($V_a = 0$).

Relationship between the percentage air voids and the air content can be obtained.

$$n_a = \frac{V_a}{V} = \frac{V_a}{V_v} \times \frac{V_v}{V}$$

$$n_a = n \times ac$$

Degree of Saturation (S) - The degree of saturation (S) is the ratio of the volume of water to the volume of voids. It is also represents as S_r.

$$S = \frac{V_w}{V_v}$$

The degree of saturation is generally expressed as a percentage.

It is equal to zero when the soil is absolutely dry and 100% when the soil is fully saturated. Degree of saturation is used as a decimal in expressions.

DENSITY INDEX:-

It the most important index property of a cohesion less soil.

It is also known as Density Index (I_D). It is also known as relative density or degree of density. It is used to express the relative compactness of a natural soil deposit.

It is the ratio of the difference between voids ratio of the soil in its loosest state and its natural voids ratio to the difference between voids ratio in the loosest and densest states.

$$I_D \text{ or } Dr = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

Where e_{max} = Maximum void ratio of the soil in the loosest condition.

e_{min} = Minimum void ratio of the soil in the densest condition.

e = void ratio of the soil in the natural state.

e_{max} will found out from γmin i,e in the loosest condition.

e_{min} will found out from γmax i,e in the densest condition.

e will found out from γd i,e in the natural condition.

$$Dr = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

$$Dr = \gamma_d = \frac{\frac{G\gamma w}{\gamma min}-1}{1+\frac{7v}{7s}} = \frac{G\gamma w}{1+e}$$

$$\gamma_d = \frac{G\gamma w}{1+e}$$

e_{max} The relative density of a soil gives a more clear idea of the denseness than does the void ratio.

BULK UNIT WEIGHTY:-

The bulk unit weight is defined as the total weight per total volume.

$$\gamma = \frac{W}{V}$$

It is also known as total unit weight (γ_t) or wet unit weight.

In SI units it is express as N/mm^3 or KN/mm^3

DRY UNIT WEIGHT:-

The dry unit weight is defined as the weight of solids per total volume.

$$\gamma_d = \frac{Ws}{V}$$

SATURATED UNIT WEIGHT:-

The saturated unit weight is the bulk unit weight when the soil is fully saturated.

$$\gamma_{sat} = \frac{Wsat}{V}$$

SUBMERGED UNIT WEIGHT: -

When the soil exists below water than it is called submerged condition.

The submerged unit weight (γ') of the soil is defined as the submerged weight per total volume.

$$\gamma_{sub} = \frac{Wsub}{V}$$

Relation between G, S, e, γ & γ_w:-

$$\gamma = \frac{W}{V} = \frac{Ws+Ww}{Vs+Vv} = \frac{GVs\gamma w + Vw\gamma w}{Vs+Vv}$$

Dividing Vs both numerator & denominator

Or, $$\gamma = \frac{G\gamma w + (7w)\gamma w}{1+\frac{7v}{7s}} = \frac{(G+\frac{7w}{7v}\frac{7v}{7s}x)\gamma w}{1+\frac{7v}{7s}} = \frac{(G+Se)\gamma w}{1+\frac{7v}{7s}}$$

Or, $\gamma = \dfrac{(G+S\,e)\gamma w}{1+e}$ ....(3)

As the degree of saturation is 100% for a saturated soil, then the S= 1

Or, $\gamma_{sat} = \dfrac{(G+e)\gamma w}{1+e}$ ....(4)

Similarly, $\gamma_d = \dfrac{Ws}{V} = \dfrac{Ws}{Vs+Vv} = \dfrac{GVs\gamma w}{Vs+Vv}$

Dividing Vs both numerator & denominator

$$\gamma d = \dfrac{G\gamma w}{1+\frac{\gamma v}{\gamma s}} = \dfrac{G\gamma w}{1+e} \quad\text{..........................(5)}$$

We known that $\gamma_{sub} = \gamma_{sat} - \gamma_w$

$$= \dfrac{(G+e)\gamma w}{1+e} - \gamma_w = \dfrac{(G-1)\gamma w}{1+e}$$

$$\gamma_{sub} = \dfrac{(G-1)\gamma w}{1+e} \quad\text{.......................(6)}$$

Relation Between e, w, G & S :-

We known that $w = \dfrac{Ww}{Ws}$ $\qquad$ (As $\gamma w = \dfrac{Ww}{Vw}$ & $\gamma s = \dfrac{Ws}{Vs}$)

$$= \dfrac{Vw\gamma w}{Vs\gamma s}$$

$$= \dfrac{Vw}{Vv} X \dfrac{Vv}{Vs} X \dfrac{\gamma w}{\gamma s}$$

$$= S \cdot e \cdot \dfrac{1}{G} \qquad (\text{As } S = \dfrac{Vw}{Vv}, e = \dfrac{Vv}{Vs} \text{ \& } G = \dfrac{\gamma s}{\gamma w})$$

$$w.G = S \cdot e$$

$$e = \dfrac{w.G}{S}$$

Relation between e, S & n_a :-

$$n_a = \dfrac{Va}{V} = \dfrac{Vv-Vw}{Vv+Vs}$$

Dividing Vs both numerator & denominator

$$= \dfrac{\frac{\gamma v}{\gamma s}-\frac{\gamma w}{\gamma s}}{\frac{\gamma v}{\gamma s}+\frac{\gamma w}{\gamma s}} = \dfrac{\frac{\gamma v}{\gamma s}-\frac{\gamma w}{\gamma v}X\frac{\gamma v}{\gamma s}}{\frac{\gamma v}{\gamma s}+\frac{\gamma w}{\gamma s}} = \dfrac{e-S\,e}{e+1}$$

$$n_a = \dfrac{(1-S)e}{1+e} \quad\text{.........(7)}$$

Relation between S & a_c :-

$$a_c = \frac{Va}{Vv} = \frac{Vv-Vw}{Vv} = 1 - \frac{Vw}{Vv} = 1 - S \dots\dots\dots\dots(8)$$

Relation between Na, a_c & n:-

$$n_a = \frac{Va}{V} = \frac{Va}{Vv} \cdot \frac{Vv}{V} = a_c \cdot n \dots\dots\dots\dots\dots(9)$$

Relation between G, n_a, e, γ_d & γ_w :-

$$V = Vs+Vv+Va$$

$$1 = \frac{Vs}{V} + \frac{Vv}{V} + \frac{Va}{V} = \frac{Vs}{V} + \frac{Vv}{V} + n_a$$

$$1 - n_a = \frac{Vs}{V} + \frac{Vv}{V} = \frac{Ws/G\gamma w}{V} + \frac{Ww/\gamma w}{V}$$

$$= \frac{\gamma d}{G\gamma w} + \frac{wWs/\gamma w}{V}$$

$$= \frac{\gamma d}{G\gamma w} + \frac{w\gamma d}{\gamma w} = (\frac{1}{G} + w) \cdot \frac{\gamma d}{\gamma w}$$

$$\gamma d = \frac{(1-na)\,G\gamma w}{1+wG} \dots\dots\dots\dots\dots(10)$$

Relation between w, γd & γ:-

$$w = \frac{Ww}{Ws}$$

$$Or, \quad 1+w = 1 + \frac{Ww}{Ws} = \frac{Ws+Ww}{Ws}$$

$$Or, W_s = \frac{W}{1+w}$$

$$Or, \gamma_{d.}V = \frac{W}{1+w}$$

$$Or, \gamma_{d.} = \frac{W}{(1+w)V} = \frac{y}{(1+w)}$$

$$Or, \gamma_{d.} = \frac{y}{(1+w)} \dots\dots.(11)$$

CHAPTER-3

DETERMINATION OF INDEX PROPERTIES

WATER CONTENT DETERMINATION:

- The water content of a soil is an important parameter that controls its behaviour.
- It is a quantitative measure of the wetness of a soil mass.
- The water content of soil mass can be determined by the following methods:
1. Oven drying method
2. Pycnometer method

(1) Oven drying method:

- The oven drying method is a standard laboratory method and this is a very accurate method.
- In this method the soil sample is taken in a small, non-corrodible, air tight container.
- The mass of the sample and that of the container are obtained using an accurate weighing balance.
- The soil sample in the container is then dried in an oven at a temperature of $110°c \pm 5°c$ for 24 hours.
- The water content of the soil sample is then calculated from the following equation:

$W = M_w/M_s$

$= \dfrac{M_2 - M_3}{M_3 - M_1} \times 100$

Where M_1 = mass of container with lid

M_2 = mass of container, lid and wet soil

M_3 = mass of container, lid and dry soil
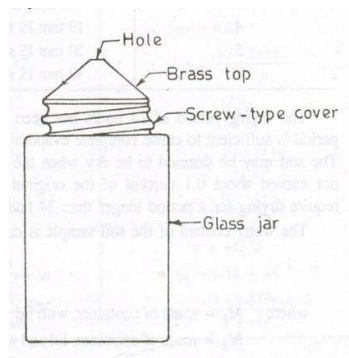
(2) Pycnometer method:

Figure 3.1
Pycnometer

- A pycnometer is a glass jar of about 1 litre capacity and fitted with a brass conical cap by means of screw type cover.
- The cap has a small hole of 6 mm diameter at its apex. A rubber or fibre washer is placed between the cap and the jar to prevent leakage.
- There is a mark on the cap and also on the jar. The cap is screwed down to the same mark such that the volume of the pycnometer used in the calculations remains constant.
- The pycnometer method for the determination of water content can be used only if the specific gravity of solid particle is known.
- A sample of wet soil about 200 to 400 gm is taken in the pycnometer and weighed.
- Water is then added to the soil in the pycnometer to make it about half full.
- The contents are thoroughly mixed using a glass rod to remove the entrapped air. More and more water is added and stirring process continued till the pycnometer is filled flush with the hole in the conical cap.
- The pycnometer is wiped dry and weighed.
- The pycnometer is then completely emptied. It is washed and filled with water, flush with the top hole.
- The pycnometer is wiped dry and weighed.
- Let M_1 = mass of pycnometer

M_2 = mass of pycnometer + wet soil

M_3 = mass of pycnometer + wet soil + water

M_4 = mass of pycnometer filled with water only

The mass M_4 is equal to mass M_3 minus the mass of solids M_s plus the mass of an equal volume of water.

$$M_4 = M_3 - M_s + \frac{M_s}{G \rho_w} \rho_w$$

$$M_4 = M_3 - M_s + \frac{M_s}{G}$$

$$= M_3 - M_s \left(1 - \frac{1}{G}\right)$$

$$M_s = (M_3 - M_4)\left(\frac{G}{G-1}\right)$$

Mass of wet soil = $M_2 - M_1$

Therefore mass of water $M_w = (M_2 - M_1) - (M_3 - M_4)\left(\frac{G}{G-1}\right)$

$$w = \frac{M_w}{M_s} \times 100$$

$$= \left[\left(\frac{M2-M1}{M3-M4}\right)\left(\frac{G-1}{G}\right) - 1\right] \times 100$$
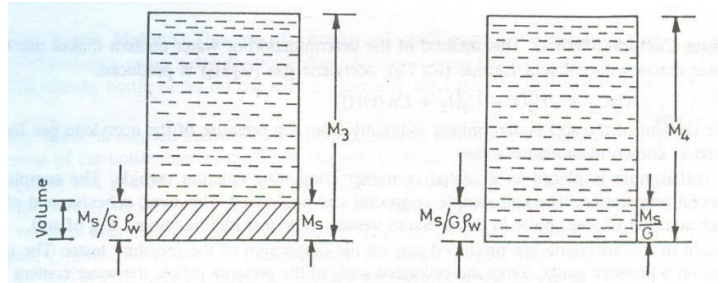
Figure 3.2

Pycnometer method Derivation

- This method for the determination of the water content is suitable for coarse grained soils from which the entrapped air can be easily removed.

SPECIFIC GRAVITY:

- The specific gravity of soil solids can be determined by :
  - (i)   A 50 ml density bottle or
  - (ii)   A 500 ml flask or
  - (iii) A pycnometer
- The density bottle method is the most accurate and is suitable for all types of soil.
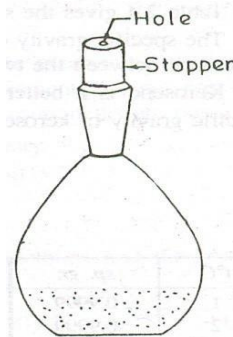
Figure 3.3

Density bottle

- The flask or pycnometer is used only for coarse grained soils.
- The density bottle method is the standard method used in the laboratory.
- In the above three methods the sequence of observation is same.
- The mass M_1 of the empty, dry, bottle is first taken.
- A sample of oven dried soil cooled in a desiccator is put in the bottle and the mass M_2 is taken.

- The bottle is then filled with distilled water gradually removing the entrapped air either by applying vacuum or by shaking the bottle.
- The mass M3 of the bottle, soil and water is taken.
- Finally, the bottle is emptied completely and thoroughly washed and clean water is filled to the top and the mass M_4 is taken.
- If the mass of solid M_s is subtracted from M_3 and replaced by the mass of water equal to the volume of solid the mass M_4 is obtained.

$M_4 = M_3 - Ms + \frac{Ms\rho w}{G\rho w}$

Or, $M_s (1 - \frac{1}{G}) = M_3 - M_4$

But $M_s = M_2 - M_1$

Therefore $(M_2 - M_1) (1 - \frac{1}{G}) = M_3 - M_4$

Or $\frac{1}{G} (M_2 - M_1) = (M_2 - M_1) - (M_3 - M_4)$

Or $G = \frac{M_2 - M_1}{(M_2 - M_1) - (M_3 - M_4)}$ ---------------- (i)

Alternatively $G = \frac{Ms}{Ms + M4 - M3}$ --------------- (ii)

Equation (1) gives the specific gravity of solids at the temperature at which the test was conducted.

Specific gravity of solids is generally reported at 27°C or at 4°C. The specific gravity at 27°C and 4°C can be determined from the following equation

$G_{27} = G_t \times \frac{\text{specific gravity of water at t°C}}{\text{specific gravity of water at 27°C}}$

And $G_4 = G_t \times$ specific gravity of water at t°C

Where G_{27} = specific gravity of particles at 27°C

G_4 = specific gravity of particles at 4°C

G_t = specific gravity of particles at t°C

PARTICLE SIZE DISTRIBUTION:
- The percentage of various sizes of particle in a given dry soil sample is found by a particle size analysis or mechanical analysis.
- Mechanical analysis means separation of a soil into its different size fractions.
- The mechanical analysis is performed in two stages
  (i)   Sieve analysis
  (ii)   Sedimentation analysis or wet mechanical analysis
- The first stage is meant for coarse grained soil only while the second stage is performed for fine grained soils.
- In general a soil sample may contain both coarse grained particles as well as fine Particles and hence both the stages of the mechanical analysis may be necessary.

Sieve Analysis:

- In the Indian standard the sieves are designated by the size of the aperature in mm.
- The sieve analysis can be divided into two parts i.e the coarse analysis and fine analysis.
- An oven dried sample of soil is separated into two fractions by sieving it through a 4.75 mm I.S sieve.
- The portion retained on it is termed as the gravel fraction and is kept for the coarse analysis while the portion passing through it is subjected to fine sieve analysis.
- The following sets of sieves are used for coarse sieve analysis: IS: 100, 63, 20, 10 and 4.75 mm.
- The sieves used for fine sieve analysis are : IS : 2 mm, 1.0 mm, 600, 425, 300, 212, 150 and 75 micron.
- Sieving is performed by arranging the various sieves one over the other in the order of their mesh openings – the largest aperature sieve being kept at the top and the smallest aperature sieve at the bottom.
- A receiver is kept at the bottom and a cover is kept at the top of whole assembly.
- The soil sample is put on the top sieve and the whole assembly is fitted on a sieve shaking machine.
- The amount of shaking depends upon the shape and the number of particles.
- At least 10 minutes of shaking is desirable for soils with small particles.
- The portion of the soil sample retained on each sieve is weighed.
- The percentage of soil retained on each sieve is calculated on the basis of the total mass of soil sample taken and from this percentage passing through each sieve is calculated.

Sedimentation Analysis:

- In the wet mechanical analysis or sedimentation analysis the soil fraction finer than 75 micron size is kept in suspension in a liquid (usually water) medium.
- The analysis is based on stokes law according to which the velocity at which grains settle out of suspension, all other factors being equal, is dependent upon the shape, weight and size of the grain.
- However in the usual analysis it is assumed that the soil particles are spherical and -have the same specific gravity.
- With this assumption the coarser particles settle more quickly than the finer ones.

- If v is the terminal velocity of sinking of a spherical particle it is given by

$$v = \frac{2r^2}{9} \frac{\gamma_s - \gamma_w}{\eta}$$

$$\text{Or } v = \frac{1}{18} \frac{D^2}{\eta} \frac{\gamma_s - \gamma_w}{\eta}$$

Where r = radius of the spherical particle (m)

D = diameter of the spherical particle (m)

v = terminal velocity (m/sec)

γ_s = unit weight of particles (KN/m^3)

γ_w = unit weight of liquid or water (KN/m^3)

= viscosity of liquid or water (KN s/m^2) = μ/g

μ = viscosity in absolute units of poise

g = acceleration due to gravity

- If water is used as the medium for suspension γ_w is equal to 9.81 KN/m^3. Similarly

$\gamma_s = G\gamma_w$. Substituting this we get

$$v = \frac{1}{18} D^2 \frac{(G-1)\gamma_w}{\eta}$$

The above formula should be expressed in the consistent units of meters, seconds and kilo newton.

If the diameter (D) of the particles is in mm we have

$$v = \frac{1}{18} (D/1000)^2 \frac{(G-1)\gamma_w}{\eta}$$

$$= \frac{D^2 \gamma_w (G-1)}{18 \times 10^6 \eta}$$

Taking γ_w = 9.81 KN/m^3

We get $v = \dfrac{D^2 (G-1)}{1.835 \times 10^6 \eta}$

$$D = \sqrt{\frac{18 \times 10^6 \eta v}{(G-1)\gamma_w}} \text{ mm}$$

$$D = 1355 \sqrt{\frac{\eta v}{G-1}} \text{ mm}$$

It should be noted that 1poise is equivalent to 0.1 Ns/m^2 or to 10^{-4} KN s/m^2

If a particle of diameter D mm falls through a height of H_e cm in t minutes.

v = H_e/60t cm/sec

= H_e/6000t m/sec

Substituting in the above equation we get

$$D = \sqrt{\frac{18 \times 10^6 \eta H_e}{(G-1)\gamma_w \times 6000t}}$$

$$= \sqrt{\frac{3000\eta}{(G-1)\gamma_w}}\sqrt{\frac{H_e}{t}}$$

$$D = 10^{-5}F\sqrt{\frac{H_e}{t}}$$

Where $F = 10^5\sqrt{\frac{3000\eta}{(G-1)\gamma_w}}$ is a constant factor for given values of ɥ and G.

At 27°C, the viscosity μ of the distilled water is approximately 0.00855 poise.

Since 1 poise is equivalent to 10^{-4} KN-s/m²

We have ɥ = 0.00855×10^{-4} KN-s/m²

Taking an average value of G = 2.68

Putting these value in $v = \frac{D^2(G-1)}{1.835 \times 10^6 \eta}$

We get $v = \frac{D^2(9.81)(2.68-1)}{18 \times 10^6 \times 0.0085 \times 10^{-4}}$

$$= 1.077\ D^2\ \text{(m/sec)}$$

This is an approximate version of Stoke's law and can be easily remembered for rough determination.

- The sedimentation analysis is done either with the help of hydrometer or a pipette.
- In both the methods a suitable amount of oven dried soil sample, finer than 75 micron size is mixed with a given volume V of distilled water.
- The mixture is shaken thoroughly and the test is started by keeping the jar containing soil water mixture, vertical.
- At the commencement of sedimentation test soil particles are assumed to be uniformly distributed throughout the suspension.
- After any time interval t, if a sample of soil suspension is taken from a height H_e (measured from the top level of suspension), only those particles will remain in the suspension which have not settled during this time interval.

The diameter of those particles which are finer than those which have already settled can be found from $D = 10^{-5}F\sqrt{\frac{H_e}{t}}$

- The greater the time interval t allowed for suspension to settle, the finer are the particles sizes retained at this depth H_e.
- Hence sampling at different time intervals, at this sampling depth H_e would give the content of particles of different sizes.
- If at any time interval t, M_D is the mass, per ml, of all particles smaller than the diameter D still in suspension at the depth H_e the percentage finer than D is given by

$$N = \frac{M_D}{M_d/V} \times 100$$

Where N = percentage finer than the diameter D

M_d = total dry mass of all particles put in the suspension

V = volume of suspension

Thus with the help of above equations we can get various diameter D and the percentage of particles finer (N %) than this diameter.

Limitation of sedimentation analysis:

(1) The sedimentation analysis gives the particle size in terms of equivalent diameter, which is less than the particle size given sieve analysis. The soil particles are not spherical. The equivalent diameter is close to the thickness (smallest dimension) rather than the length or width.

(2) As the specific gravity of solids for different particles is different, the use of an average value of G is a source of error. However as the variation of the values of G is small the error is negligible.

(3) Stokes law is applicable only when the liquid is infinite. The presence of walls of the jar affects the result to some extent.

(4) In stokes law it has been assumed that only one sphere settle and there is no interface from other spheres. In the sedimentation analysis as many particles settle simultaneously there is some interface.

(5) The sedimentation analysis cannot be used for particles larger than 0.2 mm as turbulent conditions develop and stokes law is not applicable.

(6) The sedimentation method is not applicable for particles smaller than 0.2μ because Brownian movement takes place and the particles do not as per Stokes law.

PIPETTE METHOD:

- The pipette method is the standard sedimentation method used in the laboratory.
- The equipment consists of a pipette, a jar and a number of sampling bottles.
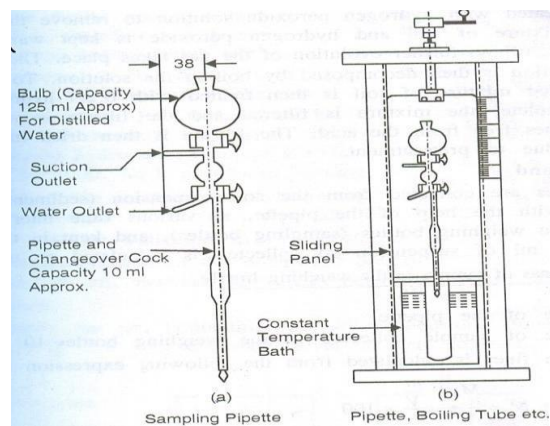- Generally a boiling tube of 500 ml capacity is used in place of a jar.

Figure 3.4

- The figure shows a pipette for extracting samples from the jar from a desired depth (H_e).
- The pipette consists of a 125 ml bulb with stop cock for keeping distilled water, a three way stop cock, suction and waste water outlets, sampling pipette of 10 ml capacity.
- The method consists in drawing off samples of soil suspension, 10 ml in volume, by means of this pipette from a depth of 10 cm (H_e) at various time intervals after the commencement of the sedimentation.
- The recommended time intervals are: ½, 1, 2, 4, 8, 15 and 30 minutes and 1, 2, 4, 8, 16 and 24 hours reckoned from the commencement of the test.
- The pipette should be inserted in the boiling tube about 25 seconds before the selected time interval and the time taken for sucking the sample should not be more than 10 to 20 seconds.
- Each sample so taken is transferred into suitable sampling bottles and dried in an oven.
- The mass M_D of solids per ml of suspension is thus found by taking the dry mass and dividing it by 10.

Method of preparing soil suspension:

- In the sedimentation analysis only those particles which are finer than 75 micron size are included.
- About 12 to 30 gm of oven dried sample is accurately weighed and mixed with distilled water in a dish or beaker to form a smooth thin paste. To have proper dispersion of soil a dispersing agent is added to the soil. Some of the common dispersing agents are sodium oxalate, sodium silicate and sodium polyphosphate compounds such as tetra sodium pyrophosphate, sodium hexametaphosphate and sodium tripolyphosphate.
- IS 2720 recommends the use of dispersing solution containing 33 gm of the sodium hexametaphosphate and 7 g of sodium carbonate in distilled water to make one litre of solution.
- 25 ml of this solution is added to the dish (containing the soil and distilled water) and the mixture is warmed gently for about 10 minutes.
- The contents are then transferred to the cup of a mechanical mixer, using a jet of distilled water to wash all traces of the soil out of the evaporating dish.
- The soil suspension is then stirred well for 15 minutes.
- The suspension is then washed through 75 micron IS sieve, using jet of distilled water and the suspension, which has passed through the sieve, is transferred to the 500 ml capacity boiling tube (sedimentation tube).
- The tube is then filled to the 500 ml mark by adding distilled water.

- The tube is then put in a constant temperature water bath.
- When the temperature is then tube has been stabilised to the temperature of the bath, the soil suspension is thoroughly shaken by inverting the tube several times, and then replaced in the bath.
- The stop was then started and the soil samples are collected at various time intervals with the help of pipette.

Calculation of D and N:

- 10 ml samples are collected from the soil suspension (sedimentation tube) from a depth of 10 cm, with the help of the pipette at various time intervals.
- The samples are collected into the weighing bottles (sampling bottles) and kept in the oven for drying.
- The mass M_D, per ml of suspension so collected is calculated as under :

M_D = dry mass of sample in the weighing bottle/V_P

Where V_P = volume of the pipette

= volume of sample collected in the weighing bottle = 10 ml

The percentage finer is calculated from the following expression

$$N' = \frac{M_D - \frac{m}{7}}{\frac{M_D}{7}} \times 100$$

Where m = mass of dispersing agent present in the in the total suspension of

volume V

V = volume of suspension = 500 ml

N' = percentage finer based on M_D

HYDROMETER METHOD:

- The hydrometer method of sedimentation analysis differs from the pipette analysis in the method of taking observation.
- In the pipette analysis the mass M_D per ml of suspension is found directly by collecting a 10 ml sample of soil suspension from the sampling depth H_e. However in the hydrometer analysis M_D is computed indirectly by reading the density of the soil suspension at a depth H_e at various time intervals.
- In the pipette test the sampling depth H_e is kept constant while in the hydrometer test, the sampling depth H_e goes on increasing as the particles settle with the increase in the time interval. It is therefore necessary to calibrate the hydrometer.
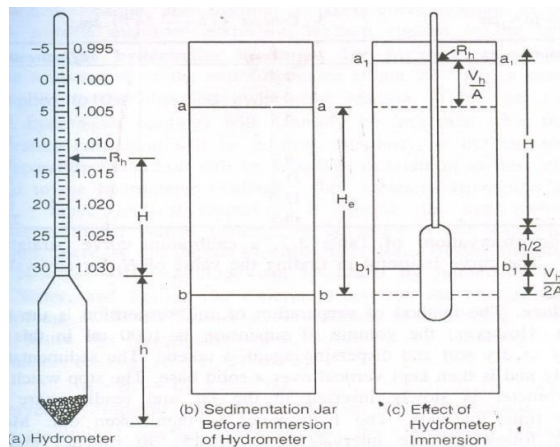
Figure 3.5

Calibration of hydrometer:

- The readings on the hydrometer stem give the density of the soil suspension situated at the centre of the bulb at any time.
- For convenience, the hydrometer readings are recorded after subtracting 1 and multiplying the remaining digits by 1000. Such a reduced reading is designated as R_h.
- For example, if the density reading at the intersection of horizontal surface of soil suspension with the stem is 1.010, it is recorded as 10 i.e $R_h = 10$.
- As indicated in figure the hydrometer reading R_h is increase in the downward direction towards the hydrometer bulb.
- Let H be the height in cm between any hydrometer reading R_h and the neck, and h the height of the bulb.
- Figure (b) shows the jar containing the soil suspension.
- When the hydrometer is immersed in the jar as shown in figure (c) the water level aa rises to a_1a_1, the rise being equal to the volume V_h of the hydrometer divided by the internal area of cross section A of the jar.
- Similarly the level bb rises to b_1b_1, where bb is the level situated at a depth H_e below the top level aa, at which the density measurements of the soil suspension are being taken.
- The rise between bb and b_1b_1 will be approximately equal to $V_h/2A$.
- The level b1b1 is now corresponding to the centre of the bulb, but the soil particles at b1b1 are of the same concentration as they were at bb.
- Therefore we have

$$H_e = (H + \frac{h}{2} + \frac{V_h}{2\text{Æ}}) - \frac{Vh}{\text{Æ}}$$

$$= H + \frac{1}{2} (h - \frac{Vh}{\text{Æ}})$$

- In the above expression there are two variables: the effective depth H_e and the depth H which depends upon the hydrometer reading R_h.
- Therefore by selecting various hydrometer reading R_h , the depth H can be measured with the help of an accurate scale and the corresponding depth H_e can be found.
- The height h of the bulb is constant. Similarly V_h and A are constant.
- To find the volume of the hydrometer it is weighed accurately. The mass of the hydrometer in grams give the volume of the hydrometer in millilitres.

Test procedure:

- The method of preparation of soil suspension is the same as indicated in the pipette test.
- However the volume of suspension is 1000 ml in this case and hence doubles the quantity of dry soil and dispersing agent is taken.
- The sedimentation jar is shaken vigorously and is then kept vertical over a solid base.
- The stop watch is started simultaneously.
- The hydrometer is slowly inserted in the jar and readings are taken at ½, 1 and 2 minutes time interval. The hydrometer is then taken out.
- More readings are then taken at the following time intervals: 4, 8, 15, 30 minutes and 1, 2, 4 hours etc.
- To take the reading, the hydrometer is inserted about 30 seconds before the given time interval, so that it is stable at the time when the reading is to be taken.
- Since the soil suspension is opaque the reading is taken corresponding to the upper level of the meniscus.

Correction to the hydrometer reading:

The hydrometer readings are corrected as under:

(i)   Meniscus correction:
- Since the suspension is opaque, the observations are taken at the top of the meniscus.
- The meniscus correction is equal to the reading between the top of the meniscus and the level of the suspension.
- As the marking on the stem increase downward the correction is positive.
- The meniscus correction (C_m) is determined from the readings at the top and bottom of meniscus in the comparison cylinder. The meniscus correction is constant for a hydrometer.
- If R_h' is the hydrometer reading of the suspension at a particular time, the corrected hydrometer R_h reading is given by

$R_h = R_h' + C_m$

(ii)   Temperature correction:

- The hydrometer is generally calibrated at 27°C. If the temperature of the suspension is different from 27°C a temperature correction (C_t) is required for the hydrometer reading.
- If the temperature is more than 27°C, the suspension is lighter and the actual reading will be less than the corrected reading. The temperature correction is positive.
- On the other hand, if the temperature is less than 27°C the temperature correction is negative.

(iii)   Dispersion agent correction:

- Addition of the dispersing agent to the soil suspension causes an increase in the specific gravity of the suspension.
- Therefore the dispersing agent correction is always negative.
- The dispersing agent correction (C_d) can be determined by noting the hydrometer reading in clear water and again in the same water after adding the dispersing agent.
- Thus the corrected reading R can be obtained from the observed reading R_h' as under

$$R = R_h' + C_m \pm C_t - C_d$$

(iv)   Composite Correction:

- Instead of finding the correction individually, it is convenient to find one composite correction.
- The composite correction (C) is the algebraic sum of all the correction.

Thus $R = R_h' \pm C$

- The composite correction is found directly from the readings taken in a comparison cylinder, which has distilled water and the dispersing agent in the same concentration and has the same temperature.

Computation of D and N:

- The particle size D is calculated from the following formula

$$D = 10^{-5}F\sqrt{\frac{H_e}{t}}$$

- To compute the percentage of the soil finer than this diameter, the mass M_D per ml of suspension at effective depth H_e is first computed as under
- Since the hydrometer readings have been recorded by subtracting 1 from the density (þ) readings and multiplying them by 1000, we have

$$R = (þ - 1)1000$$

Or, þ = 1+ R/1000 --(i)

Where þ is the density reading actually marked on the hydrometer and R is the hydrometer reading corrected for the composite correction.

- Now let us consider 1 ml of soil suspension at a time interval t at the effective depth H_e. If M_D is the mass of solids in this 1 ml suspension the mass of water in it will be

$1 – M_D/G$

Total mass of 1 ml suspension $= 1 - \dfrac{M_D}{G} + M_D$

Hence density of the suspension $= 1 - \dfrac{M_D}{G} + M_D$ ------------------ (ii)

Equating equation (i) and (ii) we get

$1+ R/1000 \quad = \quad 1 - \dfrac{M_D}{G} + M_D$

$M_D = \dfrac{R}{1000} \left(\dfrac{G}{G-1}\right)$

Where G = specific gravity of soil solids

Substituting these values in equation $N = \dfrac{M_D}{M_{D/7}} \times 100$

We get $N' = \dfrac{\frac{R}{1000}\left(\frac{G}{G-1}\right)}{Md/V} \times 100$

Taking V = 1000 ml we get

$N' = \dfrac{100GR}{Md(G-1)}$

Where N' = percentage finer with respect to M_d

Thus for various values of R, N' can be calculated

- For a combined sieve and sedimentation analysis if M is the total dry mass of soil originally taken (before sieving it over 2mm sieve) the overall percentage finer N is given by

$N = N' \times \dfrac{M'}{M}$

Where M' = cumulative mass passing 2 mm sieve

   M = total dry mass of soil sample

If the soil sample does not contain particles coarser than 2 mm size, N and N' will be equal.

CONSISTENCY OF SOIL:

- Consistency means the relative ease with which soil can be deformed.
- Consistency denotes degree of firmness of the soil which may be termed as soft, firm, stiff or hard.
- Fine grained soil may be mixed with water to form a plastic paste which can be moulded into any form by pressure.
- The addition of water reduces the cohesion making the soil still easier to mould.
- Further addition of water reduces the until the material no longer retains its shape under its own weight, but flows as a liquid.

- Enough water may be added until the soil grains are dispersed in a suspension.
- If water is evaporated from such a soil suspension the soil passes through various stages or states of consistency.
- Swedish agriculturist Atterberg divided the entire range from liquid to solid state into four stages :
  (i)   Liquid state
  (ii)   Plastic state
  (iii)   Semi – solid state
  (iv)   Solid state

He sets arbitrary limits known as consistency limit or Atterberg limit for these divisions in terms of water content

- Thus consistency limits are the water content at which the soil mass passes from one state to the next.
- The Atterberg limits which are most useful are :
  (i)   Liquid limit
  (ii)   Plastic limit
  (iii)   Shrinkage limit

Figure 3.6

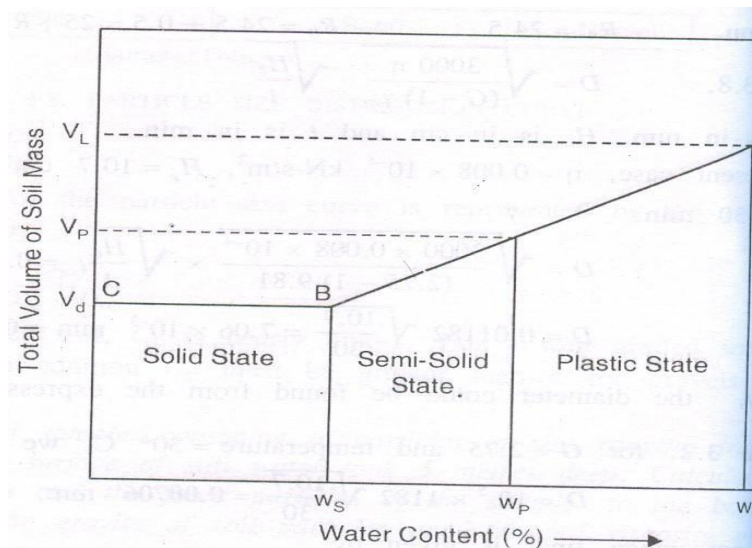**Different states of soil**

Liquid Limit:

- Liquid limit is the water content corresponding to the arbitrary limit between liquid and plastic state of consistency of a soil.
- It is defined as the minimum water content at which the soil is still in the liquid state but has a small shearing strength against flowing which can be measured by standard available means.

- With reference to the standard liquid limit device, it is defined as the minimum water content at which a part of soil cut by a groove of standard dimension, will flow together for a distance of 12 mm under an impact of 25 blows in the device.

Plastic limit:

- Plastic limit is the water content corresponding to an arbitrary limit between the plastic and the semi-solid state of consistency of a soil.
- It is defined as the minimum water content at which a soil will just begin to crumble when rolled into a thread approximately 3 mm in diameter.

Shrinkage limit:

- Shrinkage limit is defined as the maximum water content at which a reduction in water content will not cause a decrease in the volume of a soil mass.
- It is lowest water content at which a soil can still be completely saturated.

Plasticity Index:

- The range of consistency within which a soil exhibit plastic properties is called plastic range and is indicated by plasticity index.
- Plasticity index is defined is defined as the numerical difference between the liquid limit and the plastic limit of a soil.

$I_p = w_l - w_p$

- When plastic limit cannot be determined , the plasticity index is reported as NP (Non plastic).
- When the plastic limit is equal to or greater than the liquid limit the plasticity index is reported as zero.

Plasticity:

- Plasticity is defined as that property of a soil which allows it to be deformed rapidly, without rupture, without elastic rebound and without volume change.

Consistency Index:

- The consistency index or the relative consistency is defined as the ratio of the liquid limit minus the natural water content to the plasticity index of soil

$I_c = \dfrac{w_l - w}{I_p}$

Where w is the natural water content of the soil

- If consistency index of a soil is equal to unity, it is at the plastic limit.
- Similarly a soil with I_c equal to zero is at its liquid limit.
- If I_c exceeds unity the soil is in a semi – solid state and will be stiff.

- A negative consistency index indicates that the soil has natural water content greater than the liquid limit and hence behaves just like a liquid.

Liquidity index:

- The liquidity index or water plasticity ratio is the ratio, expressed as a percentage, of the natural water content of a soil minus its plastic limit to its plasticity index:

$I_l = \frac{w - w_p}{I_p}$

Where w is the natural water content of the soil.

Example – 1

- During a test for water content determination on a soil sample by pycnometer, the following observations were taken
  (1) Mass of wet soil sample   = 1000 gm
  (2) Mass of pycnometer with soil and filled with water   = 2000 gm
  (3) Mass of pycnometer filled with water only = 1480 gm
  (4) Specific gravity of solids   = 2.67

Determine the water content.

Solution:

We know that $w = [(\frac{M2 - M1}{M3 - M4})(\frac{G-1}{G}) - 1] \times 100$

$= [\frac{1000}{(2000 - 1480)} \times (\frac{2.67 - 1}{2.67}) - 1] \times 100$

$= 20.28\%$

Hence water content of the sample is 20.28%.   (Ans)

Example 2:

The mass of an empty gas jar was 0.498 Kg. When completely filled with water its mass was 1.528 Kg. An oven dried sample of soil mass 0.198 Kg was placed in the jar and water was added to fill the jar and its mass was found to be 1.653 Kg. Determine the specific gravity of particle.

Solution: we know that

$G = \frac{M_2 - M_1}{(M_2 - M_1`) - (M_3 - M_4)}$

$= \frac{0.198}{0.198 - (1.653 - 1.528)}$

= 2.71

Hence specific gravity of the sample is 2.71.

Example 3:

A soil sample consisting of particles of size ranging from 0.5 mm to 0.01 mm, is put on the surface of still water tank 5 metres deep. Calculate the time of settlement of the coarsest and the finest particle of the sample, to the bottom of the tank. Assume average specific gravity of soil particles as 2.66 and viscosity of water as 0.01 poise.

Solution:

$$v = \frac{D^2 \gamma_w (G-1)}{18 \times 10^6 \eta}$$

$$= \frac{D^2(G-1)}{1.835 \times 10^6 \times \eta}$$

Here G = 2.66 and ц = 0.01×10^{-4} = 10^{-6} KN-s/m^2

$$v = \frac{D^2}{1.835} \times \frac{2.66-1}{10^6(10^{-6})}$$

$$= 0.905 \, D^2$$

Where v is in m/sec and D is in mm

For coarsest particle D = 0.5 mm

v = $0.905(0.5)^2$ = 0.2263 m/sec

t = h/v = 5/0.2263 = 22.1 seconds

for the finest particle, D = 0.01 mm

v = $0.905(0.01)^2$ = 9.05×10^{-5} m/sec

$$t = \frac{5}{9.05 \times 10^{-5}} = 55249 \text{ sec} = 15 \text{ hours } 20 \text{ min } 49 \text{ seconds.}$$

Example 4:

50 grams of oven dried soil sample is taken for sedimentation analysis. The hydrometer reading in a 100 ml soil suspension 30 minutes after the commencement of sedimentation test is 24.5. The effective depth for R_h = 25, found from the calibration curve is 10.7 cm. The meniscus correction is found to be +0.5 and the composite correction as – 2.50 at the test temperature of 30°C. Taking the specific gravity of particles as 2.75 and viscosity of water as 0.008 poise, calculate the smallest particle size which would have settled during this interval of 30 minutes and the percentage of particles finer than this size.

Solution:

$R_h' = 24.5$ $\quad\quad\quad\quad R_h = 24.5 + 0.5 = 25$ $\quad\quad\quad\quad R = 24.5 - 2.5 = 22$

$$D = \sqrt{\frac{3000\eta}{(G-1)\gamma_w}}\sqrt{\frac{H_e}{t}}$$

Where D is in mm, H_e is in cm and t is in minute.

Here $\eta = 0.008 \times 10^{-4}$ KN-s/m²

$\quad\quad H_e = 10.7$ cm

$\quad\quad G = 2.75$ and $\gamma_w = 9.81$ KN/m³

$\quad\quad t = 30$ min.

$$D = \sqrt{\frac{3000 \times 0.008 \times 10^{-4}}{(2.75-1)9.81}} \times \sqrt{\frac{10.7}{30}}$$

$\quad = 7.06 \times 10^{-3}$ mm

$\quad = 0.00706$ mm

The percentage finer is given by

$$N' = \frac{100GR}{Md(G-1)}$$

Where M_d = mass of dry soil = 50 gm

$$N' = \frac{100 \times 2.75}{50(2.75-1)} \times 22$$

$\quad = 69.1\,\%.$ $\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad$ (Ans)

Example 5:

A soil has a liquid limit of 25 % and plastic limit is 15 %. Determine the plasticity index. If the water content of the soil in its natural condition in the field is 20 %, find the liquidity index and relative consistency.

Solution:

$w_l = 25\,\%$ $\quad\quad\quad\quad w_p = 15\,\%$ $\quad\quad\quad w = 20\,\%$

plasticity Index $I_p = w_l - w_p$

$\quad = 25 - 15 = 10\,\%$

Liquidity index $= I_l = \dfrac{w - w_p}{I_p} \times 100$

$$= \frac{0.2 - 0.15}{0.1} \times 100 = 50 \ \%$$

Relative consistency $= I_c = \frac{wl - w}{I_p} \times 100$

$$= \frac{0.25 - 0.2}{0.1} \times 100 = 50 \ \% \qquad\qquad (\text{Ans})$$

CHAPTER-4

CLASSIFICATION OF SOIL

Purpose of soil classification:

The purpose of soil classification is to arrange various types of soil into specific groups based on physical properties and engineering behavior of the soils with the objective of finding the suitability of soils for different engineering application , such as in the construction of earth dams , highway ,and foundations of building , etc.

For different areas of applications and with the need for simplicity and acceptable terminology, several soil classification system s have been developed over the year s,three of which are listed below.

1. Highway research board classification system
2. Unified soil classification system
3. Indian standard soil classification.

Highway Research Board (HRB) classification System:

The Highway Reach Board classification system, also known as Revised Public Roads Administration classification system ,is used to find the suitability of a soil , as sub grade material in pavement construction .This classification system is based on both particle size ranges and plasticity characteristic .soil are divided into 7 primary group designated as A -1 , A-2………A-7 , as shown in table 4.1.

Group A-1, is divided into two sub groups A-1 and A-1 and group A-2into four sub groups, A-2-4 , A-2-5 ,A-2-6 and A-2-7 .A characteristic group index is used to describe the performance of a soil as sub grade material.

Ground index is not used to place a soil in a particular group ; it is actually a means of rating the value of soil as a sub grade material within its own group .The higher the value of the group index, the proper is the quality of the material.

The group index of a soil depends upon.

(i)   Amount of material passing the 75-micron sieve,
(ii)   Liquid limit
(iii)   Plastic limit
        Group index is given by the following equation:
        Group index=0.2a+0.005ac+0.01bd
        Where
            a=that portion of percentage passing 75 micron sieve greater than 35 and not exceeding 75 expressed as whole number(0 to 40)
            b=that portion percentage passing 75 micron sieve greater than 15 and not exceeding 55 expressed as whole number(0 to 40)
            c=that portion of the numerical liquid limit greater than 40 and not exceeded 60 expressed as a positive whole number (0 to 20)

d=that portion of the numerical plasticity index greater than 10 and not exceeding 30 expressed as a positive whole number (0 to 20).

To classify a given soil, sieve analysis data, liquid limit and plasticity index are obtained and we proceed from left to right in the Table 4.1 and by Process of elimination find the first group from into which the test data will fit. This gives the correct Classification. The plasticity index of A-7-5 subgroup is equal to or less than liquid limit minus 30.The plasticity index of A-7-6 subgroup is greater than liquid limit minus 30.

Note: The PRA system was introduced in 1928 and revised in 1945 as HRB system. It is known as AASHTO system since 1978 after adoption by American Association of State Highway and Transportation Officials.

Unified soil classification System

The Unified soil classification system is based on the Airfield Classification system that was developed by A Casagrande .the system based on both grain size and plasticity characteristic of soil. The unified Soil classification (USC)system was adopted jointly by the Corps of Engineers ,U.S. army and U.S. Bureau of Reclamation during 1950s.

1. Coarse-grained soils – if more than 50% by Weighty is retained on No. 200 ASTM sieve.

2. Fine –grain soil- if more than 50% by weight passes through No. 200 ASTM sieve

3. Organic soils.

The soil component are assigned group symbols as indicated below :

Coarse –grained soils:

Gravel: G   Sand: S

Fine grained soils:

Silt: M   Clay's   Organic soil's

Table4.3 Unified Soil Classification System

Major Division				Group Symbols		Typical names
Coarse Grain Soils more than 50% retained on No. 200 sieve *	Gravels 50%or more of coarse fraction retained on No.4 sieves	Clear gravels -200 fraction <5%		GW	Well graded	
		Gravel with fine -200>5% fraction		GP		
	Sand more than 50% of cross fraction passes No.4 sieve	Clean sands-200<55 fraction		GM		
		Sand with fines -200>12% fraction		GC		
Fine grained soils 50%or more passes No.200 sieve*				SW		
				SP		
				SM		
				SC		
Fine-grained soils 50% or more passes No.200 sieve.						

Table 4.3 gives the details of Unified soil Classification system. The original casagrande plasticity chart used for classifing fine graind soil is given in Fig 4.3

They symbol M for sily in drived from the swedish word 'mo' for silt.

Example4.3. classify the soil with composition indicated in 4.2using USC system .

Solution : Since more than 50% of soil passes through 0.074 mm sieve the soil is fine grained.

Plasticity index = (50-40)% =10%

From fig 4.3

For w L=40% AND Ip =10% the soil can be classified as ML OR OL

INDIAN STANDARD SOIL CLASSIFICATION SYSTEM

Indian standard soil classification system (IS 1498-1970 classification and identification of soil for general engineering purpose) is esssentially based on unified soil classification system and the salient features and given in the following discussion.

In the system soils are broadly divided into 3 division

1. coarse- grained soil –if more than 50% by mass is retain on 75 micron IS sieve.

2. Fine grained soil –if more than 50% by mass passes through 75 micron IS sieve.

3. Highly organic soils and other miscelliouns soil material . The soil content larger % of fibrous organic matter such as peat and particles of decompose vegetation. In addition shorten soil containing shells ,concretions cinders and other non soil material insufficient quantities are also grouped in this division.

Coarse grained soils are grouped as gravels and sands with group symbols G and S

Gravels(G) if more than 50% by mass of the coarse grained fraction passed through 4.75 mm IS sieve .

Depending on the gradation gravels(G) and sands (S) are further described using group symbols are indicated below.

GW- Well graded gravel for which Cu >4 and Cc lies between 1&3

GP- poorly graded gravel which does not meet all graduation requirement   of GW

SW-Well graded sand for which Cu >6 and Cc lies between 1 &3

GM-silty gravel if lp > 4 for fine- grained fraction.

GC-Clayey gravel if Ip < 7 for fine-grained fraction.

SM-Silty sand if Ip < 4 for fine grained fraction.

SC- Clayey sand if Ip > 7 for fine- grained fraction.

In the case of coarse –grained soils mixed with fines if Ip lies between 4 and 7 one has to use proper judgment in dealing with this border line case. Generally non-plastic classification is favored in such cases. For example a sand with 10% fines with Cu > 6 ,Cc between 1 and 3 and Ip =

6 would be classified as SW –SM rather than SW-SC.

Fine-grained soils are grouped under following three subdivisions with respective group symbols:

Inorganic silts and very fine sands(M)

Inorganic clays(C)

Organic silts ,Organic clays and Organic matter (O)

Depending on liquid limit which is considered a good index of compressibility fine grained soils are described as possessing (i)low compressibility (L) when liquid limit is less than 35 percent .

(ii) intermediate or medium compressibility (I) when liquid limit lies between 35 percent and 50 percent

(iii) high compressibility (H) when liquid limit is greater than 50 percent.

The plasticity chart originally devised by A. Casagrande and slightly modified by IS is used to classify fine-grained soils in the laboratory.

 The A-line having the equation:

Ip=0.73 (Wl-20)

And the two vertical lines at wl=35and wl=50 divide the chart into six regions with group symbols marked as shown in Fig.4.4 if the plotted position lies below A-line, the soil has to be checked for organic odour by slight heating. If no organic odour is smelt than only it should be classified as inorganic silt. In case of doubt, the soil should be oven-dried and its liquid limit determined

again. In the case of organic soils there will be large reduction in liquid limit on drying (reduction generally>25%).

Soil	Soil component	Symbol	Particle size range and description
Coarse-grained Components	Boulder	none	Round to angular , bulky hard ,rock particle , average diameter more than 300mm.
	Cobble	None	Round to angular, bulky hard, rock particle average diameter smaller than 300 mm but retained on 80 mm sieve.
	Gravel	G	Rounded to angular, bulky hard, rock particle, passing 80 mm sieve but retained on 4.75 mm sieve. Coarse:80 mm to 20 mm sieve Fine:20 mm to 4.75 mm sieve
	Sand	S	Rounded to angular bulky hard, rocky particle, passing 4.75 mm sieve retained on 75 micron sieve Coarse:4.75 mm to 2.0 mm sieve Medium:2.0 mm to 425 micron sieve Fine: 425 micron to 75 micron sieve
	silt	M	Particles smaller than 75-micron sieve indentified by behavior; that is slightly plastic or non-plastic regardless of moisture and exhibits little or no strength when air

Fine –grained Components			dried
	Clay	C	Particles smaller than 75- micron sieve identified by behavior, than is, it can made to exhibit plastic properties within a certain range of moisture and exhibits considerable strength when air dried

CHAPTER -5

PERMEABILITY AND SEEPAGE

5.1 Concept of Permeability:-

➢ The property of soil which permits flow of water (or other any liquid) through it is called the permeability in otherworld, the permeability in the case with which water can flow through it.
➢ Permeability is very important engineering property of soil.
➢ The knowledge of permeability is essential in a number of soil engineering problems such as: Settlement of Buildings, Yield of wells, Seepage through and below the earth surface.
➢ Permeability controls the hydraulic stability of soil masses.
➢ The permeability of soils is also required in the design of filters required to prevent piping in hydraulic structure.

Darcy's Law:-

The flow of free water through soil is governed by Darcy's law. In 1856, Darcy experimentally that for laminar flow in a homogeneous soil, the velocity of flow (v) is given by

$$v= k\,i \qquad Equation\ no\ \text{-}1$$

Where, k= coefficient of permeability, i= hydraulic gradient and v = velocity of flow in laminar flow in homogeneous soil

The above equation is known as Darcy's law, which is one of the corner stones of soil engineering. The discharge 'q' is obtained by multiplying the velocity of flow (v) by the total cross-sectional area (A) normal to the direction of flow

$$Thus,\ q= vA = k\,i\,A \longrightarrow\ Equation\ no\ \text{-}\ 2$$

Note: - 1) The velocity of flow is also known as discharge velocity or superficial velocity.
2) The area A in the above equation includes both the solids and the voids.

Co-efficient of Permeability:-

➢ The coefficient of permeability can be defined using the equation 1. If the hydraulic gradient is unity, the coefficient of permeability is equal to the velocity of flow

Or,

➢ The coefficient of permeability is defined as the velocity of flow which would occur under unit hydraulic gradient. The co-efficient of permeability is equal to the velocity of flow.

➢ The coefficient permeability has the dimensions of velocity [L/T].

➢ The coefficient of permeability measured in mm/sec, cm/sec, m/sec, m/day or other velocity units.

➢ The coefficient of permeability depends upon the particle size and upon many factors.

➢ According to USBR, the soil having co-efficient permeability greater than 10^{-3}mm/sec are classified as pervious and those with a value less than 10^{-5} to 10^{-3}mm/sec are designated as semi-pervious.

5.2 Factors affecting Permeability of soils:-

The following factors affect the permeability of soils.

(1) Particle Size.
(2) Structure of soil mass.
(3) Shape of particles.
(4) Void ratio.
(5) Properties of water.
(6) Degree of saturation.
(7) Adsorbed water.
(8) Impurities in water.

(1) Particle Size: - Co-efficient of permeability of soil is proportional to the square of particle size (D). The permeability of coarse grained soils is very large as compared to that of fine-grained soils. The permeability of coarse sand may be more than one million times as much that of clay.

(2) Structure of soil mass: - The coefficient C takes into account the shape of flow passage. The size of flow passage depends upon the structural arrangement. For same void ratio, the permeability is more in the case of flocculated structure as compared to that in the disperses structure.

Stratified soil deposits have greater permeability parallel to the plane of stratification than that perpendicular to this plane. Permeability of soil deposit also depends upon shrinkage cracks, joints, fissures and shear zones. Loess deposits have greater permeability in the vertical direction than in the horizontal direction.

The permeability of natural soil deposit should be determined in undisturbed condition. The disturbance caused during sampling may destroy the original structure and affect the permeability. The effect of disturbance is more pronounces in case of fine-grained soils than in the case of coarse-grained soils.

(3) Shape of Particles: - The permeability of a soil depends upon the shape of particles. Angular particles have greater specific surface are as compared with the

rounded particles. For the same void ratio, the soils with angular particles are less permeable than those with rounded particles, as the permeability is inversely proportional to the specific surface. However, in a natural deposit, the void ratio for a soil with angular particles may be greater than that for rounded particles, and the soil with angular particles may be actually more permeable.

(4) Void Ratio: - For a given soil, the greater the void ratio, the higher is the value of the coefficient of permeability.

Based on the other concepts, it has been established that the permeability of soil varies as e^2 or $e^2/(1+e)$ (figure-2). Whatever may be the exact relationship; all oils have e versus log k plot as a straight line (figure-1).

Figure: Variation of log k with e.

Variation of k with e^2, $\dfrac{e^2}{1+e}$ and $\dfrac{e^3}{1+e}$
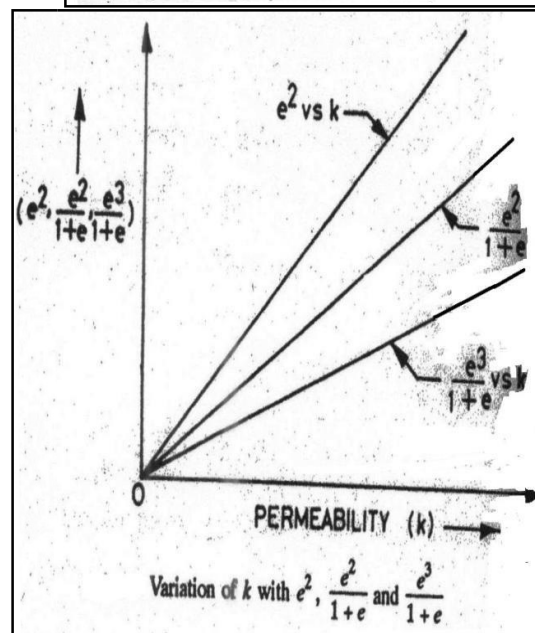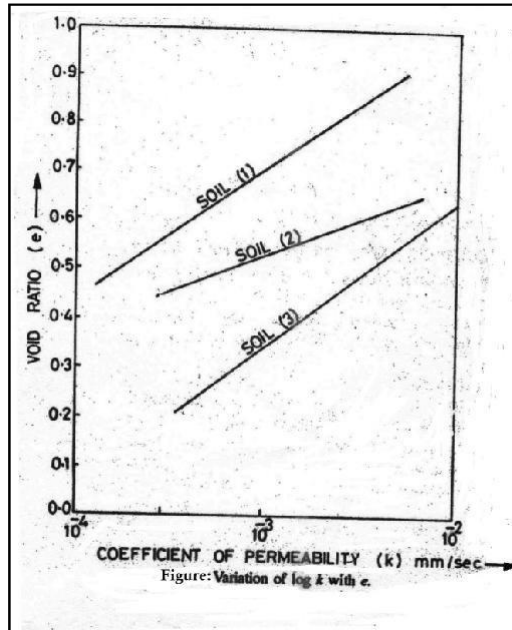
Figure (1)   Figure (2)

If the permeability of a soil at a void ratio of 0.85 is known, its value at another void ratio of 'e' can be determined using the following equation given by Casagrande:

$$k = 1.4\, k_{0.85}\, e^2$$

Where $k_{0.85}$ = permeability at void ration of 0.85, k = permeability at a void ratio of 'e'.

(5) Properties of Water: - The co-efficient of permeability is directly proportional to the unit weight of water (γ_w) and is inversely proportional to its viscosity (μ). The coefficient of permeability increases with an increase the temperature due to reduction in the viscosity.

(6) Degree of Saturation:- if the soil is not fully saturated, it contains air pocket formed due to entrapped air or due to air liberated from percolating water. Whatever may be the cause of presence of air in soils, the permeability is reduced due to presence of sir which causes blockage of passage. Consequently, the permeability of partially saturated soil is considerably smaller than that of fully saturated soil. In fact Darcy's Law not strictly applicable to such soils.

(7) Adsorbed Water: - The fine grained soils have a layer of adsorbed water strongly attached to their surface. This adsorbed water layer is not free to move under gravity. It causes an obstruction to flow of water in the pores and hence reduces the permeability of soils.

It is difficult to estimate the void occupied by the adsorbed water. According to one estimate, the void ration occupied by adsorbed water is about 0.10. The effective void ratio available for flow of water is thus about (e - 0.1) and not 'e'. In some cases, at very low hydraulic gradient, the coefficient of permeability of fine-grained soils becomes negligible small due to presence of adsorbed water.

(8) Impurities in Water: - Any foreign matter in water has a tendency to plug the flow passage and reduce the effective voids and hence the permeability of soils.

5.3 .1 - Constant Head Permeability Test:-

The coefficient of permeability of a relatively more permeable soil can be determined in a laboratory by the constant –head permeability test.
1. The test is conducted in an instrument known as constant-head Permeameter.
2. It consists of a metallic mould, 100 mm internal diameter, 127.3 mm effective height and 1000 ml capcity according to IS : 2720 (Part XVII).
3. The mould is provided with a detachable extension collar, 100 mm diameter and 60 mm high, required during compaction of soil.
4. The mould is provided with drainage base plate with recess for porous stone.
5. The mould is fitted with a drainage cap having an inlet valve and an air release valve.
6. The drainage base and cap have fittings for clamping to the mould.

Figure:-Constant Head Permeameter
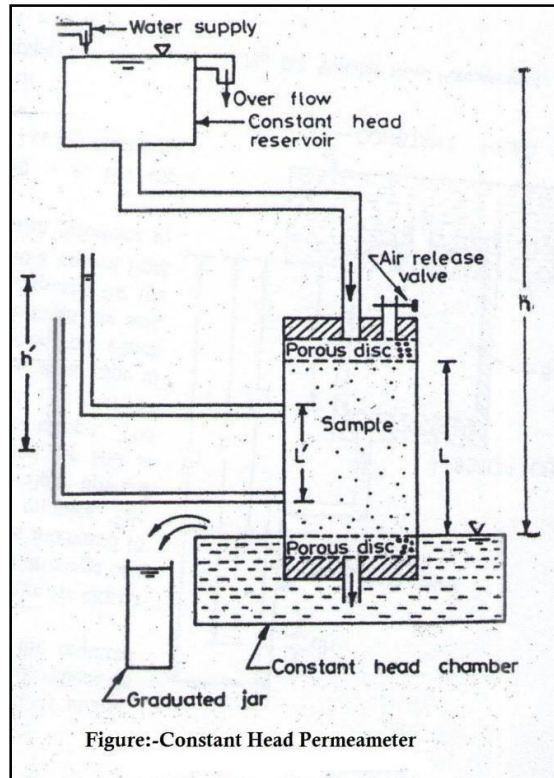
1. The above figure shows a schematic sketch.
2. The soil sample is placed inside the mould between two porous discs.
3. The porous discs should be at least ten times more permeable than the soil.
4. The porous discs should be deaired before these are placed in the mould.
5. The water tubes should also be deaired.
6. The sample can also be prepared in the permeameter by pouring the soil into it and tamping it to obtain the required density.
7. The base is provided with a dummy plate, 12 mm thick and 108 mm in diameter, which is used when the sample is compacted in the mould.
8. It is essential that the sample is fully saturated. This is done by one of the following three methods:-
   i. By pouring the soil in the permeamter filled with water and thus depositing the soil under water.
   ii. By allowing water to flow from the base to the top after the soil has been placed in the mould. This is done by attaching the constant-head reservoir to the drainage base. The upward flow is maintained for sufficient time till all the air has been expelled out.
   iii. By applying a vacuum pressure of about 700mm of mercury through the drainage cap for about 1.5 minutes after closing the drainage valve. Then the soil is saturated by allowing deaired water to enter from drainage base. The air-release valve is kept open during saturation process.
9. After the soil sample has been saturated, the constant-head reservoir is connected to the drainage cap.
10. Water is allowed to flow out from the drainage base for some time till a steady-state is established.

11. The water level in the constant-head chamber in which the mould is placed is kept constant.
12. The chamber is filled to the brim at the start of the experiment.
13. The water which enters the chamber after flowing through the sample spills over the chamber and collected in a graduated jar for convenient period.
14. The head causing flow (h) is equal to the difference in water levels between the constant-head reservoir and the constant-head chamber.
15. If the cross-sectional area of the specimen is A, the discharge is given by

$$q = k \, i \, A$$
$$or, \; q = k \, (h/l) \, A$$
$$or, \; k = (q \, l) \, / \, (A \, h)$$

where, L= Length of specimen, h= head causing flow.

The discharge q is equal to the volume of water collected divided by time.
The finer particles of soil specimen have a tendency to migrate towards the end faces when water flows through it. This results in the formation of a filter skin at the ends. The coefficient of permeability of these end portions is quite different from that of middle portion. For more accurate results, it would be preferable to measure the loss of head h' over a length L' in the middle to determine the hydraulic gradient (i). Thus $i = h' / L'$.

The temperature of permeating water should be preferably somewhat higher than that of the soil sample. This will prevent release of air during the test. It also helps in removing the entrapped air in the pores of soil. As the water cools, it has tendency to absorb air.

To reduce the chances of formation of large voids at the points where the particles of soil touch the permeameter walls, the diameter of the permeameter is kept at least 15 to 20 times the particles size.

To increase the rate of flow for the soils of low permeability, a gas under pressure is applied to the surface of water in the constant-head reservoir. The total head causing flow in that case increase to $(h + p/ \gamma_w)$, where p is pressure applied.

The bulk density of the soil in the mould may be determined from the mass of soil in the mould and its volume. The bulk density should be equal to that in the field. The undisturbed sample can also be used instead of the compacted sample. For accurate results, the specimen should have the same structure as in natural conditions.

The constant head permeability test is suitable for clean sand and gravel with $k > 10^{-2}$ mm/sec.

5.3.2- Falling Head / Variable Head Permeability Test:-

For relatively less permeability soils the quantity of water collected in the graduated jar of the constant-head permeability test is very small and cannot be accurately. For such soils, the variable-head permeability test is used.teh permeameter mould is the same as used in the constant-head permeability test.
1) A vertical, graduated stand pipe of known diameter is fitted to the top of permeameter.

2) The sample is placed between two porous discs.
3) The whole assembly is placed in a constant head chamber filled with water to brim at the start of the test. (See the below figure shows a schematic sketch).

Figure:- Variable Head Permeameter.
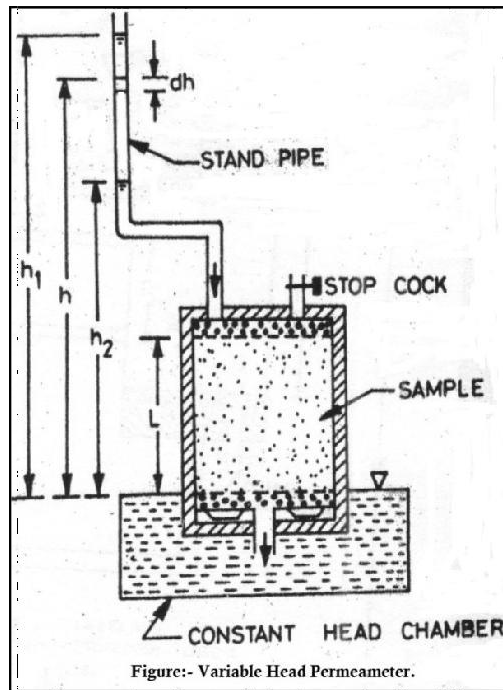
4) Tee porous discs and water tubes be de-aired before the sample is placed. If in-situ, undisturbed sample is available, the same can be used; otherwise the soil is taken in the mould and compacted to required density.
5) The valve at the drainage base (not shown in figure) is closed and vacuum pressure is applied slowly through the drainage cap to remove air from the soil.
6) The vacuum pressure is increased to 700 mm of mercury and maintained for about 15 minutes.
7) The sample is saturated by allowing deaired water to flow upward from the drainage base when under vacuum.
8) When the soil is saturated, both the top and bottom outlets are closed.
9) The standpipe is filled with water to required height.
10) The test is stated by allowing the water in the stand pipe to flo through the sample to constant-head chamber from which it overflows and spills out.
11) As the water flows through the soil, the water level in the standpipe falls.
12) The time required for the water level to fall from a known initial head (h_1) to known as final head (h_2) is determined.
13) The head is measured with reference to the level of water in the constant-head chamber.

Let us consider the instant when the head is h. For the infinitesimal small time dt, the head falls by dh.
Let the discharge through the sample be q.

From continuity of flow, *a dh = - q dt*

      Where '*a*' is cross-sectional area of standpipe.

Or ,   *a dh = - (A X k X i) X dt*

Or,   *a dh = - A X k X h/L X dt*

Or,   *A k dt / a L = -dh / h*

Integrating, $\dfrac{A\,K}{a\,L} \int_{t1}^{t2} dt = - \int_{h1}^{h2} \dfrac{dh}{h}$

$$\dfrac{A\,k}{a\,L}\,(t2 - t1) = \log_e (h_1/h_2)$$

$$k = \dfrac{A\,k}{a\,L}\,\log_e (h_1/h_2)$$

    Where, *t = (t2 –t1),* the time interval during which the head reduces from h_1 to h_2).

    Sometime $k = \dfrac{2.30\,a\,L}{A\,t} \log_{10} (h_1/h_2)$

    The rate of fall of water level in the stand pipe and te arte of flow can be adjusted by changing the area of cross-section of the standpipe. The smaller diameter pipes are required for less pervious soils.

    The coefficient of permeability is is reported at 27° C as per IS : 2720 (Part XVII). The void ratio soil is also generally determined.

The variable head permeameter is suitable for very fine sand and silt with k= 10^{-2} to 10^{-5} mm/sec.

Sometime, the permeability test is conducted using the consolidometer instead of the permeameter mould. The fixed-ring consolidometer is used a variable –head permeameter by attaching a stand.

5.4 .1 - Seepage Pressure:-

    As water flows through the soil, it extends a force on the soil. The force acts in the direction of flows. This force is known as drag force or seepage force. The pressure induced in the soil is known as seepage pressure.

    Or

    By virtue of the viscous friction exerted on water flowing through soil energy transfer is effected between the water and soil. The force corresponding to this energy transfer is called the seepage force or seepage pressure. Thus, seepage pressure is the pressure exerted by water on the soil through which percolates.

5.4.2 - The phenomenon of Quick Sand:-

    When flow takes place in an upward direction, the seepage pressure also acts in the upward direction and the effective pressure is reduced. If the seepage pressure becomes equal to the pressure due to submerged weight of the soil, the effective pressure is reduced to zero, in such case, a cohesionless soil loses all its shear strength, and the soil particles have a tendency to move up in the direction of flow. This phenomenon of lifting of soil particles is called *quick condition*, boiling condition or quick sand. Thus, during the quick condition,

$$\sigma' = z\gamma' - p_{s = 0}$$

or, $p_s = z\gamma'$

or $iz\gamma_w = z\gamma'$

From which,

$I = i_c = \gamma' / \gamma_w = \dfrac{G-1}{1+e}$

> The hydraulic gradient at such a critical state is called hydraulic gradient. For loose deposits of sand or sand or silt, if voids ratio e is taken as 0.67 and G as 2.67, the critical hydraulic gradient works out to be unity.

> It should be noted that quick sand is not a type of sand but a flow condition occurring within a cohesionless soil when its effective pressure is reduced to zero due to upward flow of water.
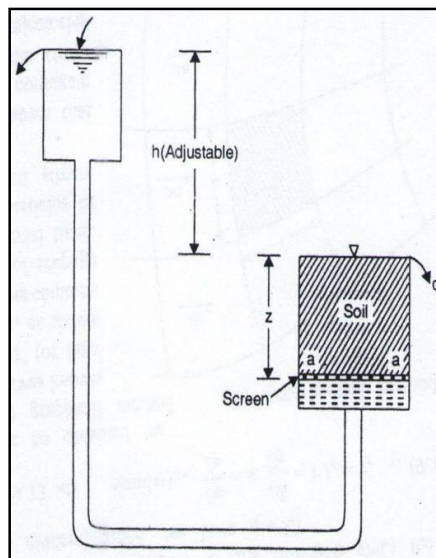

Figure: - QUICK SAND CONDITION

1. The figure shows a set-up to demonstrate the phenomenon of quick sand.
2. Water flows in an upward direction through a saturated soil sample of thickness 'z' under a hydraulic head 'h'.
3. This head can be increased or decreased by moving the supply tank in the upward or downward direction.
4. When the soil particles are in the state of critical equilibrium, the total upward force at the bottom of soil becomes equal to the total weight of all the materials above the surface considered.

Equating the upward and downward forces at the level *a-a,* we have,

$$(h + z)\,\gamma_w\,A = z\gamma_{sat}\,A$$

$$h\gamma_w = z(\gamma_{sat} - \gamma_w) = z\gamma'$$

$$\frac{h}{z} = i_c = \gamma' / \gamma_w = \frac{G-1}{1+e}$$

5.5.1- Concept of Flow-Net:-

1. The graphical method of flow net construction, first given by Forchheimer (1930), is based on trail sketching.
2. The hydraulic boundary conditions have a great effect on the general shape of the flow net, and hence must be examined before sketching is started.
3. The flow net can be plotted by trial and error by observing the properties of flow net and by following practical suggestion given by A.Casagrande.
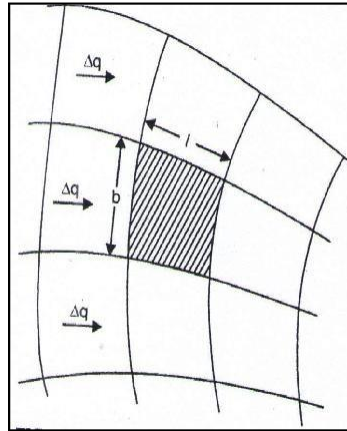

Figure: - PORTION OF A FLOW NET

5.5.2- Properties of Flow-Net:-

1. The flow lines and equipotential line meet at right angles to one another.
2. The fields are approximately squares, so that a circle can be drawn touching all the four sides of the square.
3. The quantity of water flowing through each flow channel is same, similarly, the same potential drop occurs between two successive equipotential lines
4. Smaller the dimensions of the field, greater will be the hydraulic gradient and velocity through it.
5. In a homogeneous soil, every transition in the shape of the curves is smooth, being either elliptical in shape.

5.5.3- Application of Flow-Net:-

A flow net can be utilized for the following purposes:-

   a) Determination of seepage.
   b) Determination of hydrostatic pressure.
   c) Determination of seepage pressure.

d) Determination of exist gradient.

(a) Determination of seepage:- Figure shows a portion of flow net. The portion between any two successive flow lines is known as flow channel. The portion enclosed between two successive equipotential lines and successive flow lines is known as field as that shown hatched in the figure

Let b & l be the width and length of the field

Δ h= head drop through the field

Δ q= Discharge passing through the flow channel

H= total hydraulic head causing flow = difference between upstream and downstream heads.

$$b = k H \ (N_f / N_d),$$

Where, $N_{f=}$ Total number of flow channel in the net
$N_{d=}$ Total number of potential drops in the complete net

This is required expression for the discharge passing through a flow-net and valid for isotropic soils in which $k_x = k_y = k.$

(b) Determination of hydrostatic pressure:- The hydrostatic pressure at any point within the soil mass is given by $= u = \gamma_w \ h_w$

Where u= hydrostatic pressure, h_w =Pizometric head

The hydrostatic pressure in terms of Pizometric head h_w is calculated from the relation –

$$h_{w\ =\ h\ -\ Z}$$

Where h = Hydraulic potential at the point under consideration.

Z= position head of the point above datum, considered positive upwards.

(c)Determination of seepage pressure: - The hydraulic potential h at any point located after n potential drops, each value Δ h is given by

$$h = H - n \Delta h$$

the seepage pressure at any point equals the hydraulic potential or balanced hydraulic head multiplied by unit weight of water and hence is given by $p_{s\ =} h \ \gamma_{w\ =}$ $(H - n \Delta h) \gamma_w$

The pressure acts in the direction of flow.

(d) Determination of exist gradient:- The exit gradient is hydraulic at the downstream end of the flow line where the percolating water leaves the soil mass and emerges into the free water at the downstream. The exit gradient can be calculated from the following expression, in which Δ h represents the potential drop and l the average length of last field in the flow net at exit end:

$i_e = (\Delta h / l).$

CHAPTER- 6

COMPACTION AND CONSOLIDATION

6.1 COMPACTION

Compaction is the application of mechanical energy to a soil so as to rearrange its particles and reduce the void ratio.

It is applied to improve the properties of an existing soil or in the process of placing fill such as in the construction of embankments, road bases, runways, earth dams, and reinforced earth walls. Compaction is also used to prepare a level surface during construction of buildings. There is usually no change in the water content and in the size of the individual soil particles.

The objectives of compaction are:

- To increase soil shear strength and therefore its bearing capacity.
- To reduce subsequent settlement under working loads.
- To reduce soil permeability making it more difficult for water to flow through.

LIGHT AND HEAVY COMPACTION TEST
Laboratory Compaction
The variation in compaction with water content and compactive effort is first determined in the laboratory. There are several tests with standard procedures such as:

- Indian Standard Light Compaction Test (similar to Standard Proctor Test)
- Indian Standard Heavy Compaction Test (similar to Modified Proctor Test)

Indian Standard Light Compaction Test
Soil is compacted into a 1000 cm^3 mould in 3 equal layers, each layer receiving 25 blows of a 2.6 kg rammer dropped from a height of 310 mm above the soil. The compaction is repeated at various moisture contents.

Indian Standard Heavy Compaction Test
It was found that the Light Compaction Test (Standard Test) could not reproduce the densities measured in the field under heavier loading conditions, and this led to the development of the Heavy Compaction Test (Modified Test). The equipment and procedure are essentially the same as that used for the Standard Test except that the soil is compacted in 5 layers, each layer also receiving 25 blows. The same mould is also used. To provide the increased compactive effort, a heavier rammer of 4.9 kg and a greater drop height of 450 mm are used.

OPTIMUM MOISTURE CONTENT OF SOIL,MAXIMUM DRY DENSITY,ZERO AIR VOID LINE

To assess the degree of compaction, it is necessary to use the dry unit weight, which is an indicator of compactness of solid soil particles in a given volume. The laboratory testing is meant to establish the maximum dry density that can be attained for a given soil with a standard amount of compactive effort.

In the test, the dry density cannot be determined directly, and as such the bulk density

and the moisture content are obtained first to calculate the dry density as $\gamma_d = \dfrac{\gamma_t}{1+w}$,

where γ_t = bulk density, and w = water content.

A series of samples of the soil are compacted at different water contents, and a curve is drawn with axes of dry density and water content. The resulting plot usually has a distinct peak as shown. Such inverted "V" curves are obtained for**cohesive soils** (or soils with fines), and are known as compaction curves.
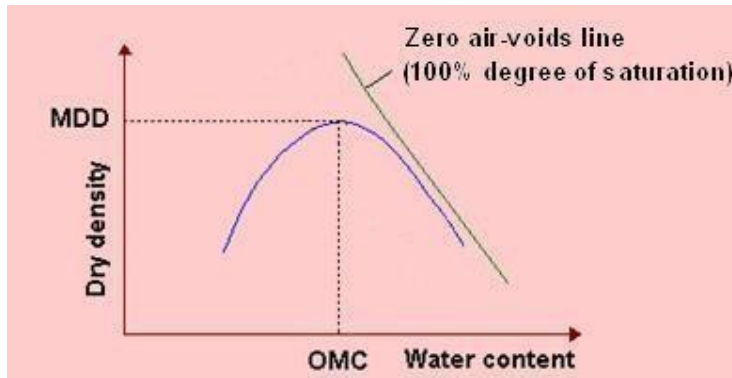
Dry density can be related to water content and degree of saturation (S) as

$$\gamma_d = \frac{G_s \cdot \gamma_w}{1+e} = \frac{G_s \cdot \gamma_w}{1+\dfrac{w \cdot G_s}{S}}$$

Thus, it can be visualized that an increase of dry density means a decrease of voids ratio and a more compact soil.

Similarly, dry density can be related to percentage air voids (n_a) as

$$\gamma_d = \frac{(1-n_a) G_s \cdot \gamma_w}{1+w G_s}$$

The relation between moisture content and dry unit weight for a saturated soil is the **zero air-voids line.** It is not feasible to expel air completely by compaction, no matter how much compactive effort is used and in whatever manner.

**Effect of Increasing Water Content**
As water is added to a soil at low moisture contents, it becomes easier for the particles to move past one another during the application of compacting force. The particles come closer, the voids are reduced and this causes the dry density to increase. As the water content increases, the soil particles develop larger water films around them.

This increase in dry density continues till a stage is reached where water starts occupying the space that could have been occupied by the soil grains. Thus the water at this stage hinders the closer packing of grains and reduces the dry unit weight.
The **maximum dry density (MDD)** occurs at an **optimum water content (OMC),** and their values can be obtained from the plot.

Effect of Increasing Compactive Effort

The effect of increasing compactive effort is shown. Different curves are obtained for different compactive efforts. A greater compactive effort reduces the optimum moisture content and increases the maximum dry density.
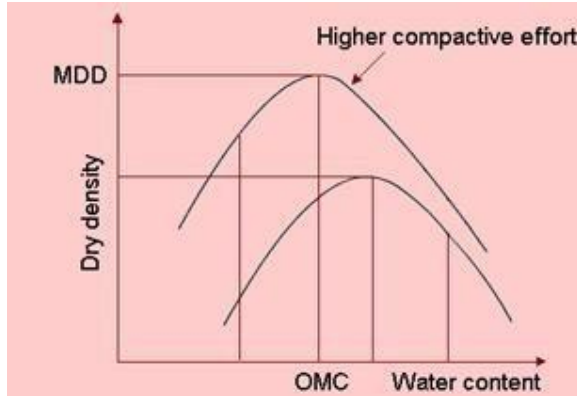
An increase in compactive effort produces a very large increase in dry density for soil when it is compacted at water contents drier than the optimum moisture content.It should be noted that for moisture contents greater than the optimum, the use of heavier compaction effort will have only a small effect on increasing dry unit weights.

It can be seen that the compaction curve is not a unique soil characteristic. It depends on the compaction effort. For this reason, it is important to specify the compaction procedure (light or heavy) when giving values of MDD and OMC.

6.2 Factors Affecting Compaction

The factors that influence the achieved degree of compaction in the laboratory are:

- Plasticity of the soil
- Water content
- Compactive effort

6.3 FIELD COMPACTION METHODS AND THEIR SUITABILITY

Compaction Equipment

Most of the compaction in the field is done with rollers. The four most common types of rollers are

1. Smooth-wheel rollers (or smooth-drum rollers)
2. Pneumatic rubber-tired rollers
3. Sheepsfoot rollers
4. Vibratory rollers

Smooth-wheel rollers are suitable for proof rolling subgrades and for finishing operation of fills with sandy and clayey soils. These rollers provide 100% coverage under the wheels, with ground contact pressures as high as 310 to 380 kN/m$_2$ (45 to 55 lb/in$_2$). They are not suitable for producing high unit weights of compaction when used on thicker layers.

Pneumatic rubber-tired rollers are better in many respects than the smooth-wheel rollers. The former are heavily loaded with several rows of tires. These tires are closely spaced—four to six in a row. The contact pressure under the tires can range from 600 to 700 kN/m$_2$ (85 to 100 lb/in$_2$), and they produce about 70 to 80% coverage. Pneumatic rollers can be used for sandy and clayey soil compaction. Compaction is achieved by a combination of pressure and kneading action.

Sheepsfoot rollers are drums with a large number of projections. The area
of each projection may range from 25 to 85 cm2 (_ 4 to 13 in2). These rollers are most effective
in compacting clayey soils. The contact pressure under the projections can range from 1400 to 7000 kN/m2 (200 to 1000 lb/in2). During compaction in the field, the initial passes compact the lower portion of a lift. Compaction at the top and middle of a lift is done at a later stage. Vibratory rollers are extremely efficient in compacting granular soils. Vibrators can
be attached to smooth-wheel, pneumatic rubber-tired, or sheepsfoot rollers to provide vibratory effects to the soil.
.
Handheld vibrating plates can be used for effective compaction of granular soils over
a limited area. Vibrating plates are also gang-mounted on machines. These plates can be used in less restricted areas..

6.4 CONSOLIDATION:

According to Terzaghi (1943), "a decrease of water content of a saturated soil without replacement of the water by air is called a process of consolidation." When saturated clayey soils-which have a low coefficient of permeability-are subjected to a compressive stress due to a foundation loading, the ore water pressure will immediately increase; however, due to the low permeability of the soil, there will be a time lag between the application of load and the extrusion of the pore water and, thus, the settlement.

DIFFERENCE BETWEEN COMPACTION AND CONSOLIDATION:

Consolidation and compaction are totally different process. Though both process results a reduction in volume, it is important to know the difference between them. These are:

a. Compaction reduces volume of soil by rapid mechanical methods like tamping, rolling and vibration; whereas consolidation process reduces volume gradually by static, sustained loading.

b. Compaction decreases volume by expelling air from partially saturated or dry soil; whereas consolidation process reduces volume by squeezing out water from saturated soil. In compaction process water content is not altered.

c. Compaction is a human generated pressing method to produce high unit weight of soil. Thus increasing other properties to have better founding soil. In contrast, consolidation is natural process where volume of saturated soil mass reduced by static loads from the weight of building or other structures that is transferred to soil through a foundation system.

6.5 SPRING ANALOGY METHOD

The process of consolidation is often explained with an idealized system composed of a spring, a container with a hole in its cover, and water. In this system, the spring represents the compressibility or the structure of the soil itself, and the water which fills the container represents the pore water in the soil.
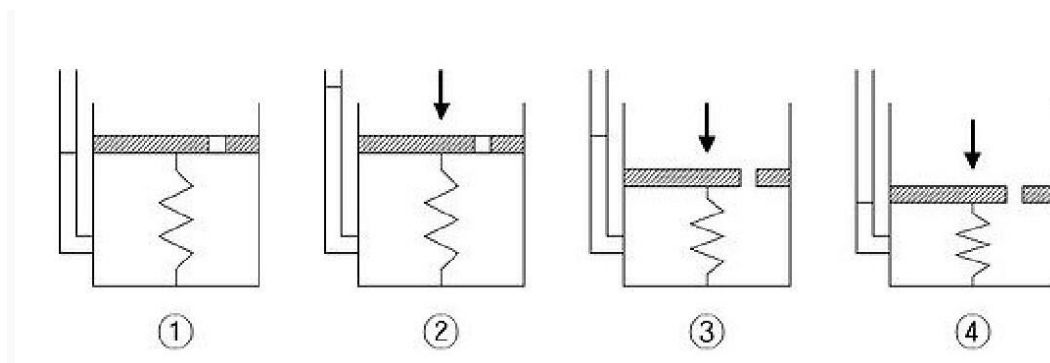
1. The container is completely filled with water, and the hole is closed. (Fully saturated soil)
2. A load is applied onto the cover, while the hole is still unopened. At this stage, only the water resists the applied load. (Development of excess pore water pressure)
3. As soon as the hole is opened, water starts to drain out through the hole and the spring shortens. (Drainage of excess pore water pressure)
4. After some time, the drainage of water no longer occurs. Now, the spring alone resists the applied load. (Full dissipation of excess pore water pressure. End of consolidation)

Pressure-Void Ratio Curve:-
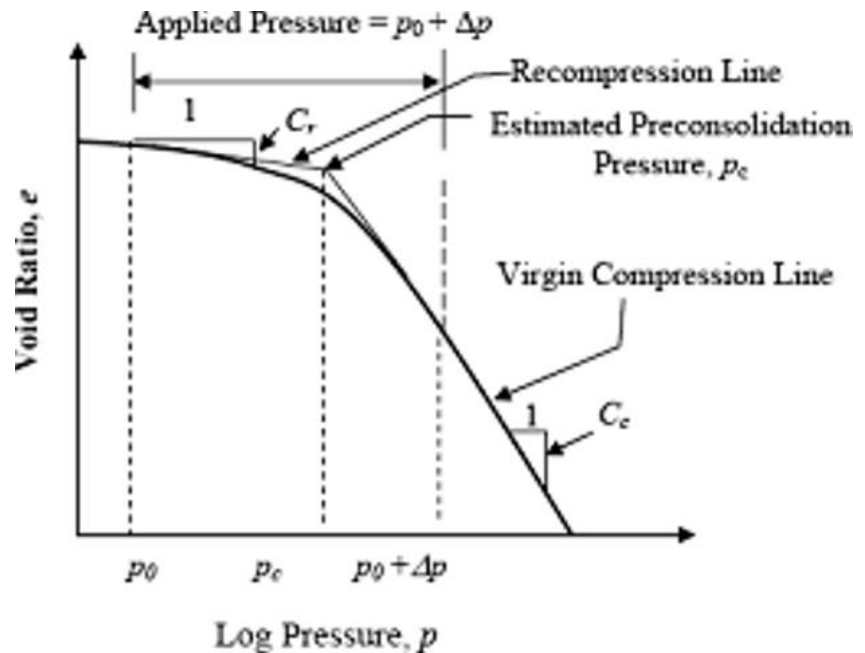
Normally consolidated, Under consolidated and Over consolidated soil,

Consolidation is a process by which soils decrease in volume. According to Karl von Terzaghi "consolidation is any process which involves a decrease in water content of saturated soil without replacement of water by air." In general it is the process in which reduction in volume takes place by expulsion of water under long term static loads. It occurs when stress is applied to a soil that causes the soil particles to pack together more tightly, therefore reducing its bulk volume. When this occurs in a soil

that is saturated with water, water will be squeezed out of the soil. The magnitude of consolidation can be predicted by many different methods. In the Classical Method, developed by Terzaghi, soils are tested with an <u>oedometer test</u> to determine their compression index. This can be used to predict the amount of consolidation.

When stress is removed from a consolidated soil, the soil will rebound, regaining some of the volume it had lost in the consolidation process. If the stress is reapplied, the soil will consolidate again along a recompression curve, defined by the recompression index. The soil which had its load removed is considered to be overconsolidated. This is the case for soils which have previously had glaciers on them. The highest stress that it has been subjected to is termed the preconsolidation stress. The over consolidation ratio or OCR is defined as the highest stress experienced divided by the current stress. A soil which is currently experiencing its highest stress is said to be normally consolidated and to have an OCR of one. A soil could be considered underconsolidated immediately after a new load is applied but before the excess pore water pressure has had time to dissipate.

Assumption of Terzaghi's theory of one-dimensional consolidation

1. The soil is homogenous (uniform in composition throughout).
2. The soil is fully saturated (zero air voids due to water content being so high).
3. The solid particles and water are incompressible.
4. Compression and flow are one-dimensional (vertical axis being the one of interest).
5. Strains in the soil are relatively small.
6. Darcy's Law is valid for all hydraulic gradients.
7. The coefficient of permeability and the coefficient of volume compressibility remain constant throughout the process.
8. There is a unique relationship, independent of time, between the void ratio and effective stress

Coefficient of Consolidation:

The Coefficient of consolidation at each pressures increment is calculated by using

the following equations:

    i. $C_v = 0.197\ d2/t_{50}$ (Log fitting method)

    In the log fitting method, a plot is made between dial readings and logarithmic of time, the time corresponding to 50% consolidation is determined

ii. $Cv = 0.848\ d2/t_{90}$ (Square fitting method)

In the square root fitting method, a plot is made between dial readings and square root of time and the time corresponding to 90% consolidation is determined.

Time Factor:-

The magnitude of consolidation settlement is often calculated using Terzaghi's expression for average degree of consolidation (U) with respect to time. Developed during a time of limited computing capabilities, Terzaghi's series solution to the one-dimensional consolidation equation was generalized using a dimensionless time factor (T), where a single U-T curve is used to describe the consolidation behavior of both singly and doubly drained strata. As a result, any comparisons between one- and two-way drainage are indirect and confined to discrete values of time. By introducing a modified time factor T* in terms of layer thickness (D) instead of the maximum drainage path length (Hdr), it is now possible to observe the effect of drainage conditions over a continuous range of time for a variety of asymmetric initial excess pore pressure distributions. Although two separate U-T plots are required (for singly and doubly drained cases), the time factor at specific times remains the same for both cases, enabling a direct visual comparison. The importance of a revised time factor is evident when observing the endpoint of consolidation, which occurs as U approaches 100%. This occurs at T*≅0.5 for two-way drainage and at T*≅2 for one-way drainage, an observation not possible using the traditional expression for time factor.

Estimation of consolidation settlements

Prediction of ground settlements have always been a big challenge for the engineers that are responsible for the design of subway tunnel projects. Since ground settlement is a crucial concept directly affecting the successfulness of a project, it must be taken seriously and should be accurately estimated.

Categories:

1. <u>Immediate settlement</u> - elastic deformation of dry soil and moist and saturated soils without change to moisture content
   a. due to high permeability, pore pressure in clays support the entire added load and no immediate settlement occurs
   b. generally, due to the construction process, immediate settlement is not important
2. <u>Primary consolidation settlement</u> - volume change in saturated cohesive soils because of the expulsion of water from void spaces
   a. high permeability of sandy, cohesionless soils result in near immediate drainage due to the increase in pore water pressure and no primary consolidation settlement occur

DIFFERENCE BETWEEN PRIMARY AND SECONDARY COSOLIDATION

Primary consolidation

This method assumes consolidation occurs in only one-dimension. Laboratory data is used to construct a plot of strain or void ratio versus effective stress where the effective stress axis is on a logarithmic scale. The plot's slope is the compression index or recompression index. The equation for consolidation settlement of a normally consolidated soil can then be determined to be:

$$\delta_c = \frac{C_c}{1 + e_0} H \log \left(\frac{\sigma'_{zf}}{\sigma'_{z0}} \right)$$

where

δ_c is the settlement due to consolidation.
C_c is the compression index.
e_0 is the initial void ratio.
H is the height of the soil.
σ_{zf} is the final vertical stress.
σ_{z0} is the initial vertical stress.

C_c can be replaced by C_r (the recompression index) for use in overconsolidated soils where the final effective stress is less than the preconsolidation stress. When the final effective stress is greater than the preconsolidation stress, the two equations must be used in combination to model both the recompression portion and the virgin compression portion of the consolidation processes, as follows,

$$\delta_c = \frac{C_r}{1 + e_0} H \log \left(\frac{\sigma'_{zc}}{\sigma'_{z0}} \right) + \frac{C_c}{1 + e_0} H \log \left(\frac{\sigma'_{zf}}{\sigma'_{zc}} \right)$$

where σ_{zc} is the preconsolidation stress of the soil.

Secondary consolidation

Secondary consolidation is the consolidation of soil that takes place after primary consolidation. Even after the reduction of hydrostatic pressure some compression of soil takes place at slow rate. This is known as secondary consolidation .Secondary consolidation is caused by creep, viscous behavior of the clay- ater system, consolidation of organic matter, and other processes. In sand, settlement caused by secondary compression is negligible, but in peat, it is very significant. Due to secondary consolidation some of the highly viscous water between the points of contact is forced out.

Secondary consolidation is given by the formula

$$S_s = \frac{H_0}{1 + e_0} C_\alpha \log\left(\frac{t}{t_{90}}\right)$$

Where

H_0 is the height of the consolidating medium

e_0 is the initial void ratio

C_a is the secondary compression index

t   is the length of time after consolidation considered

t_{90} is the length of time for achieving 90% consolidation

CHAPTER-7

SHEAR STRENGTH

Introduction

Soil mass when loaded may fail due to shear stress induced in it. Examples of such failures are sinking of soil mass under heavily loaded foundation, spalling of soil along the edge of vertical cut, slide in an earth embankment with a steep slope movement of backfill behind a weak retaining wall etc. In all the above cases, the soil fails essentially due to shear. When the shear stress induced in a mass of soil reaches limiting value, shear deformation occurs, which leads to the failure of soil mass. The resistance offered by the soil to shear is known as shear resistance.

The maximum shearing resistance of soil against continuous shear deformation along potential failure plane is known as **shear strength** of soil. The plane along which failure of soil takes place due to sliding is known as failure plane. Failure will take place on the plane on which the shear stress exceeds the shear resistance. However, if the soil has weak planes, the failure will be located in the weakest zone. Failure may not take place along the plane of maximum shear stress, i.e., the plane which makes 45^0 with the principal planes.

The shearing resistance of soil is composed of two components: Normal stress dependent and normal stress independent. Examples of the above two cases are:

1. Frictional resistance between the particles at the point of contact
2. Cohesion or force of attraction between soil particles. It is characteristic of soil state and is independent of normal stress across the plane.

The above two components can be better understood by comparing two materials, sand and clay. Considerable force is required to shear a block of clay as shown in the Figure 1(a) even when there is no external force acting on the block. This force is higher when the block is dry and lower with increase in water content of the soil sample. This component is called cohesion. On the other hand, if we take a sample of sand in a split mould and try to shear it, the force required is practically nil when there is no external normal force. Now, if we apply external normal pressure, the force

required to shear the sample increases and is proportional to the normal pressure applied. This component is called friction.

Shear strength of the cohesionless soil results from inter granular friction as above. Plastic undrained clay does not have external friction. Hence, strength of soil results from cohesion alone. In other intermediate soils, shear strength of such soil results from internal friction as well as cohesion.
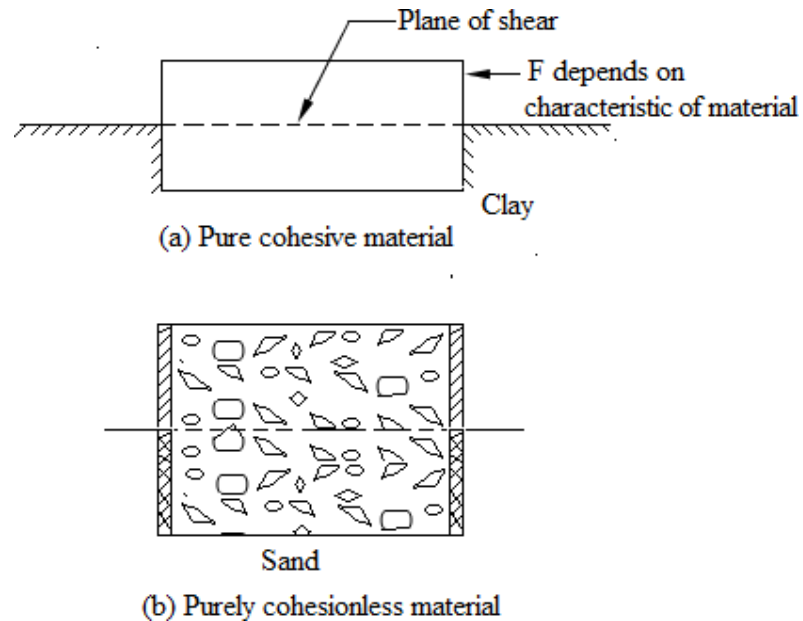

Figure 1

Theory of two dimensional stress system: Mohr's Stress Circle

Innumerable planes pass through each point in a soil mass. The stress components on each plane through the point depend upon the direction of the plane. It is known from strength of material that there exists three mutually perpendicular planes through a point on which there is no shear stress and only normal stress acts. Such planes are called principal planes and the normal stresses, the principal stresses. In order of their magnitude, these stresses are known as major principal stress (σ_1), intermediate principal stress (σ_2) and minor principal stress (σ_3). However, in most soil we deal with, failure of soil mass is independent of intermediate stress. In such problems two dimensional stress analyses gives acceptable results for the solution of such failure problems.

Consider the case of a soil element as in Figure 2 whose sides are principal planes i.e., only normal stresses are acting on the faces of the element. The stress components at a point on a give plane are given by

$$\sigma = \frac{\sigma_1 + \sigma_3}{2} + \frac{\sigma_1 - \sigma_3}{2} \cos 2\alpha$$

$$\tau = \frac{\sigma_1 - \sigma_3}{2} \sin 2\alpha$$

where σ and τ are normal stress and shear stress component on a plane inclined at an angle of α with the major principal plane.

The above results can be represented by drawing a circle with radius $\frac{\sigma_1 - \sigma_3}{2}$. The circle so drawn is known as Mohr's circle. Each point on the circumference of the circle gives two stress coordinates at that point on an inclined plane.

(a) Soil element

(b) Mohr's stress circle
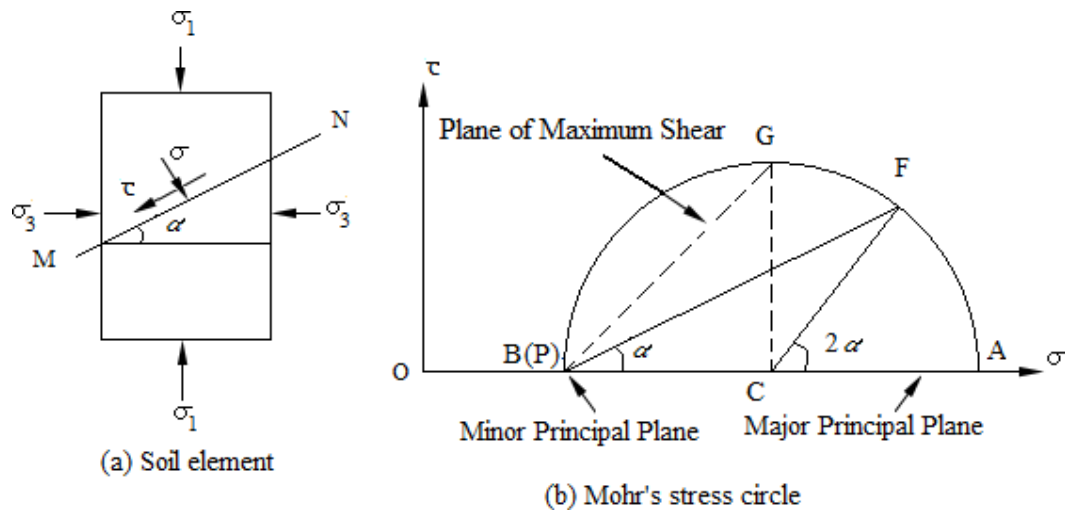
Figure 2 Mohr's stress circle

In Figure 2(a), the major principal plane is horizontal and minor principal plane is vertical. Point A in the Mohr's circle represents the major principal stress $(\sigma_1, 0)$ and B represents the minor principal stress $(\sigma_3, 0)$. To determine the stress components in a plane through the point, a point called pole is to be located on the circle. The pole is drawn by drawing a straight line parallel to the plane on which the stress conditions

are known. Hence, the pole P is located by drawing a horizontal line through the point A representing the major principal stress (σ_1). The pole can also be represented by drawing a vertical line through B representing minor principal plane (σ_3). To know the stress on the inclined plane, a straight line PF parallel to the plane is drawn through the pole P. The point F on the circle gives the coordinates of the stress on the plane inclined at an angle α with the direction of major principal plane. The shear stress is considered to be positive if its direction gives a clockwise moment about a point outside the wedge such as point E.

Consider another soil element as shown in Figure 3(a) in which major principal planes are not horizontal and vertical, but are inclined to y and x-directions. The corresponding Mohr's stress circle is drawn as shown in Figure 3(b). Point A represents principal major principal stress $(\sigma_1, 0)$ and minor principal stress $(\sigma_3, 0)$. To locate the pole, a line parallel to the major principal plane is drawn through A to intersect the circle at P. PB gives the direction of the minor principal plane. To determine the stress components on any plane MN inclined at an angle α with the major principal plane, a line making an angle of α with PA is drawn through P, to intersect the circle at F. The coordinates of point F give the stress components on the plane MN.
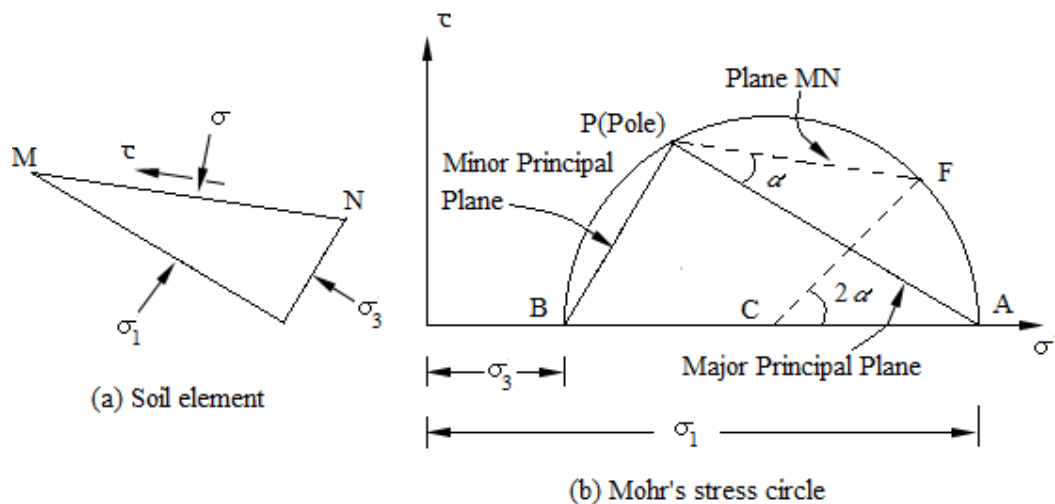

Figure 3 Mohr's stress circle

Mohr-Coulomb Theory of Failure

Various theories of failure of soil have been proposed by many soil scientists. Of these, the one proposed by Coulomb and generalised by Mohr has been the most useful for failure problems dealing with soil and hence has got wide acceptance amongst the soil scientists. This failure theory is known as Mohr-Coulomb failure theory.

According to Mohr, the failure of soli along a plane takes place when the shear stress on that plane exceeds the shear resistance of the soil. The shear resistance is a function the normal stress on the failure plane. It is expressed as

$$\tau_f = S = f(\sigma)$$

Where $\tau_f = S = $ Shear stress at failure = Shear resistance

If the normal stress and shear stress are plotted, a curve is obtained. This curve is called the shear envelope. Coulomb assumed the relationship between τ_f and σ as linear and gave the following strength equation.

$$S = C + \sigma \tan \phi$$

For most of the cases of stability of soil, Mohr's failure can be approximated as a straight line for practical purposes and thus agrees with the above strength equation given by Coulomb.
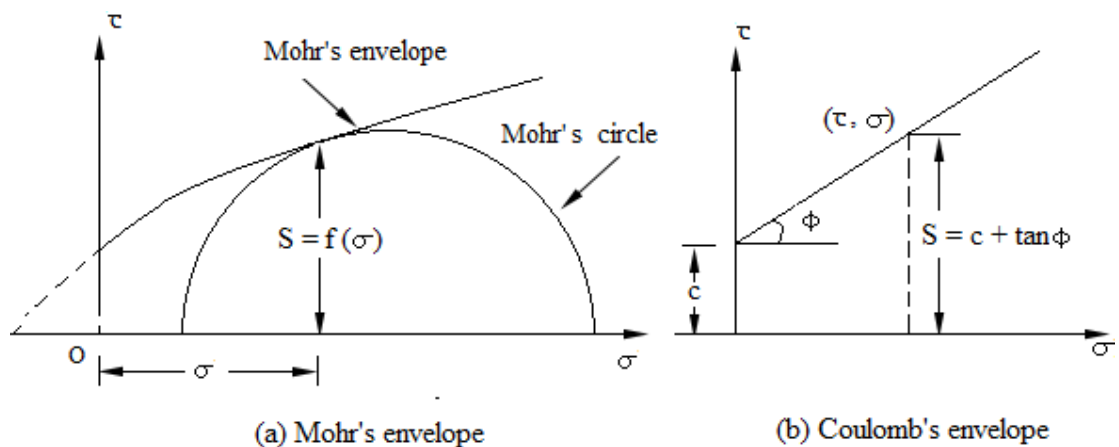

(a) Mohr's envelope   (b) Coulomb's envelope

Figure 4

C and ϕ in the expression $S = C + \sigma \tan \phi$ are empirical constants and are as cohesion and angle of friction or shearing resistance. In general the above constants are known as shear strength parameters.

Depending upon the nature of soil and the shear strength parameters, soils can be described as (i) cohesive soil, (ii) cohesion-less soil, and (iii) purely cohesive soil. The strength envelopes for the three cases are shown in the Figure 5.
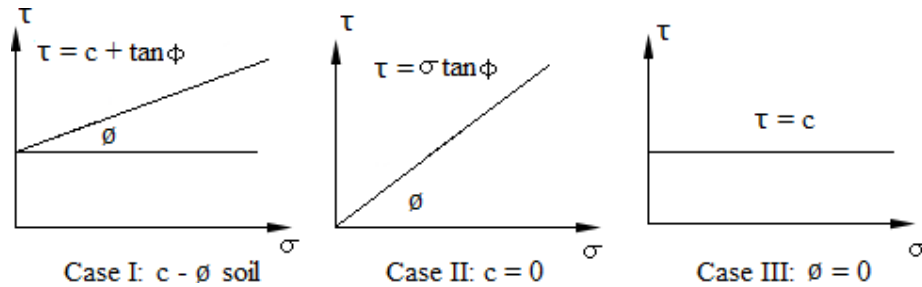

Figure 5 Strength envelopes for three types of soils

Effective stress principle

Extensive experimental studies on remoulded clays have shown the shearing strength of soil mass is controlled by the effective stress and not by the total normal stress on the plane of shear. The values of shear parameters, i.e., cohesion and angle of shearing resistance do depend upon the pore water pressure of the soil. Therefore, the Mohr-Coulomb strength equation may be expressed in terms of effective stress.

$$\tau_f = C' + \sigma' \tan \phi'$$

Where c' and ϕ' are termed as effective shear parameters.

In terms of total stresses, the equation takes the form

$$\tau_f = C_u + \sigma \tan \phi_u$$

Where C_u is the apparent cohesion and ϕ_u is the apparent angle of shearing resistance

Determination of failure plane

Failure of soil may not take place along the plane of maximum shear stress. The failure will take place along the most dangerous plane called failure plane. The failure plane is the one on which the difference between shear strength and shear stress, i.e., $(\tau_f - \tau)$ is minimum. To determine the angle of failure plane with the major principal plane, let us express the normal stress σ' and shear stress τ' on any plane inclined at an angle of α to the major principal plane.

$$\sigma' = \frac{\sigma'_1 + \sigma'_3}{2} + \frac{\sigma'_1 - \sigma'_3}{2}\cos 2\alpha$$

$$\tau' = \frac{\sigma'_1 - \sigma'_3}{2}\sin 2\alpha$$

The equation of shear strength is given by

$$\tau_f = C' + \sigma' \tan\phi'$$

$$= C' + \left[\frac{\sigma'_1 + \sigma'_3}{2} + \frac{\sigma'_1 - \sigma'_3}{2}\cos 2\alpha\right]\tan\phi'$$

So, $(\tau_f - \tau) = C' + \dfrac{\sigma'_1 + \sigma'_3}{2}\tan\phi' + \dfrac{\sigma'_1 - \sigma'_3}{2}\cos 2\alpha \tan\phi' - \dfrac{\sigma'_1 - \sigma'_3}{2}$ as

For minimum value of $(\tau_f - \tau), \dfrac{d}{d\alpha}(\tau_f - \tau) = 0$

Differentiating $(\tau_f - \tau)$ with respect to α

$$\frac{d}{d\alpha}(\tau_f - \tau) = -(\sigma'_1 - \sigma'_3)\sin 2\alpha \tan\phi' - (\sigma'_1 - \sigma'_3)\cos 2\alpha$$

$$\cos 2\alpha = -\sin 2\alpha \tan\phi'$$

$$\cot 2\alpha = \cot(90 + \phi')$$

$$\alpha = \alpha_f = 45^0 + \frac{\phi}{2}$$

where α_f is the angle of failure plane with respect to major principal plane.

The above expression for location of failure plane can be directly derived from Mohr's circle shown in Figure 6. *EF* represents the failure envelope given by the straight line $\tau_f = C' + \sigma' \tan \phi'$. *P* is the pole with stress coordinates $(\sigma'_3, 0)$. The Mohr's circle is tangential to the Mohr envelope at the point *F*. *PF* represents the direction of failure plane, inclined at an angle α_f with the direction of major principal plane. From the geometry of Figure 6, we get from triangle *EFK*.

$$2\alpha_f = 90^0 + \phi'$$

$$\alpha_f = 45^0 + \frac{\phi'}{2}$$
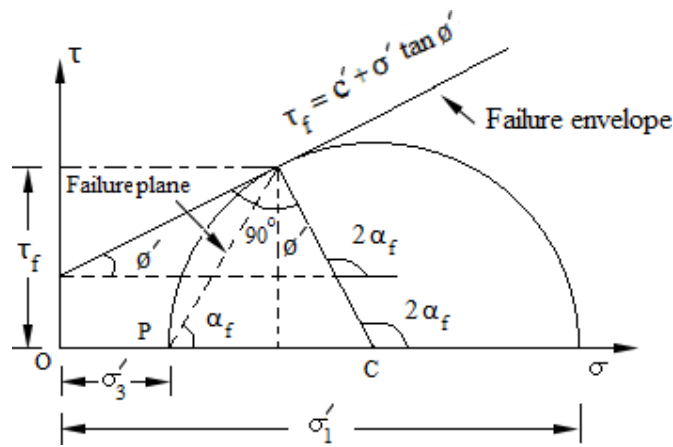
Figure 6

It may be noted that any point on the failure envelope represents two stress components σ' and τ_f at failure. And for each σ' and τ_f, there exists two values of principal effective stress on two principal planes for which failure takes place. It is evident from Figure 5 that τ_f at failure is less than the maximum shear stress, corresponding to the point *G*, and acting on the plane *PG*. Thus the failure plane does not carry maximum shear stress, and the plane which has the maximum shear stress is not the failure plane.

Determination of shear strength parameters

Shear tests are conducted on undisturbed soil samples obtained from the field. The test results are used to plot failure envelope to determine the shear strength

parameters. It is to be noted that the shear strength parameters are fundamental properties of soil and are considered as coefficients obtained from the geometry of the strength envelope drawn by using shear test results. So, during test on saturated samples, care should be taken to simulate the field drainage condition.

Following four methods of shear tests are commonly used in the laboratory.

1. Direct shear test
2. Triaxial compression test
3. Unconfined compression test
4. Vane shear test

Based on the drainage conditions, the shear tests are classified as

1. Consolidated drained test (Drained test/Quick test)
2. Consolidated undrained test
3. Unconsolidated undrained test

1. Consolidated drained test

This is also known as drained test. In this test drainage of water is allowed during the test. The soil sample is first consolidated fully under the normal load (in direct shear test) or the all round pressure (in triaxial test) and the shear stress is applied slowly enough for the water in the sample to drain away. This simulates the long term conditions in the life of a structure, i.e., the long term stability of earth dam. The effective stress parameters are used.

2. Consolidated undrained test

In this test, the soil is consolidated under the normal load or the all-round pressure but shearing is done rapidly so that drainage does not take place. This simulates the sudden effects during the life of a structure, e.g., sudden drawdown of upstream water level in an earth dam. The parameters used are C_u and ϕ_u . If pore pressure measurements are made then effective stress parameters can be used.

3. Unconsolidated undrained test

In this case, the normal load or the all-round pressure as well as shear stress are applied under conditions of no drainage. The soil is not consolidated and shearing is done rapidly. Therefore, effective stresses and hence the shear strength of the soil do

not get mobilised. This simulates short term failure conditions in a structure, e.g., stability of an earth dam immediately after construction. The total stress parameters are used for these cases.

Direct shear test

The direct shear test apparatus consists of (i) shear box of square or circular section, (ii) loading yoke for applying normal force, (iii) geared jack for applying shear force, (iv) proving ring to measure shear force, and (v) strain gauges to measure horizontal displacement and vertical displacement for volume change.

The shear box consists of two halves which can slide relative to each other. The lower half is rigidly held in position with the bottom of the shear box container, which slides on rollers when pushed by a jack provided to apply shear force. The geared jack may be driven either by electric motor or by hand. The upper half of the box is butt against a proving ring. The soil sample is placed and compacted in the shear box. The sample held in position between a pair of metal grids and porous stones or plates as shown in the Figure 7. The grid plates, provided with linear slots, are placed above the top and below the bottom of the specimen. To have proper grip with the soil specimen, the linear slots in the grid plate are aligned perpendicular to the direction of the shearing force. The soil specimen is compacted in shear box by clamping together with the help of two screws provided for the purpose. However, these screws are removed before shearing force is applied. Direct shear test may be of two types. Strain controlled shear box and stress controlled shear box. The working principles of two types of shear box are explained in the following paragraphs.

In case of strain controlled shear test a normal load N is applied on the specimen by means of loading yoke and is kept constant throughout the test. The shearing strain is increased at a constant rate by pushing the lower box through the geared jack. The movement of lower part of the box is transmitted through the specimen to the upper part of the box. The proving ring attached to the upper part reads the shear force F at failure. A number of tests are conducted on identical specimens with increased normal loads and the corresponding shear force recorded. A graph is plotted between the shear force F as ordinate and the normal load N as the abscissa. The plot so obtained is known as the failure envelope. Figure 8(b) shows the failure envelope plotted as a

function of shear stress (τ) and the normal stress (σ). The scale of both τ and σ are kept equal to measure the angle of shearing resistance (ϕ) directly from the plot.
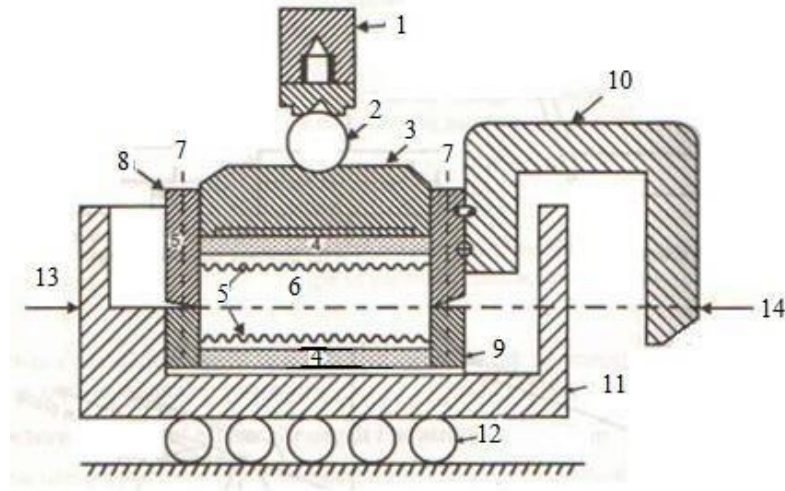

Figure 7 Shear box with accessories

1.	Loading yoke	8.	Upper part of shear box
2.	Steel ball	9.	Lower part of shear box
3.	Loading pad	10.	U-arm
4.	Porous stones	11.	Container for shear box
5.	Metal grids	12.	Rollers
6.	Soil specimen	13.	Shear force applied by jack
7.	Pins to fix two halves of shear box	14.	Shear resistance measured by proving ringdial gauge

(a) Principle of direct shear box

(b) Mohr's envelope and principal stresses during the test
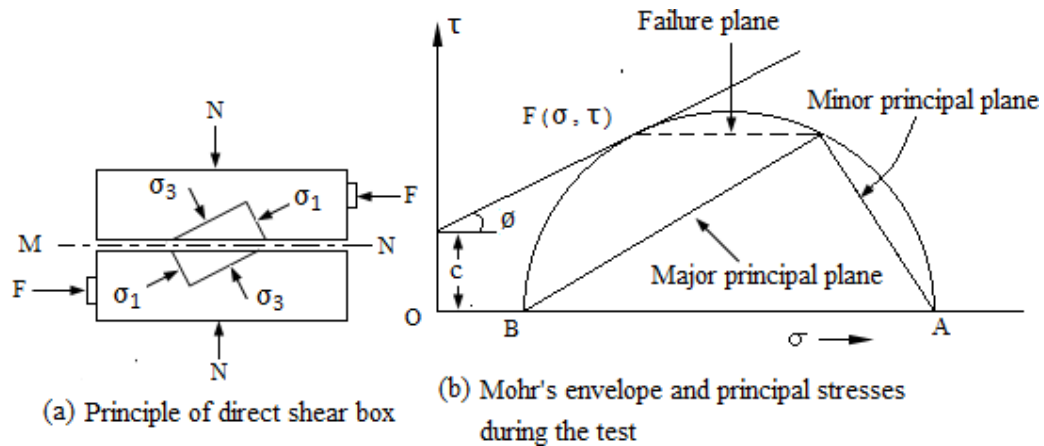
Figure 8 Shear box test

Advantages of direct shear test

1. Direct shear test is a simple test compared to the more complex triaxial test.
2. As the thickness of sample is small, it allows quick drainage and rapid dissipation of pore pressure during the test.

Disadvantages of direct shear test

1. The distribution of normal stresses and shear stresses over the potential failure plane is not uniform. The stress is more at the edges and less at the centre. Hence, progressive shear failure takes place as the shear strength is not mobilised simultaneously at all points on the failure plane.
2. The failure plane is predetermined, which may not be the weakest plane.
3. The area under shear gradually decreases as the test progresses. The corrected area at failure, A_f should be used for computing the values of normal stress σ and shear stress τ .
4. There is little control on the drainage of pore water of soil as compared to the triaxial test.
5. The stress on account of lateral restraint due to side walls of shear box is not accounted for in the test.
6. There is no provision for measurement of pore water pressure.

Problem 1.

From a direct shear test on undisturbed soil sample, following data have been obtained. Evaluate the undrained shear strength parameters. Determine shear strength, major and minor principal stresses and their planes in the case of specimen of sample subjected to a normal stress of 100 kN/m².

Normal stress (kN/m²)	70	96	114
Shear stress at failure(kN/m²)	138	156	170

Solution.

Plot the shear stress versus normal stress to obtain the failure envelope keeping the scale same for both the stresses. From the plot in Figure 9,

The angle of shearing resistance, $\phi = 36^0$; cohesion, $c = 84$ kN/m^2

The shear strength corresponding to the normal stress of 100 kN/m² is 160 kN/m². The coordinate corresponding to (100, 160) is the failure point F. Draw the Mohr's circle so that the failure envelope is tangent to the circle at F. To do so, draw FC perpendicular to the failure envelope. With C as centre and CF as radius, draw a circle so as to intersect the normal load axis at A and B. Point A corresponds to the major principal stress $\sigma_1 = 410$ kN/m^2 and point B corresponds to the minor principal stress $\sigma_3 = 20$ kN/m^2.
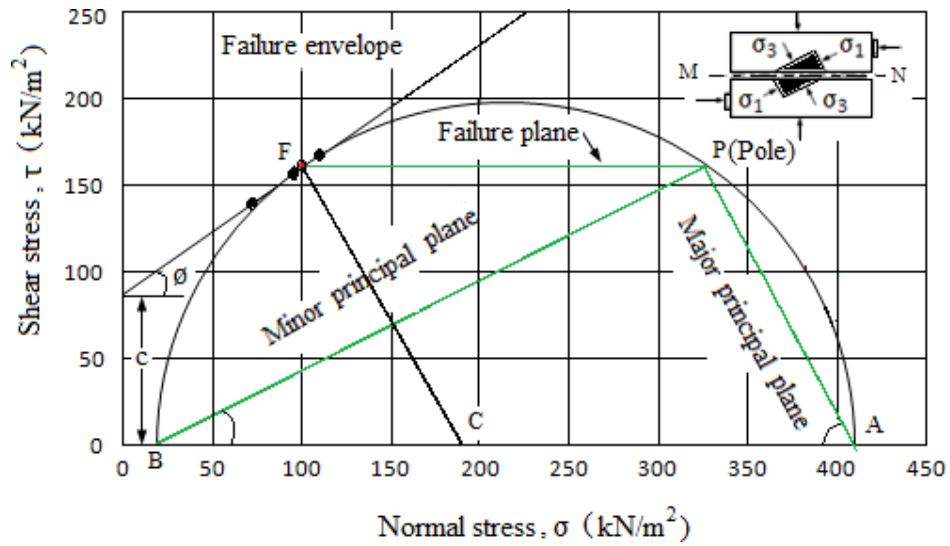
Figure 9

To locate the position of the pole, draw a line *FP* parallel to the failure plane in the shear box (horizontal). *P* is the pole. *PA* is the direction of major principal plane which makes an angle 57 in the clockwise direction with the plane of shear. *PB* is the minor principal plane, making an angle of 58 in the anticlockwise direction with the plane of shearing.

Problem 2.

A sample of cohesionless sand in a direct shear test fails under a shear stress of 160 kN/m² when the normal stress is 140 kN/m². Find the angle of shearing resistance and the principal stress at failure.

Solution.

Plot the failure envelope passing through the origin and the point with coordinate (140, 160) as normal stress and shear stress coordinates. The scale for both the stress axes are kept the same.

From the plot in the Figure 10,

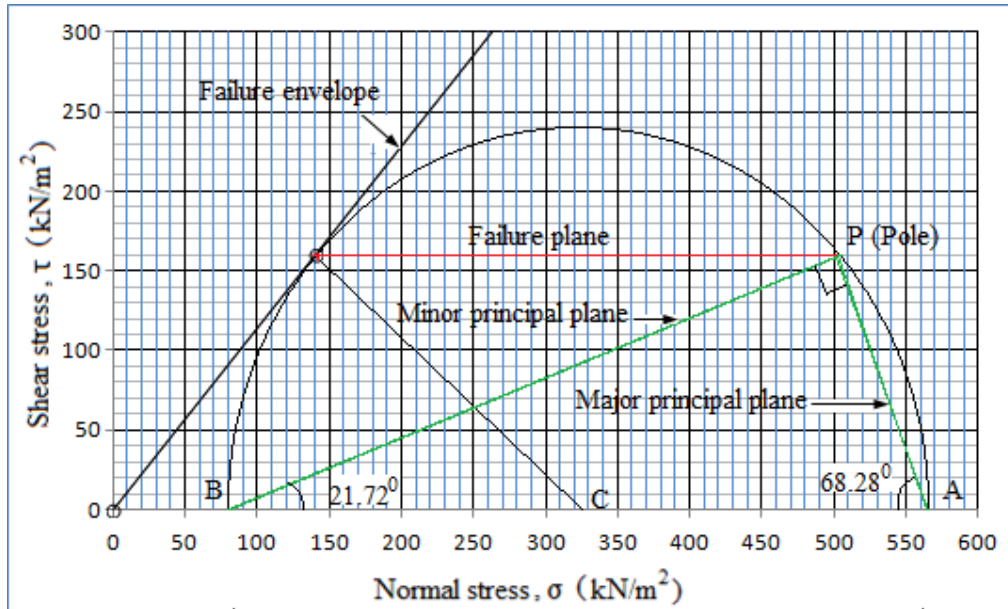The angle of shearing resistance, $\phi = 48.8^0$; cohesion, $c = 0$

Figure 10

Draw the Mohr's circle so that the failure envelope is tangent to the circle at F. To do so, draw FC perpendicular to the failure envelope. With C as centre and CF as radius, draw a circle so as to intersect the normal load axis at A and B. Point A corresponds to the major principal stress $\sigma_1 = 565..81 \text{kN/m}^2$ and point B corresponds to the minor principal stress $\sigma_3 = 80.35 \text{ kN/m}^2$.

To locate the position of the pole, draw a line FP parallel to the failure plane in the shear box (horizontal). P is the pole. PA is the direction of major principal plane which makes an angle 68.28^0 in the clockwise direction with the plane of shear. PB is the minor principal plane, making an angle of 21.72^0 in the anticlockwise direction with the plane of shearing.

Problem 3.

A cylinder of soil fails under an axial stress of 80 kN/m^2. The failure plane makes an angle of 48^0 with the horizontal. Calculate the value of cohesion and the angle of internal friction of the soil. Verify by graphical method.

Solution.

As there is only axial stress, there is no lateral stress acting on the soil, i.e., it is unconfined compression failure. Hence, minor principal stress $\sigma_3 = 0$ and major principal stress $\sigma_1 = 60$.

And $\alpha = 48^0$

We know,

$$\sigma_1 = \sigma_3 \tan^2 \alpha + 2c \tan \alpha$$

$$80 = 0 \times \tan^2 \alpha + 2c \tan 48$$

$$80 = 2c \tan 48$$

$$c = 36.02 \text{ kN/m}^2$$

Again,

$$\alpha = 45^0 + \frac{\phi}{2}$$

$$\phi = (\alpha - 45) \times 2$$

$$\phi = (48 - 45) \times 2$$

$$\phi = 6^0$$
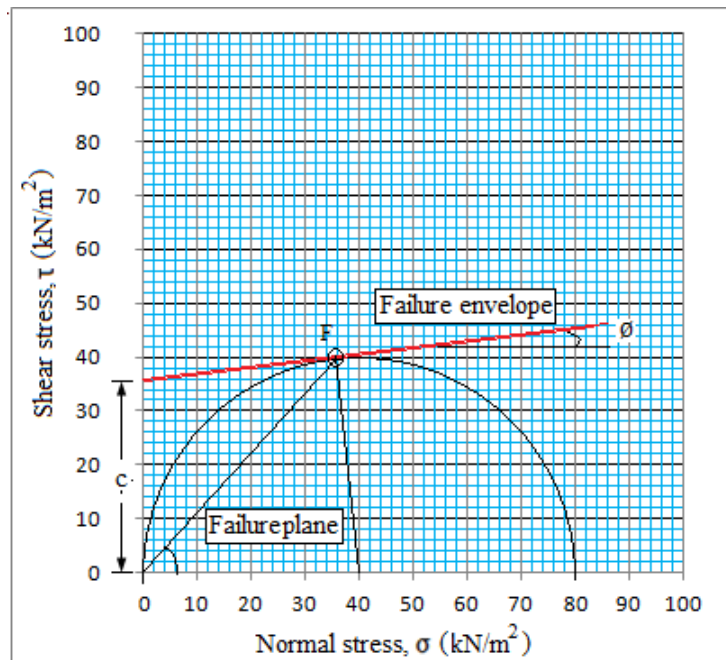
Figure 11

The Mohr's stress circle is drawn with radius $\dfrac{\sigma_1 - \sigma_3}{2} = \dfrac{80-0}{2} = 40$. The circle passes

through the origin representing the minor principal stress which is also the pole. Failure plane is drawn through the pole O so as to intersect the Mohr's circle at F. Join F with the centre C of the Mohr's circle. Draw the failure envelope by drawing a tangent at F on the circle so as to intersect the y-axis. The slope $\phi = 6^0$ of the failure plane is the angle of shearing resistance. The y-intercept $c = 36.02$ kN/m^2 is the cohesion.

Triaxial compression test

Triaxial shear test is the most extensively used for computation of shear strength parameters. In this test, the pecimen is compressed by applying all the three principal stresses, σ_1, σ_2 and σ_3.
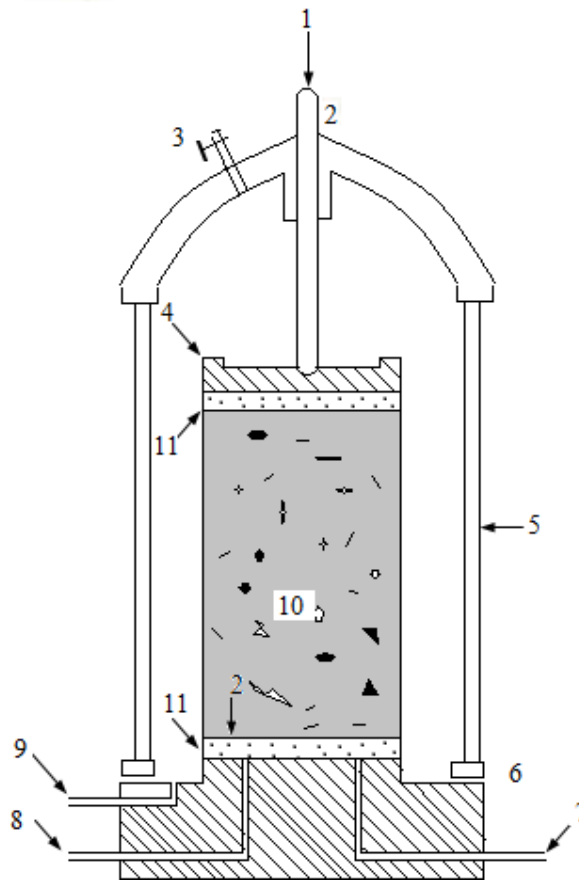

Figure 12 Triaxial cell

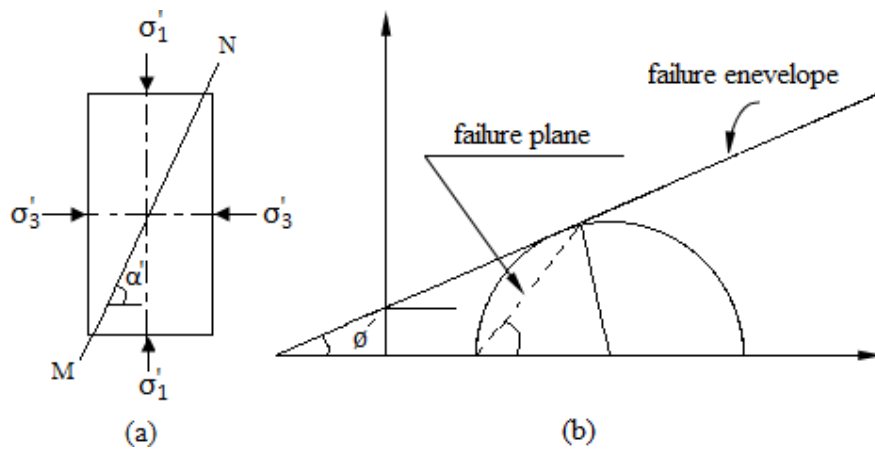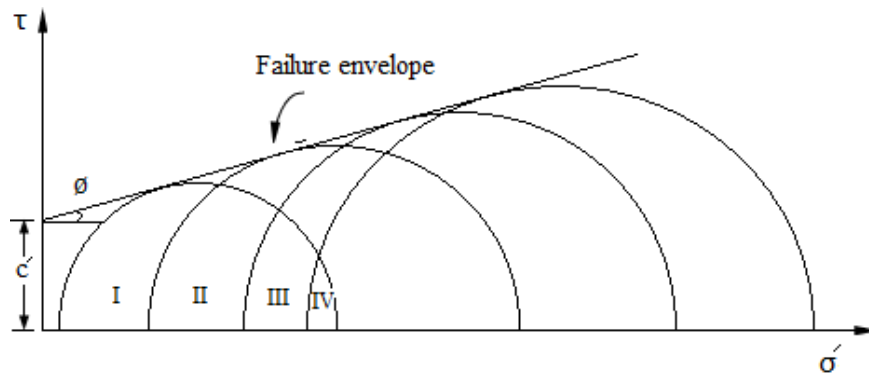| 1.	Axial load measured by proving ring dial gauge	7.	Pore water outlet
2.	Loading arm	8.	Additional pore water outlet
3.	Air release valve	9.	Cell fluid inlet
4.	Top cap	10.	Soil specimen enclosed in rubber membrane
5.	Perspex cylinder	11.	Porous disc
6.	Sealing ring		

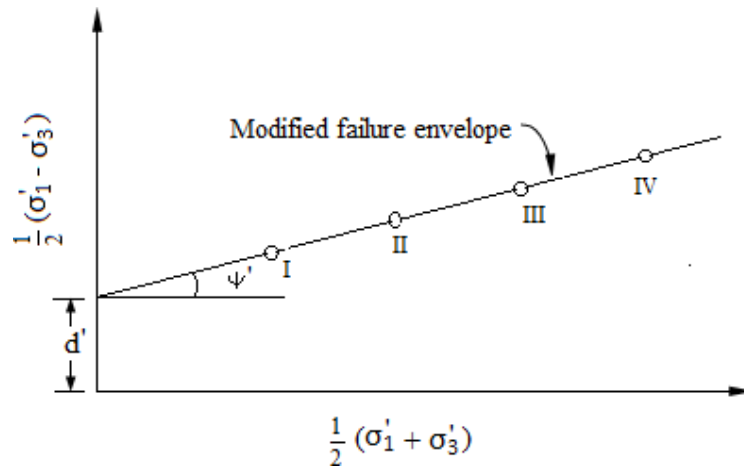Figure 13 Stress condition and failure envelope in triaxial compression test

Figure 14 Failure envelopes

Advantages of triaxial compression test

Followings are the advantages of triaxial compression test over the direct shear test.

1. Unlike the direct shear test in which the soil sample is forced to fail along a predetermined plane, the specimen in triaxial compression is free to fail along the weakest plane.

2. Distribution of stress is uniform along the failure plane is uniform. The shear strength is mobilized uniformly at all points on the failure plane.

3. The test procedure has complete control of the drainage conditions. The field drainage conditions are better simulated in triaxial compression test as compared to direct shear test.

4. Precise measurements of pore pressure and volume change are possible during the test.

5. The effect of end restraint does not have considerable effect on the result as failure usually occurs near the middle of the sample.

Unconfined compression test

Unconfined compression test is a special case of triaxial compression test in which no lateral or confining stress $\left(\sigma_2 = \sigma_3 = 0\right)$ is applied. A cylindrical soil sample of length 2 to 2.5 times the diameter is used as test sample. The soil specimen is only to

the major principal stress σ_1 till the specimen fails due to shearing along a critical failure plane.

Figure 15 shows the simplest form of compression testing machine. It consists of a small load frame fitted with a proving ring to measure the vertical stress σ_1 applied to the soil specimen. A separate dial gauge is used to measure the deformation of the sample.

The sample is conically hollowed at its ends and placed between two conical seatings attached to two metal plates. The conical seatings reduces end restraints and prevents the tendency of the specimen to become barrel shaped. The load is applied through a calibrated spring by manually operated screw jack at the top of the machine. The test sample is compressed at uniform rate of strain by the compression testing equipment. The axial deformation and the corresponding axial compressive force are measured. the sample may undergo brittle failure or plastic failure. In case of brittle failure, a definite maximum load is indicated by the proving ring which decreases rapidly with further increase of strain. However, no definite maximum load is indicated by the proving ring dial in case of a plastic failure. In such a case, the load corresponding to 20% strain is arbitrarily taken as the failure load. The maximum axial compressive stress resisted by the specimen before failure is called the unconfined compressive strength.

The unconfined compression test is a quick test in which no drainage is allowed. Since $\sigma_3 = 0$, the Mohr's circle passes through the origin, which is the pole. Figure 16 shows the stress conditions in a typical unconfined compression test. The equation plastic equilibrium may be expressed as

$$\sigma_1 = 2c_u \tan\alpha = 2c_u \tan\left(45^0 + \frac{\phi_u}{2}\right)$$

In the above equation , there are two unknowns c_u and ϕ_u , and cannot be determined by the unconfined test since a number of test on the identical specimens give the same value of σ_1 . Hence, the unconfined compression test is generally conducted on saturated clay for which the apparent angle of shearing resistance $\phi_u = 0$. Hence

$$\sigma_1 = 2c_u$$

The radius of the Mohr's circle is $\dfrac{\sigma_1}{2} = c_u$, The failure envelope is horizontal. *PF* is the failure plane, and the stresses on the failure plane are

$$\sigma = \dfrac{\sigma_1}{2} = \dfrac{q_u}{2}$$

$$\tau_f = \dfrac{\sigma_1}{2} = \dfrac{q_u}{2} = c_u$$

where q_u is the unconfined compressive strength at failure. The compressive stress $q_u = \dfrac{F}{A_c}$ is calculated on the basis of changed cross-sectional area A_c at failure, which is given by
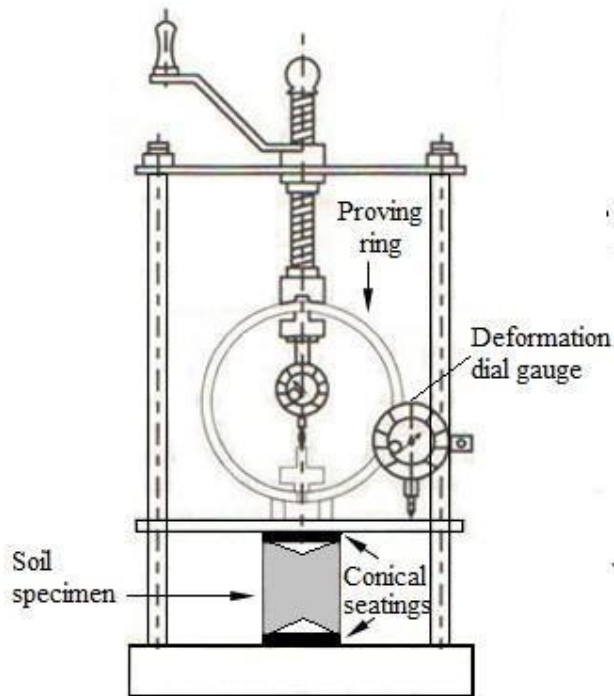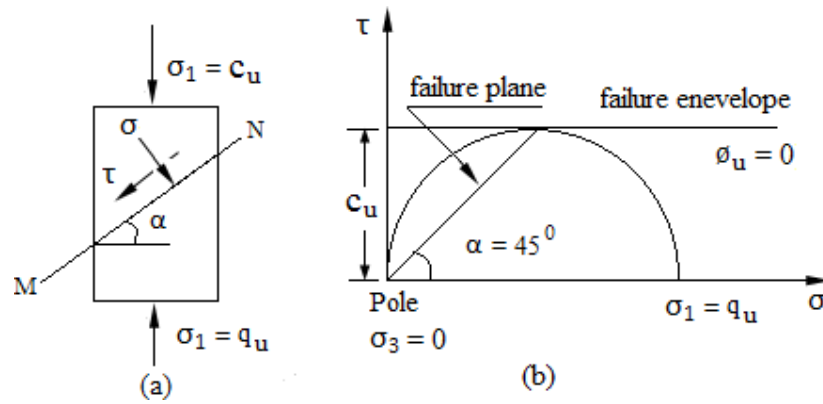

Figure 15 Unconfined Compression test set up

Figure 16 Unconfined compression test

$$A_c = \frac{V}{L_0 - \Delta L} = \frac{A_0}{1 - \dfrac{\Delta L}{L_0}}$$

$$A_c = \frac{A_0}{1 - \epsilon}$$

Where A_c = corrected area of cross section specimen

A_0 = initial area of cross section of specimen

L_0 = initial length of the specimen

V = initial volume of the specimen

ΔL = change in length at failure

$\epsilon = \dfrac{\Delta L}{L_0}$ = axial strain at failure

GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING
GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

Environmental Studies

For 3RD Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)
Prepared By
Snehasikta Tripathy
(Lecturer in Civil Engineering)

CONTENTS

UNIT - III
NATURAL RESOURCES

SYLLABUS

GE6351 ENVIRONMENTAL SCIENCE AND ENGINEERING L T P C
3 0 0 3

OBJECTIVES:

To the study of nature and the facts about environment.

- To finding and implementing scientific, technological, economic and political solutions to environmental problems.
- To study the interrelationship between living organism and environment.
- To appreciate the importance of environment by assessing its impact on the human world; envision the surrounding environment, its functions and its value.
- To study the dynamic processes and understand the features of the earth"s interior and surface.
- To study the integrated themes and biodiversity, natural resources, pollution control and waste management.

UNIT I ENVIRONMENT, ECOSYSTEMS AND BIODIVERSITY 12

Definition, scope and importance of Risk and hazards; Chemical hazards, Physical hazards, Biological hazards in the environment – concept of an ecosystem – structure and function of an ecosystem – producers, consumers and decomposers – Oxygen cycle and Nitrogen cycle – energy flow in the ecosystem – ecological succession processes – Introduction, types, characteristic features, structure and function of the (a) forest ecosystem (b) grassland ecosystem (c) desert ecosystem (d) aquatic ecosystems (ponds, streams, lakes, rivers, oceans, estuaries) – Introduction to biodiversity definition: genetic, species and ecosystem diversity – biogeographical classification of India – value of biodiversity: consumptive use, productive use, social, ethical, aesthetic and option values – Biodiversity at global, national and local levels – India as a mega-diversity nation – hot-spots of biodiversity – threats to biodiversity: habitat loss, poaching of wildlife, man-wildlife conflicts – endangered and endemic species of India – conservation of biodiversity: In-situ and ex-situ conservation of biodiversity. Field study of common plants, insects, birds Field study of simple ecosystems – pond, river, hill slopes, etc.

UNIT II ENVIRONMENTAL POLLUTION 10

Definition – causes, effects and control measures of: (a) Air pollution (Atmospheric chemistry- Chemical composition of the atmosphere; Chemical and photochemical reactions in the atmosphere - formation of smog, PAN, acid rain, oxygen and ozone chemistry;- Mitigation procedures- Control of particulate and gaseous emission, Control of SO_2, NOX, CO and HC) (b) Water pollution : Physical and chemical properties of terrestrial and marine water and their environmental significance; Water quality parameters – physical, chemical and biological; absorption of heavy metals - Water treatment processes. (c) Soil pollution - soil waste management: causes, effects and control measures of municipal solid wastes – (d) Marine pollution (e) Noise pollution (f) Thermal pollution (g) Nuclear hazards–role of an individual in prevention of pollution – pollution case studies – Field study of local polluted site – Urban / Rural / Industrial / Agricultural.

UNIT III NATURAL RESOURCES 10

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 overutilization of surface and ground water, dams-benefits and problems – Mineral resources: Use and exploitation, environmental effects of extracting and using mineral resources, case studies – Food resources: World food problems, changes caused by agriculture and overgrazing, effects of modern agriculture, fertilizer-pesticide problems, water logging, salinity, case studies – Energy resources: Growing energy needs, renewable and non renewable energy sources, use of alternate energy sources. Energy Conversion

processes – Biogas – production and uses, anaerobic digestion; case studies – Land resources: Land as a resource, land degradation, man induced landslides, soil erosion and desertification – role of an individual in conservation of natural resources – Equitable use of resources for sustainable lifestyles. Introduction to Environmental Biochemistry: Proteins –Biochemical degradation of pollutants, Bioconversion of pollutants. Field study of local area to document environmental assets – river / forest / grassland / hill / mountain.

UNIT IV SOCIAL ISSUES AND THE ENVIRONMENT 7

From unsustainable to sustainable development – urban problems related to energy – water conservation, rain water harvesting, watershed management – resettlement and rehabilitation of people; its problems and concerns, case studies – role of non-governmental organization- environmental ethics: Issues and possible solutions – 12 Principles of green chemistry- nuclear accidents and holocaust, case studies. – wasteland reclamation – consumerism and waste products – environment production act – Air act – Water act – Wildlife protection act – Forest conservation act –The Biomedical Waste (Management and Handling) Rules; 1998 and amendments- scheme of labeling of environmentally friendly products (Ecomark). enforcement machinery involved in environmental legislation- central and state pollution control boards- disaster management: floods, earthquake, cyclone and landslides. Public awareness.

UNIT V HUMAN POPULATION AND THE ENVIRONMENT 6

Population growth, variation among nations – population explosion – family welfare programme – environment and human health – human rights – value education – HIV / AIDS – women and child welfare –Environmental impact analysis (EIA)- -GIS-remote sensing-role of information technology in environment and human health – Case studies.

TOTAL : 45 PERIODS

OUTCOMES:

Environmental Pollution or problems cannot be solved by mere laws. Public participation is an important aspect which serves the environmental Protection. One will obtain knowledge on the following after completing the course.

- Public awareness of environmental is at infant stage.
- Ignorance and incomplete knowledge has lead to misconceptions
- Development and improvement in std. of living has lead to serious environmental disasters

TEXT BOOKS :

1. Gilbert M.Masters, "Introduction to Environmental Engineering and Science", 2nd edition, Pearson Education, 2004.

2. Benny Joseph, "Environmental Science and Engineering", Tata McGraw-Hill, New Delhi, 2006.

REFERENCES :

1. Trivedi.R.K., "Handbook of Environmental Laws, Rules, Guidelines, Compliances and Standards", Vol. I and II, Enviro Media, 3rd edition, BPB publications, 2010.

2. Cunningham, W.P. Cooper, T.H. Gorhani, "Environmental Encyclopedia", Jaico Publ., House, Mumbai, 2001.

3. Dharmendra S. Sengar, "Environmental law", Prentice hall of India PVT LTD, New Delhi, 2007.

4. Rajagopalan, R, "Environmental Studies-From Crisis to Cure", Oxford University Press, 2005.

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UNIT - I
ENVIRONMENT, ECOSYSTEM AND BIODIVERSITY

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1.1 ENVIRONMENT

Environmental science is the study of nature and the facts about environment. Environment can be defined as "all the social, economical, physical and chemical factors that surrounds man" or "all abiotic and biotic components around man-all living and non living things surrounds man".

1.1.1 PREREQUISITE DISCUSSIONS

The word environment is derived from the French word 'environ' which means to 'encircle or surround'.

Objective of this course is to develop concern for our own environment which will lead us to act at our own level to protect the environment we all live in.

Ever since people first recognized that their health and well-being were related to the quality of their environment, they have applied thoughtful principles to attempt to improve the quality of their environment.

There are three reasons for studying the state of the environment.

The first is the need for information that clarifies modern environmental concepts like equitable use of natural resources, more sustainable life styles etc.

Second, there is a need to change the way in which we view our own environment, using practical approach based on observation and self learning.

Third, there is a need to create a concern for our environment that will trigger pro-environmental action; including simple activities we can do in our daily life to protect it.

1.1.2 CONCEPTS

According to ancient man the environment was the Panchaboodhas (i.e.) air, water, land, sky and energy.

The human were disciples of nature. They were able to protect themselves from harmful one and protect the others. But according to modern man the environment is only air land and water.

Exploitation of various earth resources to satisfy the increasing needs of human population has resulted in 1) depletion of various resources of earth 2) pollution. Principles of environmental education:

- Examine the major environmental issues
- Discover the root cause
- Develop problem solving skills
- Promote co-operation in solving problems
- Emphasis active participation in prevention and solution to problems

1.1.3 SCOPE OF ENVIRONMENTAL SCIENCE

- Studying the interrelationship between the components of environment.
- Carrying out impact analysis and Environmental Audit
- Preventing pollution from existing and new industries
- Stopping the use of biological and nuclear weapons
- Managing unpredictable disasters etc.

1.1.4 PUBLIC AWARENESS

Environmental Pollution or problems cannot be solved by mere laws. Public participation is an important aspect which serves the environmental Protection.

- Public awareness of environmental issue is at infant stage
- 30-40% of public of developing country are aware of environmental. Problems but they do not bother about it.
- Ignorance and incomplete knowledge has lead to misconceptions.
- Development and improvement in std. of living has lead to serious environmental disasters.
- Debates on environmental Issues are treated as anti-developmental.

1.1.5 APPLICATION

- Environmental science is essentially the application of scientific methods and principles to the study of environmental issues, so it has probably been around in some forms as long as science itself.
- Environmental science is often confused with other fields of related interest, especially ecology, environmental studies, environmental education and environmental engineering.
- Environmental science is not constrained with any one discipline and it is a comprehensive field.

1.1.6 RISK AND HAZARDS IN THE ENVIRONMENT

Environmental risk due to various environmental hazards is an important topic for environmental engineers to recognise and understand in order to protect human society and ecosystems from harms or damages at local, regional or global scales. For example, to deal with contaminated soil and ground water at a brown field, risk and exposure assessment help engineers choose an optimal solution to either treat the hazard (e.g., to remove the contaminants from the soil and water) or reduce the exposure (e.g., to cover up the land with a barrier).

A hazard is a threat to life, health, property, or ecosystems, i.e., it involves something that could potentially be harmful. Therefore, when a dormant hazard comes to fruition, it will cause physical damage or destruction, loss of life, or drastic change to the environment, and result in an incident, accident, emergency event, or disaster. Hazards may be classified into:

- Chemical hazards – Combustion of Fossil fuels, industrial effluence, pesticides heavy metals.
- Physical hazards – Radioactive and UV radiations, Global warming, Chlorofluro carbons, Noise etc.
- Biological hazards – Bacteria, Viruses, Parasites.

1.2 ECOSYSTEM

Living organisms cannot be isolated from their non-living environment because the later provides materials and energy for the survival of the farmer.

An ecosystem is therefore defined as a natural functional ecological unit comprising of living organisms and their non-living environment that interact to form a stable self supporting system.

1.2.1 PREREQUISITE DISCUSSIONS

EO Wilson is an entomologist who envisioned that biological diversity was a key to human survival on Earth. He wrote 'Diversity of life' in 1993, which was awarded a prize for the best book published on environmental issues.

He emphasised the risks to mankind due to manmade disturbances in natural ecosystems that are leading to the rapid extinction of species at the global level.

An Indian ornithologist and naturalist, Salim Ali known as the "birdman of India", was among the first Indians to conduct systematic bird surveys across India.

He was instrumental in creating the Bharatpur bird sanctuary (Keoladeo National Park) and prevented the destruction of what is now the Silent Valley National Park. He was awarded India's second highest civilian honour, the Padma Vibhushan in 1976.

His autobiography, **Fall of a sparrow**, should be read by every nature enthusiast. He was our country's leading conservation scientist and influenced environmental policies in our country for over 50 years.

1.2.2 CONCEPTS

Ecology is the study of the distribution and abundance of organisms, the flows of energy and materials between abiotic and biotic components of ecosystems.

Structure of Ecosystem

1. Abiotic or non-living components or physical components
2. Biotic or Living components
3. Energy components

Function of organisms in an ecosystem

➢ Producer (autotrophy): make food; plants, algae
➢ Consumer (heterotrophy): eat other organisms
➢ Decomposer: eat dead organic matter; bacteria and fungi

Classes of Consumers

- Herbivore – primary consumer – eats plants
- Carnivores – secondary – meat eaters; eat herbivores
- Tertiary – feed on carnivores
- Omnivores – eat plants/animals

1.2.3 ENERGY FLOW IN ECOSYSTEM

- All organisms must obtain a supply of energy and nutrients from their environment in order to survive.
- The transformations of energy in an ecosystem begin first with the input of energy from the sun.
- Because, it is the first step in the production of energy for living things, it is called "Primary production".
- Photosynthesis -- Chemical reaction where green plants use water & carbon dioxide to store the sun's energy in glucose.
- ENERGY is stored in glucose.
- Glucose is stored as starch in plants
- The majority of autotrophs are photoautotrophs that harness the energy of the sun and pass some of this energy onto consumers through feeding pathways.
- The energy contained within producers and consumers is ultimately passed to the decomposers that are responsible for the constant recycling of nutrients.

- Thus, there is a one-way flow of energy through the biotic community and a cycling of nutrients between the biotic and abiotic components of the ecosystem
- Energy flow cannot occur in reverse direction.
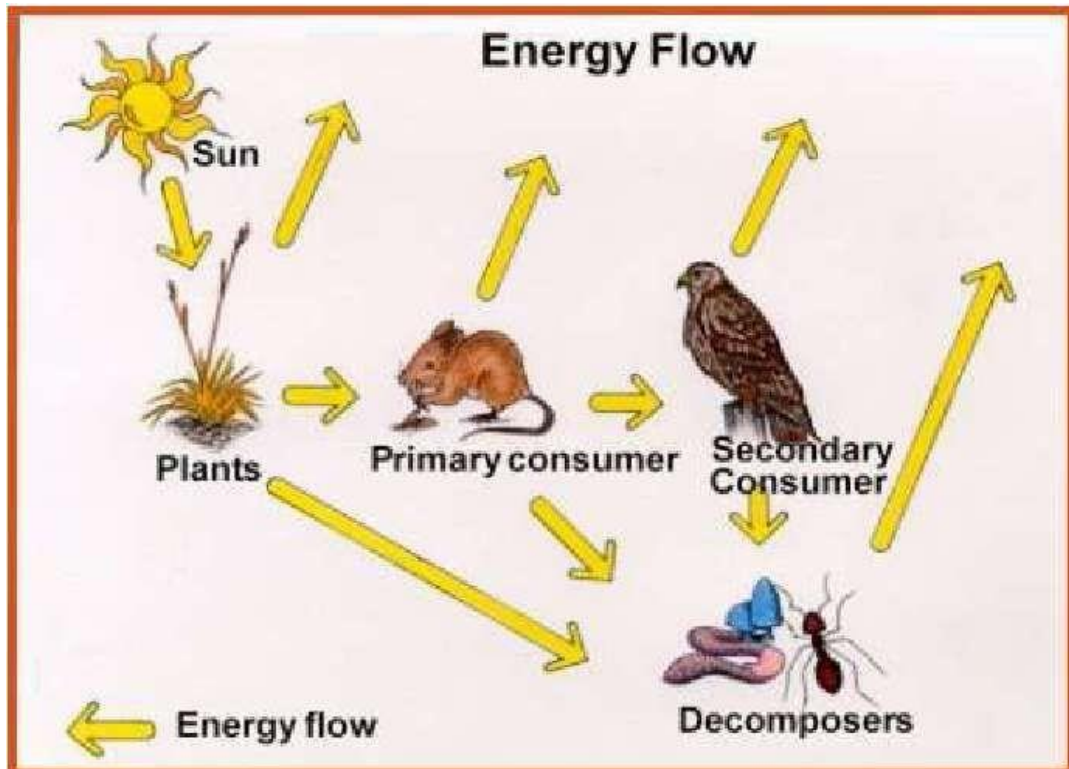
Energy Flow
- Starts from autotrophs (the producer level, i.e., first trophic level) to Heterotrophs including plant eaters or Herbivores (second trophic level) and so on.
- The amount of energy decreases with successive trophic levels.
- Only About 1% of energy from the sun is used by green plants & rest remains unutilized.
- Similarly, there is loss of energy in each trophic level.
- The transfer of food energy between the organisms in an ecosystem can be tracked by constructing food chains, food webs, pyramids of numbers, biomass and energy and energy flow diagrams.

1.2.4 FOOD CHAIN

Plants by photosynthesis convert solar energy into protoplasm. Small herbivores consume the vegetable matter and convert into animal matter which in turn eaten by large carnivores.

- A food chain may be defined as, "the transfer of energy and nutrients through a series of organisms with repeated process of eating and being eaten".
- In an ecosystem, all the organisms are linked together with one another by food relationship.
- Each organism living or dead is potential food for some other organism.
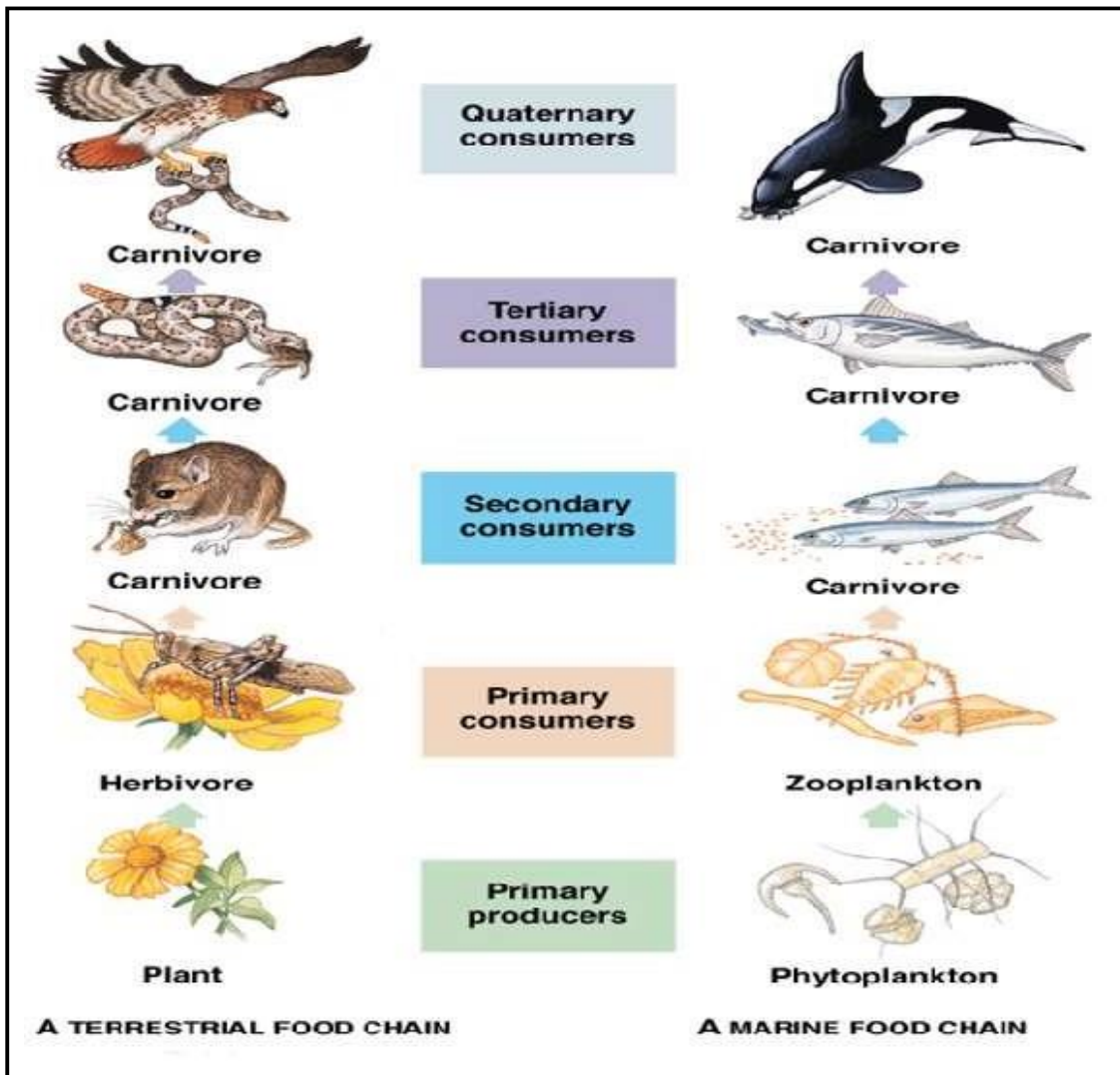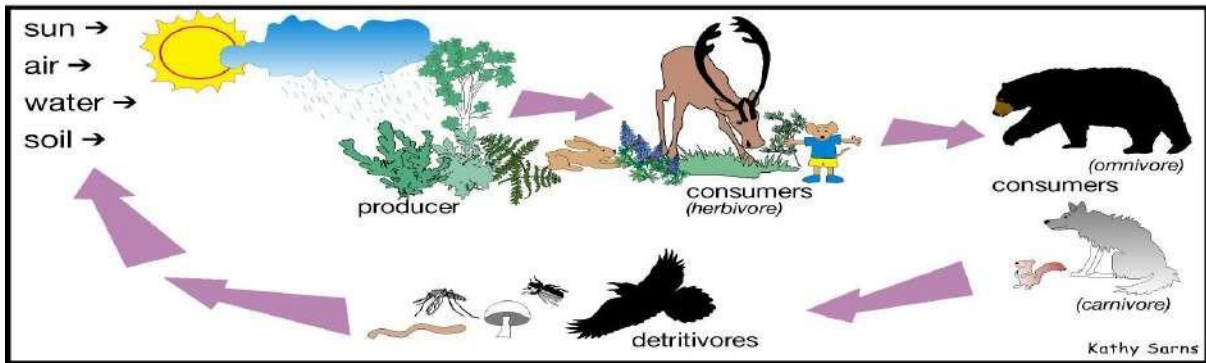
Food Chain

1.2.5 FOOD WEB

The food relationship between various organisms is being depicted by linking all the possible prey and predators of different food level. In an ecosystem linking of feeding habit relations will provide a food web or Interlocking pattern of several interlinked food chains is termed as FOOD WEB.
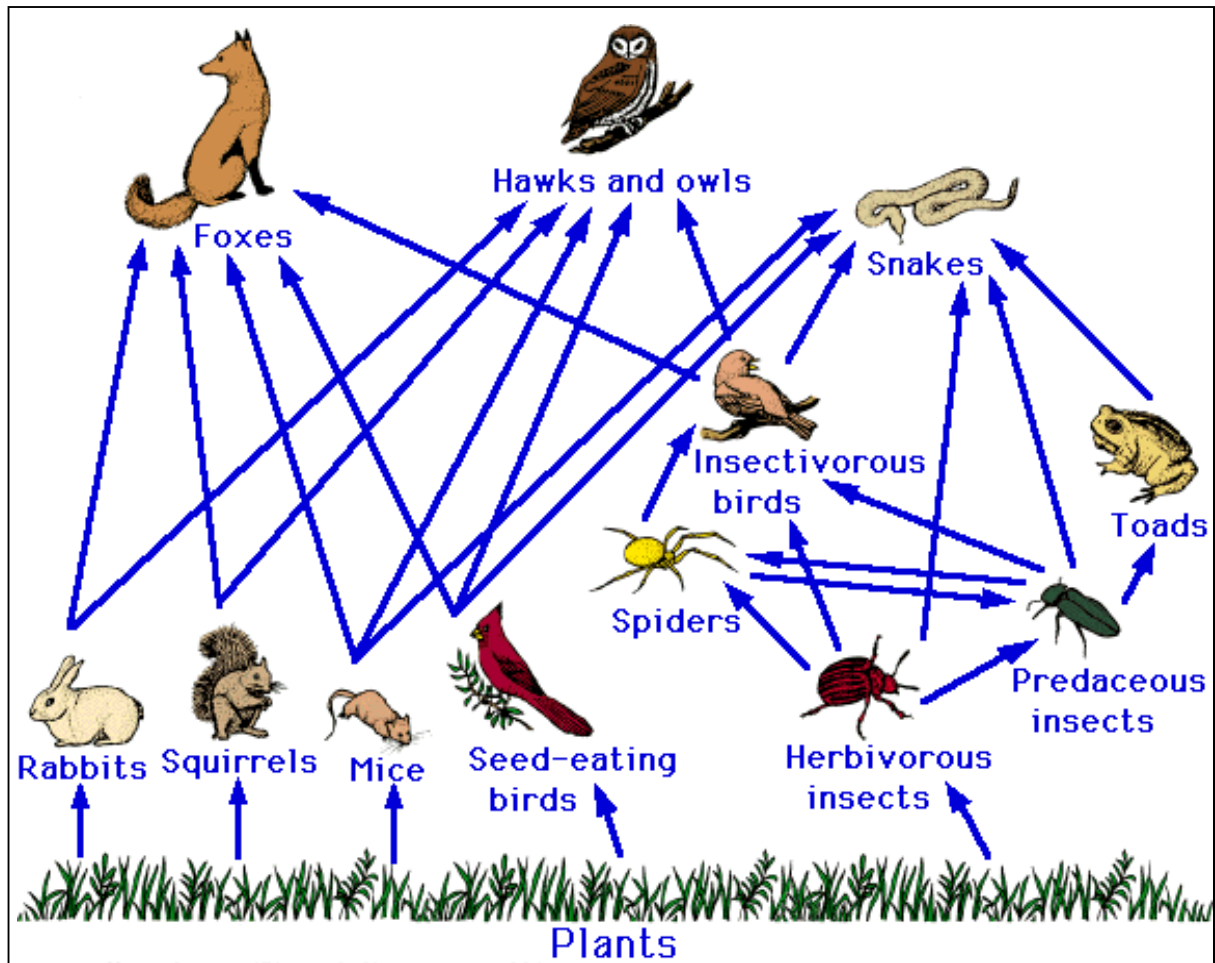
Food web in grassland ecosystem

1.2.6 ECOLOGICAL PYRAMIDS

An"Ecological pyramid" is a graphical representation that shows the relative amounts of energy or matter contained within each tropic level in a food chain or food web.

An ecological pyramid shows the relationship between consumers and producers at different tropic levels in an ecosystem.

Ecological Pyramid

Types of Ecological Pyramids
Pyramid of Numbers

Shows the relative number of individual organisms at each tropic level.

Pyramid of Numbers

Pyramid of Biomass

A pyramid of biomass represents the total dry mass (in grams per square meter of area) of all the organisms in each tropic level at a particular time.
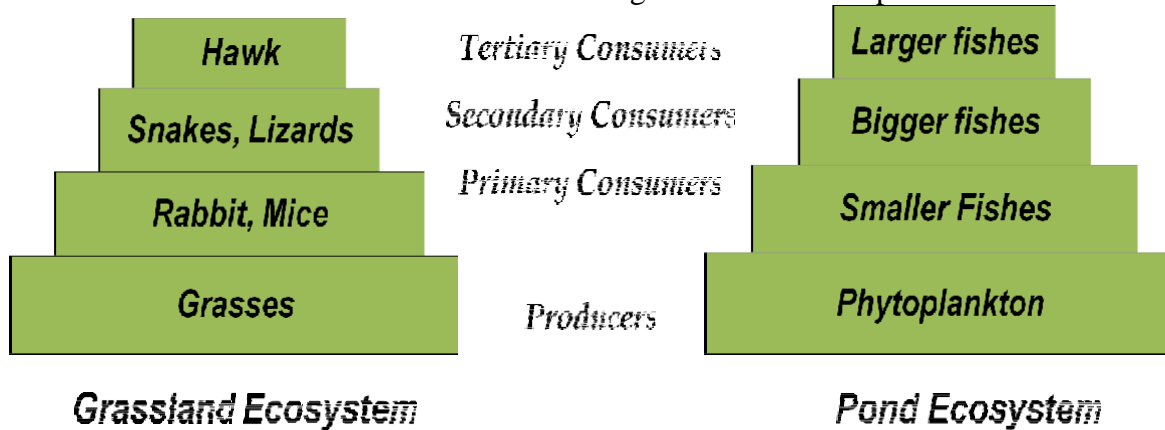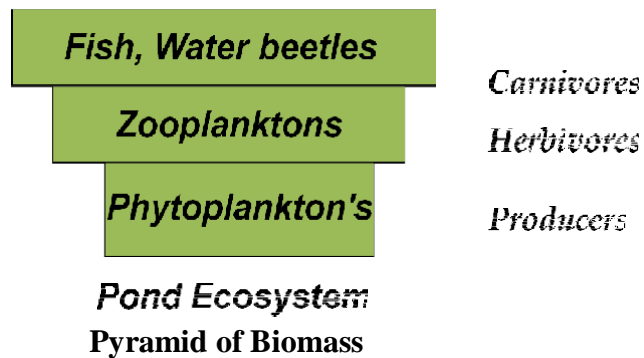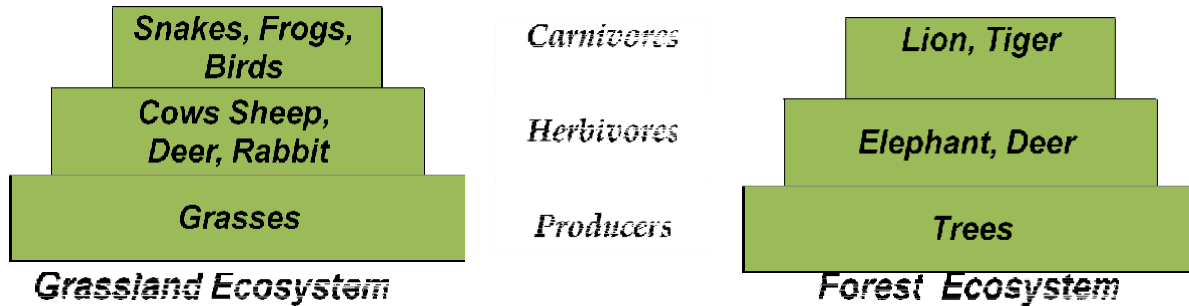
Pyramid of Biomass

Pyramid of Energy

A pyramid of biomass represents the rate of energy flow and/or productivity at successive tropic levels. The pyramids of energy are always upright.

Pyramid of Energy
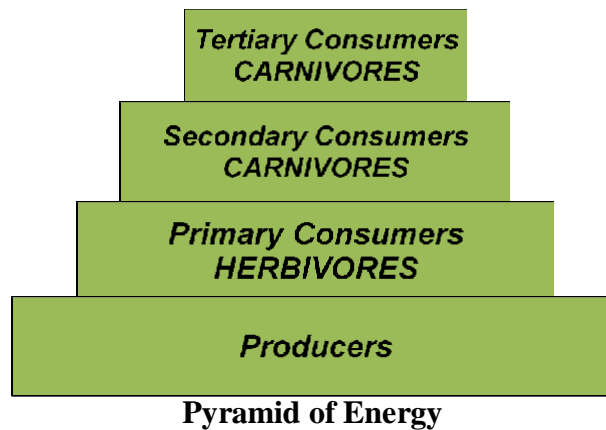
1.2.7 NITROGEN CYCLE

- Nitrogen is crucial for all organisms
    - Nucleic acids
    - Proteins
    - Chlorophyll
- Nitrogen- 78% in Atmosphere
- N2 is very stable and must be broken apart by organisms, combined with other atoms into a usable form.
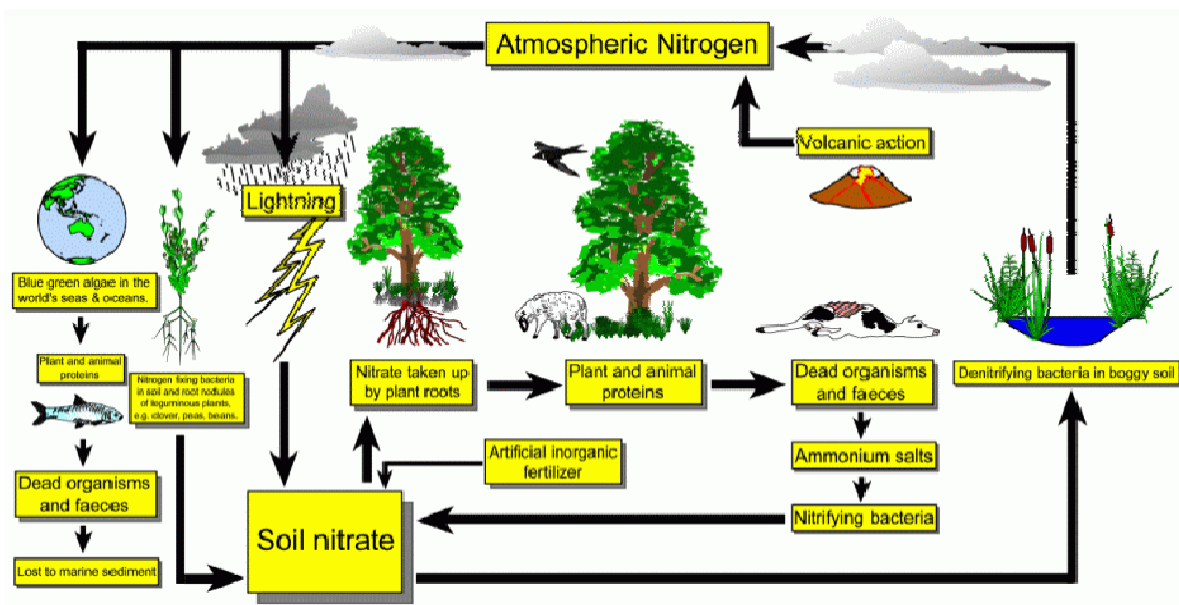
Nitrogen Cycle

Nitogen cycle completes in 5 steps:

1) Nitrogen Fixation

Conversion of $N_2 \rightarrow NH_3$

Combustion, volcanic action, Lightning, Industrial processes (making fertilizer).

Bacteria (Azotobactor, Clostridium, Nostoc etc.)

2) Nitrification

Conversion of $NH_3 \rightarrow NO_3$

Soil bacteria convert in a two step process.

3) Assimilation

Roots absorb NH_3, NH_4, or NO_3 and incorporate them into nucleic acids and protein.

4) Ammonification

Amino acids and nucleotides are broken down into waste products NH_3 or NH_4

5) Denitrification

The reduction of NO_3 to N_2 .Denitrifying bacteria return some of the nitrogen to the atmosphere

1.2.8 OXYGEN CYCLE

oxygen cycle is the circulation of oxygen in various forms through nature free in the air and dissolved in water.

Oxygen is second only to nitrogen in abundance among uncombined elements in the atmosphere.

Plants and animals use oxygen to respire and return it to the air and water as carbon dioxide (CO_2). CO_2 is then taken up by algae and terrestrial green plants and converted into carbohydrates during the process of photosynthesis, oxygen being a by-product.

The waters of the world are the main oxygen generators of the biosphere; their algae are estimated to replace about 90 percent of all oxygen used.

The generalized oxygen cycle

Oxygen is involved to some degree in all the other biogeochemical cycles. For example, over time, detritus from living organisms transfers oxygen-containing compounds such as calcium carbonates into the lithosphere.

Despite the burning of fossil fuel and the reduction of natural vegetation (on land and in the sea), the level of atmospheric oxygen appears to be relatively stable because of the increase in plant productivity resulting from agricultural advances worldwide.

1.2.9 CARBON CYCLE

- Carbon enters plants, etc., as CO2
    - Bacteria process carbon in a fashion that allows it to be recycled.
    - Obtain energy from the molecules, and convert carbohydrates to carbon dioxide as a result of respiration.
- Photosynthesis removes carbon from the abiotic environment (fixes carbon into organic molecules)
- Carbon moves through food chain through consumption of one organisms by another

- Cellular respiration, combustion, and erosion of limestone return carbon to the atmosphere, water and abiotic environment.
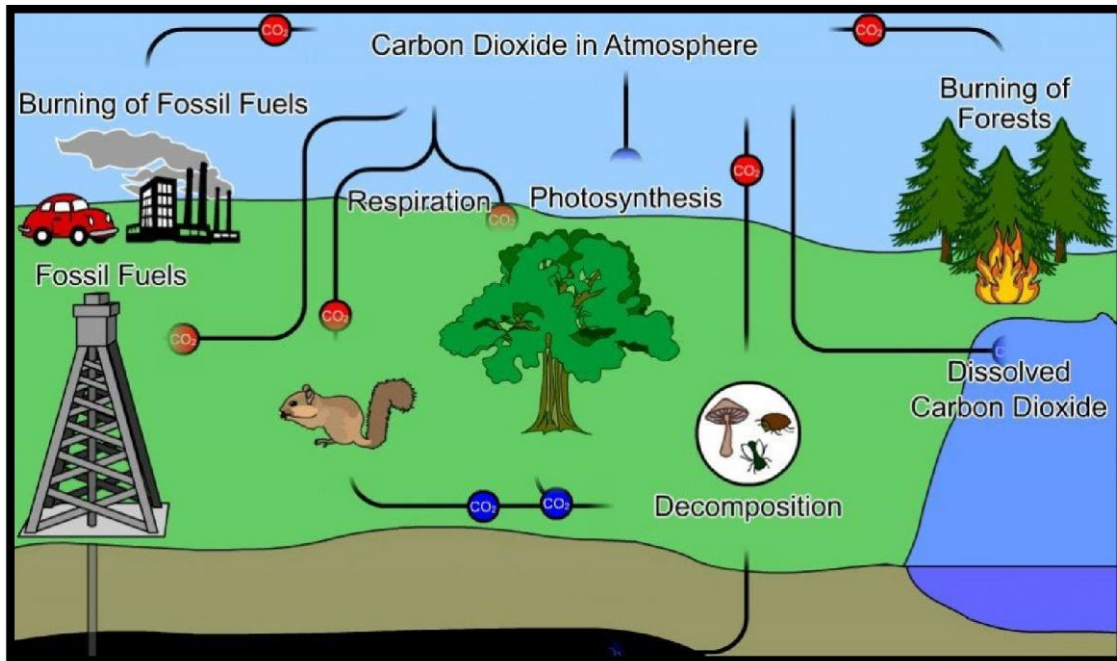
Carbon Cycle

The source of atmospheric carbon dioxide is variable but only plants can utilize atmospheric carbon directly.

1.2.10 PHOSPHOROUS CYCLE

- The only cycle that does not have a gaseous state.
- Inorganic phosphate PO_4^{3-} is released from rocks and sediments through the action of erosion.
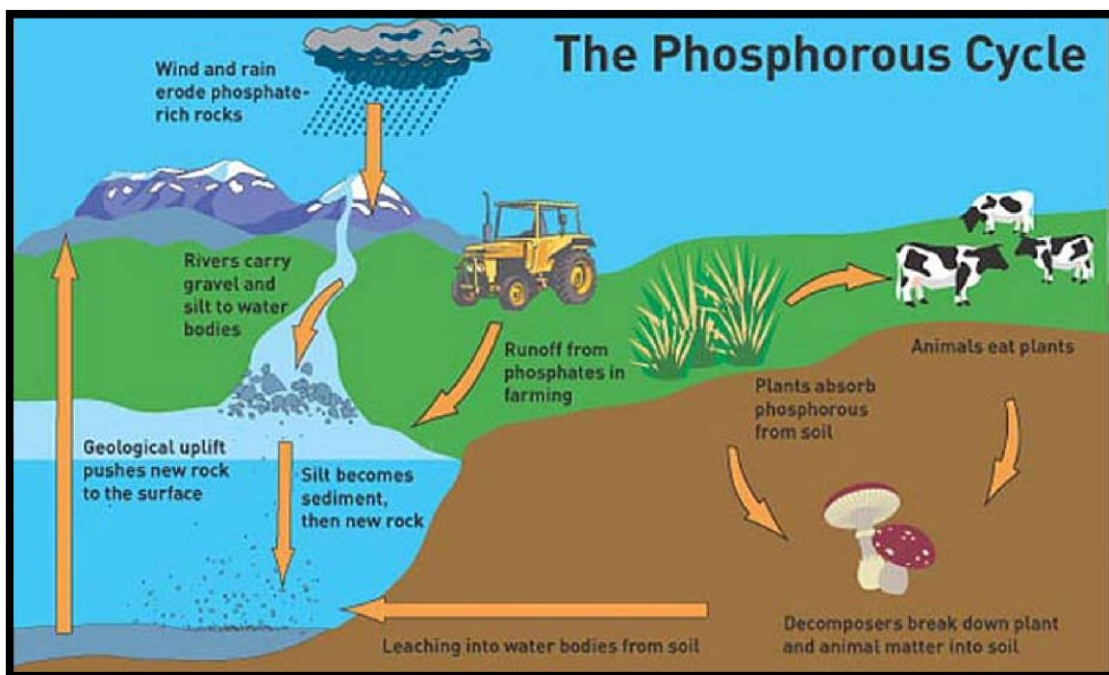
Phosphorous Cycle

GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING
GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

Estimation & cost evaluation-1

For 3rd Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)

Prepared By
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CONTENTS

Estimation & cost evaluation-1

CHAPTER -1

Introduction

WHAT IS AN ESTIMATE ......
➢ Before taking up any work for its execution, the owner or builder should have a thorough knowledge about the volume of work that can be completed within the limits of his funds or the probable cost that may be required to complete the proposed work.

➢ It is therefore necessary to prepare the probable cost or estimate for the proposed work from its plan and specification.

➢ Otherwise, it may so happen that the work has to be stopped before its completion due to the shortage of funds or of materials.

➢ Besides the above , an estimate for any public construction work is required to be prepared and submitted beforehand so that sanction of necessary funds may be obtained from the authority concerned .

➢ Thus an estimate for any construction work may be defined as the process of calculating the quantities and costs of the various items required in connection with the work .

➢ It is prepared by calculating the quantities, from the dimensions on the drawings for the various items required to complete the project and multiplied by unit cost of the item concerned.

➢ To prepare an estimate , drawing consisting of the plan , the elevation and the section through important points, along with a detailed specification giving specific description of all workmanship , properties and proportion of materials , are required.

PURPOSE OF ESTIMATING:-
➢ To ascertain the necessary amount of money required by the owner to complete the proposed work . For public construction work, estimates are required in order to obtain administrative approval , allotment of funds and technical sanction.

➢ To ascertain quantities of materials required in order to programme their timely procurement.
➢ To calculate the number of different categories of workers that is to be employed to complete the work within the scheduled time of completion.

➢ To assess the requirements of tool , plants and equipment required to complete the work according to the programmed.

Estimation & cost evaluation-1

➢ To fix up the completion period from the volume of works involved in the estimate.

➢ To draw up a construction schedule and programmed and also to arrange the funds required according to the programming.

➢ To justify the investment from benefit cost ratio.(for ideal investment ,this ratio should be more than one)

➢ To invite tenders and prepare bills for payment.

➢ An estimate for an existing property is required for valuation

TYPES OF ESTIMATE

➢ ROUGH COST ESTIMATE

➢ PLINTH AREAESTIMATE

➢ CUBICAL CONTENT ESTIMATE

➢ A QUANTITY ESTMATE

➢ APPROXIMATE QUANTITY METHOD

➢ DETAILED OR ITEM RATE ESTIMATE

➢ REVISED ESTIMATE

➢ SUPPLEMENTARY ESTIMATE

➢ REPAIR AND MAINTENANCEESTIMATE

➢ A COMPLETE ESTIMATE

ROUGH COST ESTIMATE

IT is prepared to decide the financial policy matter.it is prepared on basis of practical knowledge and cost of similar works. The competent sanctioning authority accords "Administrative approval
➢ These estimates are also referred to as rom estimate and are useful for go /no kind decision making which essential refers to whether the project should or should not be pursued
➢ Some of the methods they can be useful for such estimates are investment per annual capacity turnover and capital ratio .

PLINTH AREA ESTIMATE

➢ IT Is prepared on the basis of plinth area of the building multiplied by plinth area rate prevalent in the region.

Estimation & cost evaluation-1

➢ Plinth area rates are fixed from the cost of similar buildings constructed in the locality having similar finishing's and amenities

➢ The cost of construction is determined by multiplying plinth area with plinth area rate. The area is obtained by multiplying length and breadth (outer dimensions of building). In fixing the plinth area rate, careful observation and necessary enquiries are made in respect of quality and quantity aspect of materials and labor, type of foundation, height of building, roof, wood work, fixtures, number of stores etc.

CUBICAL CONTENT ESTIMATE

➢ This estimate is worked out on the basis of the cubical contents of proposed building to be constructed and then applying to it the rate per cubic meter.

➢ This is more accurate than plinth area estimate.

➢ The cubic content rates are deduced from the cost of similar buildings constructed in the same locality

➢ This method is generally used for multi-storied buildings. It is more accurate that the other two methods viz., plinth area method and unit base method.

➢ The cost of a structure is calculated approximately as the total cubical contents (Volume of buildings) multiplied by Local Cubic Rate.

➢ The volume of building is obtained by Length x breadth x depth or height. The length and

breadth are measured out to out of walls excluding the plinth off set
Under Review

Estimation & cost evaluation-1

A QUANTITY ESTIMATE OR QUANTITY SURVEY

➢ This is complete estimate or list of quantities for all items of work required to complete the concerned project.

➢ The quantity of each individual items of work is worked out form respective dimensions on the drawing of the structure to find the cost of an item in quantity is multiplied by the rate per unit from that item.

➢ The purpose of the bill quantity i.e. to provide a complete list of quantities necessary for the competition of any engineering project and when price given to the estimated cost of the project.

APPROXIMATE QUANTITIES

➢ Regarded as the most reliable and accurate method of estimating, provided that there is sufficient information to work on. Depending on the experience of the surveyor, measurement can be carried out fairly quickly using composite rates to save time.

➢ The rules of measurement are simple, although it must be said; they are not standardized and tend to vary slightly from one surveyor to another.

One approach involves grouping together items corresponding to a sequence of operations and relating them to a common unit of measurement; unlike the measurement for a bill of quantities,
➢ where items are measured separately

➢ Composite rates are then built up from the data available in the office for that sequence of operations
➢ All measurements are taken as gross over all but the very large openings

➢ Initially, the composite rates require time to build up, but once calculated they may be used on a variety of estimating needs
➢ Reasonably priced software packages are now available. An example for a composite is shown below for substructure:
➢ This is an approximate estimate to find out an approximate cost in the short time and thus enable the authority concerned to consider the financial aspect of the scheme for according sanctioned the same.

➢ Such an estimate is framed after knowing the rates of similar works and form practical knowledge in various ways for various types of work such as

▪ Plinth area or square meter method.

▪ Cubic rate or cubic meter method.
▪ Serve unit or unit rate method.

6

- Bay method.

- Approximate quantities with bill method.
- Cost comparison method

- Cost from materials and labor.

DETAILED OR ITEM RATE ESTIMATE

➢ This estimate is an accurate and is based on the plan and sanctions of the building.

➢ The quantity of items under each sub head of work are calculated from the dimensions taken from drawing and then total cost is worked out in a form called abstract of cost

➢ This include the detailed particulars for the quantities ,rate and cost of all the items involved for satisfactory completion of a project

➢ Quantities of all items of work are calculated from their respective dimension on the drawing on a measurement sheet .multiplying these quantities by their respective rate in separate sheet, the cost of all items of work are worked out individually and then summarized

➢ A detail estimate is accompanied by

- Report
- Specification
- Detailed drawing showing plane Design data and calculation
- Basis of rates adopted in the estimate

REVISED ESTIMATE

➢ IT Is also a detailed estimate and is prepared a fresh when the original sanctioned detailed estimate exceeds by 10% or more ,either due to rates being found insufficient or due to some other reasons

➢ It is always possible that in spite of all precaution in the planning stages it becomes clearly during execution the actual cost of a project will exceed the original estimate ,now generally a certain cushion of the cost is available ,if the exceedance is higher

➢ It is prepared on the basis on estimate on which sanction was obtained showing the existing sanction and the progress made up to date

➢ The revised estimate should be accompanied by comparative statement showing the original and revised rate and quantity

SUPPLEMENTARY ESTIMATE

➢ This is a fresh detailed estimate of the additional work in additional work in addition to the original one and is prepared when additional work is required to supplement the original work

➢ There is always a like hood that while executing a certain project it may be considered worthwhile

to carry out additional work ,which was not foreseen in initial stages and therefore not actual for the preliminary estimate

➢ Execution of such work required drawing up and approval of supplementary estimate and the exercise is essential similar to that of drawing up the estimate for the main work it is naturally expected that the cost of additional work will be much smaller than the main work

➢ In case where a substantial section of a project in abandoned or where material deviation from the original proposals are expected to result in substantial savings the estimate is revised by the department and intimated to engineer in charge for execution of work

➢ But in case where the saving is due to a material deviation of structural nature from the design originally approved supplement estimate is prepared for a revised technical sanction

➢ The method of preparation of supplementary estimate is the same as that of detail estimate and it should be accompanied by full report of the circumstances which render it necessity.

➢ The abstract must show the amount of original estimate and the total of sanctioned required including the supplementary amount.

REPAIR AND MAINTENANCE ESTIMATE

➢ In order to keep the structure roads etc in proper condition annual repairs and carried out annually for which an estimate is prepared. The estimated amount should not be more than 1.5%of the capital cost of work

➢ There are more than 10,000 maintenance, repair and preventive maintenance tasks for all types of facilities. With advances in telecommunications, computers and other workplace technologies, the list is growing rapidly. In a downsized, fiscally conservative environment, facilities are seen as vital capital assets that affect employee productivity. The demands for technical expertise and cost-effective plant operations have increased dramatically.

➢ After completion of a work it necessary to maintain the same for the proper function and for the same an estimate is prepared for items which required renewal, placement, repairs, etc in form of detailed estimate

➢ For building, such items of work like white washing, color and painting of doors and windows etc. quantities are based on the previous measurement recorded in measurement book as standard measurement books .for pretty works such as replacement of glass panes, repairs of floors patch repairs to cement plaster walls and changing roof tiles or similar natural works

➢ The total estimate coat of maintenance of structural is generally kept within the prescribe limits on percentage basis of the cost of the construction of the structure and its imp.

➢ The total estimated cost of maintenance of structure is generally kept within the prescribed

limits on percentage basis (Variable according to the age and importance of the structure) of the cost of the construction of the

Under Review

structure and its importance.

COMPLETE ESTIMATE

➢ This is an estimated cost of all items which are related to the work in addition to main contractor to the detailed estimate

➢ One may think that an estimate of a structure includes only the cost of land and the cost items to be included.
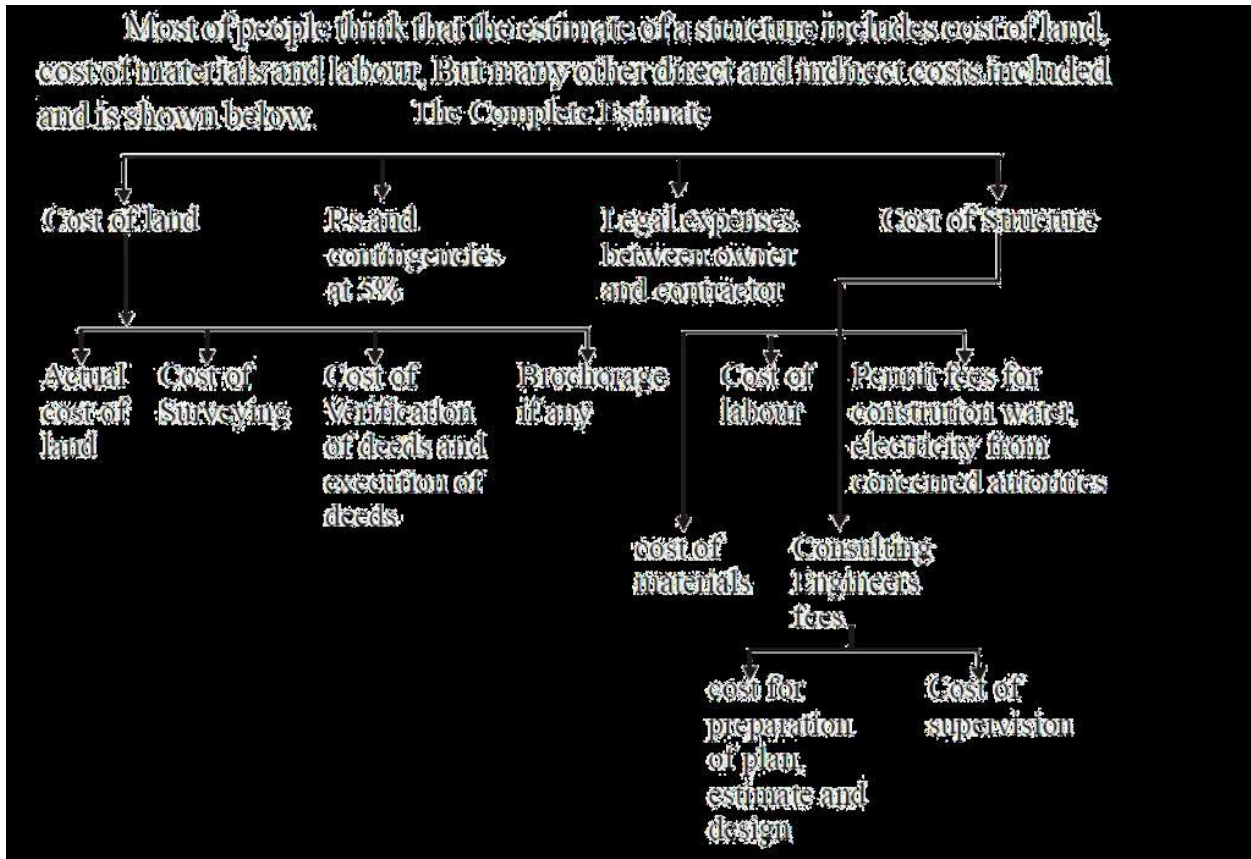
Basic SI Units

➢ Units of Length- Metre(M)- The metre is the length equal to 1650673.73 wave lengths in vacuum of the radiation corresponding to the transition between the levels 2 p $_{10}$ and 5 d $_5$ of the Krypton 86 atom.

➢ Unit of Mass-Kilogram (Kg)- The kilogram is the unit of mass and is equal to the mass of the international prototype of the Kilogram.

➢ Unit of Time- Second(s)- The second is the duration of 9192 631 770 periods of the radiation corresponding to the transition between the hyperfine levels of the ground state of the Cesium 133 atom.

➢ Unit of Electric Current- Ampere(A)- The ampere is that constant current which, if maintained in two straight parallel conductors of indefinite length, of negligible circular cross-section and placed one metre apart in vacuum, would produce between these conductors of force equal to 2 x 10-7 Newton per unit length.

9

Estimation & cost evaluation-1

➢ Unit of Thermodynamic Temperature- Kelvin (K)-The Kelvin unit of the thermodynamic temperature of the triple point of water. Kelvin may be used for expressing a temperature interval. The degree Celsius (0C) is a unit of the International

➢ Unit of Luminous Intensity- Candela (cd)- The candela is the luminous intensity, in the perpendicular direction of a surface of 1/600,000 square metre of a black body at the temperature of freezing platinum, under a pressure of 101.325 Newton's per square metre

Estimation & cost evaluation-1

CHAPTER-2

METHOD OF ESTIMATING
The quantities like earth work, foundation concrete, brickwork in plinth and super structure etc., canbe workout by any of following two methods:
a) Long wall - short wall method
b) Centre line method.
c) Partly centre line and short wall method.

LONG WALL-SHORT WALL METHOD:
In this method, the wall along the length of room is considered to be long wall while the wall perpendicular to long wall is said to be short wall. To get the Measurement of Materials and Works length of long wall or short wall, calculate first the centre line lengths of individual walls. Then the length of long wall, (out to out) may be calculated after adding half breadth at each end to its centre line length. Thus the length of short wall measured into in and may be found by deducting half breadth from its centre line length at each end. The length of long wall usually decreases from earth work to brick work in super structure while the short wall increases. These lengths are multiplied by breadth and depth to get quantities

B) CENTRE LINE METHOD:
This method is suitable for walls of similar cross sections. Here the total centre line length is multiplied by breadth and depth of respective item to get the total quantity at a time. When cross walls or partitions or verandah walls join with main all, the centre line length gets reduced by half of breadth for each junction. such junction or joints are studied carefully while calculating total centre line length. The estimates prepared by this method are most accurate and quick.

C) PARTLY CENTRE LINE AND PARTLY CROSS WALL METHOD:
This method is adopted when external (i.e., alround the building) wall is of one thickness and the internal walls having different thicknesses. In such cases, centre line method is applied to external walls and long wall-short wall method is used to internal walls. This method suits for different thicknesses walls and different level of foundations. Because of this reason, all Engineering departments are practicing this method.

HOW TO PREPARE A DETAILED ESTIMATE
⬜ ⬜Detailed Estimate:The unit-quantity method is followed to prepare a
DETAILS OF MEASUREMENT AND CALCULATION OF QUANTITIES .
MEASUREMENT FORM

Item no   Description -No   length   Breadth   - Height   Content   Remark
or   or Depth   or
particulars   quantity

Abstract of estimate form

Sl .no	Description or particulars	Quatity	Unit Rate Rs. P. rate	Unit of Rs. P.	Amount

Functions of an abstract of estimate

The main functions of an abstract of estimate are:

☐ ☐The total estimated cost and the different items of works required to complete project can be known.

☐ ☐Basis on which % rate tenders are called after excluding the amount for contingency and work-charged establishment.

☐ ☐A part of tender document and a contractor can arrive at his own rates from the schedule of work described in the description column.

☐ ☐This is the basis on which bills are prepared for payement. Comparative costs of different items of works can be known.

Data required for preparing detailed estimate
• DRAWING
• SPECIFICATIONS(both general and detailed)
• RATES
• UPDATED MODE OF MEASUREMENT
• STANDING CIRCULARS

FACTORS CONSIDERED DURING DETAILED ESTIMATION
(a) Quantity of materials

(b) Availability of materials

(c) Transportation of materials

(d) Location of site

(e) Local labour charges

Principle of Units for Various Items Of Works-

The units of different works depend on their nature, size and shape. In general, the units of different items of work are based on the following principle:-

i) Mass, voluminous and thick works shall be taken in square unit or volume. The measurement of length and breadth or height shall be taken to compute the volume or cubic contents.

Estimation & cost evaluation-1

ii) Shallow, thin and surface works shall be taken in square units or in area. The measurement of length and breadth or height shall be taken to compute the area.

iii) Long and thin work shall be taken in linear or running unit, and linear measurement shall be taken.

iv) Piece work, job work, etc.,shall be taken in number.

The units of payments and measurement of various items of work in metric system are same except for earthwork. Earthwork is measurement in cu m but payment is made per 100 cu m(per % cu m).

THE UNITS OF MEASUREMENTS AND PAYMENTS FOR VARIOUS ITEMS OF WORK AND MATERIALS

SL. NO.	Particulars of Items	Units of measurements in MKS	Units payment in MKS	Unita of payment in FPS
	Earthwork-			
1.	Earthwork in excavation in ordinary soil, earthwork in mixed soil with kankar bajri,etc.earthwork in hard soil.	cu m	Per % cu m	% cu ft
2.	Rock excavation	cu m	Per % cu m	% cu ft
3.	Earth filling in excavation in foundation.	cu m	Per % cu m	% cu ft
4.	Earth filling in foundation trenches	cu m	Per % cu m	% cu ft
5.	Earth filling in plinth	cu m	Per % cu m	% cu ft
6.	Earthwork in banking,cutting,in road and irrigation channel.	cu m	Per % cu m	% cu ft
7.	Surface dressing and levelling,cleaning,etc	sq m	Per sq m	% sq ft
8.	Cutting of trees(Girth specified)	no.	Per no.	Per no.
9.	Pudding,puddle clay core	cu m	Per % cu m	% cu ft
10.	Sand filling	cu m	Per cu m	% cu ft
11.	Quarrying of stone or boulder	cu m	Per cu m	% cu ft
12.	Blasting of rock (blasted stone stacked and then measured)	cu m	Per cu m	% cu ft

Estimation & cost evaluation-1

Concrete-

1.	Lime concrete(L.C)in foundation	cu m	Per cu m	% cu ft
2.	Lime concrete(L.C) in roof terracing,thickness specified	sq m	per sq m	% sq ft
3.	Cement concrete(C.C)	cu m	per cu m	per cu ft
4.	Reinforced cement concrete(R.C.C.)	cu m	per cu m	per cu ft
5.	C.C. or R.C.C. Chujja,sun shade	cu m	per cu m	per cu ft
6.	Precast C.C.or R.C.C.	cu m	per cu m	per cu ft
7.	Jali work or jaffri work or C.C. tracery panels(Thickness specified)	sq m	per sq m	per sq ft
8.	Cement concrete bed	cu m	per cu m	per cu ft
9.	Damp proof course-Cement concrete,rich cemen mortar.asphalt,etc.(Thicknes s specified)	sq m	per sq m	% sq ft

Brickwork-

1.	Brickworkin foundation and plinth in super structure,in arches,etc., in cement,lime or mud mortar	cu m	per cu m	% cu ft
2.	Sun dried brickwork	cu m	per cu m	% cu ft
3.	Honey-comb brickwork,thickness specified	sq m	per sq m	%sq ft
4.	Brickwork in jack arches,if measured separatly	cu m	per cu m	% cu ft
5.	Jack arch roofing including top finishing	sq m	per sq m	% sq ft
6.	Brickwork in well steining	cu m	per cu m	% cu ft
7.	Half-brickwork with or without reinforcement	sq m	per sq m	% sq ft
8.	Thin partition wall	sq m	per sq m	% sq ft
9.	Reinforced brickwork (R.B.WORK)	cu m	per cu m	% cu ft
10.	String course,drip			

course,weather   meter per m per r ft
course,coping
etc.(Projection specified)

11.   Cornice(Projection and type   meter per m per r ft
specified)

12.   Brickwork in Fire   cu m   per cu m   % cu ft
place,Chullah,Chimney

13.   Pargetting Chimney,fire   meter per m per r ft
place flue

14.   Brick edging (by road side)   meter per m per r ft

STONE WORK-

1.   Stone masonry,Random
Rubble masonry,Coursed   cu m   per cu m   % cu ft
Rubble masonry,Ashlar
masonry in walls, in
arches,etc.

2.   Cut stone work in lintel   cu m   per cu m   per cu ft
beam,etc.

3.   Stone slab in
roof,shelve,etc.,stone   sq m   per sq m   % sq ft
chujjas,stone sun shed etc.

4.   Stone work in wall facing or   sq m   per sq m   per sq ft
lining(Thickness specified)

15

16

Estimation & cost evaluation-1
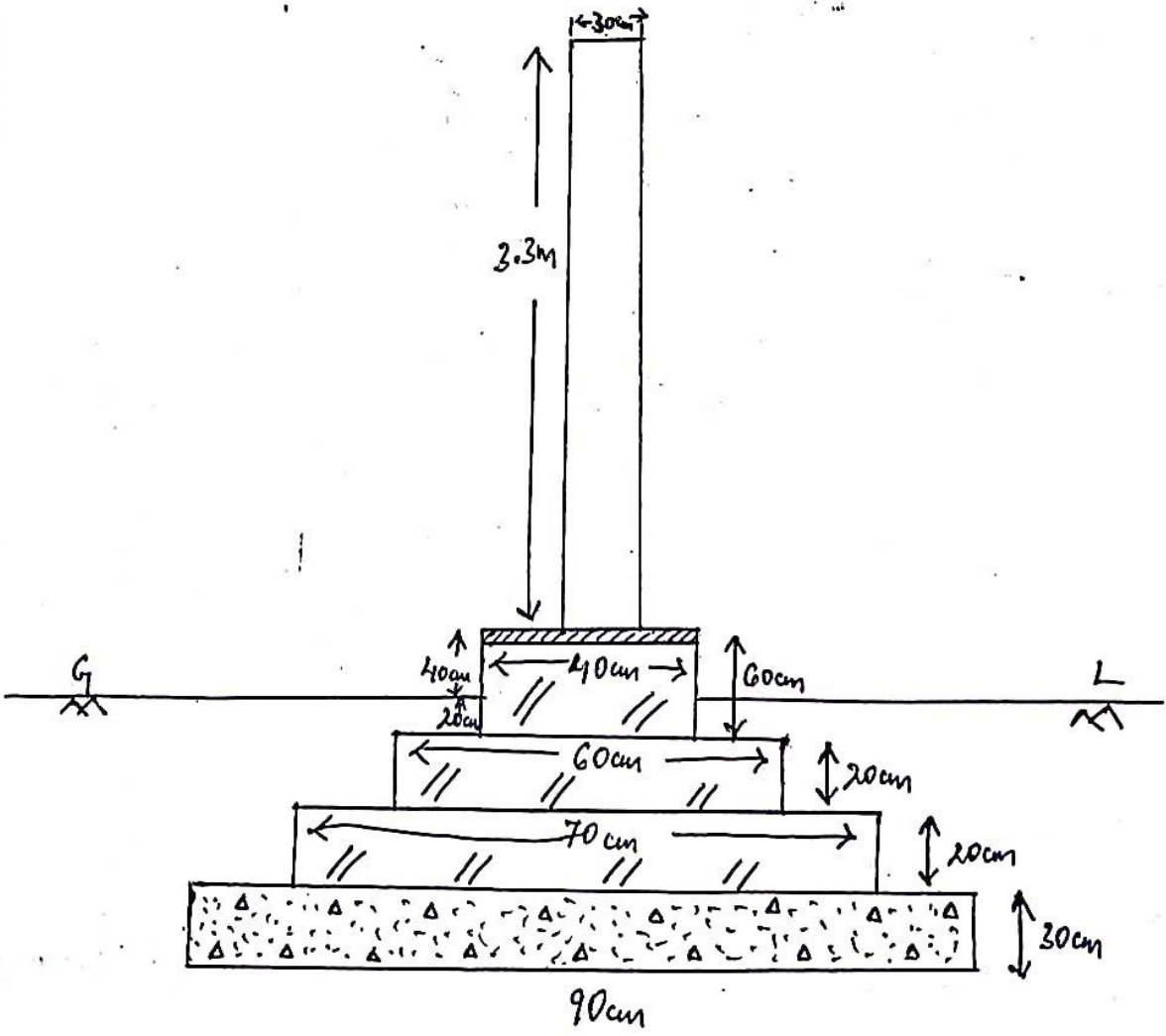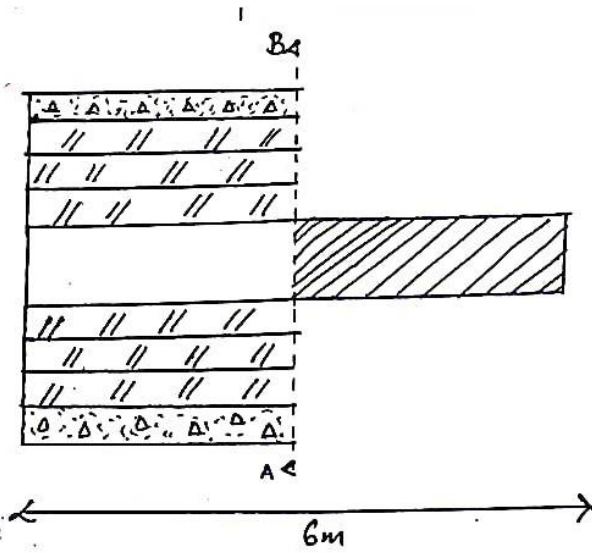
SINGLE WALL QUANTITY ESTIMATION

m no.	Particulars of Item	No.	Length	Breadth	Height	Quantity	Explanatory Note
1	E/W Excation in foundation	1	6M	0.9M	0.9M	4.86M3	H=0.3+0.2+0.2+0.2=0.9M
2	Concrete in foundation	1	6M	0.9M	0.3M	1.62M3	---------------
3	B/W in foundation & footing						
(i)	1st footing	1	6M	0.7M	0.2M	0.84M3	---------------
(ii)	2nd footing	1	6M	0.6M	0.2M	0.72M3	----------------
(iii)	Plinth	1	6M	0.4M	0.6M	1.44M3	H=0.4+0.2=0.6M
					Total=	3M3	
4	DPC	1	6M	0.4M		2.4M2	-----------------
5	B/W in Superstucture	1	6M	0.3M	3.3M	5.94M3	-----------------
6	Plastering						
(i)	Inside	1	6M	---	3.3M	19.8M3	-----------------
(ii)	Outside	1	6M	---	3.8M	22.8M3	H=3.3+0.4+0.1=3.8M
					Total=	42.6M3	

SINGLE WALL WITH WINDOW QUANTITY ESTIMATION

m no.	Particular of Item	No.	Length	Breadth	Height	Quantity	Explanatory Note
1	E/W Excavation in foundation	1	5.5M	1.1M	1.3M	7.86M3	H=0.3+0.2+0.2+0.3+0.3=1.3M
2	Concrete in foundation	1	5.5M	1.1M	0.3M	1.81M3	------------
3	B/W in foundation & footing						
(i)	1st footing	1	5.5M	0.7M	0.3M	1.15M3	----------
(ii)	2nd footing	1	5.5M	0.6M	0.2M	0.66M3	----------
(iii)	3rd footing	1	5.5M	0.5M	0.2M	0.55M3	------------
iv)	Plinth	1	5.5M	0.4M	0.9M	1.98M3	H=0.6+0.3=0.9M
					Total=	4.34M3	
4	DPC	1	5.5M	0.4M	----	2.2M2	-----------
5	B/W in Superstructure	1	5.5M	0.3M	3.2M	5.28M3	------------
	Deduction						
(i)	Window	1	1M	0.3M	2.2M	0.66M3	--------------

Estimation & cost evaluation-1

Estimation & cost evaluation-1

SINGLE ROOM BUILDING QUANTITY ESTIMATE

$(6 \times 4) \, m$

$6m$

$\dfrac{0.2}{2}$

$D = (1.2 \times 2.1) \, m$

$W = (1 \times 1.2) \, m$

20cm

3.3m

2.5cm (DPC)

45cm

30cm 300cm

20cm 40cm

20cm 50cm

20cm

70cm

(Section X–X)

(6×4) m

$6m$

$\frac{0.3}{2}$

$D = (1.2 \times 2.1)$ m

$W = (1 \times 1.2)$ m

$3.3m$

$20cm$

$2.5cm$ (DPC)

$45cm$

$30cm$

$30cm$

$20cm$

$40cm$

$20cm$

$50cm$

$20cm$

$70cm$

(Section $x - x$)

Estimation & cost evaluation-1

SINGLE ROOM BUILDING QUANTITY ESTIMATION

	C/C distance of L/W= 0.3/2+6+0.3/2 = 6.3M					
	C/C distance of S/W= 0.3/2+5+0.3/2 = 5.3M					
m no.	Particular of Item	No.	Length	Breadth	Height	Quantity	Explanatory Note
1	E/W excavation in foundation						
	Long wall	2	7.1M	0.8M	1.1M	12.496M3	H=0.4+0.2+0.2+0.3=1.1M, L=6.3+0.8=7.1M
	Short wall	2	4.5M	0.8M	1.1M	7.92M3	L=5.3-0.8= 4.5M
					Total=	20.416M3	
2	Concrete in foundation						
	Long wall	2	7.1M	0.8M	0.3M	3.408M3	L= 6.3+0.8= 7.1M
	Short wall	2	4.5M	0.8M	0.3M	2.16M3	L=5.3-0.8= 4.5M
					Total=	5.568M3	
3	B/W in footing & foundation						
	Long wall						
(i)	1st footing	2	7M	0.7M	0.2M	1.96M3	L= 6.3+0.7= 7M
(ii)	2nd footing	2	6.8M	0.5M	0.2M	1.36M3	L= 6.3+0.5= 6.8M
(iii)	Plinth	2	6.7M	0.4M	1M	5.36M3	L= 6.3+0.4=6.7M & H= 0.4+0.6=1M
	Short wall						
(i)	1st footing	2	4.6M	0.7M	0.2M	1.288M3	L= 5.3-0.7= 4.6M
(ii)	2nd footing	2	4.8M	0.5M	0.2M	0.96M3	L= 5.3-0.5= 4.8M
(iii)	Plinth	2	4.9M	0.4M	1M	3.92M3	L= 5.3-0.4= 4.9M & H= 0.4+0.6= 1M
					Total=	14.848M3	
4	DPC						
	Long wall	2	6.7M	0.4M	-----	5.36M2	L= 6.3+0.4= 6.7M
	Short wall	2	4.9M	0.4M	-----	3.92M2	L= 5.3-0.4= 4.9M
					Total=	9.28M2	
5	B/W in superstructure						
	Long wall	2	6.6M	0.3M	3.2M	12.672M3	L= 6.3+0.3= 6.6M
	Short wall	2	5M	0.3M	3.2M	9.6M3	L= 5.3-0.3= 5M
					Total=	22.272M3	
6	Plastering						
	Long wall						
	Inside	2	6M	------	3.2M	38.4M2	----------------
	Outside	2	6.6M	------	4M	52.8M2	L=6+0.3+0.3= 6.6M &H= 3.2+0.6+0.2= 4M
	Short wall						
	Inside	2	5M	-------	3.2M	32M2	--------------
	Outside	2	5.6M	-------	4M	44.8M2	L=5+0.3+0.3=5.6M & H= 3.2+0.6+0.2= 4M

Estimation & cost evaluation-1

PLAN

7.5 cm L.C. Terrace Over 13 cm R.C.C.

Room 4m × 6m

Room 6m × 6m

30 cm

30 cm

30 cm

4.00 m
D

6.00 m
D

30 cm.

6.00 m
W

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

4.20 m

30 cm

2 cm D.P.C.
2.5 cm c.c. Over 7.5 cm L.C.

60 cm

40 cm

G   L

20 cm

50 cm
60 cm
70 cm

80 cm

10 cm
10 cm
10 cm
20 cm
30 cm

1.10 m   Lime conc.

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

CROSS SECTION OF WALL ON AA.

Fig. 2-6

23

TWO ROOM BUILDING WITH WINDOW,DOOR & SHELVES QUANTITY ESTIMATION

n no.	Particular of Item	No.	Length	Breadth	Height	Quantity	Explanatory Note
	C/C distance of L/W= 0.3/2+4+0.3+6+0.3/2= 10.6M						
	C/C distance of S/W= 0.3/2+6+0.3/2= 6.3M						
1	E/W excavation in foundation						
	Long wall	2	11.7M	1.10M	1M	25.74M3	L=10.6+1.10= 11.7M & H=0.2+0.1+0.1+0.1+0.2+0.3=1M
	Short wall	3	5.2M	1.10M	1M	17.16M3	L=6.3-1.10= 5.2M
					Total=	42.9M3	
2	Concrete in foundation						
	Long wall	2	11.7M	1.10M	0.3M	7.722M3	L=10.6+1.10= 11.7M
	Short wall	3	5.2M	1.10M	0.3M	5.148M3	L=6.3-1.10= 5.2M
					Total=	12.87M3	
3	B/W in footing & foundation						
	Long wall						
(i)	1st footing	2	11.4M	0.8M	0.2M	3.648M3	L=10.6+0.8= 11.4M
(ii)	2nd footing	2	11.3M	0.7M	0.1M	1.582M3	L=10.6+0.7= 11.3M
(iii)	3rd footing	2	11.2M	0.6M	0.1M	1.344M3	L=10.6+0.6= 11.2M
(iv)	4th footing	2	11.1M	0.5M	0.1M	1.11M3	L=10.6+0.5= 11.1M
(v)	Plinth	2	11M	0.4M	0.8M	7.04M3	L=10.6+0.4= 11M & H=0.2+0.6=0.8M
	Short wall						
(i)	1st footing	3	5.5M	0.8M	0.2M	2.64M3	L=6.3-0.8= 5.5M
(ii)	2nd footing	3	5.6M	0.7M	0.1M	1.176M3	L=6.3-0.7= 5.6M
(iii)	3rd footing	3	5.7M	0.6M	0.1M	1.026M3	L=6.3-0.6= 5.7M
(iv)	4th footing	3	5.8M	0.5M	0.1M	0.87M3	L=6.3-0.5= 5.8M
(v)	Plinth	3	5.9M	0.4M	0.8M	5.664M3	L=6.3-0.4= 5.9M & H=0.2+0.6= 0.8M
					Total=	26.1M3	
4	DPC						
	Long wall	2	11M	0.4M	----	8.8M2	L=10.6+0.4= 11M
	Short wall	3	5.9M	0.4M	----	7.08M2	L=6.3-0.4= 5.9M
					Total=	15.88M2	
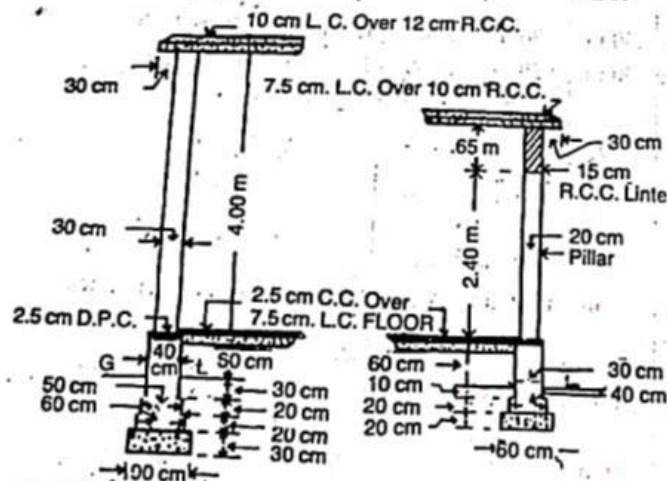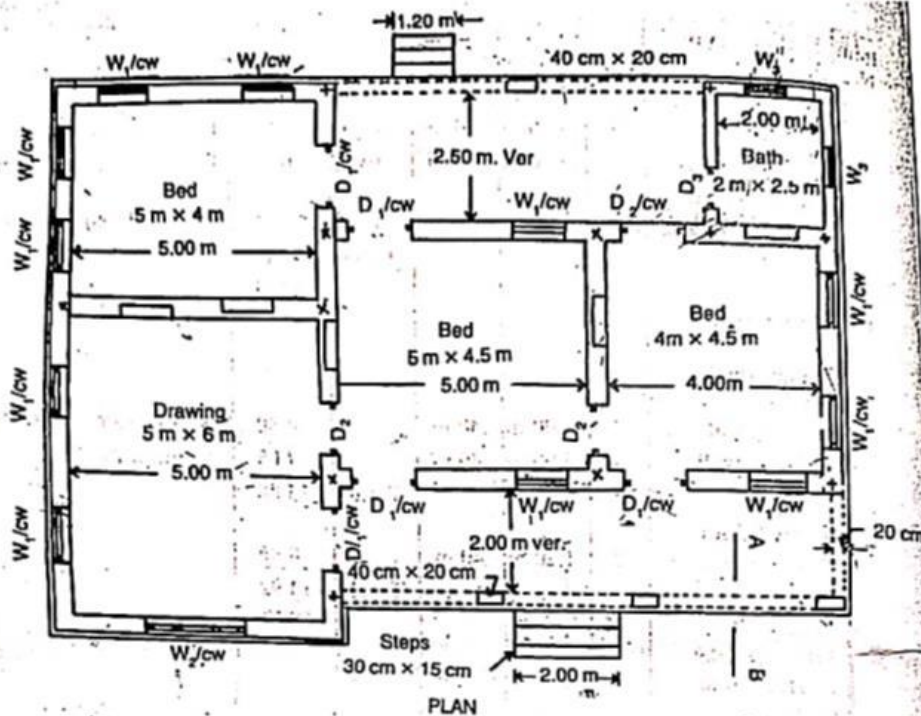
PLAN

Doors:-
D₁ – 120 cm × 210 cm (1.20 m × 2.10 m)
D₂ – 100 cm × 200 cm (1.00 m × 2.00 m)
D₃ – 75 cm × 180 cm (.75 m × 1.80 m)

Windows:-
W₁ – 100 cm × 150 cm (1.00 m × 1.50 m)
W₂ – 200 cm × 150 cm (2.00 m × 1.50 m)
W₃ – 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.

CROSS SECTION OF MAIN WALLS   CROSS SECTION AB OF VER.WALL

All walls of Drawing Rooms and   Bath Room walls have similar
Bed Rooms have same section   section.

Note— No beam has been shown in the plan.

Fig. 2-7

Estimation & cost evaluation-1

MULTIROOM BUILDING WITH DOUBLE VERANDAH QUANTITY ESTIMATION

(i)	**Leftside Bedroom & Drawing room**
	C/C distance of L/W= 0.3/2+6+0.3+4+0.3/2= 10.6M
	C/C distane of S/W= 0.3/2+5+0.3/2= 5.3M
(ii)	**Rightside two Bedroom**
	C/C distance of L/W= 0.3/2+5+0.3+4+0.3/2= 9.6M
	C/C distance of S/W= 0.3/2+4.5+0.3/2= 4.8M
(iii)	**Front Verandah**
	C/C distance of L/W= 0.3/2+5+0.3+4+0.1+0.2/2= 9.65M
	C/C distance of S/W= 0.2/2+2+0.3/2= 2.25M
(iv)	**Back Verandah & Bathroom**
	C/C distance of L/W= 0.3/2+5+0.3+4+0.1+0.2/2= 9.65M
	C/C distance of S/W= 0.3/2+2.50+0.2/2= 2.75M

n no.	Particular of Item	No.	Length	Breadth	Height	Quantity	Explanatory Note
1	**E/W excavation in foundation**						
(i)	Leftside Bedroom & Drawing room						
	Long wall	2	11.5M	0.9M	1M	20.7M3	L=10.6+0.9= 11.5M & H=0.3+0.2+0.2+0.3=1M
	Short wall	3	4.4M	0.9M	1M	11.88M3	L=5.3-0.9= 4.4M
(ii)	Rightside two Bedroom						
	Long wall	2	10.5M	0.9M	1M	18.9M3	L=9.6+0.9= 10.5M & H=0.3+0.2+0.2+0.3= 1M
	Short wall	2	3.9M	0.9M	1M	7.02M3	L=4.8-0.9= 3.9M
(iii)	Front Verandah						
	Long wall	1	9.5M	0.6M	0.5M	2.85M3	L=9.65+0.6/2-0.9/2= 9.5M & H=0.1+0.2+0.2= 0.5M
	Short wall	1	1.5M	0.6M	0.5M	0.45M3	L=2.25-0.6/2-0.9/2= 1.5M
(iv)	Back Verandah & Bathroom						
	Long wall	1	9.5M	0.6M	0.5M	2.85M3	L=9.65+0.6/2-0.9/2= 9.5M & H=0.1+0.2+0.2= 0.5M
	Short wall	2	2M	0.6M	0.5M	1.2M3	L=2.75-0.6/2-0.9/2= 2M
					Total=	65.85M3	
2	**Concrete in foundation**						

(i)	Leftside Bedroom & Drawing room						
	Long wall	2	11.5M	0.9M	0.3M	6.21M3	L=10.6+0.9= 11.5M
	Short wall	3	4.4M	0.9M	0.3M	3.564M3	L=5.3-0.9= 4.4M
(ii)	Rightside two Bedroom						
	Long wall	2	9.6M	0.9M	0.3M	5.184M3	L=9.6+0.9/2-0.9/2= 9.6M
	Short wall	2	3.9M	0.9M	0.3M	2.106M3	L=4.8-0.9/2-0.9/2= 3.9M
(iii)	Front Verandah						
	Long wall	1	9.7M	0.6M	0.2M	1.164M3	L=9.65+0.6/2-0.5/2= 9.7M
	Short wall	1	1.7M	0.6M	0.2M	0.204M3	L=2.25-0.6/2-0.5/2= 1.7M
(iv)	Back Verandah & Bathroom						
	Long wall	1	9.7M	0.6M	0.2M	1.164M3	L=9.65+0.6/2-0.5/2= 9.7M
	Short wall	2	2.2M	0.6M	0.2M	0.528M3	L=2.75-0.6/2-0.5/2= 2.2M
					Total=	18.756M3	
3	B/W in footing & foundation						
(i)	Leftside Bedroom & Drawing room						
	Long wall						
*	1st footing	2	11.2M	0.6M	0.2M	2.688M3	L=10.6+0.6= 11.2M
*	2nd footing	2	11.1M	0.5M	0.2M	2.22M3	L=10.6+0.5= 11.1M
*	Plinth	2	11M	0.4M	0.9M	7.92M3	L=10.6+0.4= 11M & H=0.6+0.3= 0.9M
	Short wall						
*	1st footing	3	4.7M	0.6M	0.2M	1.692M3	L=5.3-0.6= 4.7M
*	2nd footing	3	4.8M	0.5M	0.2M	1.44M3	L=5.3-0.5= 4.8M
*	Plinth	3	4.9M	0.4M	0.9M	5.292M3	L=5.3-0.4= 4.9M & H=0.6+0.3= 0.9M
(ii)	Rightside two Bedroom						
	Long wall						
*	1st footing	2	9.6M	0.6M	0.2M	2.304M3	L=9.6+0.6/2-0.6/2= 9.6M
*	2nd footing	2	9.6M	0.5M	0.2M	1.92M3	L=9.6+0.5/2-0.5/2= 9.6M
*	Plinth	2	9.6M	0.4M	0.9M	6.912M3	L=9.6+0.4/2-0.4/2= 9.6M & H=0.6+0.3= 0.9M
	Short wall						
*	1st footing	2	4.2M	0.6M	0.2M	1.008M3	L=4.8-0.6/2-0.6/2= 4.2M
*	2nd footing	2	4.3M	0.5M	0.2M	0.86M3	L=4.8-0.5/2-0.5/2= 4.3M

Estimation & cost evaluation-1

*	Plinth	3	4.9M	0.4M	0.9M	5.292M3	L=5.3-0.4= 4.9M & H=0.6+0.3= 0.9M
(ii)	Rightside two Bedroom						
	Long wall						
*	1st footing	2	9.6M	0.6M	0.2M	2.304M3	L=9.6+0.6/2-0.6/2= 9.6M
*	2nd footing	2	9.6M	0.5M	0.2M	1.92M3	L=9.6+0.5/2-0.5/2= 9.6M
*	Plinth	2	9.6M	0.4M	0.9M	6.912M3	L=9.6+0.4/2-0.4/2= 9.6M & H=0.6+0.3= 0.9M
	Short wall						
*	1st footing	2	4.2M	0.6M	0.2M	1.008M3	L=4.8-0.6/2-0.6/2= 4.2M
*	2nd footing	2	4.3M	0.5M	0.2M	0.86M3	L=4.8-0.5/2-0.5/2= 4.3M
*	Plinth	2	4.4M	0.4M	0.9M	3.168M3	L=4.8-0.4/2-0.4/2= 4.4M & H=0.6+0.3= 0.9M
(iii)	Front Verandah						
	Long wall						
*	1st footing	1	9.65M	0.4M	0.2M	0.772M3	L=9.65-0.4/2+0.4/2= 9.65M
*	Plinth	1	9.6M	0.3M	0.7M	2.016M3	L=9.65-0.4/2+0.3/2= 9.6M & H=0.6+0.1= 0.7M
	Short wall						
*	1st footing	1	1.85M	0.4M	0.2M	0.148M3	L=2.25-0.4/2-0.4/2= 1.85M
*	Plinth	1	1.9M	0.3M	0.7M	0.399M3	L=2.25-0.4/2-0.3/2= 1.9M & H=0.6+0.1= 0.7M
(iv)	Back Verandah & Bathroom						
	Long wall						
*	1st footing	1	9.65M	0.4M	0.2M	0.772M3	L=9.65-0.4/2+0.4/2= 9.65M
*	Plinth	1	9.6M	0.3M	0.7M	2.016M3	L=9.65-0.4/2+0.3/2= 9.6M & H=0.6+0.1= 0.7M
	Short wall						
*	1st footing	2	2.35M	0.4M	0.2M	0.376M3	L=2.75-0.4/2-0.4/2= 2.35M
*	Plinth	2	2.4M	0.3M	0.7M	1.008M3	L=2.75-0.4/2-0.3/2= 2.4M
					Total=	41.596M3	
4	DPC						
(i)	Leftside Bedroom & Drawing room						
	Long wall	2	11M	0.4M	-----	8.8M2	L=10.6+0.4= 11M
	Short wall	3	4.9M	0.4M	-----	5.88M2	L=5.3-0.4= 4.9M
(ii)	Rightside two Bedroom						
	Long wall	2	9.6M	0.4M	------	7.68M2	L=9.6+0.4/2-0.4/2= 9.6M

Estimation & cost evaluation-1

(iii)	Front & Back Verandah(Pillars)	4	8M	0.3M	------	9.6M2	
(iv)	Bathroom						
	Long wall	2	3.05M	0.3M	-------	1.83M2	L=2.75+0.3= 3.05M
	Short wall	1	1.9M	0.3M	-------	0.57M2	L=2.2-0.3= 1.9M
					Total=	37.88M2	
5	**B/W in superstructure**						
(i)	Leftside Bedroom & Drawing room						
	Long wall	2	10.9M	0.3M	4M	26.16M3	L=10.6+0.3= 10.9M
	Short wall	3	5M	0.3M	4M	18M3	L=5.3-0.3= 5M
(ii)	Rightside two Bedroom						
	Long wall	2	9.6M	0.3M	4M	23.04M3	L=9.6+0.3/2-0.3/2= 9.6M
	Short wall	2	4.5M	0.3M	4M	10.8M3	L=4.8-0.3/2-0.3/2= 4.5M
(iii)	Front Verandah (Wall above Pillar)						
	Long wall	1	9.6M	0.2M	0.50M	0.96M3	L=9.65-0.3/2+0.2/2= 9.6M & H=0.65-0.15= 0.50M
	Short wall	1	2M	0.2M	0.50M	0.2M3	L=2.25-0.2/2-0.3/2= 2M & H=0.65-0.15= 0.50M
(iv)	Back Verandah (Wall above Pillar)						
	Long wall	1	9.6M	0.2M	0.50M	0.96M3	L=9.65-0.3/2+0.2/2= 9.6M & H=0.65-0.15= 0.50M
	Short wall	2	2.5M	0.2M	0.50M	0.5M3	L=2.75-0.3/2-0.2/2= 2.5M & H=0.65-0.15= 0.50M
(v)	Pillars	4	8M	0.2M	2.40M	15.36M3	-----------------
(vi)	Bathroom						
	Long wall	2	2.7M	0.2M	2.40M	2.592M3	L=2.75+0.2/2-0.3/2= 2.7M
	Short wall	1	1.95M	0.2M	2.40M	0.936M3	L=2.2-0.3/2-0.2/2= 1.95M
					Total=	100.668M3	
	Deduction						
	Window(1)	11	1M	0.3M	1.50M	4.95M3	-----------------------------
	Lintel over Window	11	1.3M	0.3M	0.15M	0.643M3	L=1+0.15+0.15= 1.3M
	Window(2)	1	2M	0.3M	1.50M	0.9M3	-----------------------------
	Lintel over Window	1	2.3M	0.3M	0.15M	0.103M3	L=2+0.15+0.15= 2.3M
	Window(3)	2	0.75M	0.2M	1.20M	0.36M3	-----------------------------
	Lintel over Window	2	1.05M	0.2M	0.15M	0.063M3	L=0.75+0.15+0.15= 1.05M

Estimation & cost evaluation-1

Door(1)	5	1.20M	0.3M	2.10M	3.78M3	-----------------------------
Lintel over Door	5	1.5M	0.3M	0.15M	0.337M3	L=1.20+0.15+0.15= 1.5M
Door(2)	3	1M	0.3M	2M	1.8M3	-----------------------------
Lintel over Door	3	1.3M	0.3M	0.15M	0.175M3	L=1+0.15+0.15= 1.3M
Door(3)	1	0.75M	0.2M	1.80M	0.27M3	-----------------------------
Lintel over Door	1	1.05M	0.2M	0.15M	0.031M3	L=0.75+0.15+0.15= 1.05M
Shelves	5	1M	0.3M	1.50M	2.25M3	-----------------------------
Lintel over Shelves	5	1.3M	0.3M	0.15M	0.292M3	L=1+0.15+0.15= 1.3M
				Total=	15.954M3	

6	Plastering					
(i)	Leftside Bedroom & Drawing room					
	Inside	1	40M	----	4M	160M2	L=[{2(5+6)+2(5+4)}]= 40M
	Outside	1	31.8M	----	4.7M	149.46M2	L=2(10.9+5)= 31.8M & H=4+0.6+0.1= 4.7M
(ii)	Rightside two Bedroom					
	Inside	1	36M	----	4M	144M2	L=[{2(5+4.5)+2(4+4.5)}]= 36M
	Outside	1	28.2M	----	4.7M	132.54M2	L=2(9.6+4.5)= 28.2M H=4+0.6+0.1= 4.7M
(iii)	Front Verandah (Wall above Pillar)					
	Inside	1	11.3M	----	0.50M	5.65M2	L=5+0.3+4+2= 11.3M & H=0.65-0.15= 0.50M
	Outside	1	11.8M	----	0.50M	5.9M2	L=5+0.3+4+0.2+0.1+2+0.2= 11.8M & H=0.65-0.15= 0.50M
(iv)	Back Verandah (Wall above Pillar)					
	Inside	1	16.3M	----	0.50M	8.15M2	L=7.3+2(2+2.5)= 16.3M & H=0.65-0.15= 0.50M
	Outside	2	12.3M	----	0.50M	12.3M2	L=9.6+2.7= 12.3M & H=0.65-0.15= 0.50M
(v)	Pillars	4	8M	----	2.40M	76.8M2	-----------------
(iv)	Bathroom					
	Inside	1	9M	----	2.40M	21.6M2	L=2(2+2.5)= 9M
	Outside	1	9.8M	----	3.1M	30.38M2	L=2(2.4+2.5)= 9.8M & H=2.40+0.6+0.1= 3.1M
					Total=	746.78M2	

CHAPTER-3

Rate analysis

Definition: In order to determine the rate of a particular item, the factors affecting the rate of that item are studied carefully and then finally a rate is decided for that item. This process of determining the rates of an item is termed as analysis of rates or rate analysis. The rate of particular item of work depends on the following.

1. Specifications of works and material about their quality, proportion and constructional Operation method.
2. Quantity of materials and their costs.
3. Cost of labours and their wages.
4. Location of site of work and the distances from source and conveyance charges.
5. Overhead and establishment charges
6. Profit

Dry material calculation

Let us assume the mix proportion is 1 : 2: 4 (Cement : Sand : Stone = a:b:c)

Volume of wet concrete = 1 m3

Volume of dry concrete = 1 × 1.54 = 1.54 m3

Calculation For Cement

Formula, Cement = (Volume of dry concrete/a+b+c) × a

- = (1.54/a+b+c) × a = [(1.54/1+2+4)] × 1 = 0.22 cum

Now density of cement = 1440 kg/cu.m

∴ Volume of cement = 0.22 × 1440 =316.8 kg.

As we know, 1 bag of cement contains 50 kg of cement .

∴ Cement bags required = 316.8/50 = 6.33 bags.

Calculation For Sand

Formula, Sand = (Volume of dry concrete/a+b+c) × b

- = (1.54/a+b+c) × b = (1.54/1+2+4) × 2 = 0.44 cu.m.

Calculation For Aggregates

Formula, Aggregatess = (Volume of dry concrete/a+b+c) × c

- = (1.54/a+b+c) ×c = (1.54/1+2+4) × 4 = 0.88 cu.m.

Calculation For Water Content

Let us assume the water cement ratio of concrete is 0.45.

- w/c = 0.45
- Required water for 1 bag cement = 0.45 × 0.0353 = 0.0159 cu.m.

Where volume of 50 kg cement = 0.0353 cu.m

- 1 m3 water = 1000 Litre

Required water for 1 bag cement=0.0159 × 1000 = 15.9 Litre.

∴ Required water for 6.33 bags cement = 6.33 × 15.9 = 101 Litre.
Summary
- Cement = 6.33 bags.
- Sand = 0.44 cum
- Aggregates = 0.88 cum
- Water = 101 litre.

DRY MATERIAL CALCULATION OF BRICKWORK

Assuming,
1. Volume of brickwork = 1 m3
2. Grade of mortar = 1:6 (cement : sand)
3. First class brick (190 mm x 90 mm x 90 mm)
4. Thickness of mortar = 10 mm = 0.01 m

No. Of Bricks:
No. of bricks = (Volume of brickwork / Volume of one brick with mortar)
Volume of one brick without mortar = 0.19×0.09×0.09 = 0.001539 m3
Volume of brick with mortar = (0.19+0.01) x (0.09+0.1)x (0.09+0.1)
= 0.2×0.1×0.1 = 0.002 m3
No.of bricks = 1.0/ (0.002) = 500
Consider 10% to 15 % bricks as wastage.
∴ Total no. of bricks = 500 + (10 x 500)/100 = 550
Quantity Of Mortar:
Volume occupied by bricks = No. of bricks x Volume of one brick
Volume of bricks = 500 x 0.001539 = 0.7695 m3
Volume of mortar = Volume of brickwork – Volume of bricks
∴ Volume of Mortar = 1.0 – 0.7695 = 0.2305 m3
Quantity Of Cement:
Cement = (Dry volume of mortar x Cement ratio)/ Sum of the ratio
Dry volume of Mortar = 1.54 x 0.2305 = 0.35497 m3 (54% increment due to volume shrink after adding water)
Cement= (0.35497 x 1) / (1+6) = 0.35497/7= 0.05071 m3
Cement = 0.043795 x 1440 = 73.0224 kg
∴ No. of cement bags = 73.0224 / 50 = 1.45 bags (1 bag cement contains 50 kg cement)
Quantity of Sand:
Sand = (Dry volume of mortar x Sand ratio)/Sum of the ratio
Sand = (0.35497 x 6)/7 = 2.12982/7 = 0.30426 m3
∴ Sand = 0.30426 x 35.3147 = 10.7448 cft.
Summery:

Number of Bricks = 550
Cement = 1.45 bags of 50 Kg
Sand = 0.30426 m3 or10.7448 cft.

Estimation & cost evaluation-1

CHAPTER-4

DUTIES AND RESPONSIBILITIES OF JUNIOR ENGINEER

1. To assist in checking of all type of estimates, DPR in accordance of Designs and drawings and submit them to the higher authority for sanction/ process.
2. Carrying out of survey work of areas for development and preparation of Survey plans.
3. Custody of Mathematical and survey instruments, tools and plant including their receipt and issue and maintenance of their account.
4. Carrying out Laboratory test on materials such as cement, steel, timber, metal, soil, aggregates or any other material as directed by higher authorities.
5. Carrying out field tests on soils etc. as directed by higher authorities.
6. To prepare estimates as directed by his higher authority and submit them to the higher authority for sanction.
7. To assist in checking of Schedule of Rates, Analysis of rates and specification of schedule of works.
8. To assist in preparation and checking of Specifications of work.
9. To assist in checking of theoretical consumption of materials.
10. To assist in checking of supplementary, substituted and deviation item statements.

DUTIES AND RESPONSIBILITIES OF ASSISTANT ENGINEER

1. Assist in designing, developing and executing construction projects.
Work with Project Manager in reviewing project specification and in preparing project plan and design sheet.
2. Visit sites, prepare construction drawings and develop samples.
Work with engineering team in developing construction plan.
3. Determine budget, schedule, manpower and material and equipment requirements to execute construction project.
4. Report all expenses to Project Manager on timely basis.
5. Maintain quality assurance standards for projects.
6. Provide technical assistance to field staff when needed.
7. Analyze construction problems and recommend corrective actions.
8. Order and stock construction materials to avoid shortage

Estimation & cost evaluation-1

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Estimation & cost evaluation-1

Lecture notes on

STRUCTURAL MECHANICS

(TH-1, 3RD SEM)

Diploma Engineering

Prepared by-

SIPRA SURAVI BEHERA

FACULTY CIVIL ENGG.

CONTENTS...

Chapter 1

SIMPLE AND COMPLEX STRESS AND STRAIN.

SIMPLE STRESS AND STRAIN :

* **Stress :** A resistance force offered by a body against the deformation is called stress.
  (change in shape and size)

② The external force acting on the body is called load.

③ In a single line, load is applied on the body and stress is induced in the material of the body.

④ Stress is denoted by 'σ' (sigma).

⑤ Let a rod of uniform cross-sectional area is subjected to pulling force (P), to resist the deformation, then an internal force (R), will be induced inside the material.

$\therefore$ Stress $= \dfrac{R}{A}$

$\boxed{\sigma = \dfrac{P}{A}}$ $[\because R = P]$

$\rightarrow$ Unit : $\dfrac{Newton}{(metre)^2}$

$= \dfrac{N}{m^2} = Pascal$

$\boxed{R = P}$ no tearing of the body

$\begin{bmatrix} applied = resisting \\ force \qquad force \end{bmatrix}$

Stress $= \dfrac{Internal\ resisting\ force}{area}$

1

UNITS OF STRESS

$1 \text{ Pascal} = 1 \text{ N/m}^2$

$1 \text{ KiloPascal} = 10^3 \text{ N/m}^2$

$1 \text{ Mega Pascal} = 10^6 \text{ N/m}^2$

$1 \text{ Giga Pascal} = 10^9 \text{ N/m}^2$

kilo $= 10^3$
Mega $= 10^6$
Giga $= 10^9$
Tera $= 10^{12}$
Milli $= 10^{-3}$
Micro $= 10^{-6}$
Nano $= 10^{-9}$
Pico $= 10^{-12}$

* $1 m \rightarrow 10^3 mm$

  $1 mm \rightarrow \dfrac{1}{10^3} m = 10^{-3} m$

* $1 m^2 = (10^3)^2 mm^2$

  $\quad = 10^6 mm^2$

* $1 mm^2 = 10^{-6} m^2$

* $1 kN = 10^3 N$

* $1 N = 10^{-3} kN$

We know,

$$1 MPa = 10^6 \text{ N/m}^2$$

$$= 10^6 \times \frac{10^{-3} kN}{10^6 mm^2}$$

$$= 10^{-3} kN/mm^2$$

$$= 10^{-3} \times 10^3 \text{ N/mm}^2$$

$$\boxed{1 MPa = 1 N/mm^2}$$

2

* <u>strain</u>: This is the ratio of change in dimension to the original dimension.

→ It is denoted by 'e'.

$$e = \boxed{\frac{\Delta l}{l}} \Rightarrow \frac{\text{change in length}}{\text{original length}} \Rightarrow \boxed{\frac{\Delta d}{d}} \Rightarrow \boxed{\frac{\Delta v}{v}}$$

linear strain [→ tensile → compressive]   lateral strain   volumetric strain

→ Strain has no units i.e. it is unitless.

* <u>Types of Stress</u>:

A material is capable of offering three types of stresses:

① Tensile stress
② Compressive stress
③ Shear stress

① <u>Tensile stress</u>: When the offering resistance by a section of a member is against an increase in length. the section is said to offer a tensile stress.
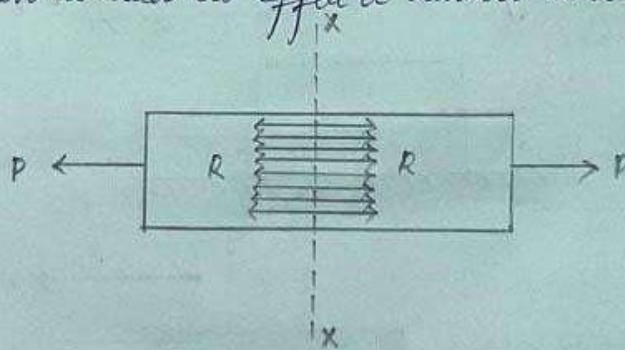
(ii) **Compressive Stress:** When the offering resistance by a section of a member is against a decrease in length, the section is said to offer a compressive stress.
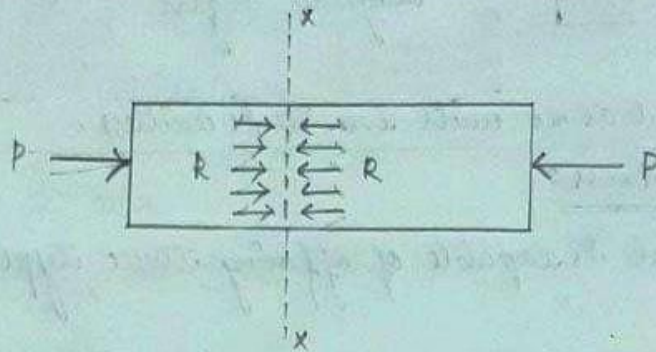
(iii) **Shear Stress: (τ)** A load P is applied tangentially along the top of the face of the body of which, bottom face is fixed. Such force acting tangentially along a surface is called as shear force.
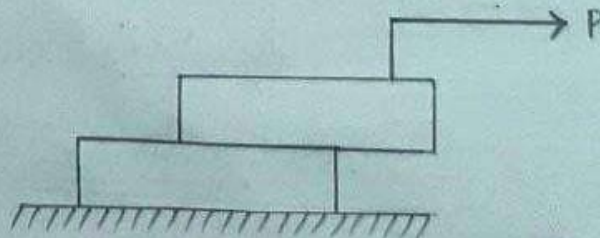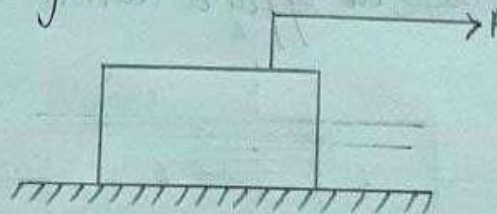
→ The resistance provided in this case is called shear resistance.

→ Shear stress is defined as the ratio of shear resistance to the shear area.

mathematically,
$$\tau = \frac{R}{A}$$

* Types of strain:

Strain can be of 4 types:

(i) __Tensile strain__: The ratio of increase in length to its original length, is called tensile strain.

$$e = \frac{\Delta l}{l}$$
→ here, Δl → change in __increase in length__

(ii) __Compressive Strain__: The ratio of decrease in length to the original length, is called compressive strain.

$$e = \frac{\Delta l}{l}$$
→ here, Δl → change in __decrease in org length__

(iii) __Lateral strain__: The ratio of the change in lateral dimension to the original lateral dimension, is called lateral strain.

$$e = \frac{\Delta d}{d}$$
→ here, Δd → change in lateral dimension

5

7

(iv) **Volumetric strain:** The ratio of the change in volume to its original volume, is called as volumetric strain.

$$e = \frac{\Delta v}{v}$$ → here, Δv → change in volume

* **Hooke's Law:** (most imp)
→ 2 marks regular question

It states that "when a material is loaded such that the intensity of stress within a certain limit, the ratio of the intensity of the stress to the corresponding strain is a constant."

$$\frac{stress}{strain} = \frac{\sigma}{e} = constant$$

$$\Rightarrow \frac{\sigma}{e} = constant$$

$$\Rightarrow \sigma = constant \times e$$

$$\Rightarrow \boxed{\sigma \propto e}$$

→ It also can be stated as "Stress is directly proportional to strain, within a certain limit."

6

8

* Modulus of Elasticity (or) Young's Modulus:

In case of axial loading, the ratio of the intensity of tensile on compressive stress to the corresponding strain is constant.

→ This ratio is called modulus of elasticity on young's modulus and is denoted by 'E'.

$$\frac{\sigma}{e} = constant = E$$

→ $$\boxed{E = \frac{\sigma}{e}}$$ $\left[\because constant \ is \ nothing \ but \ the \ young's \ modulus \ 'E'. \right]$

* Deformation of prismatic bar due to uni-axial load:

→ throughout the length no change in dimension or no change in cross-sectional area.

→ 1 or some
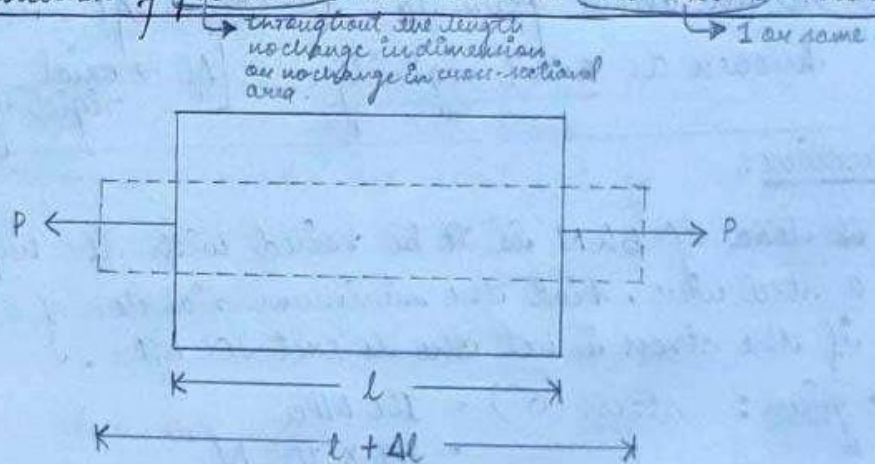
Consider a prismatic bar is subjected to axial tensile load ⓟ → same as an external force.

The stress will be introduced due to load P is given by, $$\boxed{\sigma = \frac{P}{A}}$$

7

9

we know, strain : $e = \dfrac{\Delta l}{l}$

According to Hooke's law,

$\sigma \propto e$

$\Rightarrow \sigma = Ee$

$\Rightarrow \dfrac{P}{A} = E \dfrac{\Delta l}{l}$

$\left[\because \sigma = \dfrac{P}{A} \text{ and } e = \dfrac{\Delta l}{l} \right]$

$\overset{***}{\underset{**}{\Rightarrow}} \boxed{\Delta l = \dfrac{Pl}{AE}}$

→ In the above formula, the item 'AE' is known as <u>axial rigidity</u>. $[AE \rightarrow \text{axial rigidity}]$

Questions :

① A load of 5kN is to be raised with the help of a steel wire. Find the minimum diameter of a wire, if the stress is not the to exit 100 MPa.

→ given : stress $(\sigma) = 100$ MPa
$= 100 \times 10^6 \dfrac{N}{m^2}$.

external force $(P) = 5kN$
$= 5 \times 10^3 N$

P=5kN ←○▭○→ P = 5kN

8

10

$$\sigma = \frac{P}{A}$$

$$\Rightarrow 100 \times 10^6 = \frac{5 \times 10^3}{\frac{\pi}{4} d^2}$$

$$\Rightarrow d^2 = \frac{5 \times 10^3}{100 \times 10^6 \times \frac{\pi}{4}}$$

$$\Rightarrow d = \sqrt{\frac{5 \times 10^3}{100 \times 10^6 \times \frac{\pi}{4}}}$$

$$\Rightarrow \boxed{d = 7.97 \times 10^{-3}_{m}}$$

$$\Rightarrow d = 7.97 \times 10^{-3} \times 10^3 \, mm$$

$$\Rightarrow \boxed{d = 7.97 \, mm}$$

② A steel rod 500 mm long and 20 mm × 10 mm in cross-section is subjected to axial pull of 300 kN. If modulus of elasticity is $2 \times 10^5 \frac{N}{mm^2}$. Calculate the elongation of the rod.
Also calculate the strain induced in the bar.

→ given : $l = 500 \, mm$
$$P = 300 kN = 300 \times 10^3 \, N$$
$$E = 2 \times 10^5 \, N/mm^2$$
$$A = 20 \, mm \times 10 \, mm$$
$$= 200 \, mm^2$$

9

$\therefore \Delta l$

$$\therefore \Delta l = \frac{Pl}{AE} = \frac{300 \times 10^3 \times 500}{200 \times 2 \times 10^5} = 3.75 \, mm$$

$$\text{and strain} = \frac{\Delta l}{l} = \frac{3.75}{500} = 7.5 \times 10^{-3}.$$

② A hollow cylinder 2m long has an outside diameter of 50 mm and inside diameter of 30 mm. If the cylinder is carrying a load of 25 kN, find the stress in cylinder, also find deformation of the cylinder. Take E = 100 GPa.

→ given: $l = 2m = 2 \times 1000 \, mm$
$= 2000 \, mm$

$D = 50 \, mm$
$d = 30 \, mm$
$E = 100 \, GPa$
$P = 25 \, kN$
$= 25 \times 10^3 \, N$

$\therefore$ Area of hollow cylinder: $A = \frac{\pi}{4}(D^2 - d^2)$

$\Rightarrow A = \frac{\pi}{4}((50)^2 - (30)^2)$

$\Rightarrow \boxed{A = 1256 \, mm^2}$

$$E = 100 \text{ GPa}$$
$$= 100 \times 10^9 \, \frac{N}{m^2}.$$

ight margin:

$1m \to 10^3 mm$

$1m^2 \to (10^3)^2 mm^2$

$\boxed{\to 1m^2 = 10^6 mm^2}$

$$= 100 \times 10^9 \, \frac{N}{10^6}$$
$$= 100 \times 10^9 \times 10^6 \, \frac{N}{mm^2}$$
$$E = 100 \times 10^3 \, \frac{N}{mm^2}$$

$$\therefore \quad \sigma = \frac{P}{A} = \frac{25 \times 10^3}{1256} = 19.90 \, \frac{N}{mm^2}$$

$$\text{and } \Delta l = \frac{Pl}{AE} = \frac{25 \times 10^3 \times 2000}{1256 \times 100 \times 10^3} = 0.398 \, mm.$$

* POISSON'S RATIO $\left(\mu / \frac{1}{m} \right)$

05.11.2021

$$\text{linear strain} = + \frac{l'}{l}$$

$$\text{lateral strain} = - \frac{d'}{d}$$

→ Poisson's ratio is defined as "the ratio between the lateral strain to the longitudinal strain."

→ It is denoted as μ or $\frac{1}{m}$ and the value of μ is always less than 1.

11
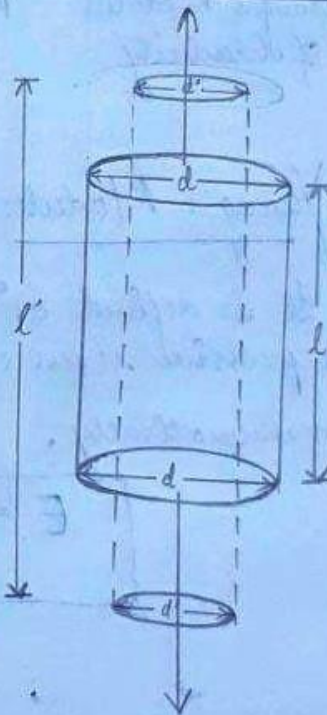
mathematically,

$$\mu = -\frac{\text{lateral strain}}{\text{longitudinal strain}}$$

→ It is a constant and it has no unit.

→ ex: ① Poisson ratio of concrete = 0.15 to 0.3
    ⑪ Poisson ratio of steel = 0.27 to 0.3

→ Poisson ratio varies from $\boxed{-1 \text{ to } 0.5}$

Elastic Constant

```
                    ┌──────────────┼──────────────────┐
                    ▼              ▼                   ▼
              E: Young's      K: Bulk           C: Shear
              Modulus         Modulus           Modulus (or)
              of Elasticity                     Modulus of
                                                Rigidity
```
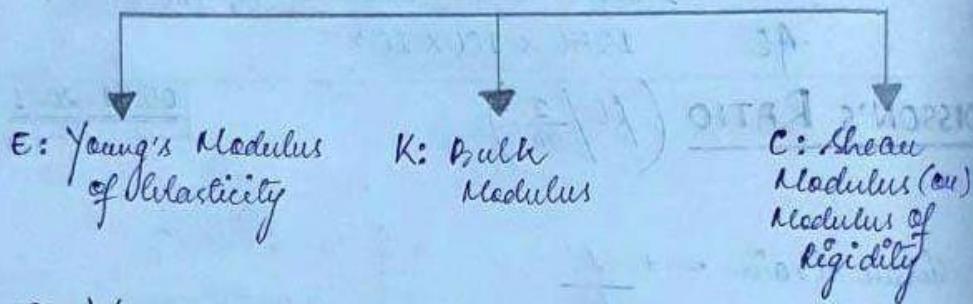
① Young's Modulus (E):

It is defined as "the ratio of tensile or compressive stress to the corresponding strain."

mathematically,

$$\boxed{E = \frac{\sigma}{e}}$$

(ii) Bulk Modulus (K):

when an uniform element is subjected to equal stress in 3 mutually perpendicular directions then "the ratio of direct stress to the volumetric strain is known as bulk modulus."

mathematically,

$$K = \frac{\sigma}{e_v}$$

$$\left[\because e_v = \frac{\Delta v}{v} \right]$$
volumetric strain

(iii) Shear Modulus (C or G):

It is defined as "the ratio of shear stress to the shear strain."

→ Higher the value of shear modulus, shows that the body is highly rigid.

mathematically,

$$C = \frac{\tau}{\epsilon}$$

where, τ → shear stress
ϵ → shear strain

13

15

* Relationship between Elastic Constants E, K, C:

① Relationship between E and K:

$$E = 3K \left(1 - \frac{2}{M} \right)$$

② Relationship between E and C:

$$E = 2C \left(1 + \frac{1}{m} \right)$$

where, $\frac{1}{m}$ is poisson's ratio

③ Relationship between E, K and C:

$$E = \frac{9KC}{3K + C}$$

* __Proof:__

$$E = \dfrac{9KC}{3K+C}$$

we know,

$$E = 3K\left(1 - \dfrac{2}{M}\right) \Rightarrow \boxed{1 - \dfrac{2}{M} = \dfrac{E}{3K}} \quad —①$$

$$E = 2C\left(1 + \dfrac{1}{m}\right) \Rightarrow \boxed{1 + \dfrac{1}{m} = \dfrac{E}{2C}} \quad —②$$

multiplying 2 in eqⁿ ②

$$2\left(1 + \dfrac{1}{M}\right) = 2\left(\dfrac{E}{2C}\right)$$

$$\Rightarrow \boxed{2 + \dfrac{2}{M} = \dfrac{E}{C}} \quad —③$$

now adding ① and ③

$$1 - \dfrac{2}{M} + 2 + \dfrac{2}{M} = \dfrac{E}{3K} + \dfrac{E}{C}$$

$$\Rightarrow 1 + 2 = \dfrac{E}{3K} + \dfrac{E}{C}$$

$$\Rightarrow 3 = \dfrac{EC + E3K}{3KC}$$

$$\Rightarrow 3(3KC) = EC + E3K$$

15

$\Rightarrow 9KC = E(C + 3K)$

$$\Rightarrow \boxed{E = \frac{9KC}{3K+C}} \quad \text{hence} \atop \text{proved}$$

Questions:

① The modulus of rigidity of a material is $0.8 \times 10^5 \frac{N}{mm^2}$. find the poisson's ratio if the modulus of elasticity of that material is $2.1 \times 10^5 \frac{N}{mm^2}$.

$\rightarrow$ given: $C = 0.8 \times 10^5 \frac{N}{mm^2}$

$\qquad E = 2.1 \times 10^5 \frac{N}{mm^2}$

$\qquad \mu$ ou $\frac{1}{M} = ?$

$\therefore E = \left\{ 2C \left(1 + \frac{1}{M}\right) \right.$

$\Rightarrow 2.1 \times 10^5 \frac{N}{mm^2} = 2 \times 0.8 \times 10^5 \frac{N}{mm^2} \left(1 + \frac{1}{M}\right)$

$\Rightarrow \dfrac{2.1 \times 10^5}{2 \times 0.8 \times 10^5} = 1 + \dfrac{1}{M}$

$\Rightarrow \dfrac{2.1 \times 10^5}{2 \times 0.8 \times 10^5} - 1 = \dfrac{1}{M}$

$\Rightarrow \dfrac{1}{M}$ ou $\mu = \dfrac{2.1 \times 10^5}{2 \times 0.8 \times 10^5} - 1.$

$\Rightarrow \boxed{\mu = 0.3125}$ Ans

16

② The modulus of rigidity of material is $0.8 \times 10^5 \frac{N}{mm^2}$. When a $6mm \times 6mm$ rod of this material was subjected to an axial pull of $3600 N$. It was found that the lateral dimension of the rod changed to $5.9991 mm \times 5.9991 mm$. Find the poisson's ratio and modulus of elasticity of that rod.

→ given: $C = 0.8 \times 10^5 \frac{N}{mm^2}$.

$A = 6 mm \times 6 mm$
$\quad = 36 mm^2$

$P = 3600 N$

$\sigma = \frac{P}{A} = \frac{3600}{36} = 100 \frac{N}{mm^2}$

longitudinal strain, $E = \frac{\sigma}{e} \; \rightarrow) \; \boxed{e = \frac{\sigma}{E}}$

lateral strain $= \dfrac{\text{change in lateral dimension}}{\text{original dimension}}$

$\quad = \dfrac{6 - 5.9991}{6} = 0.00015$

17

Poisson's ratio:

$$\frac{1}{M} = \frac{lateral\ strain}{longitudinal\ strain}$$

$$\Rightarrow \frac{1}{M} = \frac{0.00015}{\frac{\sigma}{E}}$$

$$\Rightarrow \frac{1}{M} = \frac{0.00015}{100} \times E$$

$$\Rightarrow \frac{1}{EM} = \frac{0.00015}{100}$$

$$\Rightarrow ME = \frac{100}{0.00015}$$

$$\Rightarrow \boxed{ME = \frac{2 \times 10^6}{3}} \quad \text{①}$$

we know, $E = 2C\left(1 + \frac{1}{M}\right)$

$$\Rightarrow ME = 2C \cdot \frac{(M+1)}{M}$$

$$\Rightarrow ME = 2C \cdot (M+1)$$

$$\Rightarrow \boxed{ME = 2 \times 0.8 \times 10^5 (M+1)} \quad \text{②}$$

equating eqn ① and ②

$$\Rightarrow 2 \times 0.8 \times 10^5 (M+1) = \frac{2 \times 10^6}{3}$$

18

20

$) M + 1 = \dfrac{2 \times 10^6}{3(2 \times 0.8 \times 10^5)}$

$\Rightarrow) M = \dfrac{2 \times 10^6}{3(2 \times 0.8 \times 10^5)} - 1$

$\Rightarrow) \boxed{M = 3.167}$

$\because \quad \mu = \dfrac{1}{M} = \dfrac{1}{3.167}$

$\Rightarrow) \boxed{\mu = 0.315}$

$\therefore \quad E = 2C\left(1 + \dfrac{1}{M}\right)$

$\Rightarrow) E = 2 \times 0.8 \times 10^5 (1 + 0.315)$

$\Rightarrow) E = 210400$

$\Rightarrow) \boxed{E = 2.1 \times 10^5 \dfrac{N}{mm^2}}$

* <u>Axial deformation of bar due to its self-weight:</u>

Considering a prismatic bar
of uniform cross-sectional of
length 'ℓ' suspended freely.

weight of the bar = ω
unit weight of the bar = Y

$\left[\begin{array}{l} Y = \dfrac{\omega}{volume} \\[2mm] \Rightarrow) Y = \dfrac{\omega}{Ax} \end{array}\right]$ $\begin{array}{l} A \to area \\ x \to length \\ Ax \to volume \end{array}$

19
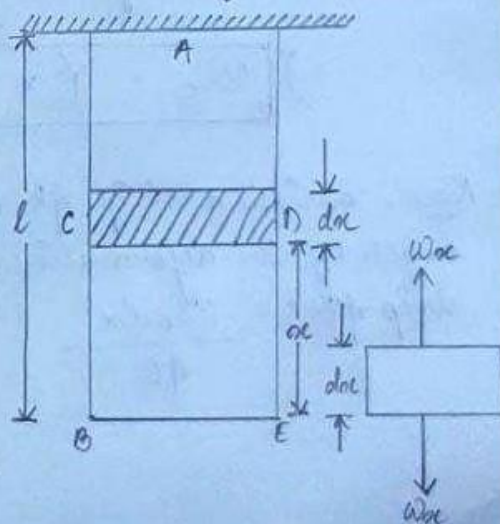
Assuming cross-sectional area = 'A'
Consider a small element of length 'dx' at a distance 'x' from the free end.

The element is subjected to weight of bar below element (i.e. BCDE)

weight of bar below element dx
$$\Rightarrow W_x = Y(Ax)$$

Elongation of element al length dx
$$\Rightarrow \boxed{d\Delta = \frac{W_x \, dx}{AE}}$$

let E, ρ, Y, be the Young's Modulus, density and unit weight of the body respectively.

we know, (volume) $V_x = A \times x$

$$Y = \frac{W_x}{V_x}$$

$$\Rightarrow \boxed{W_x = Y \times V_x}$$
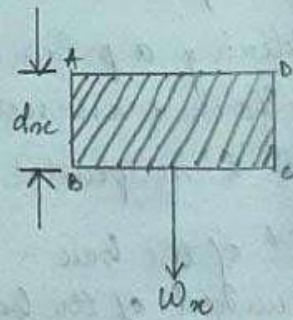
Now, consider a strip ABCD of length by the deformation for the strip ABCD = $\frac{W_x \, dx}{AE}$

Total deformation of the bar $= \int_0^l \dfrac{\omega x \times dx}{AE}$

$$= \int_0^l \dfrac{Y \times V_x}{AE}\, dx$$

$$= \int_0^l \dfrac{Y \times A \times x}{A E}\, dx$$

$$= \int_0^l \dfrac{Y\, x}{E}\, dx$$

$$= \dfrac{Y}{E} \int_0^l x\, dx \qquad \left[\because Y \text{ and } E \text{ are constants} \right]$$

$$= \dfrac{Y}{E} \left(\dfrac{x^2}{2} \right)_0^l$$

$$= \dfrac{Y}{E} \left[\dfrac{l^2}{2} - \dfrac{0^2}{2} \right]$$

$$= \dfrac{Y}{E} \times \dfrac{l^2}{2}$$

21

$$\Rightarrow \boxed{\Delta = \frac{\gamma l^2}{2E}}$$

$\Delta l \rightarrow$ total deformation of the bar.

✳ NOTES :

① Ultimate stress = $\dfrac{\text{maximum load}}{\text{Area}}$

② Yield stress = $\dfrac{\text{yield point load}}{\text{Area}}$

③ Safe stress = $\dfrac{\text{yield stress}}{\text{load factor}}$

<u>Questions :</u>

① A following data refers to a mild steel specimen tested in a laboratory :

ⓔ diameter of specimen = 25 mm

ⓘⓘ length of specimen = 300 mm

ⓘⓘⓘ extension under a load 50 kN = 0.045 mm

ⓘⓥ load at yield point = 127.65 kN

ⓥ maximum load = 208.60 kN

✗ⓥⓘ length of the specimen after failure = 375 mm

✗ⓥⓘⓘ neck diameter = 17.75 mm

Now, determine : ⓘ Young's Modulus

ⓘⓘ Yield point stress

ⓘⓘⓘ Ultimate stress

ⓘⓥ Safe stress adopting a factor of safety of 2.

22

24

→ given: $d = 25\ mm$

$l = 300\ mm$

$P = 50\ kN$

$= 50 \times 10^3\ N$

$\Delta l = 0.045\ mm$

yield point load $= 127.65\ kN$

$= 127.65 \times 10^3\ N$

maximum load $= 208.60\ kN$

$= 208.60 \times 10^3\ N$

load factor $= 2$.

(i) $\Delta l = \dfrac{P l}{A E}$

$\Rightarrow E = \dfrac{P l}{A\ \Delta l}$

$\Rightarrow E =$

(ii) Yield point stress $= \dfrac{\text{yield point load}}{\text{area}}$

$$= \dfrac{127.65 \times 10^3 \, N}{\dfrac{\pi}{4} (25)^2 \, mm^2}$$

$$= 260.05 \, \dfrac{N}{mm^2}$$

(iii) Ultimate stress $= \dfrac{\text{maximum load}}{\text{area}}$

$$= \dfrac{208.60 \times 10^3 \, N}{\dfrac{\pi}{4} (25)^2 \, mm^2}$$

$$= 424.96 \, \dfrac{N}{mm^2}$$

(iv) Safe stress $= \dfrac{\text{yield stress}}{\text{load factor}}$

$$= \dfrac{260.05}{2} \, \dfrac{N}{mm^2}$$

$$= 130.025 \, \dfrac{N}{mm^2}$$

24

Mechanical Properties Of Materials

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MECHANICAL PROPERTIES OF MATERIALS :

① **Rigidity:**

ⓘ This is defined as the property possessed by a solid body to change its shape.

ⓘⓘ It means when an external force is applied to the solid material, there wont be any change in its shape due to intermolecular attraction by the closely packed particles.

ⓘⓘⓘ This is the property of the material to resistance the bending.

② **Elasticity:**

ⓘ This is the property of a body by virtue of which in return its original shape after removal of external force causing deformation which is applied on it.

ⓘⓘ Elastic property entirely depends upon the type of material and not on the shape and size.

③ **Plasticity:**

ⓘ This is the ability of a solid material to undergo permanent deformation where force is applied to it.

ⓘⓘ The ability of a material to retain the changed shape under application of load is known as plasticity.

ⓘⓘⓘ Plastic deformation is the property of ductile and malleable solids.

25

(4) **Compressibility:**

(i) This is the property of material by virtue of which it tends to flatten and reduce in size under pressure.

(ii) This nature or property of material changes the inter-molecular structure of the material.

(5) **Hardness:**

(i) The property of material by virtue of which it resists the local surface deformation when undergoes abrasion, drilling, impact, etc.

(ii) It is the state of material, being hard for which it can withstand friction.

(6) **Toughness:**

(i) The amount of energy per unit volume that a material can absorb before rupture is called <u>toughness</u>.

(ii) It can be defined as the ability of a material to resist breaking when force is applied to it.

(iii) This property allows the material to deform before rupture or fracture.

(7) **Stiffness:**

(i) The property of material which resists deformation when a force is applied to it. This is the rigidity of a material.

(ii) The material having more flexibility has less stiffness.

(iii) A stiff material has high long modulus.

28

⑧ Brittleness:

ⓘ The property of a material by which it fractures when subjected to stress without deformation.

ⓘ Brittle material has a little tendency to deform before rupture.

ⓘ It has small plastic region.

ex: Bone, concrete, ceramic, cast Iron, glass products, etc.

⑨ Ductility:

ⓘ It is defined as the ability of a material to undergo permanent deformation through elongation and reduction in cross-sectional area or bending at room temperature without fracturing.

ⓘ This or is an ability to undergo last permanent deformation in tension.

  Example of ductile material → copper, aluminium, steel.
★ Opposite to ductile is brittle.

⑩ Malleability:

ⓘ This is the property of material by which it can be beaten to form its thin shots.

ex: lead, tin, gold, silver, aluminium, copper, Iron.

⑪ Creep:

ⓘ This is the permanent change in shape and size of a material which increases as a function of time under application of load and elevated temperature.

ⓘ Creep is time independent.

ⓘ Creep begins at different temperature for different material.

27

(12) <u>Fatigue</u>:

(i) This is the deterioration of the material subjected to repeated cycle of stress and strain resulting in progressive cracking, eventually producing fracture.

(ii) This is the weakening of a material caused by cyclic loading that results in progressive damage and the growth of cracks.

(iii) This is responsible for 90% of mechanical failure.

(13) <u>Durability</u>:

(i) The ability of a material to remain serviceable during the useful time without damaging the material.

(ii) It represents how long the material works.

(14) <u>Tenacity</u>:

→ The property of material to resist the breaking or is known as <u>tenacity</u>.

★ <u>NOTES</u>:

(1) <u>Percentage of Elongation</u>:

(a) It is a measure of ductility.

(b) This can be obtained as $= \dfrac{\text{final length} - \text{initial length}}{\text{initial length}} \times 100$

$$\text{or} \quad \boxed{\dfrac{\Delta l}{l} \times 100}$$

where, Δl = change in length.

28

31

② Percentage of reduction in area:

ⓐ This is the measure of the specimen that how much that specimen narrowed when it undergoes some load application.

ⓑ It is obtained as follows:

$$\frac{\text{final area} - \text{initial area}}{\text{initial area}} \times 100$$

* Signification of Percentage Elongation and Percentage Reduction in area:

ⓘ Percentage elongation is a property of material which prod provides a value of its ductility.

ⓘⓘ The greater deformation before breaking shows the material is more ductile.

ⓘⓘⓘ Percentage elongation is a measure of ductility.

ⓘⓥ Percentage reduction area is also a measure of ductility.

* Stress – Strain diagram for mild steel: (Imp)
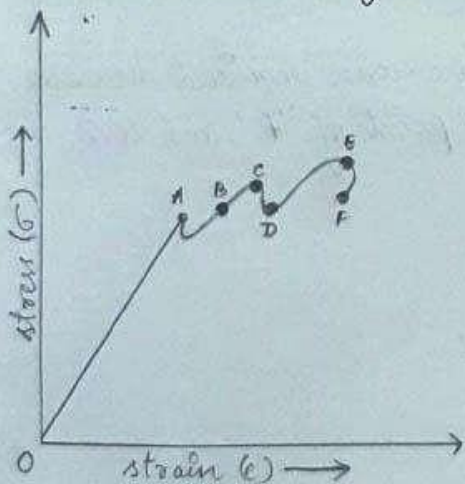
→ This curve is obtained when a mild steel specimen undergoes a tensile test. The plot from O to A is a straight line, this portion obeys Hooke's law and the straight line is called the limit of proportionality.

→ In this range of extension, the stress is proportional to strain i.e. $\boxed{\sigma \propto e}$

29

→ If the specimen is extended beyond the limit of proportionality ny upto the point 'B', the material still remains elastic.

→ But from A to B, stress–strain relation is not linear.

→ The stress at 'B' is called elastic limit.

→ Again if the specimen is extended beyond the elastic limit, plastic deformation occurs.

→ In the range of 'B' to 'C' strain increases without increase in stress.

→ At point 'C' the material goes extended with decrease in load and the stress at point C is called upper-yield point.

→ At point 'D' the material again offers resistance to greater extension and the stress corresponding to this point is called lower-yield point.

→ As the load is increased, the extension increases and the point 'E' indicates the necking of the specimen and the stress corresponding to this point is called ultimate tensile stress.

→ As the extension is increased, the load required decreases and the specimen breaks at the point of 'F' and this point is called stress failure.

30

* <u>Elongation in bar of (varying) section:</u>

change in cross section

E is same, material to material
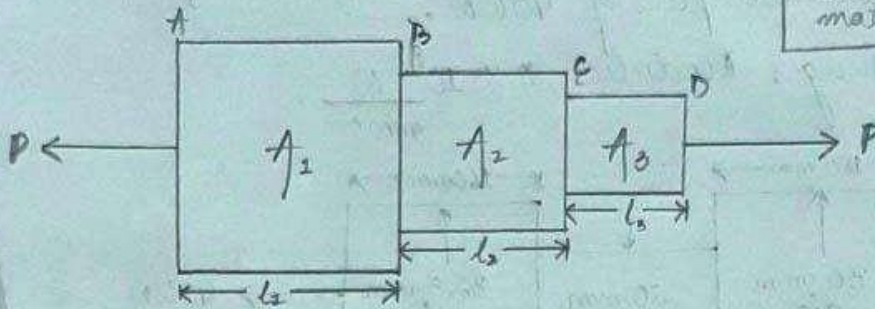
The elongation for the portion AB : $\boxed{\Delta l_1 = \dfrac{P\,l_1}{A_1\,E}}$

The elongation for the portion BC : $\boxed{\Delta l_2 = \dfrac{P\,l_2}{A_2\,E}}$

The elongation for the portion CD : $\boxed{\Delta l_3 = \dfrac{P\,l_3}{A_3\,E}}$

So total elongation : $\boxed{\Delta l = \Delta l_1 + \Delta l_2 + \Delta l_3}$

$= \dfrac{P\,l_1}{A_1 E} + \dfrac{P\,l_2}{A_2 E} + \dfrac{P\,l_3}{A_3 E}$

$= \dfrac{P}{E}\left(\dfrac{l_1}{A_1} + \dfrac{l_2}{A_2} + \dfrac{l_3}{A_3}\right)$

31

① A bar consisting of 3 lengths. Find the stresses in the three parts and the total elongation of the bar for an axial pull of 40 kN.
Take Young's Modulus 2 × 10⁵ $\frac{N}{mm^2}$.
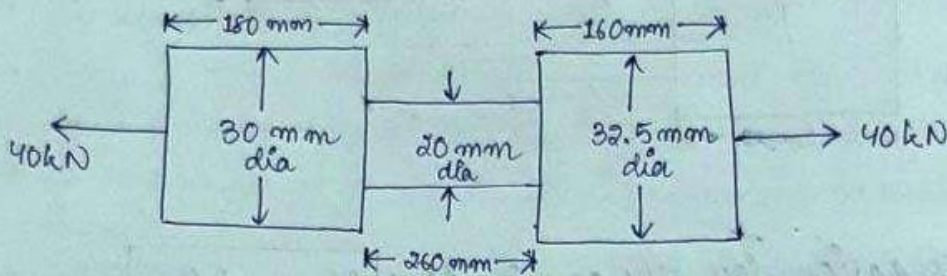
→ given: P = 40 kN = 40 × 10³ N
           E = 2 × 10⁵ $\frac{N}{mm^2}$.

$A_1 = \frac{\pi}{4}(30)^2 = 706.85\ mm^2$.

$A_2 = \frac{\pi}{4}(20)^2 = 314.15\ mm^2$.

$A_3 = \frac{\pi}{4}(32.5)^2 = 829.579\ mm^2$.

$\Delta l_1 = \dfrac{40 \times 10^3 \times 180}{706.85 \times 2 \times 10^5} = 0.50\ mm$

$\Delta l_2 = \dfrac{40 \times 10^3 \times 260}{314.15 \times 2 \times 10^5} = 0.165\ mm$

$\Delta l_3 = \dfrac{40 \times 10^3 \times 160}{829.579 \times 2 \times 10^5} = 0.038\ mm$

32

③ A steel rod ABCD of 1000 mm² cross sectional area and 8m long is subjected to forces as shown in the figure bto below. Find the total deformation if young's modulus of the bar is 200 GPa. All forces are are in KN.
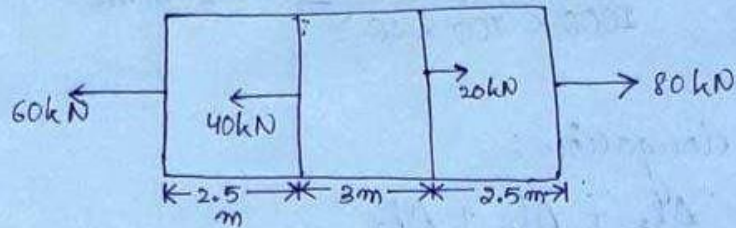
FBD:
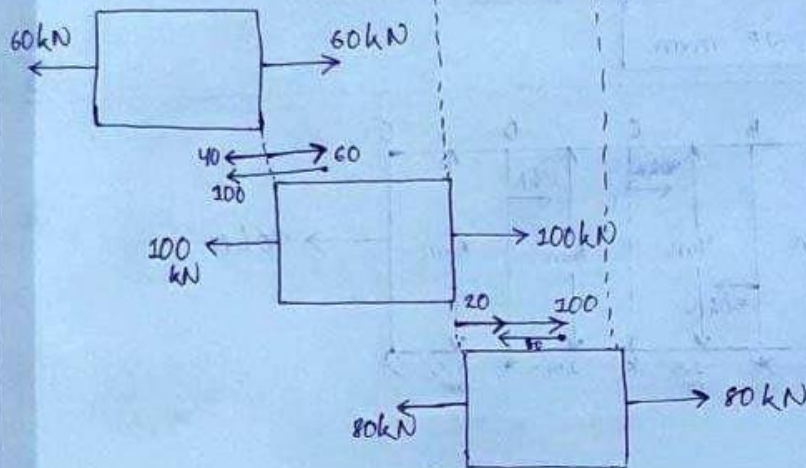
$P_1 = 60kN = 60 \times 10^3 N$

$P_2 = 100kN = 100 \times 10^3 N$

$P_3 = 80kN = 80 \times 10^3 N$

$l_1 = 2.5m = 2.5 \times 10^3 mm$

$l_2 = 3m = 3 \times 10^3 mm$

$l_3 = 2.5m = 2.5 \times 10^3 mm$

$E = 200 \, GPa$

$= 200 \times 10^9 \dfrac{N}{m^2}$

$= 200 \times 10^9 \dfrac{N}{10^6 \, mm^2}$

$= 200 \times 10^{9-6} \dfrac{N}{mm^2}$

$= 200 \times 10^3 \dfrac{N}{mm^2}$

$A = 1000 \, mm^2$

$$\Delta l_1 = \frac{P_1 l_1}{AE} = \frac{60 \times 10^3 \times 2.5 \times 10}{1000 \times 200 \times 10^3} = 0.75 \text{ mm}$$

$$\Delta l_2 = \frac{P_2 l_2}{AE} = \frac{100 \times 10^3 \times 3 \times 10^3}{1000 \times 200 \times 10^3} = 1.5 \text{ mm}$$

$$\Delta l_3 = \frac{P_3 l_3}{AE} = \frac{80 \times 10^3 \times 2.5 \times 10^3}{1000 \times 200 \times 10^3} = 1 \text{ mm}$$

$\therefore$ Total elongation:

$$\Delta l = \Delta l_1 + \Delta l_2 + \Delta l_3$$

$\Rightarrow$ $\Delta l = 0.75 \text{mm} + 1.5 \text{mm} + 1 \text{mm}$

$\Rightarrow$ $\boxed{\Delta l = 3.25 \text{ mm}}$
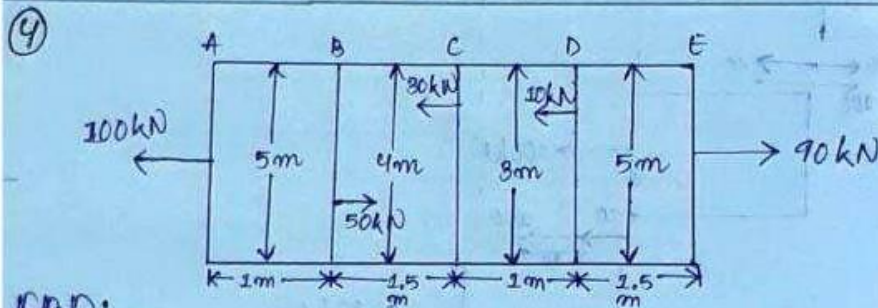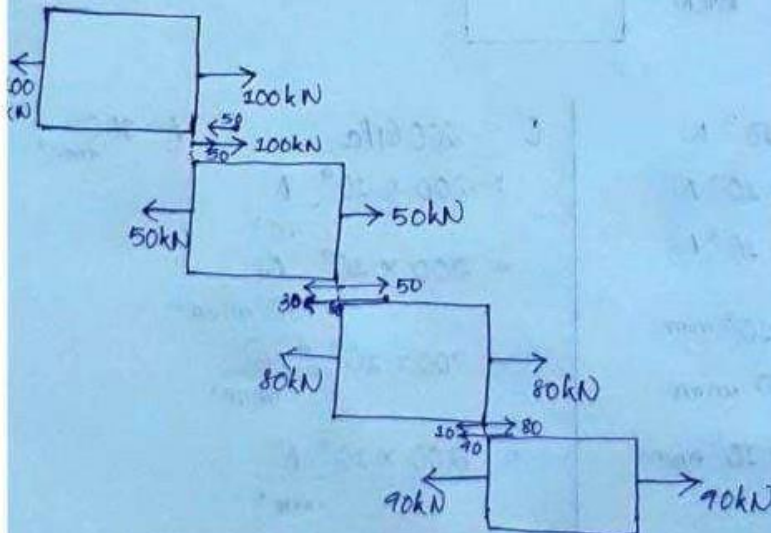
(4)

FBD:

34

$P_1 = 100\,kN = 100 \times 10^3\,N$ $l_1 = 1\,m$ $d_1 = 5\,m$

$P_2 = 50\,kN = 50 \times 10^3\,N$ $l_2 = 1.5\,m$ $d_2 = 4\,m$

$P_3 = 80\,kN = 80 \times 10^3\,N$ $l_3 = 1\,m$ $d_3 = 3\,m$

$P_4 = 90\,kN = 90 \times 10^3\,N$ $l_4 = 1.5\,m$ $d_4 = 5\,m$

$A_1 = \dfrac{\pi}{4}(5)^2 = 19.63\,m^2$ $E = 2.5\,MPa$

$A_2 = \dfrac{\pi}{4}(4)^2 = 12.56\,m^2$ $= 2.5 \times 10^6\,\dfrac{N}{m^2}$

$A_3 = \dfrac{\pi}{4}(3)^2 = 7.06\,m^2$

$A_4 = \dfrac{\pi}{4}(5)^2 = 19.63\,m^2$

$\Delta l_1 = \dfrac{P_1 l_1}{A_1 E} = \dfrac{100 \times 10^3 \times 1}{19.63 \times 2.5 \times 10^6} = 2.037 \times 10^{-3}\,m$

$\Delta l_2 = \dfrac{P_2 l_2}{A_2 E} = \dfrac{50 \times 10^3 \times 1.5}{12.56 \times 2.5 \times 10^6} = 2.38 \times 10^{-3}\,m$

$\Delta l_3 = \dfrac{P_3 l_3}{A_3 E} = \dfrac{80 \times 10^3 \times 1}{7.06 \times 2.5 \times 10^6} = 4.53 \times 10^{-3}\,m$

$\Delta l_4 = \dfrac{P_4 l_4}{A_4 E} = \dfrac{90 \times 10^3 \times 1.5}{19.63 \times 2.5 \times 10^6} = 2.75 \times 10^{-3}\,m$

$\therefore$ Total elongation:

$\Delta l = \Delta l_1 + \Delta l_2 + \Delta l_3 + \Delta l_4$

$\Rightarrow \Delta l = (2.037 \times 10^{-3}) + (2.38 \times 10^{-3}) + (4.53 \times 10^{-3}) + (2.75 \times 10^{-3})\,m$

$\Rightarrow \Delta l = 0.011697\,m$

$\Rightarrow \boxed{\Delta l = 11.697\,mm}$

35

⑤ An aluminium bar is rigidly attached between steel bar and a bronze bar shown in figure. Axial forces are applied at the position indicated. Determine the maximum value of P that will not exceed stress of 160 MPa in steel; 100 MPa in aluminium and 120 MPa in bronze.

$A_1 = 600 \text{ mm}^2$

① → Steel
$\sigma_1 = 160 \text{ MPa}$

4P

$A_2 = 400 \text{ mm}^2$

② → Aluminium
$\sigma_2 = 100 \text{ MPa}$

$A_3 = 200 \text{ mm}^2$

P

③ → Bronze
$\sigma_3 = 120 \text{ MPa}$

2P

FBD:

5P

①

5P

4P
5P

P

②

P

P

P
2P

2P

③

2P

36
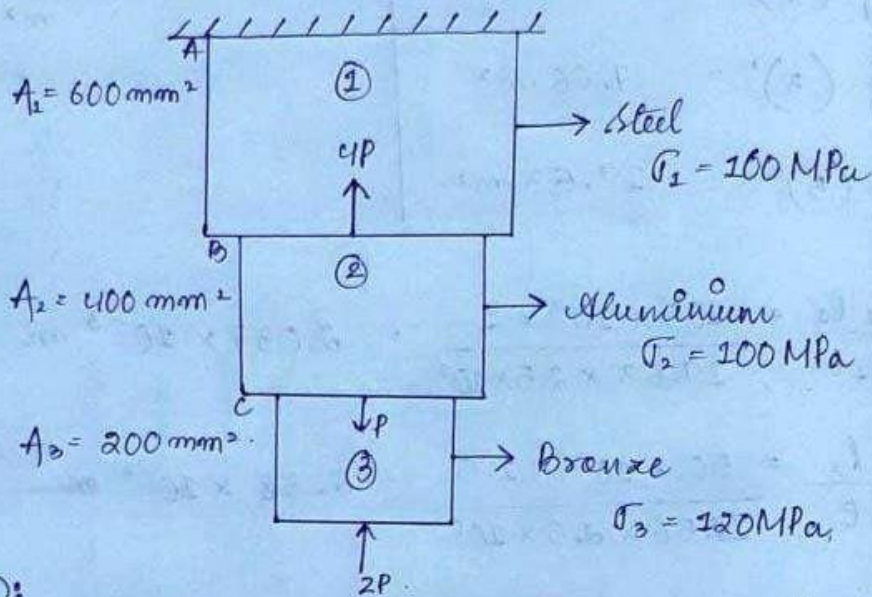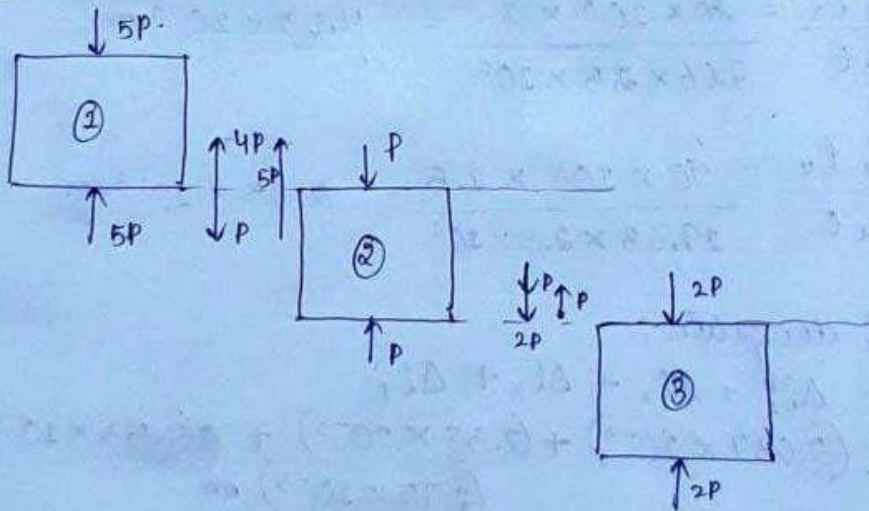
$\sigma_1 = 160\ MPa = 160 \times 10^6\ \dfrac{N}{m^2} = 160 \times 10^6\ \dfrac{N}{10^6\ mm^2}$

$$\Rightarrow \boxed{\sigma_1 = 160\ \dfrac{N}{mm^2}}$$

$\sigma_2 = 100\ MPa = 100 \times 10^6\ \dfrac{N}{m^2} = 100 \times 10^6\ \dfrac{N}{10^6\ mm^2}$

$$\Rightarrow \boxed{\sigma_2 = 100\ \dfrac{N}{mm^2}}$$

$\sigma_3 = 120\ MPa = 120 \times 10^6\ \dfrac{N}{m^2} = 120 \times 10^6\ \dfrac{N}{10^6\ mm^2}$

$$\Rightarrow \boxed{\sigma_3 = 120\ \dfrac{N}{mm^2}}$$

$A_1 = 600\ mm^2$ $P_1 = 5P$

$A_2 = 400\ mm^2$ $P_2 = P$

$A_3 = 200\ mm^2$ $P_3 = 2P$

$\therefore$ we know that :

for section ① :

$$\sigma_1 = \dfrac{P_1}{A_1}$$

$$\Rightarrow 160 = \dfrac{5P}{600} \qquad \Rightarrow P_1 = \dfrac{160 \times 600}{5}$$

$$\Rightarrow \boxed{P_1 = 19200\ N}$$

37

for section ② :

$$\sigma_2 = \frac{P_2}{A_2}$$

$\Rightarrow 200 = \dfrac{P}{400} \Rightarrow P_2 = 100 \times 400$

$\Rightarrow \boxed{P_2 = 40000 \ N}$

for section ③ :

$$\sigma_3 = \frac{P_3}{A_3}$$

$\Rightarrow 120 = \dfrac{2P}{200} \Rightarrow P_3 = \dfrac{120 \times 200}{2}$

$\Rightarrow \boxed{P_3 = 12000 \ N}$

38

① Volumetric strain of a Rectangular Section Under the action of a stress along the longitudinal strain:
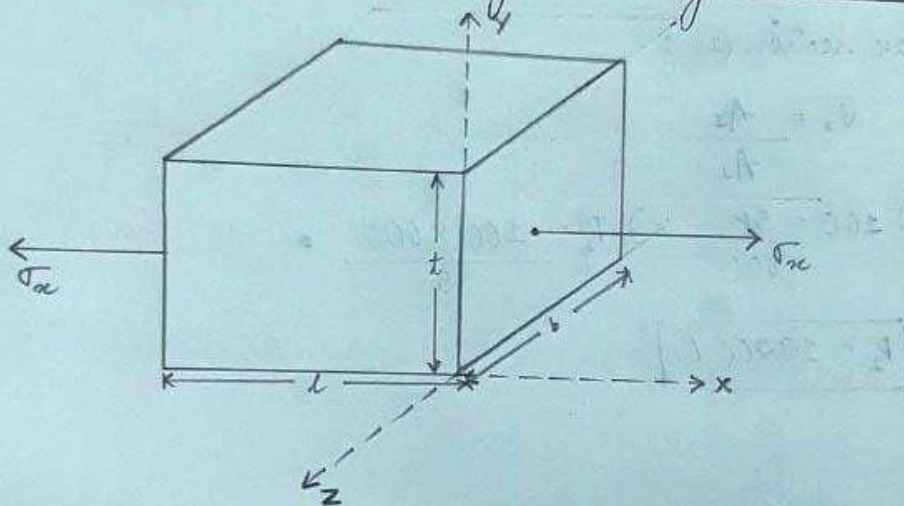
Let l, b, t are the length, breadth and thickness of the rectangular block.

σ_x = tensile stress in x - direction

E = Young's Modulus

μ = Poisson's ratio

$= \dfrac{- \text{lateral strain}}{\text{longitudinal strain}}$

longitudinal strain : $\boxed{e_x = \dfrac{\sigma_x}{E}}$

$$\mu = - \dfrac{e_y}{e_x}$$

$\Rightarrow e_y = - \mu \cdot \boxed{e_x}$

$\Rightarrow \boxed{e_y = - \mu \dfrac{\sigma_x}{E}}$ $\left[\because e_x = \dfrac{\sigma_x}{E} \right]$

Similarly, $\boxed{e_z = - \mu \dfrac{\sigma_x}{E}}$

But we know that,

$\boxed{e_x = \dfrac{\delta l}{l}}$, $\boxed{e_y = \dfrac{\delta b}{b}}$, $\boxed{e_z = \dfrac{\delta t}{t}}$

40

43

we know that,

volume of a rectangular block is given by : (3D)

$$V = l \times b \times t$$

Now, derivating both the sides :

$$\frac{dv}{v} = \frac{dl}{l} + \frac{db}{b} + \frac{dt}{t}$$

$$\left[\because \frac{d}{l}(l \times b \times t) + \frac{d}{b}(l \times b \times t) + \frac{d}{t}(l \times b \times t) \right.$$

$$\left. = \frac{dl}{l} + \frac{db}{b} + \frac{dt}{t} \right]$$

$$\Rightarrow \boxed{e_v = e_x + e_y + e_z}$$

$$\left[\because e_v = \frac{dv}{v} = \text{volumetric strain} \right.$$

$$e_x = \frac{dl}{l} \qquad e_z = \frac{dt}{t}$$

$$\left. e_y = \frac{db}{b} \right]$$

41

44

Now put the value of e_x, e_y and e_z

$$e_v = \frac{\sigma_x}{E} - \mu\frac{\sigma_x}{E} - \mu\frac{\sigma_x}{E}$$

$$\Rightarrow e_v = \frac{\sigma_x}{E} - 2\mu\frac{\sigma_x}{E}$$

$$\Rightarrow \boxed{e_v = \frac{\sigma_x}{E}\left(1 - 2\mu\right)} \quad {}^{***}_{**}$$

② Volumetric strain of a rectangular block subjected to three mutually perpendicular stress:
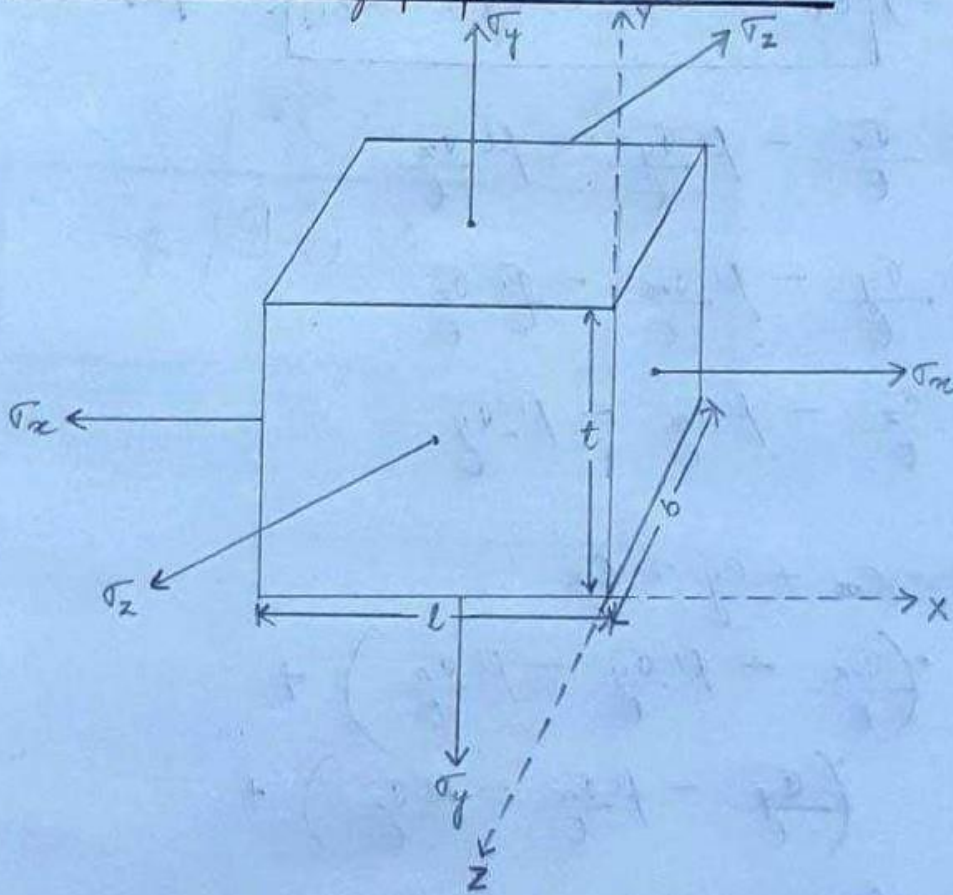
Let l, b, t are the length, breadth and thickness of the rectangular block.

σ_x, σ_y, σ_z are the three stresses act in the direction of x, y, z.

We know that,

$$V = l \times b \times t$$

and $\dfrac{dv}{v} = \dfrac{dl}{l} + \dfrac{db}{b} + \dfrac{dt}{t}$

$$\Rightarrow \boxed{e_v = e_x + e_y + e_z}$$

- $e_x = \dfrac{\sigma_x}{E} - \mu \dfrac{\sigma_y}{E} - \mu \dfrac{\sigma_z}{E}$

- $e_y = \dfrac{\sigma_y}{E} - \mu \dfrac{\sigma_x}{E} - \mu \dfrac{\sigma_z}{E}$

- $e_z = \dfrac{\sigma_z}{E} - \mu \dfrac{\sigma_x}{E} - \mu \dfrac{\sigma_y}{E}$

$\therefore e_v = e_x + e_y + e_z$

$$= \left(\dfrac{\sigma_x}{E} - \mu \dfrac{\sigma_y}{E} - \mu \dfrac{\sigma_z}{E} \right) +$$

$$\left(\dfrac{\sigma_y}{E} - \mu \dfrac{\sigma_x}{E} - \mu \dfrac{\sigma_z}{E} \right) +$$

$$\left(\dfrac{\sigma_z}{E} - \mu \dfrac{\sigma_x}{E} - \mu \dfrac{\sigma_y}{E} \right)$$

43

$$\Rightarrow e_v = \frac{\sigma_x}{E} - \mu\frac{\sigma_y}{E} - \mu\frac{\sigma_z}{E} + \frac{\sigma_y}{E} - \mu\frac{\sigma_x}{E} - \mu\frac{\sigma_z}{E} +$$

$$\frac{\sigma_z}{E} - \mu\frac{\sigma_x}{E} - \mu\frac{\sigma_y}{E}$$

$$\Rightarrow e_v = \frac{1}{E}\left[\sigma_x - \mu\sigma_y - \mu\sigma_z + \sigma_y - \mu\sigma_x - \mu\sigma_z - \sigma_z - \mu\sigma_x - \mu\sigma_y\right]$$

$$\Rightarrow e_v = \frac{1}{E}\left[\sigma_x - \mu\sigma_x - \mu\sigma_x + \sigma_y - \mu\sigma_y - \mu\sigma_y + \sigma_z - \mu\sigma_z - \mu\sigma_z\right]$$

$$\Rightarrow e_v = \frac{1}{E}\left[\sigma_x - 2\mu\sigma_x + \sigma_y - 2\mu\sigma_y + \sigma_z - 2\mu\sigma_z\right]$$

$$\Rightarrow e_v = \frac{1}{E}\left[\sigma_x(1-2\mu) + \sigma_y(1-2\mu) + \sigma_z(1-2\mu)\right]$$

$$\Rightarrow \boxed{e_v = \frac{1}{E}\left[(1-2\mu)(\sigma_x + \sigma_y + \sigma_z)\right]}$$

③ <u>Volumetric strain for a Circular Rod:</u>
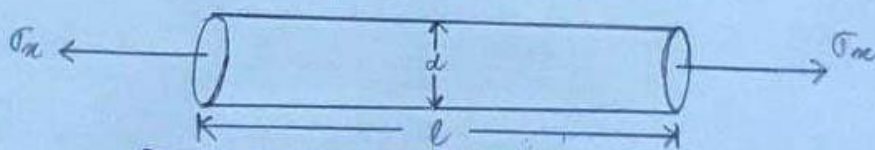
$$\boxed{e_v = 2e_d + e_l}$$

where, e_v = volumetric strain

e_d = lateral strain

e_l = longitudinal strain

44

Complex Stress and Strain .

④ Volumetric strain for a sphere:

$$\ell_v = 3\,\ell_d$$
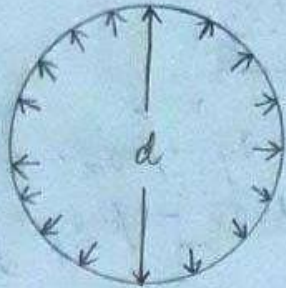
CHAPTER - 3

COMPLEX STRAIN AND STRESS :
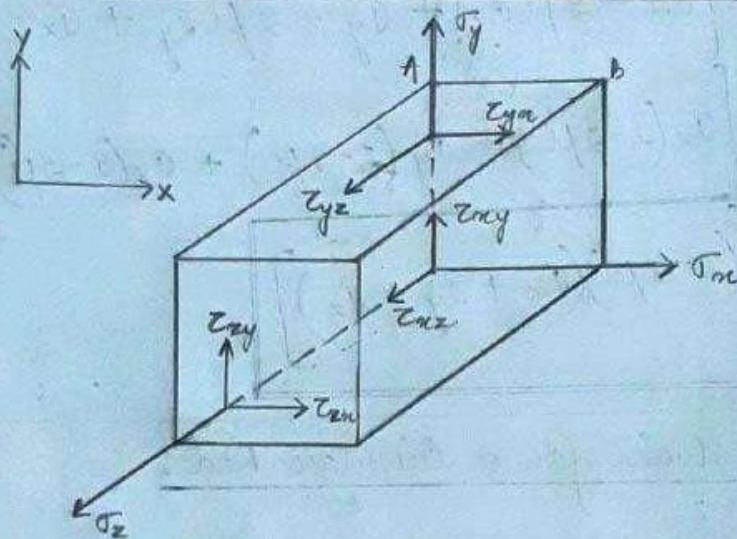
3-D representation of stresses

(or)

3-dimensional stress system

45
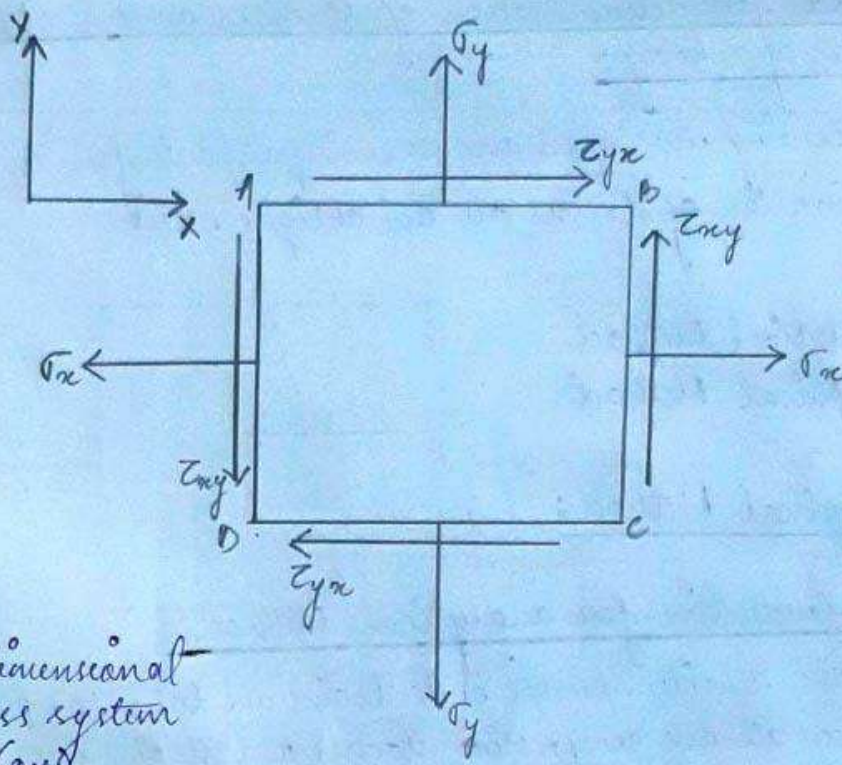
2-dimensional
stress system
(and
Plane stress condition

* Principle Plane :

If has been observed that any point in a strength material,
there are 3 planes mutually perpendicular to each other.
The plane which carry direct stresses only and half an
no shear stresses are known as _principle plane_.

* Principle Stress :

The magnitude of direct stress, across a principle plane is
known as _principle stress_.

46

* Methods for determination of stresses on an oblique section of a body:

The following two methods are important for the determination of stresses on an oblique section i.e.

① Analytical Method
② Graphical Method.

① Analytical Method:

* Sign Convention for analytical method:

① All the tensile stresses and strains are taken as positive, whereas all the compressive stresses and strains are taken as negative.

② The shear stress which tends to rotate the element in the clockwise direction is taken as positive, whereas that which tends to rotate in an anticlockwise direction is taken as negative.

Here σ_x is compressive stress

In the element the shear stress in the vertical face is taken as positive, whereas the shear stress on the horizontal face is taken as negative.

Here σ_y is tensile stress

47
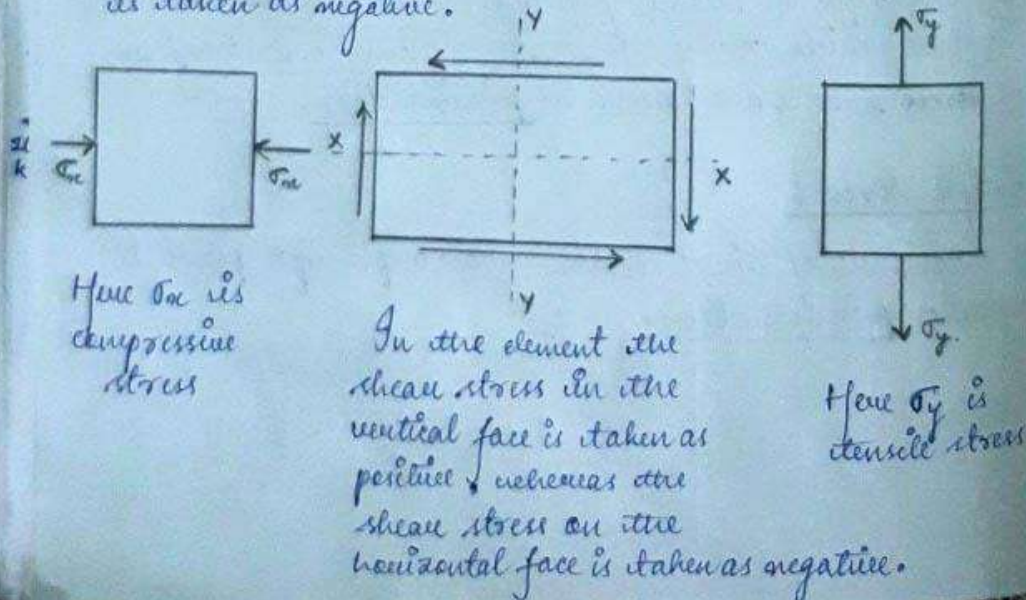
① CASE-1

Stresses on an oblique section of a body subjected to a direct stress in one plane.
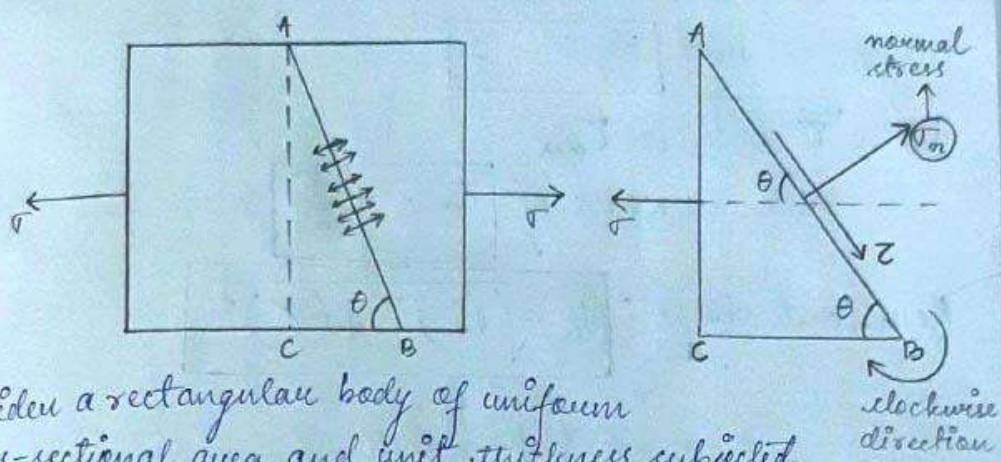
Consider a rectangular body of uniform cross-sectional area and unit thickness subjected to a direct tensile stress along X-X axis.

Now consider an oblique section AB inclined with the X-X axis.

Let σ = tensile stress across the face AC

θ = angle, which the oblique section AB makes with BC i.e. with the X-X axis in clockwise direction.

The normal stress across section AB is given by:

$$\sigma_n = \frac{\sigma}{2} - \frac{\sigma}{2}\cos 2\theta$$

Shear stress across the section AB,

$$\tau = \frac{\sigma}{2} \sin 2\theta$$

when $\theta = 45°$, then $2\theta = 90°$

and $\boxed{\sin \theta = 1}$

then `τ' becomes τ_{max}

and the value of $\boxed{\tau_{max} = \dfrac{\sigma}{2}}$

Resultant stress : $\boxed{\sigma_R = \sqrt{(\sigma_m)^2 + (\tau)^2}}$

↳ resultant of normal stress
and shear stress

Questions:

① A wooden bar is subjected to tensile stress of 5MPa. What will be the values of normal and shear stresses across a section, which makes an angle of 25° with the direction of the tensile stresses?

→ given: $\sigma = 5MPa$

$\sigma_m = \dfrac{\sigma}{2} - \dfrac{\sigma}{2} \cos 2\theta$

$\Rightarrow \sigma_m = \dfrac{5}{2} - \dfrac{5}{2} \cos(2 \times 25)$

$\Rightarrow \sigma_m = 0.89 \dfrac{N}{mm^2}$

$\Rightarrow \boxed{\sigma_m = 0.89 MPa}$

$\tau = \dfrac{\sigma}{2} \sin 2\theta$

$\Rightarrow \tau = \dfrac{5}{2} \sin(2 \times 25)$

$\Rightarrow \tau = 1.915 \dfrac{N}{mm^2}$

$\Rightarrow \boxed{\tau = 1.915 MPa}$

49

53

② Two wooden pieces 100mm × 100mm in cross-section are joined together along a line AB. Find the maximum force P, which can be applied if the shear stresses along the joint AB is 1.3 MPa.

→ given : $\tau = 1.3$ MPa

$P = ?$

$\theta = 60°$
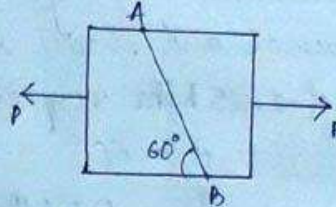
$\tau = \dfrac{\sigma}{2} \sin(2 \times 60°)$

=) $1.3 = \dfrac{\sigma}{2} \sin(2 \times 60°)$

=) $\sigma = \dfrac{1.3}{\sin(2 \times 60°)} \times 2$. =) $\sigma = 3.002 \dfrac{N}{mm^2}$.

=) $\boxed{\sigma = 3.002 \text{ MPa}}$

$\sigma = \dfrac{P}{A}$

=) $3.002 = \dfrac{P}{100 \times 100}$

=) $P = 3.02 \times 100 \times 100$

=) $\boxed{P = 30020 N}$

=) $P = \dfrac{30020}{10^3}$ kN =) $\boxed{P = 30.02 \text{ kN}}$

50

③ A tension member is formed by connecting two
wooden members 200mm × 100mm as shown in
the figure:

minimum value.

Determine the (safe value) of the force P if permeable
normal and shear stresses in the joints are 0.5 MPa
and 1.25 MPa respectively.

→ given: $\theta = 60°$

$\sigma_m = 0.5\,MPa$

$\tau = 1.25\,MPa$

A = 200mm × 100mm
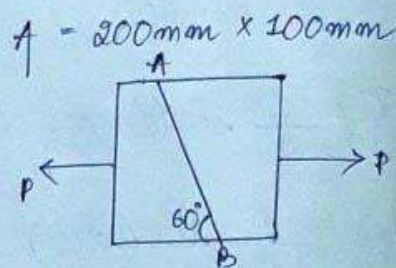
$\tau = \dfrac{\sigma}{2}\sin(2\times 60)$

$\Rightarrow 1.25 = \dfrac{\sigma}{2}\sin(2\times 60)$

$\Rightarrow \sigma = \dfrac{1.25}{\sin(2\times 60)}\times 2.$

$\Rightarrow \boxed{\sigma = 2.88\,MPa}$

$\sigma = \dfrac{P}{A}$

$\Rightarrow 2.88 = \dfrac{P}{200\times 100}$

$\Rightarrow P = 2.88\times 200\times 100$

$\Rightarrow \boxed{P = 57600\,N}$

$\Rightarrow P = \dfrac{57600}{10^3}\,kN \Rightarrow \boxed{P = 57.6\,kN}$

51

55

$$\sigma_n = \frac{\sigma}{2} - \frac{\sigma}{2}\cos 2\theta$$

$$\Rightarrow 0.5 = \frac{\sigma}{2}(1 - \cos 2\theta)$$

$$\Rightarrow 0.5 = \frac{\sigma}{2}(1 - \cos(2 \times 60))$$

$$\Rightarrow \sigma = \frac{0.5}{(1 - \cos(2 \times 60))} \times 2$$

$$\Rightarrow \boxed{\sigma = 0.67 \, MPa}$$

$$\sigma = \frac{P}{A}$$

$$\Rightarrow 0.67 = \frac{P}{200 \times 100}$$

$$\Rightarrow P = 0.67 \times 200 \times 100$$

$$\Rightarrow \boxed{P = 13400 \, N}$$

$$\Rightarrow P = \frac{13400}{10^3} \, kN$$

$$\Rightarrow \boxed{P = 13.4 \, kN}$$

② CASE-2 (Imp)

Stresses on an oblique section of a body subjected to direct stresses in 2 mutually perpendicular directions:
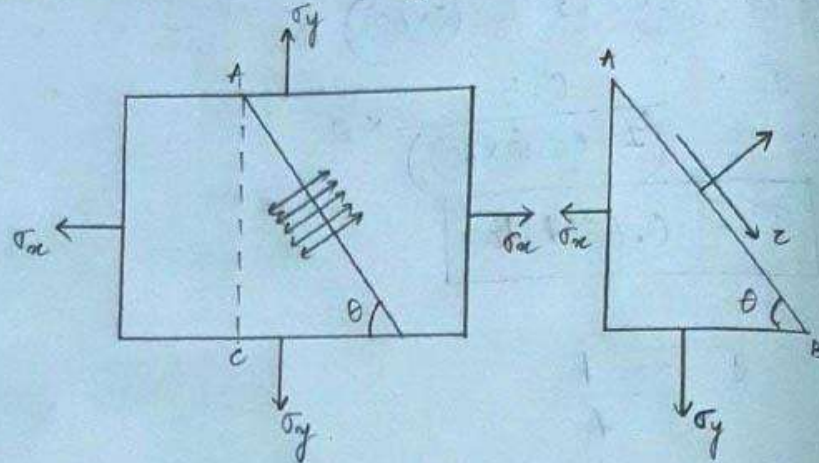
- Consider a rectangular body of uniform cross-sectional area and unit thickness subjected to direct tensile stresses in two mutually perpendicular directions along X-X axis and Y-Y axis.

Now, let us consider an oblique section AB inclined with X-X axis.

σ_x = tensile stress along X-X axis
    (major tensile stress)

σ_y = tensile stress along Y-Y axis
    (minor tensile stress)

θ = angle, which the oblique section AB makes with X-X axis in clockwise direction.

normal stress:

$$\sigma_n = \left(\frac{\sigma_x + \sigma_y}{2}\right) - \left(\frac{\sigma_x - \sigma_y}{2}\right) \cos 2\theta$$

shear stress: $$\tau = \left(\frac{\sigma_x - \sigma_y}{2}\right) \sin 2\theta$$

τ_{max} : when $\theta = 45$
$$2\theta = 90$$
$$\sin 90° = 1$$

$$\tau_{max} = \frac{\sigma_x - \sigma_y}{2}$$

$$\sigma_R = \sqrt{(\sigma_n)^2 + (\tau)^2}$$ (σ_R - resultant stress)

Direction of resultant stress:

$$\tan \alpha = \frac{\tau}{\sigma_n}$$

$$\alpha = \tan^{-1}\left(\frac{\tau}{\sigma_n}\right)$$

54

Questions:

① The stresses at a point in a component are 100 MPa (tensile) and 50 MPa (compressive).

Determine the magnitude of the normal and shear stresses on a plane inclined at an angle of 25° with tensile stress.

Also determine the direction of the resultant stress and the magnitude of the maximum intensity of shear stress.

→ given: $\sigma_x = 100\, MPa$ $\sigma_y = 50\, MPa$
                (tensile)   (compressive)
                                          = −50

$$\theta = 25°$$

$$\therefore \sigma_n = \left(\frac{100 + (-50)}{2}\right) - \left(\frac{100 - (-50)}{2}\right)\cos(2 \times 25)$$

$$\Rightarrow \sigma_n = \left(\frac{100 - 50}{2}\right) - \left(\frac{100 + 50}{2}\right)\cos(2 \times 25)$$

$$\Rightarrow \boxed{\sigma_n = -23.20\, MPa}$$

$$\tau = \left(\frac{100 - (-50)}{2}\right)\sin(2 \times 25)$$

$$\Rightarrow \tau = \left(\frac{100 + 50}{2}\right)\sin(2 \times 25)$$

$$\Rightarrow \boxed{\tau = 57.45\, MPa}$$

$$\sigma_R = \sqrt{(\sigma_m)^2 + (\tau)^2}$$

$$\Rightarrow \sigma_R = \sqrt{(-23.20)^2 + (57.45)^2}$$

$$\Rightarrow \boxed{\sigma_R = 61.95 \text{ MPa}}$$

$$\alpha = \tan^{-1}\left(\frac{\tau}{\sigma_m}\right)$$

$$\Rightarrow \alpha = \tan^{-1}\left(\frac{57.45}{-23.20}\right)$$

$$\Rightarrow \boxed{\alpha = \tan^{-1}(-68.009)}$$

$$\tau_{max} = \frac{\sigma_x - \sigma_y}{2} = \frac{100 - (-50)}{2}$$

$$\Rightarrow \boxed{\tau_{max} = 75 \text{ MPa}}$$

② The stresses at a point of a machine component are 150 MPa and 50 MPa both tensile. Find the intensities of normal, shear and resultant stresses on a plane inclined at an angle of 55° with the axis of major tensile stress. (x-x axis)
Also find the magnitude of the maximum shear stress in the component.

→ given: $\sigma_x = 150 \text{ MPa } (+)$ $\theta = 55°$
$\sigma_y = 50 \text{ MPa } (+)$

56

60
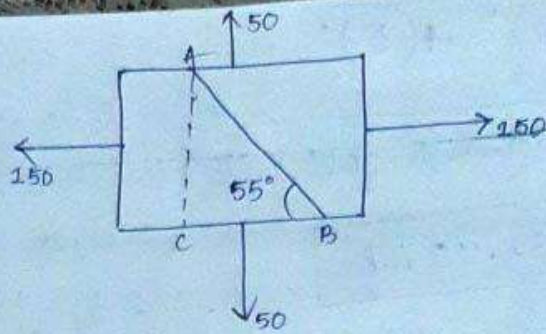
$$\sigma_n = \left(\frac{150 + 50}{2}\right) - \left(\frac{150 - 50}{2}\right) \cos(2 \times 55)$$

$$\Rightarrow \boxed{\sigma_n = 117.10 \text{ MPa}}$$

$$\tau = \left(\frac{150 - 50}{2}\right) \sin(2 \times 55)$$

$$\Rightarrow \boxed{\tau = 46.98 \text{ MPa}}$$

$$\sigma_R = \sqrt{(117.10)^2 + (46.98)^2}$$

$$\Rightarrow \boxed{\sigma_R = 126.17 \text{ MPa}}$$

$$\tau_{max} = \frac{150 - 50}{2} \Rightarrow \boxed{\tau_{max} = 50 \text{ MPa}}$$

$$\alpha = \tan^{-1}\left(\frac{46.98}{117.10}\right)$$

$$\Rightarrow \boxed{\alpha = \tan^{-1}(21.86)}$$

57

61

③ A point in a strained material is subjected to two
Imp mutually perpendicular tensile stresses of 200 MPa
and 100 MPa. Determine the intensities of normal,
shear and resultant stresses on a plane inclined at
30° with the axis of minor tensile stress. (Y-Y axis)

→ given: $\sigma_x = 200\ MPa\ (+)$
$\sigma_y = 100\ MPa\ (+)$

angle with the minor
tensile stress = 30°

angle with the major
tensile stress $= 90° - 30°$
$= 60°$
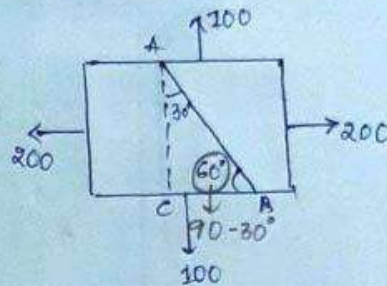
$$\sigma_n = \left(\frac{200 + 100}{2}\right) - \left(\frac{200 - 100}{2}\right) \cos(2 \times 60)$$

$\Rightarrow \boxed{\sigma_n = 175\ MPa}$

$$\tau = \left(\frac{200 - 100}{2}\right) \sin(2 \times 60)$$

$\Rightarrow \boxed{\tau = 43.30\ MPa}$

$\sigma_R = \sqrt{(175)^2 + (43.30)^2} \Rightarrow \boxed{\sigma_R = 180.27\ MPa}$

$\tau_{max} = \frac{200 - 100}{2} \Rightarrow \boxed{\tau_{max} = 50\ MPa}$

$\alpha = \tan^{-1}\left(\frac{43.30}{175}\right) \Rightarrow \boxed{\alpha = \tan^{-1}(13.89)}$

58

③ CASE-3 ($\underline{Imp}$)

Stresses on an oblique section of a body subjected to a simple shear stress only.
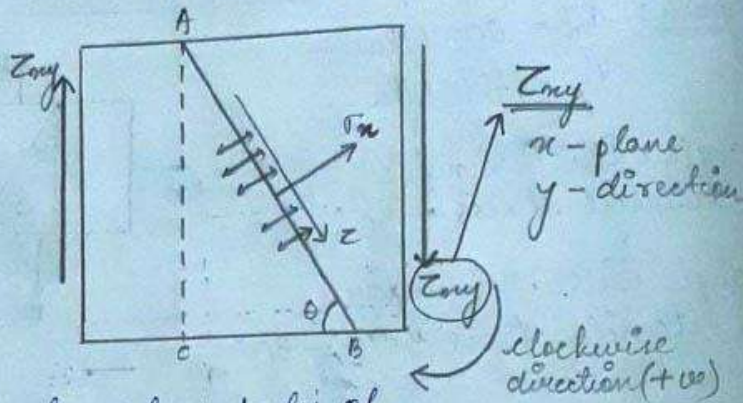
τ_{xy}
x-plane
y-direction

clockwise direction (+ve)

Consider a rectangular body of uniform cross-sectional area and unit thickness subjected to a positive (clockwise) shear stress along X-X axis.

Let us consider an oblique section AB inclined with X-X axis on which we are required to find out the axis.

Let, τ_{xy} = positive shear stress along X-X axis.

θ = angle, which the oblique section AB makes with X-X axis.

normal stress: $\boxed{\sigma_n = \tau_{xy} \sin 2\theta}$

shear stress: $\boxed{\tau = -\tau_{xy} \cos 2\theta}$

59

63

Resultant stress: $\boxed{\sigma_R = \sqrt{(\sigma_n)^2 + (\tau)^2}}$

④ CASE-4 (Imp)   (Combo of Case 1 and Case-3)

Stresses on an oblique section of a body subjected to direct stress in one plane accompanied by a simple shear stress.
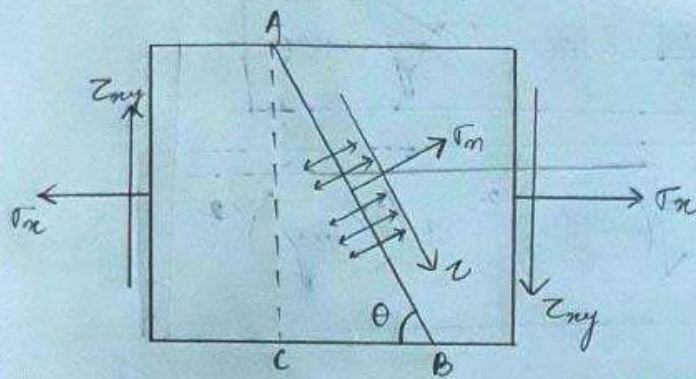
Consider a rectangular body of uniform cross-sectional area and unit thickness subjected to tensile stress along X-X axis accompanied with a positive (clockwise) shear stress along X-X axis.

Now let us consider an oblique section AB inclined with X-X axis on which we are required to find out the stresses.

Let, σ_x = tensile stress along X-X axis

τ_{xy} = positive (clockwise) shear stress along X-X axis

θ = angle, which the oblique section AB makes with X-X axis in clockwise direction.

normal stress:

$$\sigma_n = \frac{\sigma_x}{2} - \frac{\sigma_x}{2}\cos 2\theta - \tau_{xy}\sin 2\theta$$

shear stress:

$$\tau = \frac{\sigma_x}{2}\sin 2\theta - \tau_{xy}\cos 2\theta$$

$$\tau_{max} = \sqrt{\left(\frac{\sigma_x}{2}\right)^2 + \left(\tau_{xy}\right)^2}$$

Maximum Principle Stress:

$$\sigma_{P_1} = \frac{\sigma_x}{2} + \sqrt{\left(\frac{\sigma_x}{2}\right)^2 - \left(\tau_{xy}\right)^2}$$

Minimum Principle Stress:

$$\sigma_{P_2} = \frac{\sigma_x}{2} - \sqrt{\left(\frac{\sigma_x}{2}\right)^2 - \left(\tau_{xy}\right)^2}$$

61

Questions:

1. A plane element in a body is subjected to the tensile stress of 100 MPa accompanied with a shear stress of 25 MPa.

Find: (i) the normal and shear stress on a plane inclined at an angle of 20° with the tensile stress.

(ii) Maximum shear stress on the plane.

→ given: $\theta = 20°$

$$\sigma_x = 100 \, MPa$$

$$\tau_{ny} = 25 \, MPa$$

(i) normal stress:

$$\sigma_m = \frac{100}{2} - \frac{100}{2} \cos(2 \times 20) - 25 \sin(2 \times 20)$$

$$\Rightarrow \boxed{\sigma_m = -4.37 \, MPa}$$

shear stress:

$$\tau = \frac{100}{2} \sin(2 \times 20) - 25 \cos(2 \times 20)$$

$$\Rightarrow \boxed{\tau = 12.98 \, MPa}$$

(ii) $\tau_{max} \sqrt{\left(\frac{100}{2}\right)^2 + (25)^2}$

$$\Rightarrow \boxed{\tau_{max} = 55.90 \, MPa}$$

62

② An element in a strain body is subjected to a compressive stress 200 MPa and a clockwise shear stress of 50 MPa on the same plane.

Calculate the values of normal and shear stresses on a plane inclined at 35° with the compressive stress. Also calculate the value of maximum shear stress in the element.

→ given:

$$xy = 200 \, MPa \, (compressive)$$
$$= (-200) \, MPa$$
$$\tau_x = 50 \, MPa$$
$$\theta = 35°$$

$$\sigma_n = \left(\frac{-200}{2}\right) - \left(\frac{-200}{2}\right) \cos(2 \times 35) - 50 \sin(2 \times 35)$$

$$\Rightarrow \boxed{\sigma_n = -112.78 \, MPa}$$

$$\tau = \frac{-200}{2} \sin(2 \times 35) - 50 \cos(2 \times 35)$$

$$\Rightarrow \boxed{\tau = -111.07 \, MPa}$$

$$\tau_{max} = \sqrt{\left(\frac{-200}{2}\right)^2 + (50)^2}$$

$$\Rightarrow \boxed{\tau_{max} = 111.80 \, MPa}$$

63

67

③ An element in a strained body is subjected to tensile stress of 150 MPa and a shear stress of 50 MPa tending to rotate the element in an anticlockwise direction.

Find: ⓘ the magnitude of the normal and shear stresses on a section inclined at 40° with the tensile stress.

ⓘⓘ the magnitude of the shear maximum shear stress that can exist in the element.

→ given: $\sigma_x = 150$ MPa $\theta = 40°$

$\tau_{xy} = 50$ MPa (in anticlockwise
$= (-50$ MPa)   direction)

$\sigma_m = \dfrac{150}{2} - \dfrac{150}{2} \cos(2 \times 40) - (-50) \sin(2 \times 40)$

$\Rightarrow \boxed{\sigma_m = 111.21 \text{ MPa}}$

$\tau = \dfrac{150}{2} \sin(2 \times 40) - (-50) \cos(2 \times 40)$

$\Rightarrow \boxed{\tau = 82.54 \text{ MPa}}$

$\tau_{max} = \sqrt{\left(\dfrac{150}{2}\right)^2 + (-50)^2}$

$\boxed{\tau_{max} = 90.13 \text{ MPa}}$

84

⑤ CASE – 5 ***** VVV Imp.   (Combo of case – 2 and case – 3)

<u>Stresses on an oblique section of a body subjected to direct stresses in two mutually perpendicular direction accompanied by a simple shear stresses.</u>
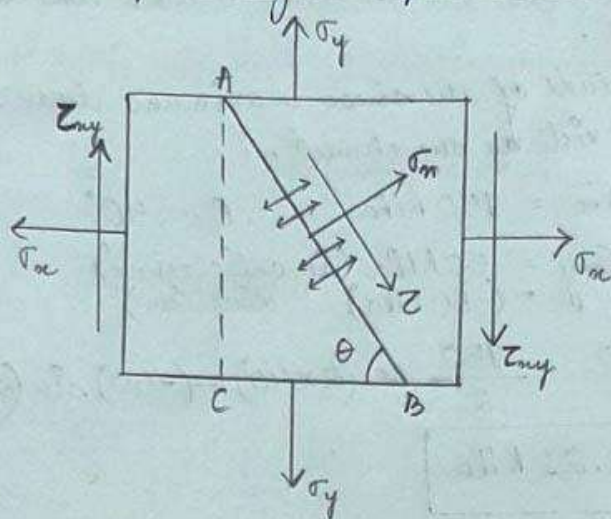
Consider a rectangular body of uniform cross-sectional area and unit thickness subjected to tensile stresses along X – X axis and Y – Y axis respectively and accompanied by a positive (clockwise) shear stress along X – X axis.

Now, consider an oblique section AB inclined with X – X axis on which we are required to find out the stresses.

Let, σ_x = tensile stress along X – X axis

σ_y = tensile stress along Y – Y axis

τ_{xy} = positive (clockwise) shear stress along X – X axis

65

θ = angle, which the oblique section AB makes with X-X axis in an anti-clockwise direction

normal stress:

$$\sigma_n = \left(\frac{\sigma_x + \sigma_y}{2}\right) - \left(\frac{\sigma_x - \sigma_y}{2}\right)\cos 2\theta - \tau_{xy}\sin 2\theta$$

shear stress:

$$\tau = \left(\frac{\sigma_x - \sigma_y}{2}\right)\sin 2\theta - \tau_{xy}\cos 2\theta$$

resultant stress:

$$\sigma_R = \sqrt{(\sigma_n)^2 + (\tau)^2}$$

Maximum Principle stress:

$$\sigma_{P_1} = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + (\tau_{xy})^2}$$

Minimum Principle stress:

$$\sigma_{P_2} = \frac{\sigma_x + \sigma_y}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + (\tau_{xy})^2}$$

maximum shear:

$$\tau_{max} = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \left(\tau_{xy}\right)^2}$$

Questions:

① A point is subjected to tensile stress of 250 MPa in horizontal direction and another tensile stress of 100 MPa in the vertical direction. The point is also subjected to a simple stresses shear stress of 25 MPa such that when it is associated with the major tensile stress, it tends to rotate the element in clockwise direction. what is the magnitude of normal and shear stress on the section inclined at an angle 70° with the minor tensile stress. Also find out the principle stresses.

given:) $\sigma_x = 250$ MPa

$\sigma_y = 100$ MPa

$\tau_{xy} = 25$ MPa (+ve)

angle with the minor tensile stress = 70°

angle with the major tensile stress = 90° - 70°

$\qquad = 20°$

$\sigma_m = \left(\dfrac{250 + 100}{2}\right) - \left(\dfrac{250 - 100}{2}\right) \cos(2 \times 20) -$

$\qquad\qquad\qquad\qquad 25 \sin(2 \times 20)$

$\Rightarrow \boxed{\sigma_m = 101.47 \text{ MPa}}$

67

71

$$\tau = \left(\frac{250-100}{2}\right) \sin(2 \times 20) - 25 \cos(2 \times 20)$$

$$\Rightarrow \boxed{\tau = 29.05 \, MPa}$$

$$\sigma_R = \sqrt{(101.47)^2 + (29.05)^2}$$

$$\Rightarrow \boxed{\sigma_R = 105.54 \, MPa}$$

$$\tau_{max} = \sqrt{\left(\frac{250-100}{2}\right)^2 + (25)^2}$$

$$\Rightarrow \boxed{\tau_{max} = 79.05 \, MPa}$$

$$\sigma_{P_1} = \frac{250+100}{2} + \sqrt{\left(\frac{250-100}{2}\right)^2 + (25)^2}$$

$$\Rightarrow \boxed{\sigma_{P_1} = 254.05 \, MPa}$$

$$\sigma_{P_2} = \frac{250-100}{2} - \sqrt{\left(\frac{250-100}{2}\right)^2 + (25)^2}$$

$$\Rightarrow \boxed{\sigma_{P_2} = 95.94 \, MPa}$$

68

② A plane is element is subjected to tensile stresses of 400 MPa on one plane and 150 MPa on the another plane. Each of the above stress is accompanied by a shear stress of 100 MPa such that when associated with the minor tensile stress, tends to rotate the element in anti-clockwise direction.

find :ⓐ principle stresses and their direction.
ⓑ maximum shearing stresses and the direction of the plane on which they act.

→ given: $\sigma_x = 400 MPa$ $\tau_{xy} = 100 MPa$
           $\sigma_y = 150 MPa$ (anticlockwise direction)
                                    $= -100 MPa.$

$$\sigma_{P_1} = \frac{400 + 150}{2} + \sqrt{\left(\frac{400 - 150}{2}\right)^2 + (-100)^2}$$

$$\Rightarrow \boxed{\sigma_{P_1} = 435.07 \, MPa}$$

$$\sigma_{P_2} = \frac{400 + 150}{2} - \sqrt{\left(\frac{400 - 150}{2}\right)^2 + (-100)^2}$$

$$\Rightarrow \boxed{\sigma_{P_2} = 114.92 \, MPa}$$

69

73

$$\tau_{max} = \sqrt{\left(\frac{400-150}{2}\right)^2 + (-100)^2}$$

$$\Rightarrow \boxed{\tau_{max} = 160.07 \text{ MPa}}$$

* **Direction of Principle Stress:**

$$\boxed{\tan 2\alpha = \frac{2\tau_{xy}}{\sigma_x - \sigma_y}}$$

* **Direction of Maximum Shear Stress:**

$$\boxed{\tan 2\alpha = \frac{\sigma_x - \sigma_y}{2\tau_{xy}}}$$

Questions:

①
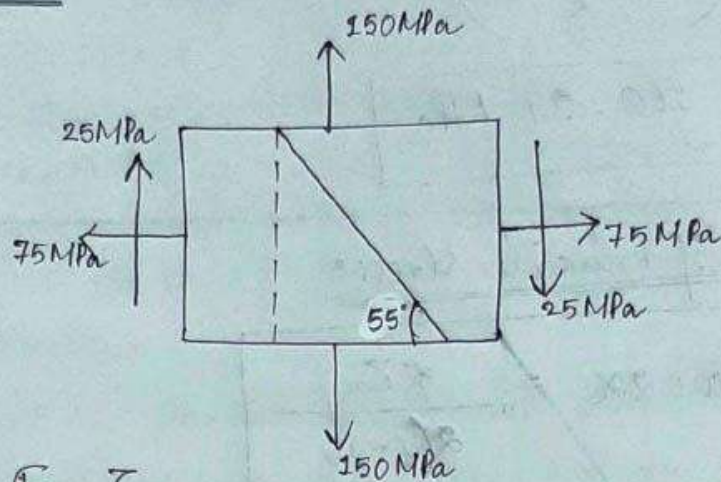
find σ_n, τ,
τ_{max}, σ_{P_1}, σ_{P_2}.

→ given: $\sigma_x = 75\,MPa$ $\theta = 55°$
    $\sigma_y = 150\,MPa$ $\tau_{xy} = 25\,MPa$

$$\sigma_n = \left(\frac{75 + 150}{2}\right) - \left(\frac{75 - 150}{2}\right) \cos(2 \times 55)$$

$$- 25 \sin(2 \times 55)$$

$$\Rightarrow \boxed{\sigma_n = 76.18\,MPa}$$

$$\tau = \left(\frac{75 - 150}{2}\right) \sin(2 \times 55) - 25 \cos(2 \times 55)$$

$$\Rightarrow \boxed{\tau = -26.68\,MPa}$$

71

75

$$\tau_{max} = \sqrt{\left(\frac{75-150}{2}\right)^2 + (25)^2}$$

$$\Rightarrow \boxed{\tau_{max} = 45.06 \, MPa}$$

$$\sigma_{P_1} = \frac{75+150}{2} + \sqrt{\left(\frac{75-150}{2}\right)^2 + (25)^2}$$

$$\Rightarrow \boxed{\sigma_{P_1} = 157.56 \, MPa}$$

$$\sigma_{P_2} = \frac{75+150}{2} - \sqrt{\left(\frac{75-150}{2}\right)^2 + (25)^2}$$

$$\Rightarrow \boxed{\sigma_{P_2} = 67.43 \, MPa}$$

②

50MPa
25MPa
100MPa
25MPa
100MPa
60°
50MPa

find σ_m, τ,
τ_{max}, σ_{P_1}, σ_{P_2}

72

76
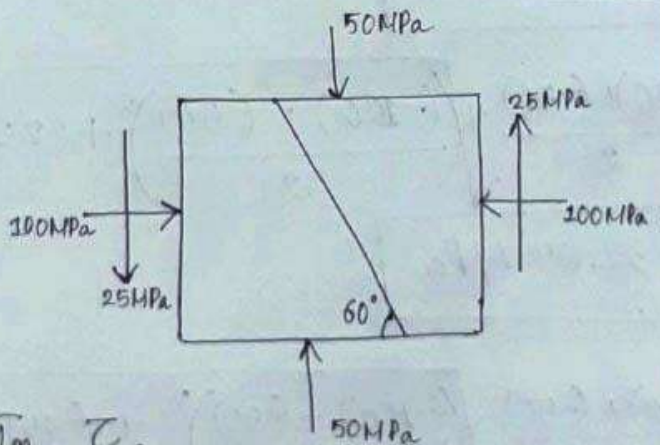
$\rightarrow$ điểm: $\sigma_x = -100\,MPa$ $\quad \tau_{xy} = -25\,MPa$

$\sigma_y = -50\,MPa$ $\quad \theta = 60°$

$$\sigma_n = \left(\frac{(-100)+(-50)}{2}\right) - \left(\frac{(-100)-(-50)}{2}\right) .$$

$$\cos(2\times60) - (-25)\sin(2\times60)$$

$$\Rightarrow \boxed{\sigma_n = -65.84\,MPa}$$

$$\tau = -\left(\frac{(-100)-(-50)}{2}\right)\sin(2\times60) -$$

$$(-25)\cos(2\times60)$$

$$\Rightarrow \boxed{\tau = -34.15\,MPa}$$

$$\sigma_{P_1} = \frac{(-100)+(-50)}{2} + \sqrt{\left(\frac{(-100)-(-50)}{2}\right)^2 + (25)^2}$$

$$\Rightarrow \boxed{\sigma_{P_1} = -39.64\,MPa}$$

$$\sigma_{P_2} = \frac{(-100)+(-50)}{2} - \sqrt{\left(\frac{(-100)-(-50)}{2}\right)^2 + (-25)^2}$$

$$\Rightarrow \boxed{\sigma_{P_2} = -110.35\,MPa}$$

73

$$\tau_{max} = \sqrt{\left(\frac{(-100)-(-50)}{2}\right)^2 + (-25)^2}$$

$$\Rightarrow \boxed{\tau_{max} = 35.35 \; MPa}$$

Chapter 2

REVIEW OF BASIC CONCEPT

CG and M·I

Centroid :-

It is the Point at which the total area of the plane figure (namely rectangle, square, triangle, circle etc) is assumed to be concentrated.

Centre of gravity :-

It is Point through which the resultant of the distributed gravity force (weights) act irrespectic of the orientation of the body.

Centre of mass

→ It is the Point where the entire mass of the body may be assumed to be concentrated.

→ For all Practical Purpose the Centroid and centre of gravity are assumed to be the same.

Moment of Inertia :-

The moment of inertia of a body about an axis is defined as the resistance offered by the body to rotation about that axis.

• it is also defined as the Product of the area and the square of the distance of the center of gravity of the area from that axis.

• Moment of is denoted by I.

• Hence the moment of inertia the x axis is represented by I_{xx} and about the Y axis is represented by I_{yy}.

Parallel axis Theorem :-

Parallel axis theorem states that the moment of inertia of an area with respect to any axis in its plane is equal to the moment of inertia of the area with to a Parallel Centroidal axis Plus the Product of the area and the distance between the two axes.

Perpendicular axis theorem (Polar moment Of Inertia)

Perpendicular axis theorem States that the moment of inertia of an area with respect to an axis Perpendicular to the $x-y$ Plane (z axis) and Passing through a Pole o is equal to the Sum of the moment of inertia of the area about the other two axes (x & y axis) Passing through Pole.

• It's also called as Polar moment of inertia and is denoted by the letter J. $J = I_{zz} = I_{xx} + I_{yy}$

Lamina :-

Lamina Ex:-
Sheet of a Paper
Then layer
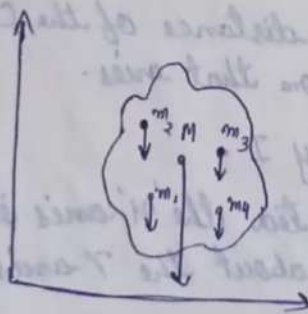
Center of gravity :-

$$\bar{X} = \frac{a_1 x_1 + a_2 x_2 + a_3 x_3}{a_1 + a_2 + a_3} \quad , \quad \bar{Y} = \frac{a_1 y_1 + a_2 y_2 + a_3 y_3}{a_1 + a_2 + a_3}$$

$$\bar{X} = \frac{\Sigma a x}{\Sigma a} \qquad\qquad \bar{Y} = \frac{\Sigma a y}{\Sigma a}$$

① Find the centroid of the lamina as shown in figure.

Ans: In fig ① $A_1 = 150 \times 20 = 3000\,mm^2$

$X_1 = \dfrac{150}{2} = 75\,mm$

$Y_1 = \dfrac{20}{2} = 10\,mm$

In fig ②

$A_2 = 100 \times 20 = 2000\,mm^2$

$X_2 = 65 + \dfrac{20}{2} = 75\,mm$

$Y_2 = 20 + \dfrac{100}{2} = 70\,mm$
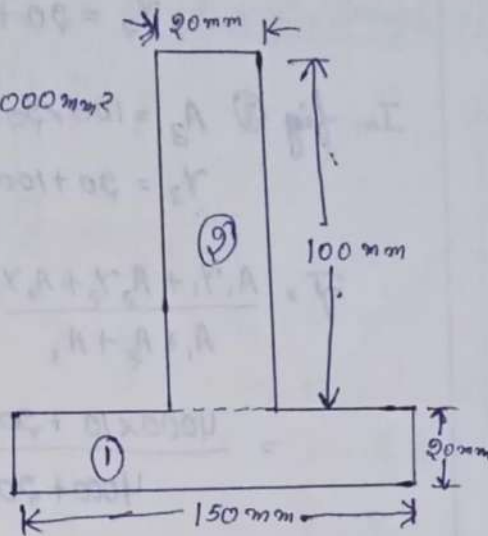
As the Section Simetrycal about Y-Y axis. Then the C.G will lie on that axis. After that we have to find out $\overline{y} = \dfrac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2}$

$\overline{Y} = \dfrac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2} = \dfrac{3000 \times 10 + 2000 \times 70}{3000 + 2000} = 34\,mm$

② Find the centroid of the lamina as shown in figure.

Ans: As the Section Simetrycal about Y-Y axis. Then the C.G will lie on that axis. After that we have to fing out

$\overline{Y} = \dfrac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2}$

In fig ①

$A_1 = 200 \times 20 = 4000\,mm^2$

$Y_1 = \dfrac{20}{2} = 10\,mm$
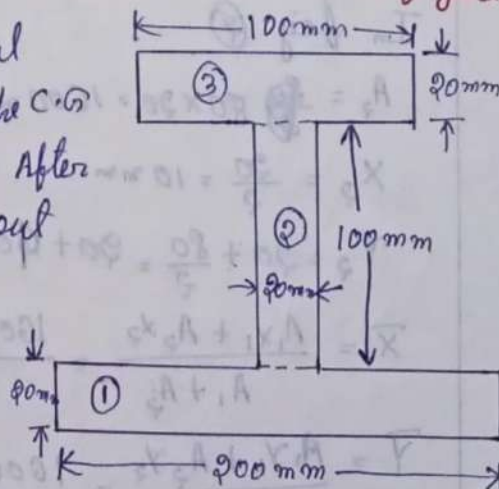
In fig ② $A_2 = 100 \times 90 = 2000 \, mm^2$

$$Y_2 = 20 + \frac{100}{2} = 70 \, mm$$

In fig ③ $A_3 = 100 \times 20 = 2000 \, mm^2$

$$Y_3 = 20 + 100 + \frac{20}{2} = 130 \, mm$$

$$\overline{Y} = \frac{A_1 Y_1 + A_2 Y_2 + A_3 Y_3}{A_1 + A_2 + A_3}$$

$$= \frac{4000 \times 10 + 2000 \times 70 + 2000 \times 130}{4000 + 2000 + 2000}$$

$$= 55 \, mm.$$

Find the Centroid of the lamina as shown in figure.

In fig ①

$A_1 = 80 \times 20 = 1600 \, mm^2$

$X_1 = \frac{80}{2} = 40 \, mm$

$Y_1 = \frac{20}{2} = 10 \, mm$

In fig ②

$A_2 = 80 \times 20 = 1600 \, mm^2$

$X_2 = \frac{20}{2} = 10 \, mm$

$Y_2 = 20 + \frac{80}{2} = 20 + 40 = 60 \, mm$

$$\overline{X} = \frac{A_1 X_1 + A_2 X_2}{A_1 + A_2} = \frac{1600 \times 40 + 1600 \times 10}{1600 + 1600} = 25 \, mm$$

$$\overline{Y} = \frac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2} = \frac{1600 \times 10 + 1600 \times 60}{1600 + 1600} = 95 \, mm$$

C.G For following Section :-

Triangle
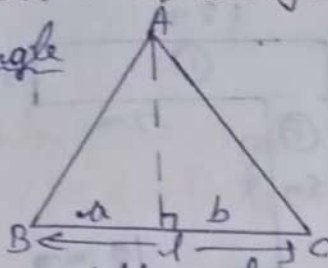
from left end
$$\bar{x} = \frac{d+a}{3}$$

form right end
$$\bar{x} = \frac{d+b}{3}$$

Right-angle-triangle
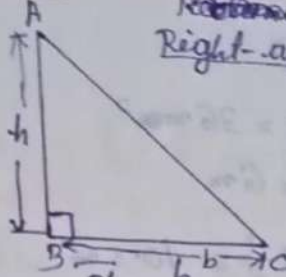
$$\bar{y} = \frac{h}{3}$$

$$\bar{x} = \frac{b}{3}$$

$$\bar{x} = \frac{2}{3}b$$

$$\bar{y} = \frac{h}{3}$$
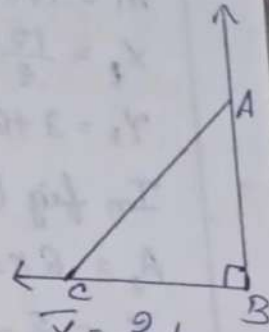
Trapizeum
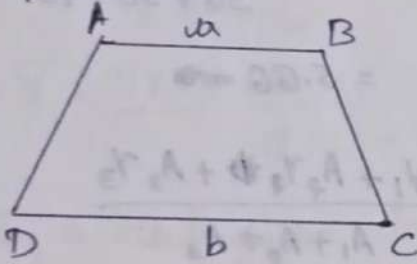
C.G (for DC

$$C.G \ (from \ DC) = \left(\frac{2a+b}{a+b}\right)\frac{h}{3}$$

$$From \ AB = \left(\frac{a+2b}{a+b}\right)\frac{h}{3}$$

$$Area = \frac{1}{2}(a+b)\times h$$

Quater circle
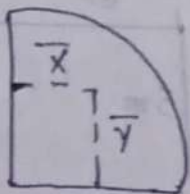
$$\bar{x} = \frac{4r}{3\pi}$$

$$\bar{y} = \frac{4r}{3\pi}$$

$$Area = \frac{\pi r^2}{4}$$

Semicircle

$$Area = \frac{\pi r^2}{2}$$

$$\bar{y} = \frac{4r}{3\pi} \ (from \ bo$$

$$\bar{x} = R$$

① Find the centroid of the lamina as shown in fig

Ans: In fig ①

$A_1 = 12 \times 3 = 36 \, m^2$

$X_1 = \frac{12}{2} = 6 \, m$

$Y_1 = 3 + 6 + \frac{3}{2} = 10.5 \, m$

In fig ②

$A_2 = 6 \times 5 = 30 \, m$

$X_2 = \frac{5}{2} = 2.5 \, m$

$Y_2 = 3 + \frac{6}{2} = 3 + 3 = 6 \, m$

In fig ③

$A_3 = 15 \times 3 = 45 \, m$

$X_3 = \frac{15}{2} = 7.5 \, m$

$Y_3 = \frac{3}{2} = 1.5 \, m$
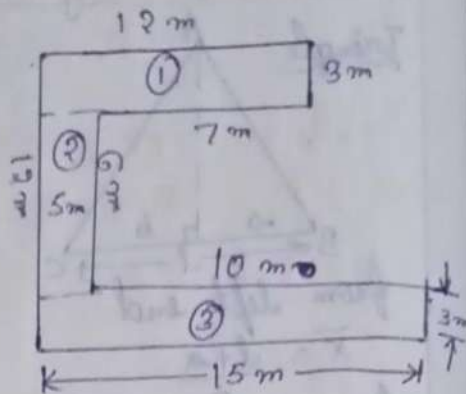
$$\overline{X} = \frac{A_1 X_1 + A_2 X_2 + A_3 X_3}{A_1 + A_2 + A_3}$$

$$= \frac{36 \times 6 + 30 \times 2.5 + 45 \times 7.5}{36 + 30 + 45}$$

$$= 5.66 \, m$$

$$\overline{Y} = \frac{A_1 Y_1 + A_2 Y_2 + A_3 Y_3}{A_1 + A_2 + A_3}$$

$$= \frac{36 \times 10.5 + 30 \times 6 + 45 \times 1.5}{36 + 30 + 45}$$

$$= 5.635 \, m$$

② Find the centroid of the lamina as shown in fig.

Ans: In fig ①

$A_1 = 12 \times 4 = 48 \, m$

$X_1 = \frac{4}{2} = 2 \, m$

$Y_1 = \frac{12}{2} = 6 \, m$

In fig ②

$A_2 = 5 \times 3 = 15 \, m$

85
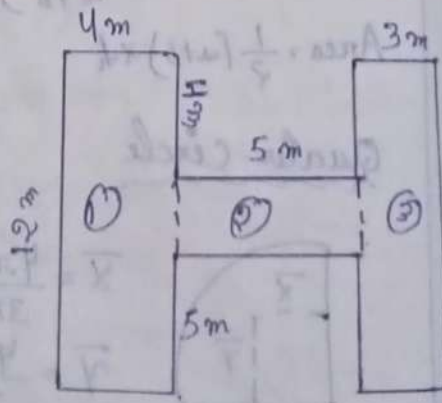
$$X_2 = 4 + \frac{5}{2} = 4 + 2 \cdot 5 = 6 \cdot 5 \, m$$

$$Y_2 = 5 + \frac{3}{2} = 6 \cdot 5 \, m$$

In fig ③

$$A_3 = 12 \times 3 = 36 \, m$$

$$X_3 = 4 + 5 + \frac{3}{2} = 10 \cdot 5 \, m$$

$$Y_3 = \frac{12}{2} = 6 \, m$$

$$\overline{X} = \frac{A_1 X_1 + A_2 X_2 + A_3 X_3}{A_1 + A_2 + A_3}$$

$$= \frac{48 \times 2 + 15 \times 6 \cdot 5 + 36 \times 10 \cdot 5}{48 + 15 + 36}$$

$$= 5 \cdot 77 \, m$$

$$\overline{Y} = \frac{48 \times 6 + 15 \times 6 \cdot 5 + 36 \times 6}{48 + 15 + 36} = 6 \cdot 07 \, m$$

③ In a rectangular lamina 100 mm × 120 mm, a rectangular Opening PQRS 30mm × 40mm is made as shown in fig.

$$A_1 = 100 \times 120 = 12000 \, mm^2$$

$$X_1 = \frac{100}{2} = 50 \, mm$$

$$Y_1 = \frac{120}{2} = 60 \, mm$$

Deducted area

$$A_2 = 40 \times 30 = 1200 \, mm^2$$

$$X_2 = 50 + 10 + \frac{30}{2} = 75 \, mm$$

$$Y_2 = 20 + \frac{40}{2} = 40 \, mm$$

$$\overline{X} = \frac{12000 \times 50 - 1200 \times 75}{12000 - 1200} = 47 \cdot 22 \, mm$$
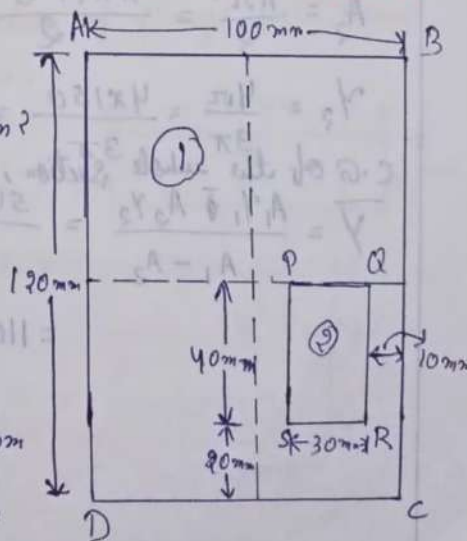
$$\overline{y} = \frac{12000 \times 60 - 1200 \times 40}{12000 - 1200} = 62.22 \, mm$$

(4) A semi circle of 150 mm radius is cut out from a trapizium as shown as the figure. Find the C.G. of the section.

Ans: In fig ①

$$A_1 = \frac{1}{2}(a+b)\,h$$

$$= \frac{1}{2}(200+400) \times 180$$

$$= 54000 \, mm^2$$

$$Y_1 = \frac{h}{3}\left(\frac{b + 2a}{b+a}\right)$$

$$= \frac{180}{3}\left(\frac{400 + 2 \times 200}{400 + 200}\right) = 80 \, mm$$
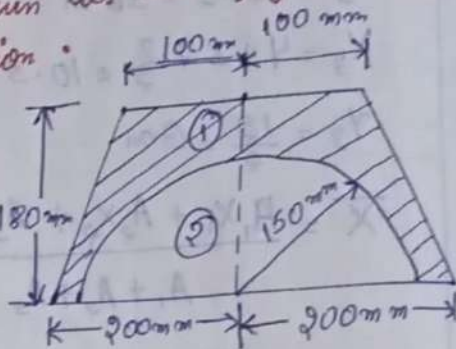
In fig ②

$$A_2 = \frac{\pi r^2}{2} = \frac{\pi \times (150)^2}{2} = 35343 \, mm^2$$

$$Y_2 = \frac{4r}{3\pi} = \frac{4 \times 150}{3\pi} = 63.66 \, mm$$

C.G of the whole section,

$$\overline{Y} = \frac{A_1 Y_1 - A_2 Y_2}{A_1 - A_2} = \frac{54000 \times 80 - 35343 \times 63.66}{54000 - 35343}$$

$$= 110.95 \, mm$$
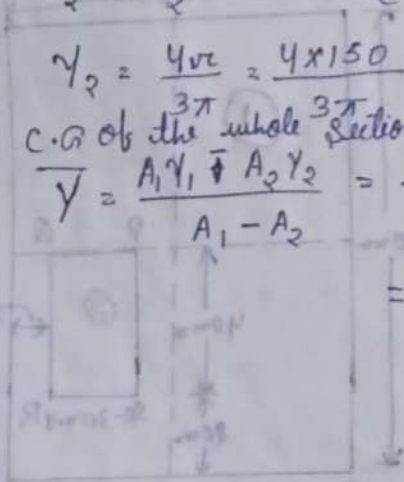
⑤ Find C.G of this section?

In fig ①

$A_1 = 1.5 \times 3 = 4.5 \ m^2$

$Y_1 = 2 + \frac{3}{2} = 3.5 \ m$

In fig ②

$A_2 = \frac{1}{2}(a+b)h$

$= \frac{1}{2} \times (1.5+3) \times 2$

$= 4.5 \ m^2$

$Y_2 = \frac{h}{3}\left(\frac{b+2a}{b+a}\right)$

$= \frac{2}{3}\left(\frac{3+2\times1.5}{3+1.5}\right) = 0.89 \ m$

$\overline{Y} = \frac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2}$

$= \frac{4.5 \times 3.5 + 4.5 \times 0.89}{4.5 + 4.5} = 2.19 \ m$
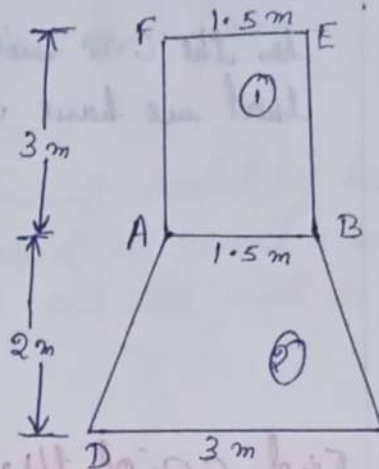
⑥ Find Centroid of T section of flange 160×20 mm and web 120 mm × 20 mm.

Ans: In fig ①

$A_1 = 120 \times 20 = 2400 \ mm^2$

$X_1 = 70 + \frac{20}{2} = 70 + 10 = 80 \ mm$

$Y_1 = \frac{120}{2} = 60 \ mm$

In fig ②

$A_1 = 160 \times 20 = 3200 \ mm^2$

$X_1 = \frac{160}{2} = 80 \ mm$

$Y_1 = 120 + \frac{20}{2} = 120 + 10 = 130 \ mm$
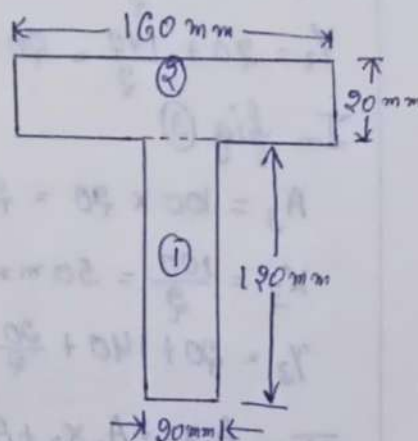
As the section Simetrycal about $y-y$ axis. Then the C.G will lie on that axis after that we have to find $\bar{y} = \dfrac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2}$

$$= \dfrac{2400 \times 60 + 3200 \times 130}{2400 + 3200}$$

$$= 100 \, mm$$

⑦ Find C.G of this section?

Ans: In fig ①

$A_1 = 100 \times 20 = 2000 \, mm^2$

$X_1 = 180 + \dfrac{100}{2}$

$\quad = 130 \, mm$

$Y_1 = \dfrac{20}{2} = 10 \, mm$
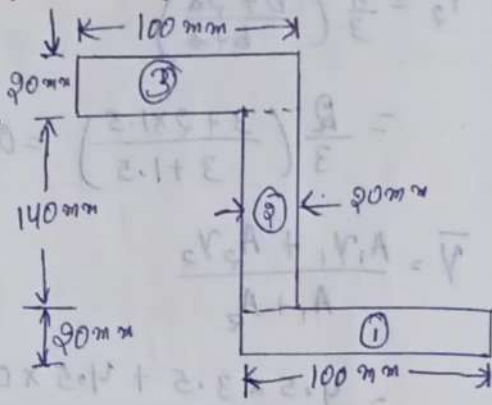
In fig ②

$A_2 = 140 \times 20 = 2800 \, mm^2$

$X_2 = 80 + \dfrac{20}{2} = 90 \, mm$

$Y_2 = 20 + \dfrac{140}{2} = 20 + 70 = 90 \, mm$

In fig ③

$A_3 = 100 \times 20 = 2000 \, mm^2$

$X_3 = \dfrac{100}{2} = 50 \, mm$

$Y_3 = 20 + 140 + \dfrac{20}{2} = 170 \, mm$

$$\overline{X} = \dfrac{A_1 X_1 + A_2 X_2 + A_3 X_3}{A_1 + A_2 + A_3} = \dfrac{2000 \times 130 + 2800 \times 90 + 2000 \times 50}{2000 + 2800 + 2000}$$

$$= 90 \, mm$$

$$\overline{y} = \frac{A_1 y_1 + A_2 y_2 + A_3 y_3}{A_1 + A_2 + A_3} = \frac{2000 \times 10 + 2800 \times 90 + 2000 \times 170}{2000 + 2800 + 2000}$$

$$= 90 \, mm$$

* From area Point of view moment created by top Portion about its C.G is equal to moment created by bottom area about C.G. and this will be simetrycal about both axis.

⑧ Find C.G of this section ?

In fig ①

$A_1 = 10 \times 1.5 = 15 \, m^2$

$X_1 = \frac{1.5}{2} = 0.75 \, m$

$Y_1 = \frac{10}{2} = 5 \, m.$

In fig ②

~~$A_2 = \frac{1}{2} \times 1.5 \times 6$~~

$A_2 = \frac{1}{2} \times 2.5 \times 6$

$= 7.5 \, m^2$

$X_1 = 1.5 + \frac{2.5}{3} = 2.33 \, m.$

$Y_2 = \frac{h}{3} = \frac{6}{3} = 3 \, m.$

In fig ③

$A_3 = 4.5 \times 1 = 4.5 \, m^2$

$X_3 = \frac{4.5}{2} = 2.25 + 1.5 = 3.75 \, m$

$Y_3 = \frac{1}{2} = 0.5 \, m.$
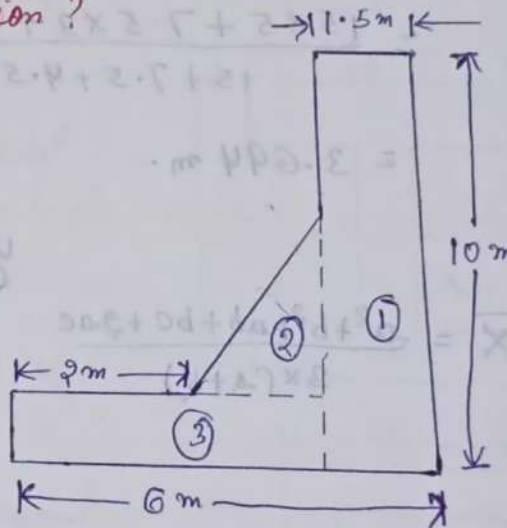
$$\overline{X} = \frac{A_1 x_1 + A_2 x_2 + A_3 x_3}{A_1 + A_2 + A_3}$$

$$= \frac{15 \times 0.75 + 7.5 \times 2.33 + 4.5 \times 3.75}{15 + 7.5 + 4.5}$$

$$= 1.69 \, m$$

$$\overline{Y} = \frac{A_1 y_1 + A_2 y_2 + A_3 y_3}{A_1 + A_2 + A_3}$$

$$= \frac{15 \times 5 + 7.5 \times 3 + 4.5 \times 0.5}{15 + 7.5 + 4.5}$$

$$= 3.694 \, m.$$

$$\overline{X} = \frac{a^2 + b^2 + ab + bc + 2ac}{3 \times (a+b)}$$
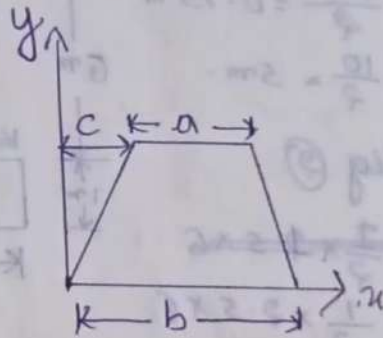
1) Reactangular Lamina :-

Moment of Inertia about
x-axis $I_{xx} = \dfrac{bd^3}{12}$

Moment of Inertia about
y-axis $I_{yy} = \dfrac{db^3}{12}$
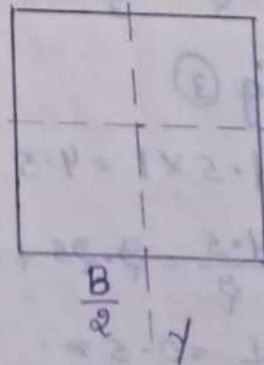
2) Reclangular lamina with a Centrally situated hollow rectangular :-

Moment of Inertia about x-axis is given by

$$I_{xx} = \frac{BD^3 - bd^3}{12}$$

Moment of Inertia about Y-axis is given by $I_{yy} = \frac{DB^3 - db^3}{12}$
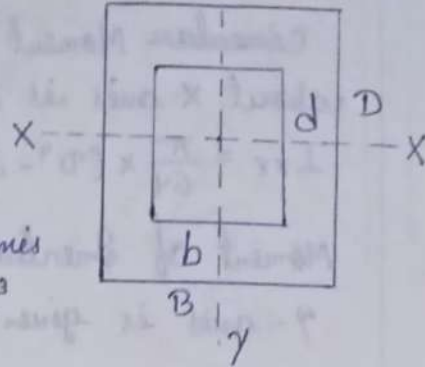
3) 'I' Section :-

Moment of Inertia about x-axis is given by

$$I_{xx} = \frac{BD^3}{12} - \frac{(B-b)d^3}{12}$$

$$= \frac{1}{12}\{BD^3 - (B-b)d^3\}$$
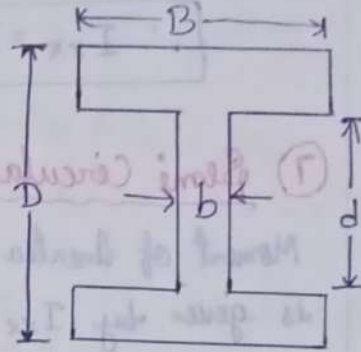
4) 'C' Section :-

Moment of Inertia about x-axis is given by

$$I_{xx} = \frac{BD^3}{12} - \frac{(B-b)d^3}{12}$$
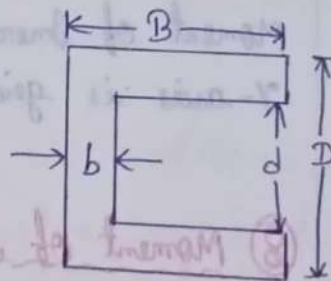
5) Circular Lamina :-

Moment of Inertia about x-axis is given by

$$I_{xx} = \frac{\pi \times D^4}{64}$$

Moment of Inertia about Y-axis given by $I_{yy} = \frac{\pi \times D^4}{64}$

92
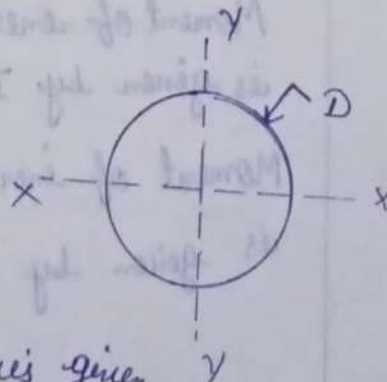
⑥ Circular lamina with a centerally situated hollow circular :-

Circular Moment of Inertia about x-axis is given by

$$I_{xx} = \frac{\pi}{64} \times (D^4 - d^4)$$

Moment of Inertia about y-axis is given by $I_{YY} = \frac{\pi}{4} \times (D^4 - d^4)$
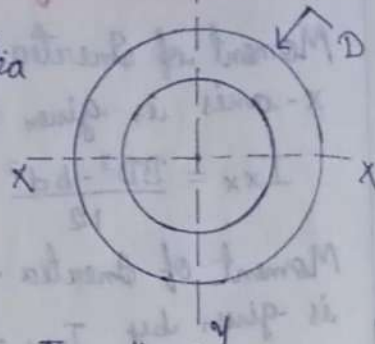
$$\boxed{\therefore I_{xx} = I_{YY} = \frac{\pi}{4} \times (D^4 - d^4)}$$

⑦ Semi Circular lamina about its Base or diameter :-

Moment of Inertia about x-axis is given by $I_{xx} = \frac{\pi d^4}{4} = 0.11 r^4$

Moment of Inertia about y-axis is given by $I_{YY} = \frac{\pi d^4}{128}$ or $\frac{\pi r^4}{8}$
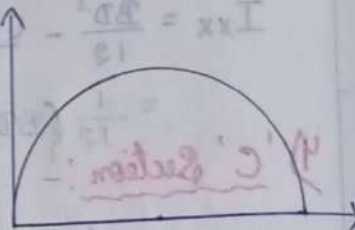
⑧ Moment of inertia qualar circle :-

Moment of inertia about x-axis is given by $I_{xx} = \frac{1}{2} \times 0.11 \times R^4$

Moment of inertia about y-axis is given by $I_{YY} = \frac{1}{2} \times \frac{\pi r^4}{8}$
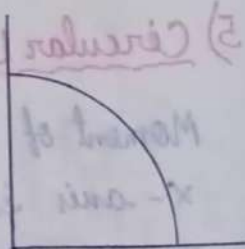
① Find the moment of inertia of the area Shaded in fig about the axis AB·

Ans: Moment of inertia of right angle triangle about its base AB;

is $\dfrac{bh^3}{12}$

$= \dfrac{80 \times (80)^3}{12}$

$= 3413333·33 \ mm^4$
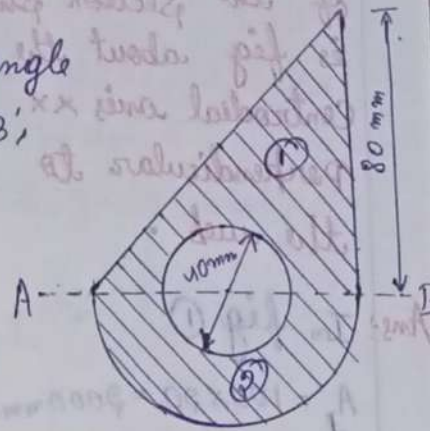
Moment of Inertia of the Semi circle about its diameter AB,

$I = \dfrac{\pi d^4}{128} = \dfrac{\pi \times (80)^4}{128} = 1005309·65 \ mm^4$

Total moment of Inertia of the given ① & ② fig about the axis $AB = 3413333·33 + 1005309·6$

$= 4418642·98 \ mm^4$

Now moment of Inertia of the circle of 40 mm dia about $AB = \dfrac{\pi}{64} \times D^4 = \dfrac{\pi}{64} \times (40)^4 = 125663·71 \ mm$

Hence the M·I of the Shaded Portion

$= 4418642·98 - 125663·71$

$= 4292979·27 \ mm^4$

② Find the moment of inertia of the section show is fig about the centrodial axis xx perpendicular to the web.
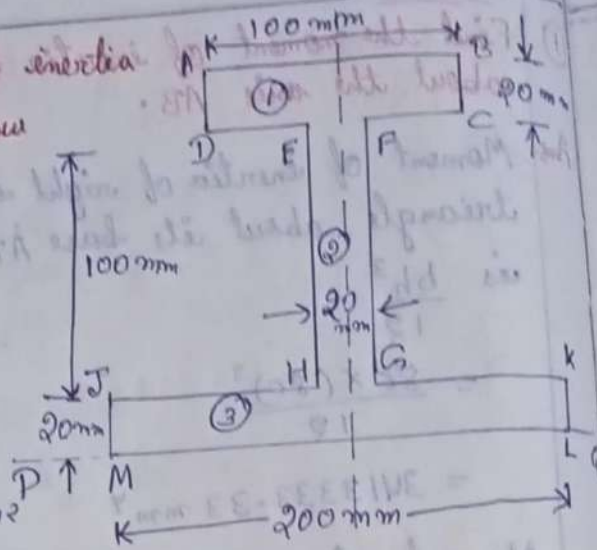
Ans: In fig ①

$$A_1 = 100 \times 20 = 2000 \, mm^2$$

$$y_1 = 20 + 100 + \frac{20}{2} = 130 \, mm$$

$$I_{G_1} = \frac{bd^3}{12} = \frac{100 \times (20)^3}{12} = 66666.67 \, mm^4$$

In fig ②

$$A_2 = 100 \times 20 = 2000 \, mm^2$$

$$y_2 = 20 + \frac{100}{2} = 70 \, mm$$

$$I_{G_2} = \frac{bd^3}{12} = \frac{20 \times (100)^3}{12} = 1666666.67 \, mm^4$$

In fig ③

$$A_3 = 200 \times 20 = 4000 \, mm^2$$

$$y_3 = \frac{20}{2} = 10 \, mm$$

$$I_{G_3} = \frac{bd^3}{12} = \frac{200 \times (20)^3}{12} = 133333.33 \, mm^4$$

$$\bar{y} = \frac{A_1 y_1 + A_2 y_2 + A_3 y_3}{A_1 + A_2 + A_3} = \frac{2000 \times 130 + 2000 \times 70 + 4000 \times 10}{2000 + 2000 + 400}$$

$$= 55 \, mm.$$

Now, distance of x-x axis from PQ = 55 m
$(\bar{y})$

M.I about axis PQ = 666667 + 133333.33
$\qquad$ + 1666666.67
$\qquad$ = 1866666.67 mm⁴

$\sum ay^2 = a_1 y_1^2 + a_2 y_2^2 + a_3 y_3^2$

$\qquad$ = 44000000 mm⁴

$\sum M.I = 1866667$ mm⁴

M.I about the axis PQ = $\sum M.I + \sum ay^2$

$\qquad$ = 1866667 + 44000000

$\qquad$ = 45866667 mm⁴

We have $I_{PQ} = I_{xx} + \sum a\bar{y}^2$

$\Rightarrow 45866667 = I_{xx} + 8000 \times (55)^2$

$\Rightarrow I_{xx} = 45866667 - 8000 \times (55)^2$

$\qquad$ = 21666667 mm⁴.

③ Find the M.I of the Centrodial axis xx and YY of the section shown in the fig.

Ans. In fig ①

$A_1 = 60 \times 20 = 1200 mm^2$

$X_1 = 20 + \dfrac{60}{2} = 50 mm$

$Y_1 = \dfrac{20}{2} = 10 mm$
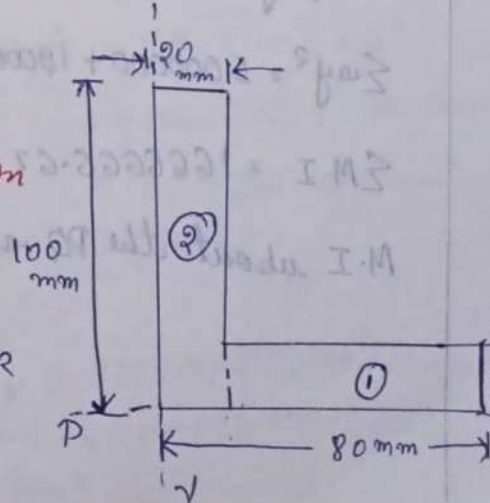
In fig ②

$$A_2 = 100 \times 20 = 2000 \text{ mm}^2$$

$$X_2 = \frac{20}{2} = 10 \text{ mm}$$

$$Y_2 = \frac{100}{2} = 50 \text{ mm}$$

$$\overline{X} = \frac{A_1 X_1 + A_2 X_2}{A_1 + A_2} = \frac{1200 \times 60 + 2000 \times 10}{1200 + 2000} = 28.75 \text{ mm}$$

$$\overline{Y} = \frac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2} = \frac{1200 \times 10 + 2000 \times 50}{1200 + 2000} = 35 \text{ mm}$$

M·I of fig (1) about its centrodial axis I_{G_1}

$$\frac{bd^3}{12} = \frac{60 \times (20)^3}{12} = 40000 \text{ mm}^4$$

M·I of fig (2) about its centrodial axis I_{G_2}

$$\frac{bd^3}{12} = \frac{20 \times (100)^3}{12} = 1666666 \cdot 67 \text{ mm}^4$$

In fig (i) $ay^2 = 1200 \times (10)^2 = 120000 \text{ mm}^4$

In fig (ii) $ay^2 = 2000 \times (50)^2 = 5000000 \text{ mm}^4$

$$\sum ay^2 = 5000000 + 120000 = 5120000 \text{ mm}^4$$

$$\sum M·I = 1666666 \cdot 67 + 40000 = 1706666 \cdot 67 \text{ mm}^4$$

M·I about the PQ axis $= \sum M·I + \sum ay^2$

$$= 1706666 \cdot 67 + 5120000$$

$$= 6826666 \cdot 67 \text{ mm}^4$$
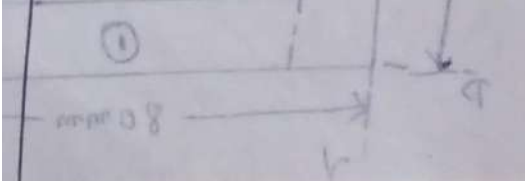
we have M.I about the axis (PQ) = $I_{xx} + \Sigma \, a \, y^2$

$\Rightarrow 6826666 \cdot 67 = I_{xx} + 3200 \times (35)^2$

$\Rightarrow I_{xx} = 6826666 \cdot 67 - 3200 \times (35)^2$

$\Rightarrow \boxed{I_{xx} = 2906666 \cdot 67 \, mm^4}$

we known that moment of inertia of rectangle
(1) about an axis through its center of gravity
and parallel to $\gamma - \gamma$ axis.

$I_{G_1} = \dfrac{db^3}{12} = \dfrac{20 \times (60)^3}{12} = 360000 \, mm^4$

M.I about rectangular (2)

$I_{G_2} = \dfrac{db^3}{12} = \dfrac{100 \times (20)^3}{12} = 66666 \cdot 67 \, mm^4$

In fig (1) $a \, x^2 = 1200 \times (50)^2 = 3000000 \, mm^4$

In fig (2) $a \, x^2 = 2000 \times (10)^2 = 200000 \, mm^4$

$\Sigma \, a \, y^2 = 3000000 + 200000 = 3200000 \, mm^4$

$\Sigma \, M.I = 360000 + 66666 \cdot 67 = 426666 \cdot 67$

M.I about the $X \gamma$ axis $= \Sigma M.I + \Sigma \, a \, y^2$

$= 426666 \cdot 67 + 3200000$

$= 3626666 \cdot 67 \, mm^4$

we have M.I about the axis $(PQ \times \gamma)$

$= I_{\gamma \gamma} + \Sigma \, a \, \bar{y}^2$

$\Rightarrow 3626666 \cdot 67 = I_{xy} + 3200 \times (28.75)^2 (25)^2$

$\Rightarrow I_{xy} = 3626666 \cdot 67 - 3200 \times (28.75)^2 (25)^2$

$\Rightarrow I_{xy} = 1626666 \cdot 67 \, mm^4$

98

4) Find the M.I of the centroid axis xx and YY of the section show in the fig.
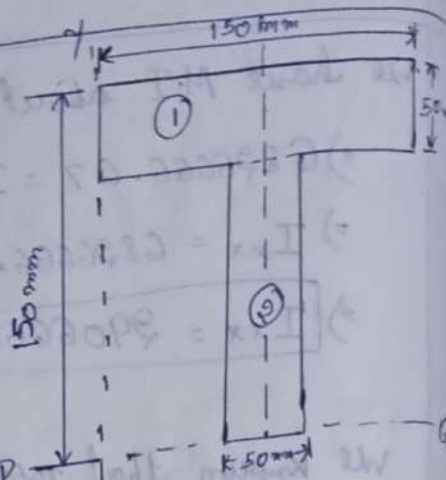
Ans: In fig ①

$A_1 = 150 \times 50 = 7500 \text{ mm}^2$

$X_1 = \dfrac{150}{2} = 75 \text{ mm}$

$Y_1 = 100 + \dfrac{50}{2} = 125 \text{ mm}$

In fig ②

$A_2 = 100 \times 50 = 5000 \text{ mm}^2$

$X_2 = 50 + \dfrac{50}{2} = 75 \text{ mm}$

$Y_2 = \dfrac{100}{2} = 50 \text{ mm}$

$\overline{X} = \dfrac{A_1 X_1 + A_2 X_2}{A_1 + A_2} = \dfrac{7500 \times 75 + 5000 \times 75}{7500 + 7500}$

$= 75 \text{ mm}$

$\overline{Y} = \dfrac{A_1 Y_1 + A_2 Y_2}{A_1 + A_2} = \dfrac{7500 \times 125 + 5000 \times 50}{7500 + 7500} = 95 \text{ mm}$

In Ixx axis

M.I of fig (i) about its centroid axis I_{G_1}

$= \dfrac{bd^3}{12} = \dfrac{150 \times (50)^3}{12} = 1562500 \text{ mm}^4$

M.I of fig (ii) about its centroid axis I_{G_2}

$= \dfrac{bd^3}{12} = \dfrac{50 \times (100)^3}{12} = 4166666 \cdot 67 \text{ mm}^4$

$\Sigma M.I = 1562500 + 4166666 \cdot 67 = 5729166 \cdot 67 \text{ mm}^4$

$$\sum ay^2 = a_1 y_1^2 + a_2 y_2^2$$

$$= 129687500 \, mm^4$$

M.I. about PQ axis $= \sum M.I + \sum ay^2$

$$= \cancel{D} 5729166.67 + 129687500$$

$$= 135416666.7 \, mm^4$$

$$I_{xx} = I_{PQ} - \sum a\bar{y}^2$$

$$= 135416666.7 - 12500 \times (95)^2$$

$$= 22604166.7 \, mm^4$$

Im I_{YY} axis

M.I of fig (i) about its centroid axis I_{G_1}

$$\frac{db^3}{12} = \frac{50 \times (150)^3}{12} = 14062500 \, mm^4$$

M.I of fig (ii) about its centroid axis I_{G_2}

$$\frac{db^3}{12} = \frac{100 \times (50)^3}{12} = 1041666.67 \, mm^4$$

$$\sum M.I = 15104166.67 \, mm^4$$

$$\sum ax^2 = a_1 x_1^2 + a_2 x_2^2 = 70312500 \, mm^4$$

M.I about XY axis $= \sum M.I + \sum ax^2$

$$= 15104166.67 + 70312500$$

$$= 85416666.67 \, mm^4$$

$$I_{YY} = I_{XY} - \sum a\bar{x}^2$$

$$= 85416666.67 - 12500 \times (75)^2$$

$$= 15104166.67 \, mm^4$$

Chapter 3

Stresses in

Beams .

* Stresses in beam due to bending:

→ A member subjected to bending moment and shear forces undergoes certain deformation, so that the material of the member will offer resistance on stresses against these deformation.

→ A bending moment bends a member. So stresses induced by bending moment are called <u>bending stresses</u>.

→ Similarly stresses induced due to shear forces are called <u>shear stresses</u>.

* <u>Pure bending</u>: (2 marks)

When any portion of a beam is absolutely free from shear force but subjected to bending moment, in that case the bending is called <u>pure bending</u> (or) <u>simple bending</u>.

* Moment of Resistance:

It is defined as " The maximum bending moment, a section can resist."

(OR)

It is defined as " The couple produced by the internal forces in a beam subjected to bending under the maximum permissible stress."

* **Assumption in the theory of simple bending** *
Imp. **or pure bending:** (6 marks)

① The beam is subjected to pure bending and is therefore free from shear force.

② The material of the beam is homogeneous and isotropic.

③ The value of the Young's Modulus is the same for the beam material in tension as well as in compression.

④ A transverse section of the beam which is a plane before bending will remain a plane after bending.

⑤ The resultant pull or thrust on a transverse section of the beam is zero.

⑥ The transverse section of the beam is symmetrical about an axis passing through the centroid of the section and parallel to the plane of bending.
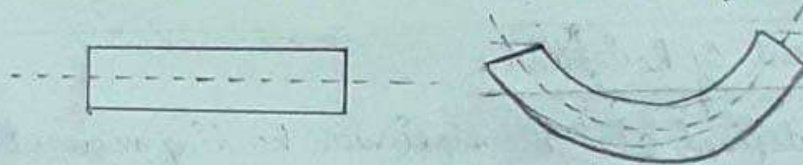
⑦ The radius of curvature of the deflected beam is very large compared with the dimensions of the cross-section of the beam.

[neutral axis → divides the tension and compression]

137

103

* <u>Simple Bending Equation:</u>

$$\frac{M}{I} = \frac{f}{y} = \frac{E}{R}$$

where, M → Bending Moment

F → Stress on the beam

I → Moment of Inertia about the neutral axis

y → distance of neutral axis

E → Young's Modulus of Elasticity

R → Radius of Bending Arc
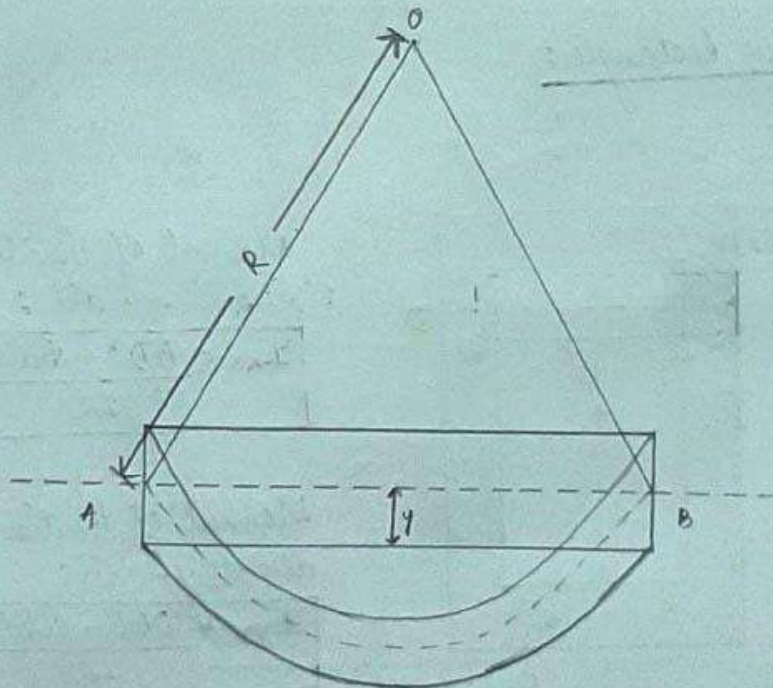
* Some formula Related to moment of Inertia :

① Rectangle :
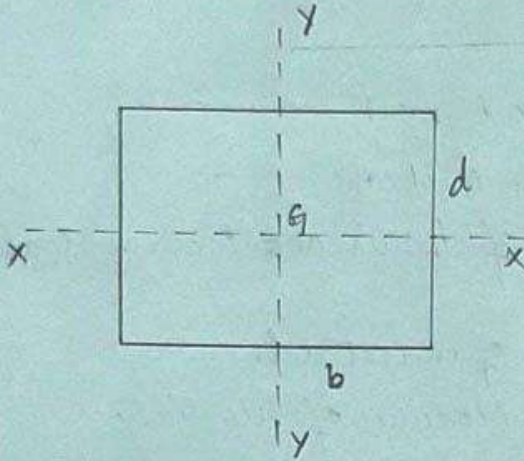
Moment of Inertia about x $\perp$ axis :

$$I_{xx} = \frac{bd^3}{12}$$

Moment of Inertia about y-axis :

$$I_{yy} = \frac{db^3}{12}$$

② Hollow Rectangle :
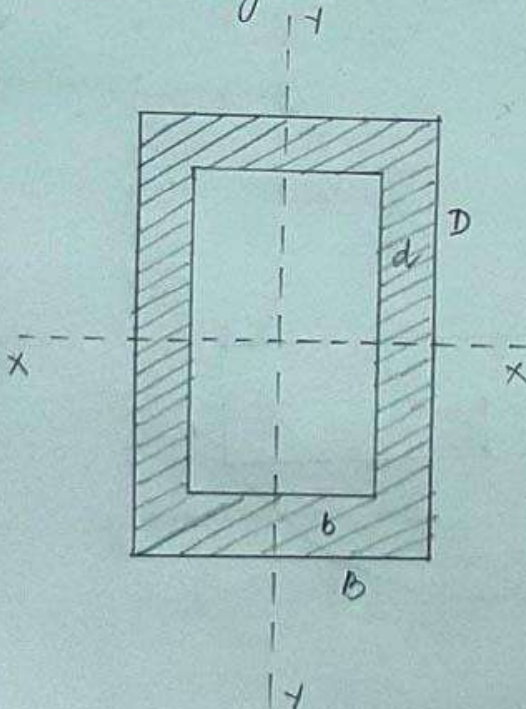
Moment of Inertia about x $\perp$ axis :

$$I_{xx} = \frac{BD^3 - bd^3}{12}$$

Moment of Inertia about y-axis :

$$I_{yy} = \frac{DB^3 - db^3}{12}$$

139
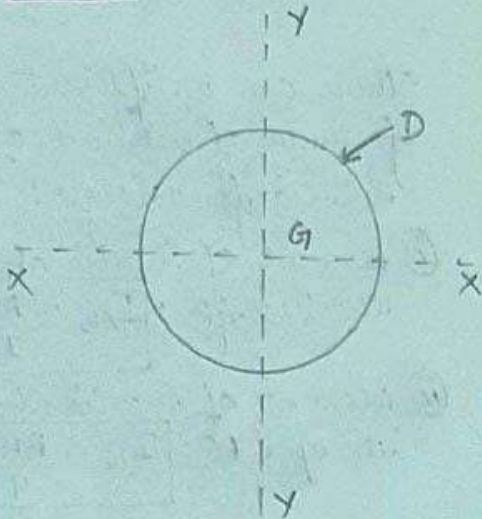
③ Circle:

Moment of Inertia about the x-axis and y-axis will be same, which is given by:

$$I_{XX} = I_{YY} = \frac{\pi}{64} \times D^4$$

④ Hollow Circle:
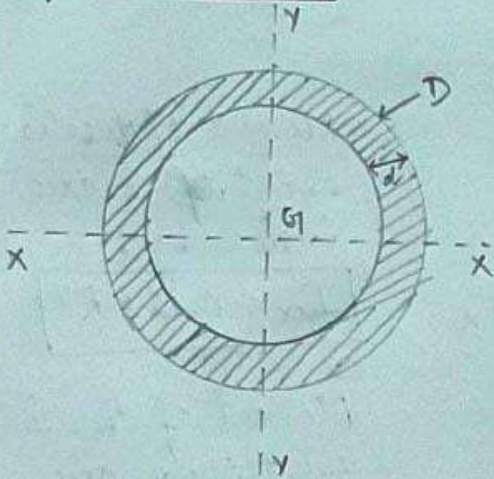
Moment of Inertia about the x-axis and y-axis will be same, which is given by:

$$I_{XX} = I_{YY} = \frac{\pi}{64}(D^4 - d^4)$$

140

106
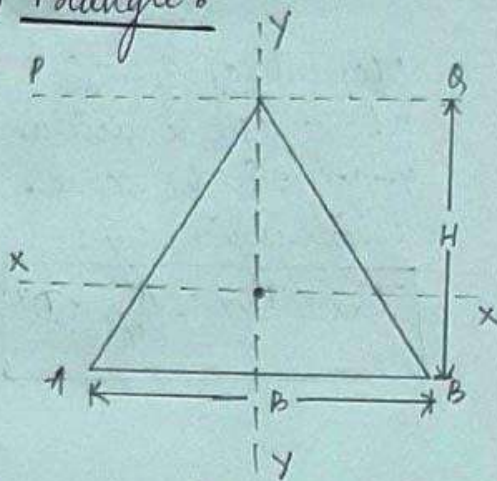
(5) Triangle:

There are 3 different formula of moment of inertia for triangle:

(i) Moment of inertia about the base AB : $I_{AB} = \dfrac{BH^3}{12}$

(ii) Moment of inertia about the apex PQ : $I_{PQ} = \dfrac{BH^3}{4}$

(iii) Moment of Inertia about its centroid i.e : $I_{xx} = \dfrac{BH^3}{36}$

(6) Semi-Circle:
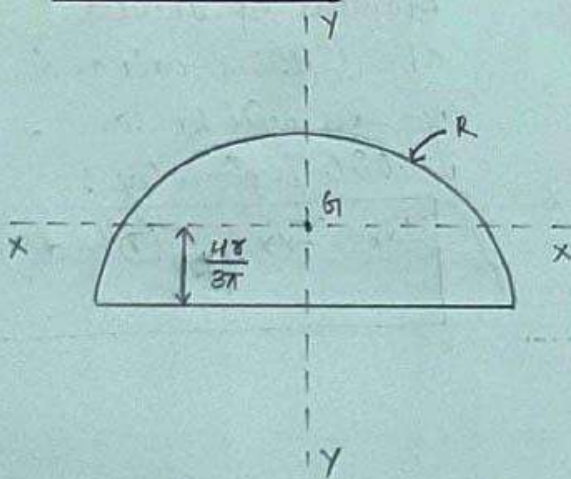
Moment of Inertia about the x-axis is given by :
$I_{xx} = 0.11 \times R^4$

Moment of Inertia about the y-axis is given by :
$I_{yy} = \dfrac{\pi}{8} \times R^4$

$\Rightarrow I_{yy} = 0.393 \times R^4$

141

107

⑦ Quantan Circle :
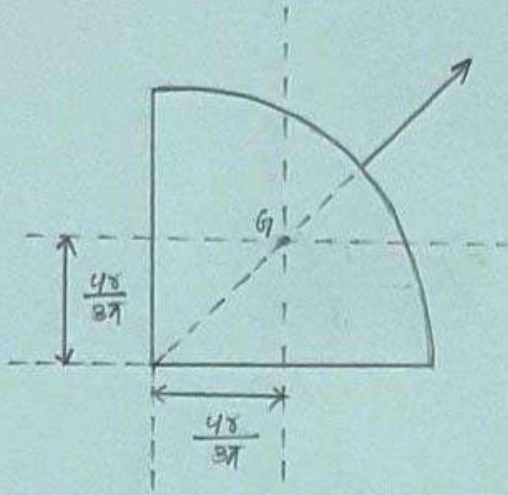
Moment of Inertia about the y-axis same, which is given by :

$$I_{XX} = I_{YY} = \frac{0.11}{2} \times R^4$$

* **Maximum Bending Moment :**

①

$$BM = \frac{Wl}{4}$$

②
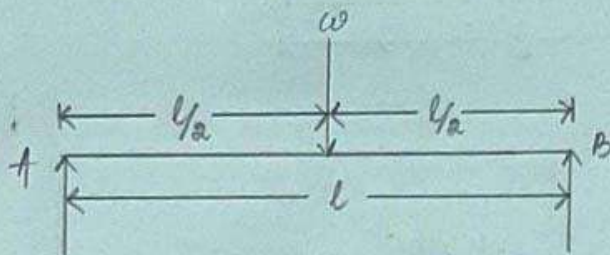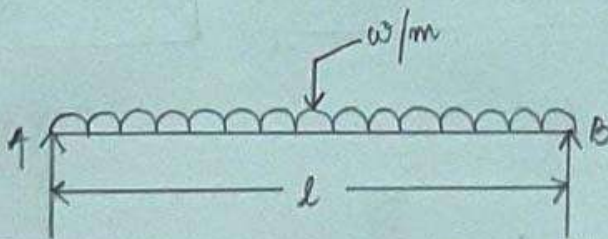
$$BM = \frac{wl^2}{8}$$ (or) $$BM = \frac{Wl}{8}$$ $$\left[\because W = wl \right]$$ where, W is the total load.

$$\Rightarrow \left(\frac{wl}{2} \times \frac{l}{2} \right) - \left(\frac{wl}{2} \times \frac{l}{4} \right)$$

$$\Rightarrow \frac{wl^2}{4} - \frac{wl^2}{8}$$

$$\Rightarrow \frac{2wl^2 - wl^2}{8} \Rightarrow \frac{wl^2}{8}.$$

③
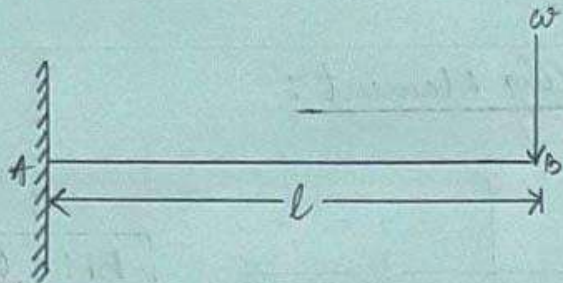
$$\boxed{BM = wl}$$

④
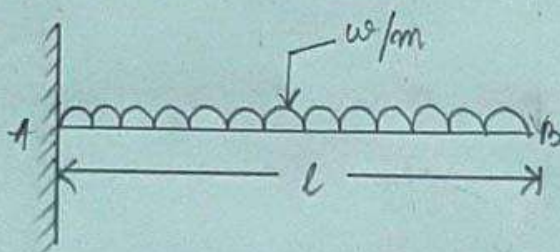
$$\boxed{BM = \frac{wl^2}{2}}$$

⑤
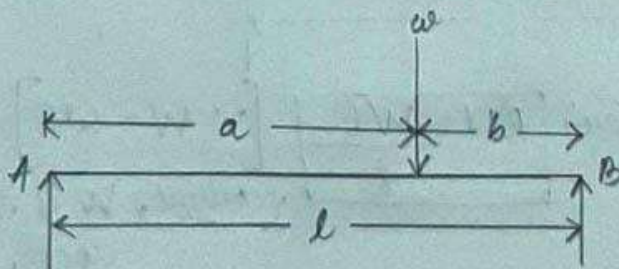
$$\boxed{BM = \frac{wab}{l}}$$

* <u>Neutral Axis:</u>   <u>28.12.21</u>

→ The imaginary axis in a beam which divides a beam into two zones, tension and compression.

→ This is also called <u>zero stress line</u> (or) <u>zero stress axis.</u>

* <u>Section Modulus:</u>

→ It is defined as " The ratio of moment of Inertia about the neutral axis to the distance of the most distant point of the section from the neutral axis."

→ It is denoted as 'z'.

→ Mathematically,

$$Z = \frac{I}{Y_{max}}$$

* <u>Section Modulus for Various Shape of Beam:</u>

① <u>Rectangular Section:</u>

$$Y_{max} = \frac{d}{2}$$

$$I = \frac{bd^3}{12}$$

$$Z = \frac{I}{Y_{max}} = \frac{\frac{bd^3}{12}}{\frac{d}{2}}$$

$$= \frac{bd^2}{12_6} \times \frac{2}{d} \quad \boxed{Z = \frac{bd^2}{6}}$$

144
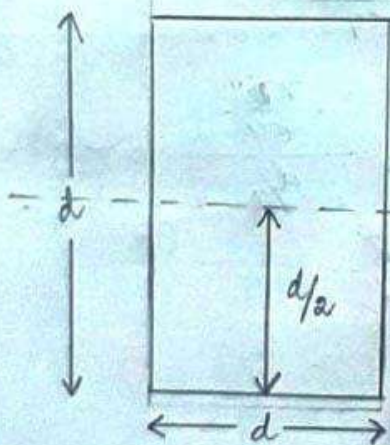
② Hollow Rectangular Section:
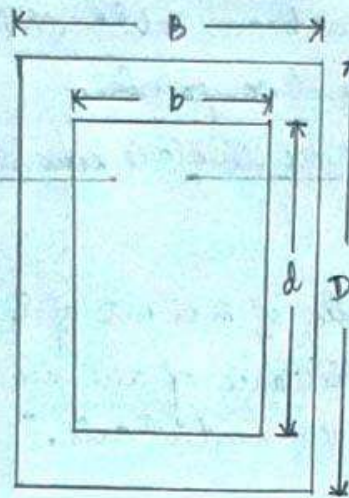
$$Z = \frac{BD^3 - bd^3}{6D}$$

③ Circular Section:

$$Y_{max} = \frac{d}{2}$$

$$I = \frac{\pi d^4}{64}$$

$$Z = \frac{I}{Y_{max}} = \frac{\frac{\pi d^4}{64}}{d/2}$$
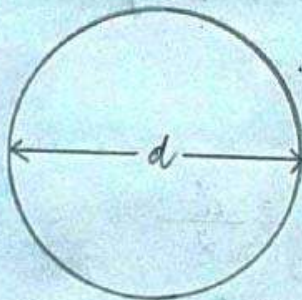
$$\Rightarrow Z = \frac{\pi d^{4^3}}{64_{32}} \times \frac{2}{d}$$

$$\Rightarrow \boxed{Z = \frac{\pi d^3}{32}}$$

④ Hollow Circular Section:

$$Y_{max} = \frac{d}{2}$$

$$I = \frac{\pi(D^4 - d^4)}{64}$$

$$Z = \frac{I}{Y_{max}} = \frac{\frac{\pi(D^4 - d^4)}{64}}{\frac{d}{2}}$$
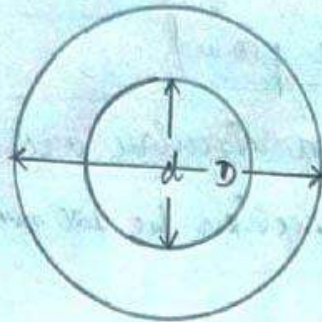
$$\Rightarrow Z = \frac{\pi(D^4 - d^4)}{64} \times \frac{2}{d}$$

$$\Rightarrow \boxed{Z = \frac{\pi(D^4 - d^4)}{32\,d}}$$

* **Relationship Between Maximum Bending Moment (M), Stress (F) and Section Modulus (Z):**

$$\boxed{M = F \times Z}$$

Proof: In pure bending equation,

$$\Rightarrow \frac{M}{I} = \frac{F}{y}$$

$$\Rightarrow \frac{M}{F} = \frac{I}{y}$$

146

112

$$\Rightarrow \frac{M}{I} = Z \qquad \left[\because Z = \frac{I}{y} \right]$$

$$\Rightarrow \boxed{M = F \times Z}$$

hence proved

① A steel plate is bent into a circular arc of radius 10m. If the plane section be 120 mm wide and 20 mm thick.

Find the maximum stress induced and the bending moment which can produce this stress.

Take $E = 2 \times 10^5 \, N/mm^2$.

given:) radius $(R) = 10m$

$$= 10 \times 10^3 \, mm$$

breadth $(B) = 120 \, mm$

thickness $(D) = 20 \, mm$

$E = 2 \times 10^5 \, N/mm^2$.

$$I = \frac{bd^3}{12} = \frac{120 \times (20)^3}{12}$$

$$I = 80000 = 8 \times 10^4 \, mm^4$$

147

113
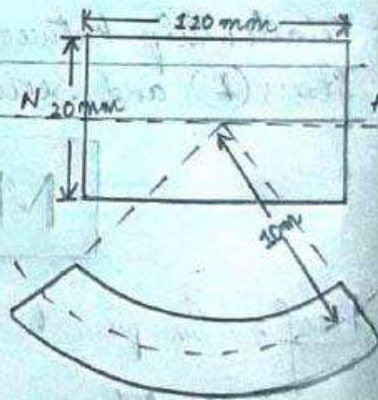
we know for pure bending,

$$\boxed{\frac{M}{I} = \frac{F}{y} = \frac{E}{R}}$$

$\Rightarrow \dfrac{M}{I} = \dfrac{E}{R}$

$\Rightarrow M = \dfrac{E \times I}{R}$

$\Rightarrow M = \dfrac{2 \times 10^5 \times 8 \times 10^4}{10 \times 10^3}$

$\Rightarrow \boxed{M = 1.6 \times 10^6 \ N-mm}$

$\therefore \ Y_{max} = \dfrac{d}{2} = \dfrac{20}{2} = 10 \, mm$

$\dfrac{F}{y} = \dfrac{E}{R}$

$\Rightarrow F = \dfrac{E \times y}{R}$

$\Rightarrow F = \dfrac{2 \times 10^5 \times 10}{10 \times 10^3}$

$\Rightarrow \boxed{f = 200 \ N/_{mm^2}}$

148

114

② A cast iron cantilever of length 1.5m fails when a load of 1920 N is applied at the free end. Determine the stress at failure if the section of cantilever is 40 mm × 60 mm.
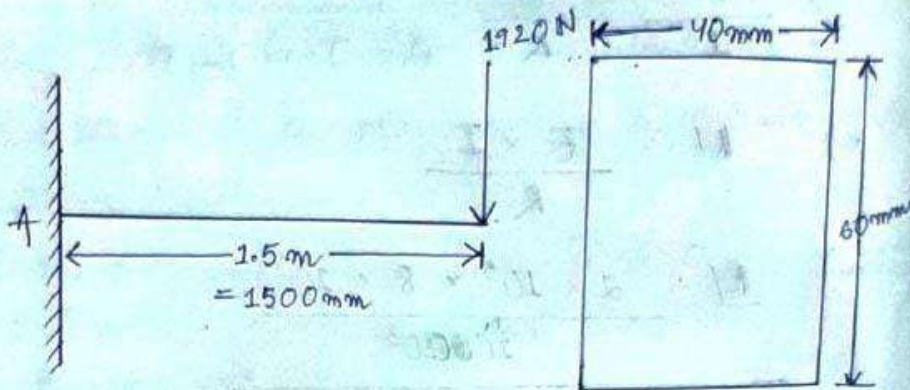
$$BM = \left(1920 \times 1500\right)$$
$$= 2.88 \times 10^6 \text{ N-mm}$$

given) Beam size = 40 mm × 60 mm

point load = 1920 N

length of the beam = 1.5 m

Maximum BM = 2.88 × 10⁶ N-mm

we know, $\boxed{M = fZ}$

$$Z = \frac{bd^2}{6} = \frac{40 \times (60)^2}{6}$$

$$= 24000 \text{ mm}$$

$$= 24 \times 10^3 \text{ mm}$$

149

115

$$f = \frac{M}{Z} = \frac{2.88 \times 10^6}{24 \times 10^3}$$

$$\Rightarrow \boxed{f = 120 \frac{N}{mm^2}}$$

③ A wooden beam 200 mm × 200mm is simply supported on a span of 6m. When the beam is loaded with 14kN load at each $\frac{1}{3}$rd span point, it failed. Find the stress at failure.
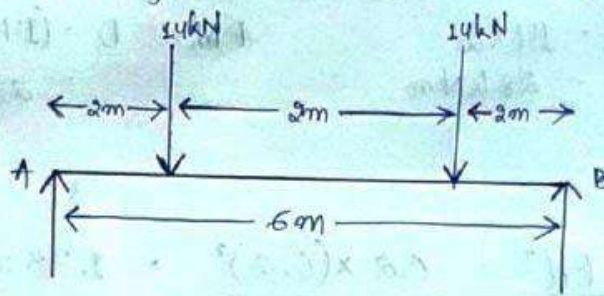
*Reaction Calculation:

Taking moment about $A = 0$

All the clockwise moments $=$ All the anticlockwise moments

$\Rightarrow (14 \times 2) + (14 \times 4) = R_B \times 6$

$\Rightarrow R_B = \frac{(14 \times 2) + (14 \times 4)}{6}$

$\Rightarrow \boxed{R_B = 14kN}$

150

116

$\therefore$ Total load $= 14 + 14$

$\Rightarrow R_A + R_B = 28$

$\Rightarrow R_A = 28 - R_B$

$\Rightarrow R_A = 28 - 14$

$\Rightarrow \boxed{R_A = 14 \, kN}$

(ii) Bending Moment Calculation:

BM at A = 0   BM at B = 0

BM at C = 14×2 BM at D = $(14 \times 4) - (14 \times 2)$

$= 28 \, kNm$ $= 28 \, kNm$

$$Z = \frac{bd^2}{6} = \frac{0.2 \times (0.2)^2}{6} = 1.33 \times 10^{-3} \, m^3$$

$\therefore M = f \times Z$

$\Rightarrow f = \dfrac{M}{Z}$

$\Rightarrow f = \dfrac{28}{1.33 \times 10^{-3}} \dfrac{kN}{m^2}$

$\Rightarrow \boxed{f = 21052.68 \dfrac{kN}{m^3}}$

151

117

④ A steel beam of symmetrical section is subjected to a uniform bending moment which produces a maximum stress of $100\ N/mm^2$. If the beam is 250mm deep. Find the radius to which the longitudinal axis of the beam will be bend.

Take $E = 2 \times 10^5\ N/mm^2$.

Soln⁾ given: $F = 100\ N/mm^2$.

$$d = 250mm$$
$$E = 2 \times 10^5\ N/mm^2.$$

$$Y = \frac{d}{2} = \frac{250}{2} = 125$$

$$\therefore \frac{F}{y} = \frac{E}{R}$$

$$\Rightarrow R = \frac{E \times y}{F}$$

$$\Rightarrow R = \frac{2 \times 10^5 \times 125}{100}$$

$$\Rightarrow R = 25 \times 10^4\ mm$$

$$\Rightarrow R = \frac{25 \times 10^4}{10^3}$$

$$\Rightarrow \boxed{R = 250\ m}$$

⑤ Find the maximum stress produced in a round steel bar 50mm in diameter and 9m leng. Due to its own weight when it is simply supported at its two ends; steel weights at 77000 N/m^3.

Soln:) given:) $d = 50\ mm$

$\qquad\qquad l = 9m$
$\qquad\qquad\quad = 9000\ mm$

$V = A \times l$

$\qquad = \dfrac{\pi}{4}(50)^2 \times 9000$

$\qquad = 17.67 \times 10^6\ mm^3$

specific weight $= 77000\ \dfrac{N}{m^3}$

$\Rightarrow$ $\dfrac{weight}{volume} = 77000\ \dfrac{N}{m^3}$

$\Rightarrow$ $\dfrac{weight}{17.67 \times 10^6\ mm^3} = 77000\ \dfrac{N}{m^3}$

$$\left[\because 77000\ \dfrac{N}{m^3} = \dfrac{77000\ N}{(10^3)^3\ mm^3} \right.$$
$$\left. = 77000 \times 10^{-9}\ \dfrac{N}{mm^3} \right]$$

$$\Rightarrow \quad \frac{weight}{17.67 \times 10^6 \, mm^3} = 77000 \times 10^{-9} \frac{N}{mm^3}$$

$$\Rightarrow \quad weight = 77000 \times 10^{-9} \times 17.67 \times 10^6$$

$$\Rightarrow \boxed{weight = 1360.59 \, N}$$

$$\xrightarrow{\quad} 1360.59 \, N$$

$$A \qquad\qquad\qquad\qquad B$$

$$\xleftarrow{\quad 9m \quad}$$
$$= 9000 \, mm$$
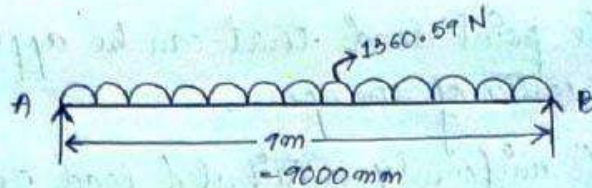
we know,

$$BM = \frac{wl^2}{8}$$

$$= \frac{1360.59 \times (9 \times 10^3)^2}{8}$$

$$\boxed{M = 1.37 \times 10^{10} \, N\text{-}mm}$$

$$\therefore \quad M = f \times Z$$

$$\Rightarrow \quad f = \frac{M}{Z}$$

$$\Rightarrow \quad f = \frac{1.37 \times 10^{10}}{\frac{\pi}{32}(50)^3}$$

$$\Rightarrow \boxed{f = 1.11 \times 10^6 \, N/mm^2}$$

⑥ A simply supported beam of span 10m is 85 350 mm deep. The section of the beam is symmetrical. The moment of inertia of the section is 9.5×10^7 mm⁴. If the permissible bending stress is 120 N/mm², find:

① the safe point load that can be applied at the centre of the span.

② the safe uniformly distributed load that can be applied on the span.

(Neglect the dead load of the beam).

Solu?) given: $l = 10m$
$= 10 \times 10^3$ mm

$d = 350$ mm

$I = 9.5 \times 10^7$ mm⁴

$f = 120$ N/mm²

let $M =$ maximum bending moment for the beam

$$\frac{M}{I} = \frac{f}{y}$$

$$\Rightarrow M = \frac{f \times I}{y}$$

155

$$\Rightarrow M = \dfrac{120 \times 9.5 \times 10^7}{\dfrac{350}{2}}$$

$$\Rightarrow \boxed{M = 65.14 \times 10^6 \ N\text{-}mm}$$

(i)
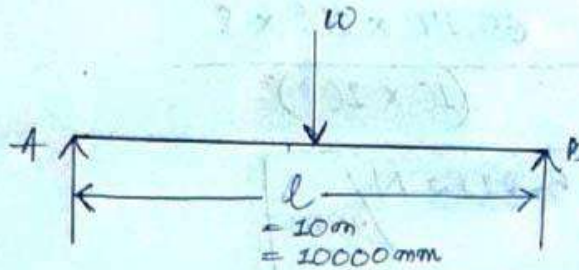
$$M = \dfrac{wl}{4}$$

$$\Rightarrow 65.14 \times 10^6 = \dfrac{W \times (10 \times 10^3)}{4}$$

$$\Rightarrow W = \dfrac{65.14 \times 10^6 \times 4}{10 \times 10^3}$$

$$\Rightarrow \boxed{W = 26.056 \times 10^3 \ N}$$

$$\Rightarrow \boxed{W = 26.056 \ kN}$$

(ii)
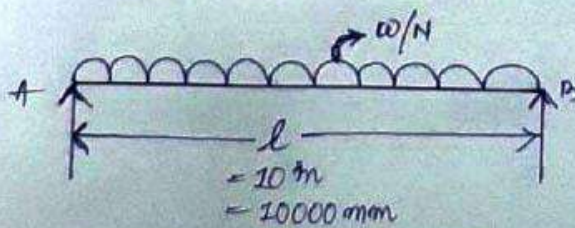
$$M = \frac{Wl^2}{8}$$

$$\Rightarrow 65.14 \times 10^6 = \frac{W \times (10 \times 10^3)^2}{8}$$

$$\Rightarrow W = \frac{65.14 \times 10^6 \times 8}{(10 \times 10^3)^2}$$

$$\Rightarrow \boxed{W = 5.2112 \, N/mm}$$

$$\Rightarrow W = \frac{5.2112}{10^3 \times 10^{-3}} \, \frac{kN}{m}$$

$$\Rightarrow \boxed{W = 5.2112 \, \frac{kN}{m}}$$

* Derivation:

STRESSES IN SHAFTS DUE TO TORSION

* <u>Torsion:</u> The twisting of a body by the application of the forces tending to turn one end or part, about a longitudinal axis while the other end is held fixed or turned in the opposite direction is known as torsion.

* <u>Pure Torsion:</u> The shaft of circular section is said to be impure torsion when it is subjected to equal and opposite end couples whose axis coincide with the axis of the shaft.

179

124
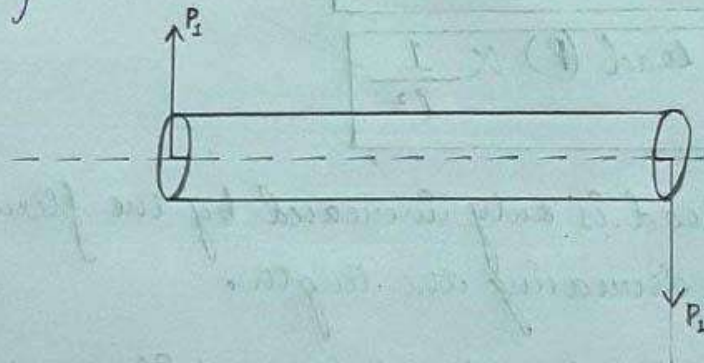
* Assumption of pure torsion:

① The material of the shaft is uniform throughout.

② The twist along the shaft is uniform.

③ The shaft is of uniform circular section throughout.

④ Cross-section of the shaft which are plain before twist, remain plain after twist.

⑤ All the radii which are straight before twist, remain straight after twist.

① **Torsion of Solid shaft:**

$$T = \frac{\pi}{16} \, \tau \times D^3$$

where, $T \rightarrow$ torque transmitted by shaft

$\tau \rightarrow$ shearing stress in N/mm^2.

$D \rightarrow$ diameter of shaft

② **Torsion of Hollow shaft:**

$$T = \frac{\pi}{16} \, \tau \left[\frac{D^4 - d^4}{D} \right]$$

where, $D \rightarrow$ external diameter

$d \rightarrow$ internal diameter

180

125

① A hollow shaft of external and internal diameter 80mm and 50mm respectively is required to transmit torque from one end to the other end. what is the safe torque it can be transmit of the allowable shear stress in 45 MPa.

given:) $d = 50mm$
            $D = 80mm$
            $\tau = 45\ MPa$
            $= 45\ N/mm^2$

$$T = \frac{\pi}{16}\ \tau \left[\frac{D^4 - d^4}{D}\right]$$

$$\Rightarrow T = \frac{\pi}{16} \times (45) \times \left[\frac{(80)^4 - (50)^4}{80}\right]$$

$$\Rightarrow \boxed{T = 3.83 \times 10^6\ N\text{-}mm}$$

$$\Rightarrow T = \frac{3.83 \times 10^6}{10^3 \times 10^3}\ kN\text{-}m$$

$$\Rightarrow \boxed{T = 3.83\ kN\text{-}m}$$

181

126

② A steel shaft is to transmit a torque of 10 kNm. If the shearing stress is not to exceed 45 MPa, find the maximum diameter of the shaft.

given :) $T = 10\ kNm$

$\quad = 10 \times 10^3 \times 10^3\ Nmm$

$\quad = 10^7\ Nmm$

$\tau = 45\ MPa$

$\quad = 45\ N/mm^2$

$\therefore\ T = \dfrac{\pi}{16}\ \tau \times D^3$

$\Rightarrow\ D^3 = \dfrac{T \times 16}{\pi \times \tau}$

$\Rightarrow\ D^3 = \dfrac{10^7 \times 16}{\pi \times 45}\ mm$

$\Rightarrow\ D^3 = 1.13 \times 10^6\ mm$

$\Rightarrow\ D = \sqrt{1.13 \times 10^6}\ mm$

$\Rightarrow\ \boxed{D = 104.15\ mm}$

$\Rightarrow\ D = \dfrac{104.15}{10^3}\ m$

$\Rightarrow\ \boxed{D = 0.10415\ m}$

* **Power Transmitted by a shaft :**

$$\boxed{P = \dfrac{2\pi NT}{60}}$$

→ Its unit :
  w, kW
  (watt, kilowatt)

where, P → power transmitted

N → no. of revolution per minute

T → average torque

182

127

① A circular shaft of 60 mm diameter is rotating at 150 rpm. If the shear stress is not to exceed 50 MPa, find the power which can be transmitted by the shaft.

given:) $d = 60$ mm

$N = 150$ rpm

$Z = 50$ MPa

$= 50$ N/mm²

$$T = \frac{\pi}{16} Z d^3$$

$$\Rightarrow T = \frac{\pi}{16} \times (50) \times (60)^3$$

$$\Rightarrow \boxed{T = 2.12 \times 10^6 \text{ N-mm}}$$

$$\therefore P = \frac{2\pi N T}{60}$$

$$\Rightarrow P = \frac{2\pi \times 150 \times 2.12 \times 10^6}{60}$$

$$\Rightarrow \boxed{P = 33.3 \times 10^6 \text{ N-m}}$$

183

$$\Rightarrow P = \frac{33.3 \times 10^{-6}}{10^3 \times 10^3} \quad kN\text{-}m$$

$$\Rightarrow \boxed{P = 33.3 \; kN\text{-}m}$$

② A hollow shaft of external and internal diameter as 100mm and 40mm respectively is transmitting power at 120rpm. Find the power that the shaft can transmit, if the shearing stress is not to exceed 50 MPa.

given:) $D = 100\,mm$ $N = 120\,rpm$
$d = 40\,mm$ $\tau = 50\,MPa$
               $= 50\,N/mm^2$

$$T = \frac{\pi}{16} \times \tau \left[\frac{D^4 - d^4}{D} \right]$$

$$\Rightarrow T = \frac{\pi}{16} (50) \times \left[\frac{(100)^4 - (40)^2}{100} \right]$$

$$\Rightarrow \boxed{T = 9.56 \times 10^6 \; N\text{-}mm}$$

$$\therefore P = \frac{2\pi NT}{60}$$

$$\Rightarrow P = \frac{2\pi \times 120 \times 9.56 \times 10^6}{60}$$

$$\Rightarrow \boxed{P = 120.13 \times 10^6 \ W}$$

③ A solid circular shaft of 100 mm diameter is transmitting 120 kW power at 150 rpm. Find the intensity of shear stress in the shaft.

given:) $d = 100 \ mm$

$N = 150 \ rpm$

$P = 120 \ kW$

$= 120 \times 10^3 \ W$

$$P = \frac{2\pi NT}{60}$$

$$\Rightarrow T = \frac{P \times 60}{2\pi N}$$

$$\Rightarrow T = \frac{120 \times 10^3 \times 60}{2\pi \times 150}$$

$$\Rightarrow \boxed{T = 7.63 \times 10^3 \ N\text{-}mm}$$

185

130

$$\therefore \ T = \frac{\pi}{16} \ \tau \ d^3$$

$$\Rightarrow \ \tau = \frac{T \times 16}{\pi \times d^3}$$

$$\Rightarrow \ \tau = \frac{7.63 \times 10^3 \times 16}{\pi \times (100)^3}$$

$$\Rightarrow \boxed{\tau = 38.85 \times 10^{-3} \ N/mm^2}$$

or

$$\Rightarrow \boxed{\tau = 0.03 \ N/mm^2}$$

* Polar Moment of Inertia :

The moment of inertia of a plane area with respect to an axis perpendicular to the plane of the figure is called polar moment of inertia with respect to the point where the axis interporte the plane.

→ In a circular plane, the point is always the centre of the circle.

→ Polar moment of inertia is denoted as 'J'.

186

$$\boxed{J = \frac{\pi}{32} D^4}$$ and $$\boxed{J = \frac{\pi}{32}\left(D^4 - d^4\right)}$$

(for solid shaft)   (for hollow shaft)

* Polar Modulus (Z_p) :

The ratio of polar moment of inertia of a shaft section to the maximum radius is called polar modulus.

mathematically,

$$\boxed{Z_p = \frac{\pi}{16} D^3}$$ and $$\boxed{Z_p = \frac{\pi}{16D}\left[D^4 - d^4\right]}$$

(for solid shaft)   (for hollow shaft)

① A solid steel shaft has to transmit 100 kN power at 160 rpm.

Taking allowable shear stress as 70 MPa, find the suitable diameter of the shaft.

The maximum torque transmitted in each revolution exceeds the torque by 20%.

given :) P = 100 kW

       N = 160 rpm

       τ = 70 MPa

187

132

the maximum torque transmitted in each revolution exceeds the torque by 20% :

So, $2T = T + 20\%$ of T

$$\Rightarrow T = \left(1 + \frac{20}{100}\right) T$$

$$\Rightarrow T = (1 + 0.2) T$$

$$\Rightarrow T = 1.2 T$$

$$P = \frac{2\pi N T}{60}$$

$$\Rightarrow 100 = \frac{2\pi \times 160 \times 1.2 T}{60}$$

$$\Rightarrow T = \frac{100 \times 60}{2\pi \times 160 \times 1.2}$$

$$\Rightarrow \boxed{T = 4.97 \ kN\text{-}m}$$

$$T = \frac{\pi}{16} \ \tau D^3$$

$$\Rightarrow 4.97 = \frac{\pi}{16} \times 70 \times 10^6 \times D^3$$

$$\Rightarrow D^3 = \frac{4.97 \times 16}{\pi \times 70 \times 10^6} \ mm$$

$$\Rightarrow D^3 = 3.61 \times 10^{-7} \ mm$$

$$\Rightarrow D = \sqrt[3]{3.61 \times 10^{-7}} \ mm$$

$$\Rightarrow \boxed{D = 7.12 \times 10^{-3} \ mm}$$

188

133

* Torsional Equation:

$$\frac{T}{J} = \frac{C\theta}{l}$$

where, $T \rightarrow$ torque transmitted

$J \rightarrow$ polar moment of inertia

$C \rightarrow$ modulus of rigidity

$\theta \rightarrow$ angle of twist

$l \rightarrow$ length of the shaft

* Torsional Rigidity:

It is defined as "The torque required to produce a twist of one radian per unit length of the given shaft."

$\rightarrow$ It is given by, $\boxed{T = CJ}$

where, $CJ \rightarrow$ torsional rigidity

189

134

1) Find the maximum torque that can be applied safely to a shaft of 300 mm diameter. The angle of twist is $1.5°$ in length of 7.5m and the shear stress is not to exceed 42 N/mm^2. Take $C = 84.4 \text{ kN/mm}^2$.

given:) $d = 300 \text{ mm}$
$\quad\quad C = 84.4 \text{ kN/mm}^2$
$\quad\quad\quad = 84.4 \times 10^3 \text{ N/mm}^2$

$\tau = 42 \text{ N/mm}^2$
$l = 7.5 \text{ m}$
$\quad = 7.5 \times 10^3 \text{ mm}$

$\theta = 15°$
$\quad = \dfrac{\pi}{180} \times 15° \text{ rad}$
$\quad = 0.026 \text{ rad}$

$$\left[\because \; \theta \to rad = \dfrac{\pi}{180} \right]$$

$$J = \dfrac{\pi}{32} d^4$$

$$\Rightarrow J = \dfrac{\pi}{32} \times (300)^4$$

$$\Rightarrow \boxed{J = 795.21 \times 10^6}$$

$$\therefore \dfrac{T}{J} = \dfrac{C\theta}{l}$$

$$\Rightarrow T = \dfrac{C\theta J}{l}$$

190

135

$$\Rightarrow T = \frac{84 \times 10^3 \times 0.026 \times 795.21 \times 10^9}{7.5 \times 10^3}$$

$$\Rightarrow \boxed{T = 231.56 \times 10^6 \ N\text{-}mm}$$

② A bar of magnesium alloy 28 mm in diameter was tested in a gauge length of 25 cm in tension and torsion. The tensile load of 5000 N produced an extension of 0.4 mm and a torque of 1250 N-cm produce a twist of 1.51°.

Determine the Young's Modulus (E), Bulk Modulus (K), Modulus of Rigidity (C) and the poisson's ratio (μ).

given:) $d = 28 mm$
$\quad = 2.8 cm$

$\Delta l = 0.4 mm$
$\quad = 0.04 cm$

$P = 5000 N$

$l = 25 cm$

$T = 1250 \ N\text{-}cm$

$\theta = 1.51°$
$\quad = \frac{\pi}{180} \times 1.51° \ rad.$
$\quad = 0.026 \ rad$

① $\Delta l = \frac{Pl}{AE} \Rightarrow E = \frac{Pl}{A \Delta l}$

191

136

$$\Rightarrow E = \frac{5000 \times 25}{\frac{\pi}{4}(2.8)^2 \times 0.04}$$

$$\Rightarrow \boxed{E = 507.50 \times 10^3 \ N/cm^2}$$

$$J = \frac{\pi}{32} d^4$$

$$\Rightarrow J = \frac{\pi}{32}(2.8)^4$$

$$\Rightarrow \boxed{J = 6.03 \ mm^4}$$

(ii) Modulus of rigidity (C):

$$\frac{T}{J} = \frac{C\theta}{l}$$

$$\Rightarrow C = \frac{Tl}{J\theta}$$

$$\Rightarrow C = \frac{1250 \times 25}{6.03 \times 0.026}$$

$$\Rightarrow \boxed{C = 199.32 \times 10^3 \ N/cm^2}$$

192

137

(iii) **Poisson's Ratio** $\left(\mu \text{ ou } \frac{1}{m} \right)$:

$$E = 2C \left(1 + \frac{1}{m} \right)$$

$$\Rightarrow \frac{E}{2C} = 1 + \frac{1}{m}$$

$$\Rightarrow \frac{1}{m} \text{ ou } \mu = \frac{E}{2C} - 1$$

$$\Rightarrow \mu = \frac{507.50 \times 10^3}{2(199.32 \times 10^3)} - 1$$

$$\Rightarrow \boxed{\mu = 0.27}$$

(iv) **Bulk Modulus** (K):

$$E = 3K \left(1 - \frac{2}{m} \right)$$

$$\Rightarrow E = 3K \left(1 - 2 \cdot \frac{1}{m} \right)$$

$$\Rightarrow E = 3K \left(1 - 2 \cdot \mu \right)$$

$$\Rightarrow E = 3K \left(1 - (2 \times 0.27) \right)$$

193

138

$\Rightarrow) \quad E = 3K \times 0.46$

$\Rightarrow) \quad K = \dfrac{E}{3 \times 0.46}$

$\Rightarrow) \quad K = \dfrac{507.50 \times 10^3}{3 \times 0.46}$

$\Rightarrow) \quad K = 367.75 \times 10^3 \ N/cm^2$

194

139

Chapter 4

COLUMN

AND STRUTS

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CHAPTER-7

COLUMNS AND STRUTS

* **Columns:**

→ It is a vertical member made up of brick RCC, steel, timber which is only subjected to compression.

→ It takes loads from neither lintel or roof slab.

* **Types of Columns:**

Columns may be of 2 types:

① Short Column
② Long Column

① **Short Column:**

A column is said to be a short column if the ratio of effective length to its least lateral dimension is less than or equal to 12.

(OR)

A column is said to be a short column when the ratio of effective length to the least radius of gyration is less than 45. (i.e. slenderness ratio)

A column is said to be a short column when the length of the column is less as compared to its cross-sectional dimension.

② Long Column :

A column is said to be a long column if the ratio of effective length to its least lateral dimension is not less than 12.

<div align="center">(OR)</div>

A column is said to be a long column if the ratio of effective length to the least radius of gyration is greater than 45 (i.e. slenderness ratio)

✳ Slenderness Ratio :

→ It is defined as "The ratio of effective length to least radius of gyration".

→ It is denoted by 'λ' (lambda) and mathematically,

$$\lambda = \frac{L_{eff}}{K}$$

where, $K \to$ least radius of gyration
$L_{eff} \to$ effective length.

* <u>Effective length of a column :</u> (2marks or 6marks)
This is the distance between the successive points of zero moment.

* <u>Effective length of column for various end condition :</u> $\boxed{\text{Actual length} = l}$

<u>Case : 1</u> → <u>Both ends hinged</u>
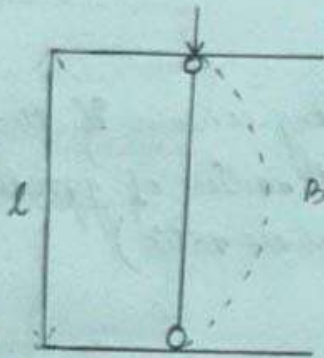
l Buckling $\boxed{L_{eff} = l}$

<u>Case : 2</u> → <u>One end fixed, one end free</u>

l $\boxed{L_{eff} = 2l}$

161

143
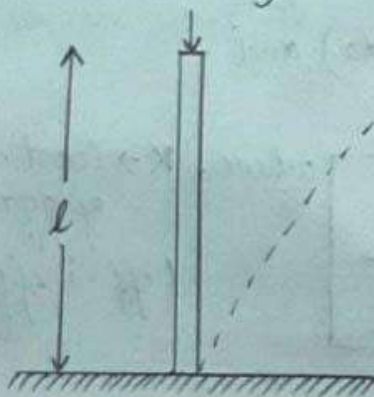
Case:3 → Both ends fixed
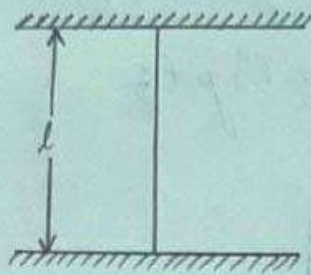
$$L_{eff} = \frac{l}{2}$$

Case:4 → One end fixed, one end hinged
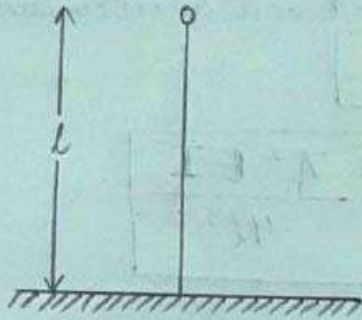
$$L_{eff} = \frac{l}{\sqrt{2}}$$

* Euler's Theory of Long Column: 29.12.21

Euler's Theory of long column is given by:

$$P = \frac{\pi^2 EI}{(l_{eff})^2}$$

where, P → cripling load
       E → Young's Modulus
       I → Moment of Inertia
       l_eff → effective length

162

→ Euler's theory of long column is based on different end conditions of column are given below:

Case 1: When both ends are hinged;

$$\boxed{l_{eff} = l}$$

$$\boxed{P = \frac{\pi^2 EI}{l^2}}$$

$\leftarrow \beta = 3 \, cm$

Case 2: When one end fixed and another end is free; $\boxed{l_{eff} = 2l}$

$$P = \frac{\pi^2 EI}{(2l)^2} \Rightarrow \boxed{P = \frac{\pi^2 EI}{4l^2}}$$

Case 3: When both ends are fixed;

$$\boxed{l_{eff} = \frac{l}{2}}$$

$$P = \frac{\pi^2 EI}{(l/2)^2} \Rightarrow P = \frac{\pi^2 EI}{\left(\frac{l^2}{4}\right)}$$

$$\Rightarrow \boxed{P = \frac{4\pi^2 EI}{l^2}}$$

163

145

Case 4 : When one end is fixed and another is hinged ;

$$l_{eff} = \frac{l}{\sqrt{2}}$$

$$P = \frac{\pi^2 EI}{\left(\frac{l}{\sqrt{2}}\right)^2} \implies P = \frac{\pi^2 EI}{\left(\frac{l^2}{2}\right)}$$

$$\implies P = \frac{2\pi^2 EI}{l^2}$$

Questions :

① A mild steel tube 4m long, 30mm internal diameter and 4mm thick is used as a column with both ends hinged. Find the collapsing load.

Take $E = 2.1 \times 10^5 \, N/mm^2$.

given :) $l = 4m$
$$= 4000 \, mm = 4 \times 10^3 \, mm$$

(d) internal diameter $= 30mm$

$E = 2.1 \times 10^5 \, N/mm$

thickness $= 4mm$
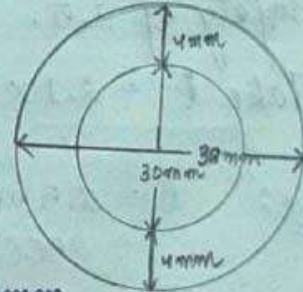
(D) external diameter $= 30mm + 4mm$
$$+ 4mm$$
$$= 38 \, mm$$

$$I = \frac{\pi}{64}(D^4 - d^4)$$

$$= \frac{\pi}{64}\left((38)^4 - (30)^4\right) \implies I = 62.59 \times 10^3 \, mm^4$$

$$\therefore P = \dfrac{\pi^2 EI}{l^2}$$

$$\Rightarrow P = \dfrac{\pi^2 \times 2.1 \times 10^5 \times 62.59 \times 10^3}{(4 \times 10^3)^2}$$

$$\Rightarrow \boxed{P = 8.1 \, N}$$

$$\Rightarrow \boxed{P = 8.1 \times 10^{-3} \, kN}$$

② A strut 2.5 m long is 60 mm in diameter, One end of strut is fixed while its other end is hinged.

Find the safe compressive load for the member using Euler's formula, allowing a factor of safety of 3.5.

Take $E = 2.1 \times 10^5 \, N/mm^2$

given: $l = 2.5 \, m$
$\qquad = 2500 \, mm$
$\qquad d = 60 \, mm$

$$I = \dfrac{\pi}{64} d^4 = \dfrac{\pi}{64} (60)^4$$

$$\Rightarrow I = 636.17 \times 10^3 \, mm^4$$

$$P = \frac{2\pi^2 EI}{l^2}$$

$$\Rightarrow P = \frac{2 \times \pi^2 \times 2.1 \times 10^5 \times 636.17 \times 10^3}{(2500)^2}$$

$$\Rightarrow \boxed{P = 421.93 \times 10^3 \, N}$$

$$\Rightarrow \boxed{P = 421.93 \, kN}$$

∴ Safe Permissible load

$$= \frac{P}{\text{factor of safety}} = \frac{421.93}{3.5}$$

$$\Rightarrow \boxed{SPL = 120.55 kN}$$

③ A solid circular compressive member 50mm in diameter is to be replaced by a hollow circular section of the same material.
Find the size of the hollow section if the internal diameter is 0.6 times the external diameter.

given:) A solid circular compression member of
diameter = 50mm

let the external diameter of the hollow circular section = 'x' mm

166

148

then we have the internal diameter of the hollow circular section = $0.6 \times x$ mm

Therefore due to same moment of inertia:

$$I_{solid} = I_{hollow}$$

$$\Rightarrow \frac{\pi}{64}(d)^4 = \frac{\pi}{64}(D^4 + d^4)$$

$$\Rightarrow d^4 = D^4 - d^4$$

$$\Rightarrow (50)^4 = x^4 - (0.6x)^4$$

$$\Rightarrow (50)^4 = x^4 - 0.1296\, x^4$$

$$\Rightarrow (50)^4 = x^4(1 - 0.1296)$$

$$\Rightarrow \frac{(50)^4}{(1 - 0.1296)} = x^4$$

$$\Rightarrow x^4 = \frac{(50)^4}{1 - 0.1296}$$

167

149

$$\Rightarrow x^4 = 7.18 \times 10^6$$

$$\Rightarrow x = \sqrt[4]{7.18 \times 10^6}$$

$$\Rightarrow \boxed{x = 51.76 \text{ mm}}$$

$\therefore$ internal diameter $= 0.6 \times x$

$$\Rightarrow d = 0.6 \times 51.76$$

$$\Rightarrow \boxed{d = 31.06 \text{ mm}}$$

168

150

④ Find the shortest length (l) for a pin ended steel column having a cross-section of 60mm × 100mm for which Euler's formula applies.

Take $E = 2 × 10^5$ N/mm² and critical proportional limit is 250 N/mm².

given:) cross-section of a steel column:
$$= 60mm × 100mm$$

$$E = 2 × 10^5 \, N/mm²$$

critical proportional limit = 250 N/mm².
(or) critical stress

$$critical \ stress = \frac{critical \ load (P)}{Area}$$

$$=) \quad 250 = \frac{P}{60 × 100}$$

$$\Rightarrow P = 250 \times 60 \times 100$$

$$\Rightarrow \boxed{P = 1.5 \times 10^6 \, N}$$

For both ends hinged, $\boxed{l_{eff} = l}$

$$P = \frac{\pi^2 E I}{l^2}$$

$$\Rightarrow l^2 = \frac{\pi^2 E I}{P}$$

$$\Rightarrow l^2 = \frac{\pi^2 \times 2 \times 10^5 \times 100 \times (60)^3}{1.5 \times 10^6 \times 12}$$

$$\Rightarrow l^2 = 2.36 \times 10^6$$

$$\Rightarrow l = \sqrt{2.36 \times 10^6} \, mm$$

$$\Rightarrow \boxed{l = 1.53 \times 10^3 \, mm}$$

$$\Rightarrow \boxed{l = 1.53 \, m}$$

⑤ A hollow alloy tube 4m long with external diameter of 40mm and internal diameter of 25mm respectively, was found to extend 4.8mm under a tensile load of 80 kN. Find the buckling load for the tubes with both ends pinned.

Also find the safe load on the tube, taking a factor of safety of 5.

given:) $l = 4000 mm = 4 \times 10^3 mm$

$D = 40 mm$

$d = 25 mm$

$A = \dfrac{\pi}{4}(D^2 - d^2)$

$\Rightarrow A = \dfrac{\pi}{4}\left((40)^2 - (25)^2\right)$

$\Rightarrow \boxed{A = 765.76 \ mm^2}$

$I = \dfrac{\pi}{64}(D^4 - d^4)$

$\Rightarrow I = \dfrac{\pi}{64}\left((40)^4 - (25)^4\right)$

$\Rightarrow \boxed{I = 106.48 \times 10^3 \ mm^4}$

171

153

$$\Delta l = 4.8 \, mm \quad (given)$$

we know, $E = \dfrac{\sigma}{e}$

$$\sigma (stress) = \dfrac{P}{A}$$

$$\Rightarrow \sigma = \dfrac{60 \times 10^3}{765.76} \dfrac{N}{mm^2}$$

$$\Rightarrow \boxed{\sigma = 76.35 \, N/mm^2}$$

$$e(strain) = \dfrac{\Delta l}{l}$$

$$\Rightarrow e = \dfrac{4.8}{4 \times 10^3}$$

$$\Rightarrow \boxed{e = 1.2 \times 10^{-3}}$$

$$\therefore E = \dfrac{\sigma}{e} = \left(\dfrac{76.35}{1.2 \times 10^{-3}} \right) \dfrac{N}{mm^2}$$

$$\Rightarrow \boxed{E = 65.29 \times 10^3 \dfrac{N}{mm^2}}$$

172

154

① $P = \dfrac{\pi^2 E I}{l^2}$

$\Rightarrow) P = \dfrac{\pi^2 \times 65.29 \times 10^3 \times 106.48 \times 10^3}{(4 \times 10^3)^2}$

$\Rightarrow) \boxed{P = 4.28 \times 10^3 N}$

② Safe load $= \dfrac{P}{\text{factor of safety}} = \dfrac{P}{5}$

$= \dfrac{4.28 \times 10^3}{5}$

$\Rightarrow) \boxed{\text{safe load} = 856 \, N}$

$\Rightarrow) \boxed{\text{safe load} = 0.856 \, kN}$

⑥ A `T' section 150 mm × 120mm × 20 mm is used as a strut of 4m long with hinged at its both ends. Calculate the crippling load, if the Young's Modulus for the material is 200 GPa.

given) $E = 200 \, GPa$
$= 200 \times 10^9 \dfrac{N}{m^2}$
$= 200 \times 10^9 \dfrac{N}{10^6 mm^2}$

$\Rightarrow) E = 200 \times 10^{9-6} \dfrac{N}{mm^2}$

$\Rightarrow) E = 200 \times 10^3 \dfrac{N}{mm^2}$

Centre of Gravity:
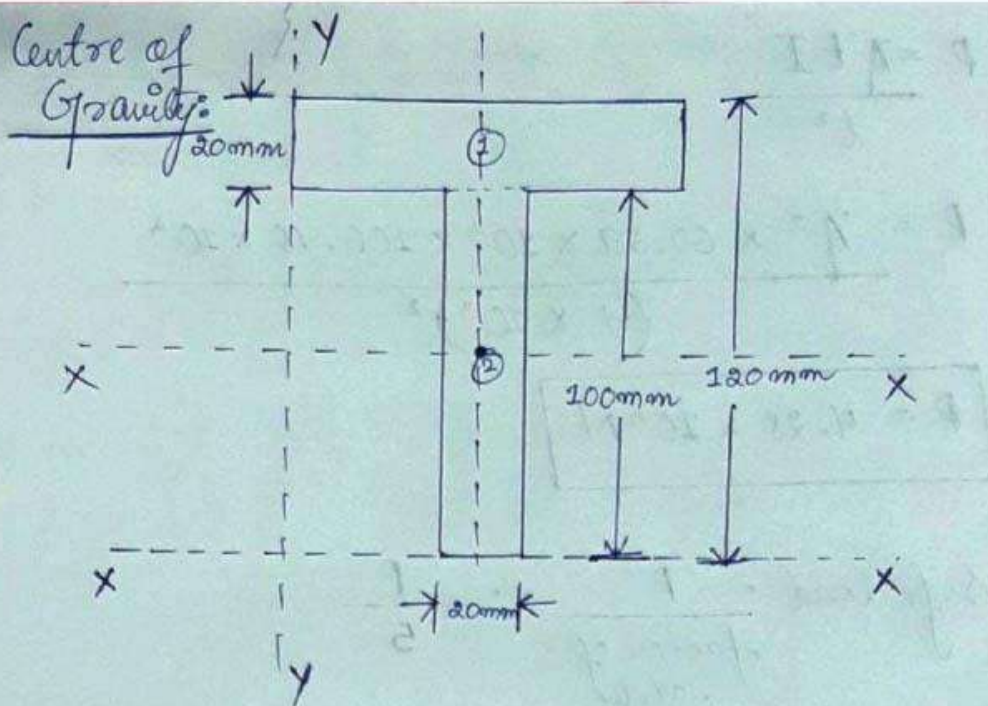
Section ①

$a_1 = 150 \times 20 = 3000 \, mm^2$

$x_1 = \dfrac{150}{2} = 75 \, mm$

$y_1 = \dfrac{20}{2} + 100 = 110 \, mm$

Section ②

$a_2 = (120 - 20) \times 20 = 2000 \, mm^2$

$x_2 = \dfrac{150}{2} = 75 \, mm$

$y_2 = \dfrac{100}{2} = 50 \, mm$

174

156

$$\overline{y} = \frac{a_1 y_1 + a_2 y_2}{a_1 + a_2}$$

$$\Rightarrow \overline{y} = \frac{(3000 \times 110) + (2000 \times 50)}{3000 + 2000}$$

$$\Rightarrow \boxed{\overline{y} = 86 \, mm}$$

$$\overline{x} = \frac{a_1 x_1 + a_2 x_2}{a_1 + a_2}$$

$$\Rightarrow \overline{x} = \frac{(3000 \times 75) + (2000 \times 75)}{3000 + 2000}$$

$$\Rightarrow \boxed{\overline{x} = 75 \, mm}$$

Moment of Inertia:

* Radius of Gyration:

→ The radius of gyration of a body about an axis of rotation is defined as "The radial distance of a point from the axis of rotation at which if the whole mass of the body is assumed to be concentrated."

→ Its moment of inertia about the given axis would be the same as with its actual distribution of mass.

Note:

① We know : $P = \dfrac{\pi^2 EI}{l^2}$

where, $EI \rightarrow$ flexural rigidity

mass of the body is concentrated
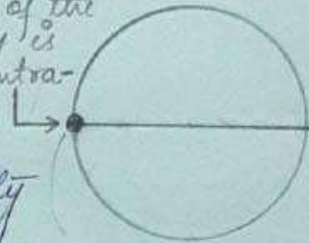
② So, $\boxed{\text{crippling load}\,(P) \propto EI}$

→ $\boxed{\text{crippling load}\,(P) \propto \dfrac{1}{l^2}}$

→ critical load is only increased by the flexural rigidity, decreasing the length.

③ Euler's Formula should not be used if the length of the member is less than 99.34 times the least radius of gyration.

177

* <u>Struts</u>: It is a composite member of truss which is subjected to compressive load or compression.

Chapter 5

SHEAR FORCE AND BENDING MOMENT .

CHAPTER - 4

* Types of loads and beams:

Beam:

→ These are horixontal structural element that withstand vertical loads, shear forces and bending moments.

→ Beams transfer loads that impacts along their length to their end points such as walls, columns, foundations, etc.

* Based on support condition, beams are classified as:

① Simply Supported Beam   ⑥ Propped Cantilever Beam

② Fixed Beam

③ Cantilever Beam

④ Continuous Beam

⑤ Overhanging Beam

① <u>Simply Supported Beam:</u>

→ It is one of the simplest structure elements that both end rest on supports but it is free to rotate.

→ It contains (pinned) support at one end and a roller support at the other end.

_{hinge}

pinned
on
hinged
support.

roller
support
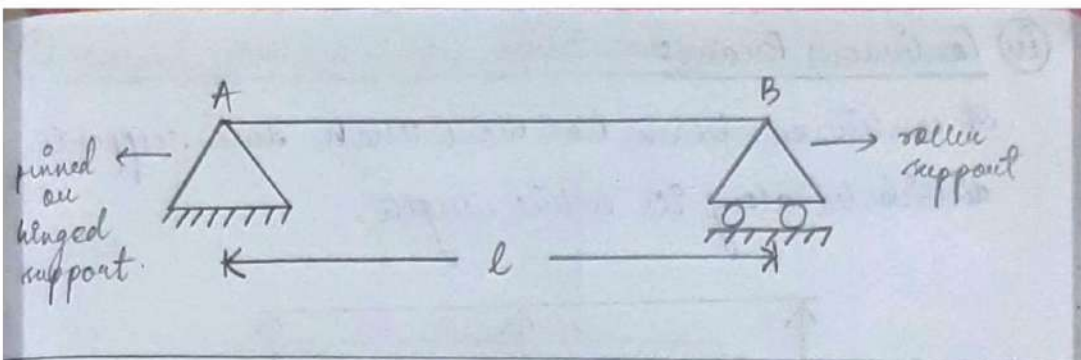
(ii) Fixed Beam:

It is supported at both ends and fixed to resist rotation
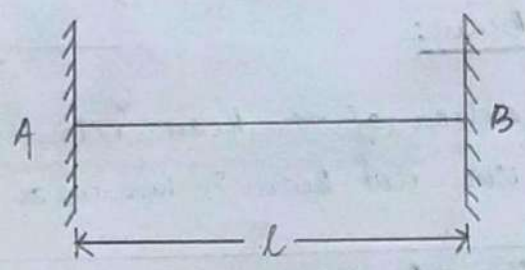
(iii) Cantilever Beam:

If a beam is fixed at one end and set to be free at the other end it is termed as cantilever beam.

ex: chhajja

free end

76

162
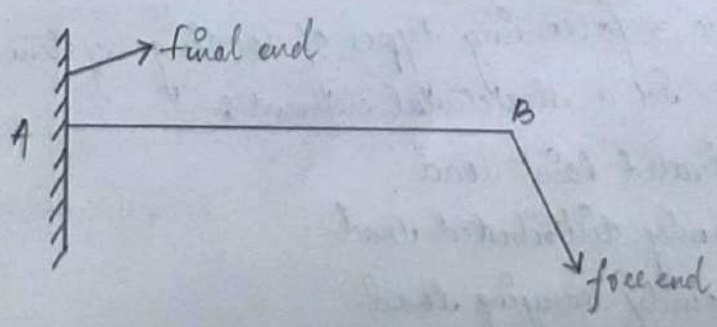
(iv) Continuous Beam:

A continuous beam has more than two supports distributed along its entire length.

support

span   span
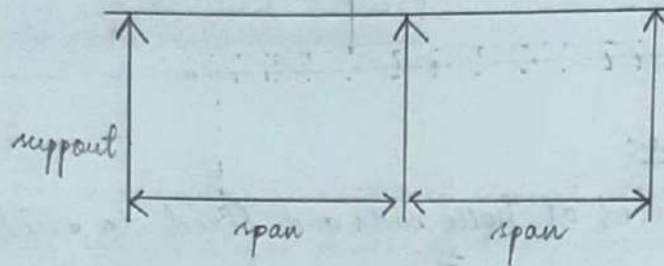
(v) Overhanging Beam:

A   B

If the end portion of the beam extends beyond the support then the beam is known as overhanging beam.
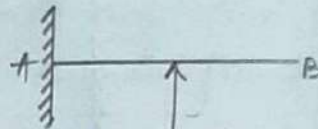
(vi) Propped Cantilever Beam:

A cantilever beam whose one end is fixed and the other end is provided with a simple support, in order to resist deflection is called propped cantilever beam.

A   B
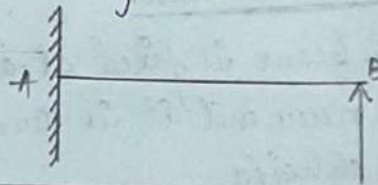
* Types of loading:

There are 3 following types of loading system can be possible for a structural element:

(i) Concentrated Point load

(ii) Uniformly distributed load

(iii) Uniformly varying load

(i) Concentrated load or point load:
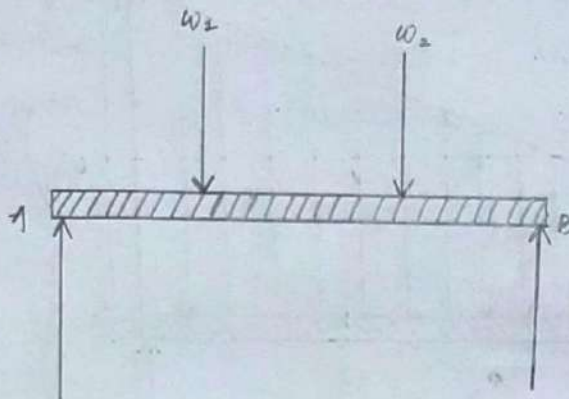
A load acting at a point on a beam is known as concentrated or point load.

(ii) Uniformly Distributed Load (UDL): $\left[\begin{array}{l} 1m \rightarrow w\,kN \\ l_m \rightarrow w \times l\,\,kN \end{array} \right]$

load is applied throughout the length

→ w/kN per length
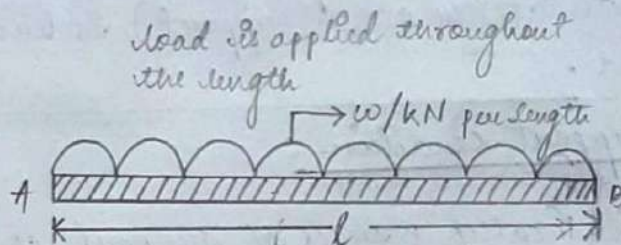
→ A load which is spread over a beam, in such a manner that each unit length is loaded to the same extent is known as uniformly distributed load.

→ The total uniformly distributed load is assumed to be acting at the centre of gravity of the load for the calculation purpose.

78

164

(iii) <u>Uniformly Varying Load (UVL)</u>:

loads
varying
uniformly
entire beam

W_2/length

W_1/length

A   B
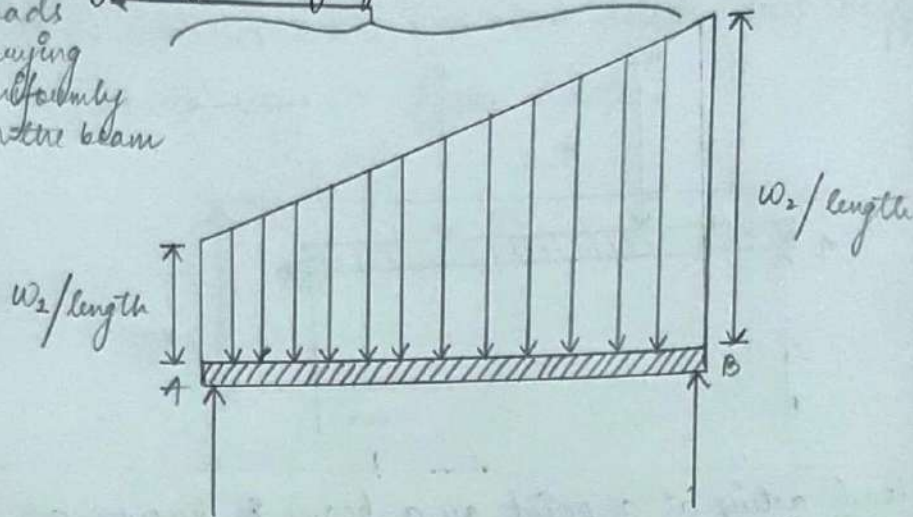
A load which is spread over a beam in such a manner that its extent varies uniformly on each unit length (say, from W_1/length at one support to W_2/length at the other support) is known as varying load.

* <u>Types of support condition</u>:

There are basically 3 types of support conditions acting against the loading:

(i) Hinge support
(ii) Roller support
(iii) fixed support

79

165

(i) **Hinge Support :**

The hinge support is capable of resisting forces acting in any direction of the plane.

→ This support doesn't provide any resistance to rotation.

→ The horizontal and vertical component of reaction can be determined using equilibrium equations (or) equation of equilibrium.

(condition of equilibrium : $\Sigma H = 0$
$\Sigma V = 0$
$\Sigma M = 0$)

(ii) **Roller Support :**

The roller support are free to rotate and translate along the surface upon which the roller rests.

→ The surface can be horizontal, vertical or sloped.

(iii) **Fixed Support :**

A fixed support is the most rigid type of support or connection.

→ It constrains the member in all translations and rotation i.e. it cannot move or rotate in any direction.

80

166

* Calculation of Support Reaction:

@ for simply supported beam with point load:

$R_A, R_B = ?$
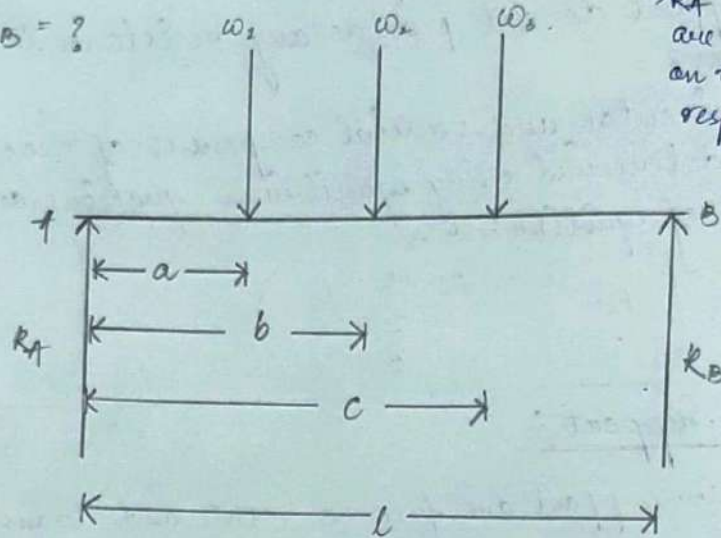
$\rightarrow R_A$ and R_B are reactions on A and B respectively.

Consider a simply supported beam AB of span l subjected to point loads.

$\omega_1, \omega_2, \omega_3$ at a distances of A, B, C respectively for the support A.

Let, R_A = reaction at A

R_B = reaction at B.

for calculating R_A and R_B, taking moment about $A = 0$ (if clockwise (positive) moments and anticlockwise (negative) moments are equal)

That means all the clockwise moments about A is equal to all the anticlockwise moment about A.

81

167

$$\Rightarrow (\omega_1 \times a) + (\omega_2 \times b) + (\omega_3 \times c) = R_B \times l$$

$$\Rightarrow \boxed{R_B = \frac{\omega_1 a + \omega_2 b + \omega_3 + c}{l}}$$

Total load $= \omega_1 + \omega_2 + \omega_3$

$$\therefore R_A + R_B = \omega_1 + \omega_2 + \omega_3$$

$$\Rightarrow \boxed{R_A = (\omega_1 + \omega_2 + \omega_3) - R_B}$$

Questions:

①
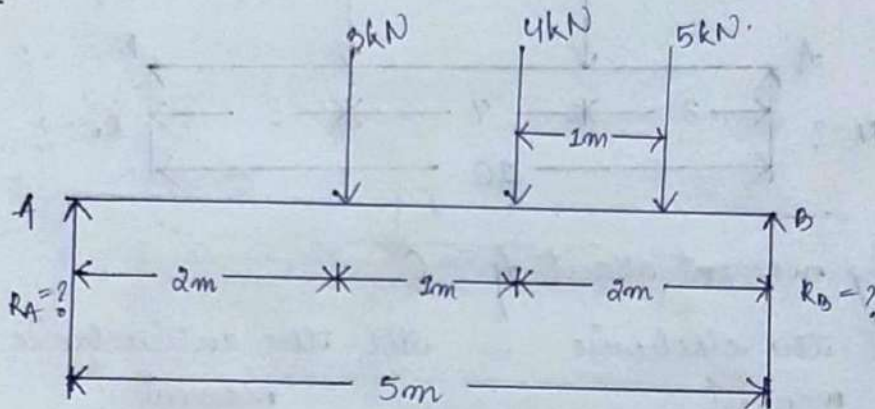
Taking moment about $A = 0$

All the clockwise $=$ All the anticlockwise
   moment   moment

$$\Rightarrow (3 \times 2) + (4 \times 3) + (5 \times 4) = R_B \times 5$$

$$\Rightarrow R_B = \frac{(3 \times 2) + (4 \times 3) + (5 \times 4)}{5}$$

$$\Rightarrow \boxed{R_B = 7.6 \, kN}$$

82

Total load = 3 + 4 + 5

$\Rightarrow R_A + R_B = 12 kN$

$\Rightarrow R_A = 12 - R_B$

$\Rightarrow R_A = 12 - 7.6$

$\Rightarrow \boxed{R_A = 4.4 \, kN}$

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②
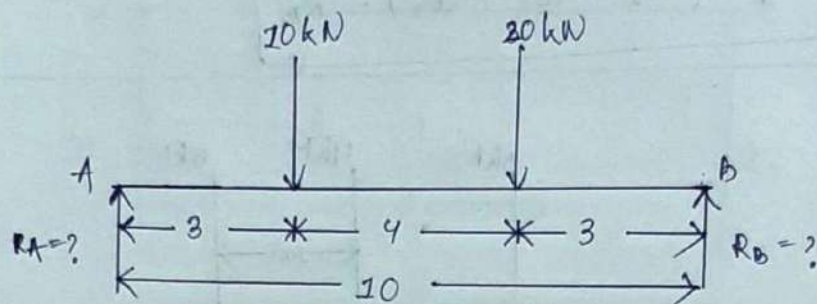
Taking moment about A = 0

All the clockwise = All the anticlockwise
moment   moment

$\Rightarrow (10 \times 3) + (20 \times 7) = R_B \times 10$

$\Rightarrow R_B = \dfrac{(10 \times 3) + (20 \times 7)}{10}$

$\Rightarrow \boxed{R_B = 17 \, kN}$

Total load = 10 + 20

$\Rightarrow R_A + R_B = 30$

$\Rightarrow R_A = 30 - R_B$

$\Rightarrow R_A = 30 - 17$ $\Rightarrow \boxed{R_A = 13 \, kN}$

83

ⓑ For simply supported beam with UDL:
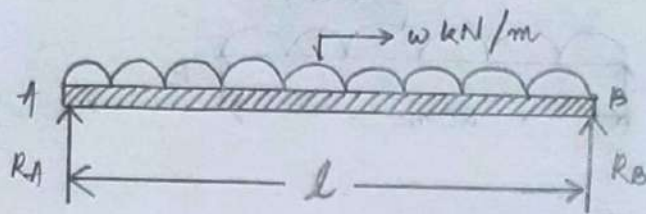
→ we will assume (UDL), the uniformly distributed load to be a point load.
→ The total uniformly distributed load is $\omega \times l$ assumed to be at the centre of gravity.
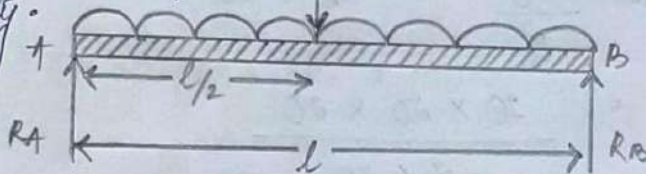
Taking moment about $A = 0$

All the clockwise moments = All the anticlockwise moments

$\Rightarrow \quad Wl \times \dfrac{l}{2} = R_B \times l$

$\Rightarrow \quad R_B = \dfrac{Wl \times \dfrac{l}{2}}{l}$

$\Rightarrow \quad R_B = \dfrac{Wl \times l}{2 \times l} \qquad \Rightarrow \quad \boxed{R_B = \dfrac{Wl}{2}}$

84

170

Question:

①

$R_A = ?$ A
$R_B = ?$

10kN/m

B

—20m—

Solⁿ)
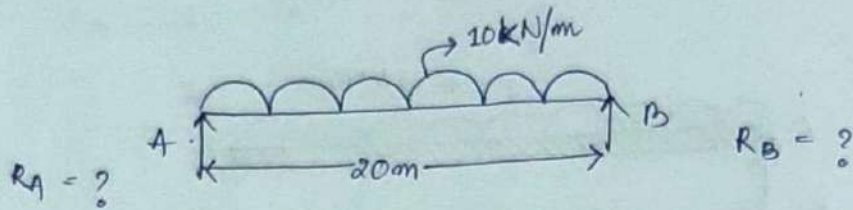
Taking moment about $A = 0$

All the clockwise moments = All the anticlockwise moments.

$\Rightarrow$ $10 \times 20 \times \dfrac{20}{2} = R_B \times 20$

$\Rightarrow$ $R_B = \dfrac{10 \times 20 \times 20}{2 \times 20}$

$\Rightarrow$ $\boxed{R_B = 100 \ kN}$

$\therefore$ Total load $= 20 \times 10$

$\Rightarrow$ $R_A + R_B = 200 \ kN$

$\Rightarrow$ $R_A = 200 - R_B$

$\Rightarrow$ $R_A = 200 - 100$

$\Rightarrow$ $\boxed{R_A = 100 \ kN}$
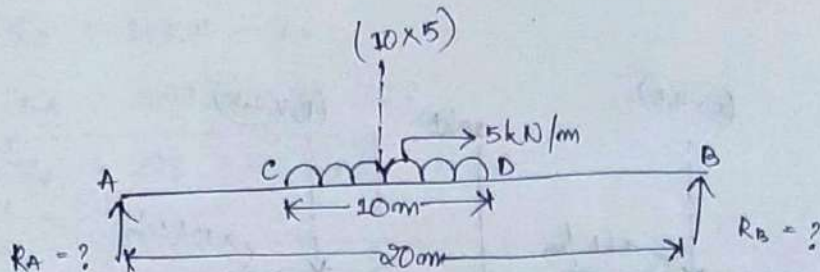
85

171

②

(10×5)

$5kN/m$

A   C   D   B

←10m→

$R_A = ?$ ←20m→ $R_B = ?$

Soln?)

Total load $(\omega) = 5 \times 10$

$= 50 kN$

Taking moment about $A = 0$

All the clockwise ___ All the anticlockwise
moments   moments.

$\Rightarrow$ $50 \times 10 = R_B \times 20$

$\Rightarrow$ $R_B = \dfrac{50 \times 10}{20}$

$\Rightarrow$ $\boxed{R_B = 25 kN}$

$\therefore R_A + R_B = 50 kN$

$\Rightarrow R_A = 50 - 25$

$\Rightarrow \boxed{R_A = 25 kN}$

86

172

③

(5×7.5)   10kN.   (10×7.5)

-5kN/m   10kN/m
A   B
7.5 m   *2.5 m *2.5 m K   7.5
RA=?   RB=?
20 m
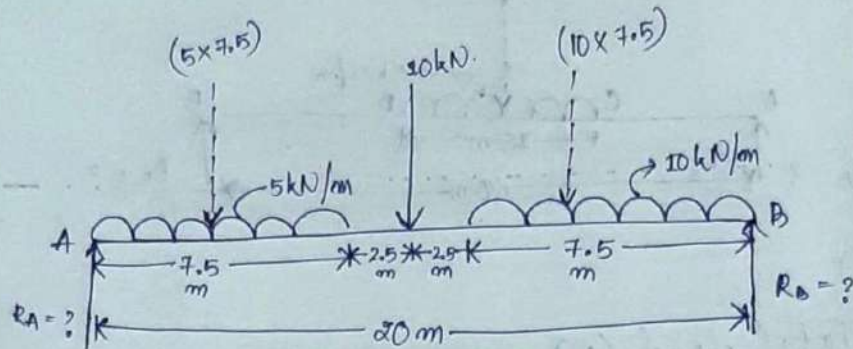
Solⁿ:) Total load = 10 + (5 × 7.5) + (10 × 7.5)

$$= 122.5 \, kN$$

Taking moment about, A = 0

All the clockwise = All the anticlockwise
    moment   moment .

$$=) (5 \times 7.5 \times 3.75) + (10 \times 10) + \qquad = R_B \times 20$$
$$(10 \times 7.5 \times (3.75 + 5 + 7.5))$$

$$=) \quad R_B = \frac{(5 \times 7.5 \times 3.75) + (10 \times 10) + (10 \times 7.5 \times (3.75 + 5 + 7.5))}{20}$$

$$=) \quad R_B = \frac{534.375}{20}$$

$$=) \boxed{R_B = 26.718 \, kN}$$

87

173

$\therefore R_A + R_B = 122.5 \, kN$

$\Rightarrow R_A = 122.5 - R_B$

$\Rightarrow R_A = 122.5 - 26.718$

$\Rightarrow \boxed{R_A = 95.782 \, kN}$

④
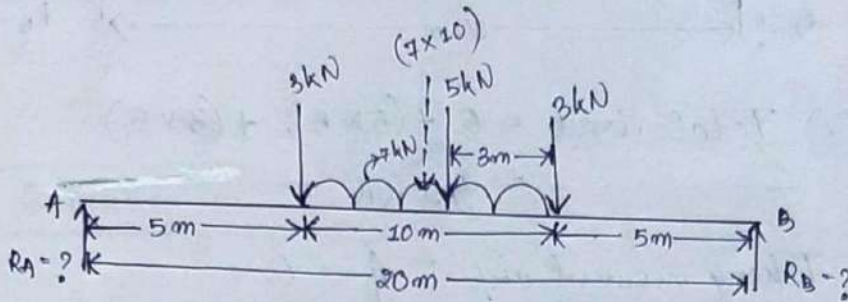
Solu?) Total load $= 3 + 3 + 5 + (7 \times 10)$

$= 81 \, kN$

Taking moment about $A = 0$

All the clockwise moments $=$ All the anticlockwise moments

$\Rightarrow (10 \times 7 \times (5 + 5)) + (3 \times (10 + 5))$
$\quad + (5 \times (7 + 5)) + (3 \times 5)$
$\hspace{4cm} = R_B \times 20$

$\Rightarrow \quad 820 = R_B \times 20$

$\Rightarrow \quad R_B = \dfrac{820}{20}$

$\Rightarrow \boxed{R_B = 41 \, kN}$

$\therefore R_A + R_B = 81 \, kN$

$\Rightarrow R_A = 81 - R_B$

$\Rightarrow R_A = 81 - 41$

$\Rightarrow \boxed{R_A = 40 \, kN}$

88

174

(5)
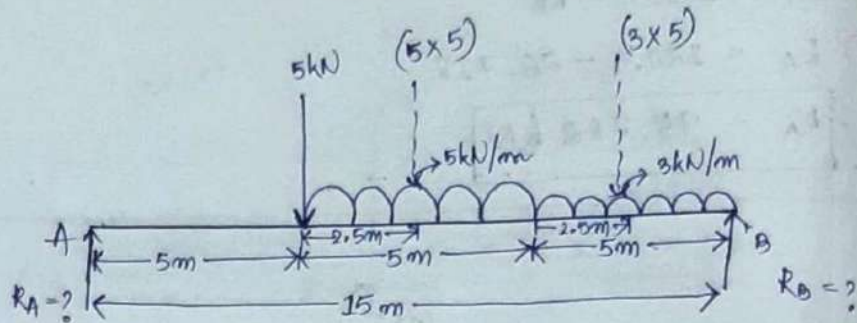
Solⁿ:) Total load $= 5 + (5 \times 5) + (3 \times 5)$

$$= 45 \, kN$$

Taking moment about $A = 0$

All the clockwise moment $=$ All the anticlockwise moments.

$\Rightarrow) (5 \times 5) + (5 \times 5 \times (2.5 + 5))$
$\quad + (3 \times 5 \times (2.5 + 5 + 5)) \quad = R_B \times 15$

$\Rightarrow) \quad 400 \quad = R_B \times 15$

$\Rightarrow) \quad R_B = \dfrac{400}{15}$

$\Rightarrow) \boxed{R_B = 26.67 \, kN}$

$\therefore R_A + R_B = 45$
$\Rightarrow) R_A = 45 - R_B$
$\Rightarrow) R_A = 45 - 26.67$
$\Rightarrow) \boxed{R_A = 18.33 \, kN}$

89

175

* Shear Forces:

The shear force at the cross-section of a beam may be defined as "the algebraic sum of unbalanced vertical forces to the right or left of the section."

* Bending Moment:

The bending moment at the cross-section of the beam may be defined as "the algebraic sum of the moments of the vertical forces, to the right or left of the section."

* Sign Convention for Shear Force and Bending Moment:

① The shear force is said to be positive at a section, when the left hand portion tends to slide upward and the right hand portion tends to slide downward or, in other words all the downward forces to the left of the section cause negative shear and those acting upward cause positive shear.

② Similarly the shear force is said to be negative at a section, when the left hand portion slide downward and the right hand portion tends to upward or, in other words all the upward forces to the left of the section cause positive shear and those downwards cause negative shear.

90

176

SFD
Shear Force Diagram
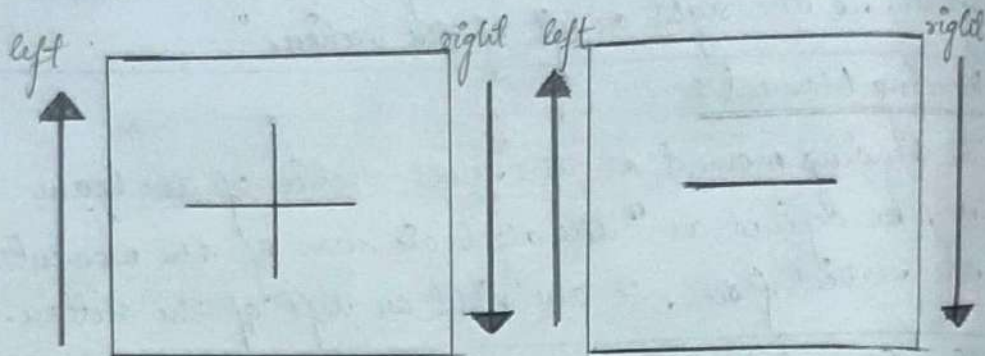
③ At a section where the bending moment is such that it tends to bend the beam at that point to a curvature having <u>concavity</u> at the top, is taken as <u>positive</u>.

④ On the other hand where the bending moment in such that it tends to bend the beam at that point to a curvature having <u>convexity</u> at the top, is taken as <u>negative</u>.

⑤ The positive bending moment is often called <u>sagging moment</u> and negative bending moment is <u>hogging moment</u>

91

(concavity)

+ve

sagging moment

(convexity)

−ve

hogging moment

left   right   left   right

$+$

$-$
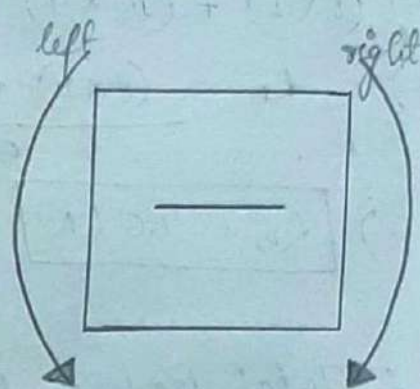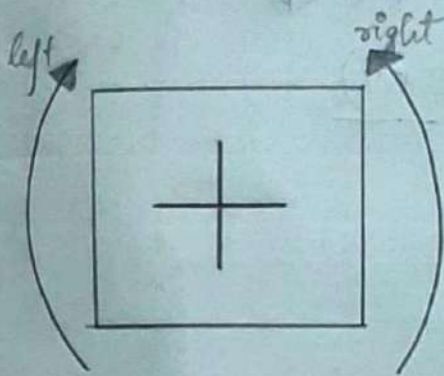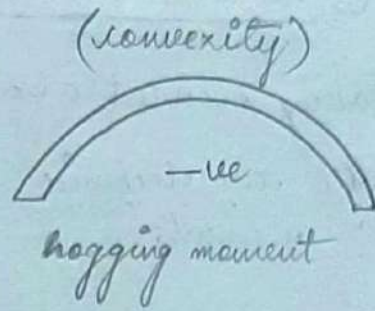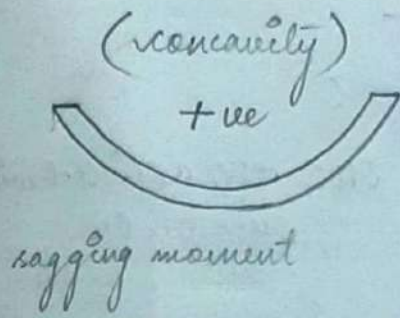
Questions:

① Calculate SF and BM. Draw SFD and BMD.

8kN   10kN

A ⟶   ⟵ 1m ⟶*⟵ 3m ⟶*⟵ 1m ⟶   B

$R_A = ?$ $R_B = ?$

⟵ 5m ⟶
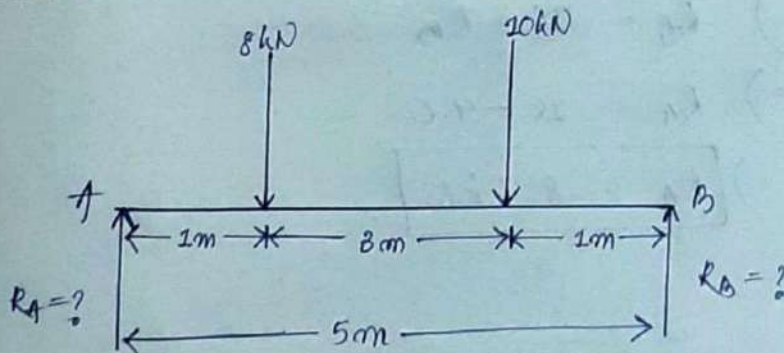
Steps: ① Reaction calculation
② Shear force Calculation
③ Bending Moment Calculation

92

178

Solu^n.) ① Reaction Calculation :

Taking moment about $A = 0$

All the clockwise = All the anticlockwise
  moments   moments

$\Rightarrow (8 \times 1) + (10 \times 4) = 5 \times R_B$

$\Rightarrow R_B = \dfrac{(8 \times 1) + (10 \times 4)}{5}$

$\Rightarrow \boxed{R_B = 9.6 \ kN}$

$\therefore$ Total load $= 8 + 10 \ kN$

$\Rightarrow R_A + R_B = 18 \ kN$

$\Rightarrow R_A = 18 - R_B$

$\Rightarrow R_A = 18 - 9.6$

$\Rightarrow \boxed{R_A = 8.4 \ kN}$

93

179

8kN   10kN

A   C   D   B
|←1m→|←3m→|←1m→|
R_A = 8.4kN
←5m→
R_B = 9.6kN

8.4   8.4kN
0.4kN   0.4kN
O
B
A
-9.6 kN   -9.6kN

Shear Force Diagram

9.6kN
8.4kN
A   B

Bending Moment Diagram

94
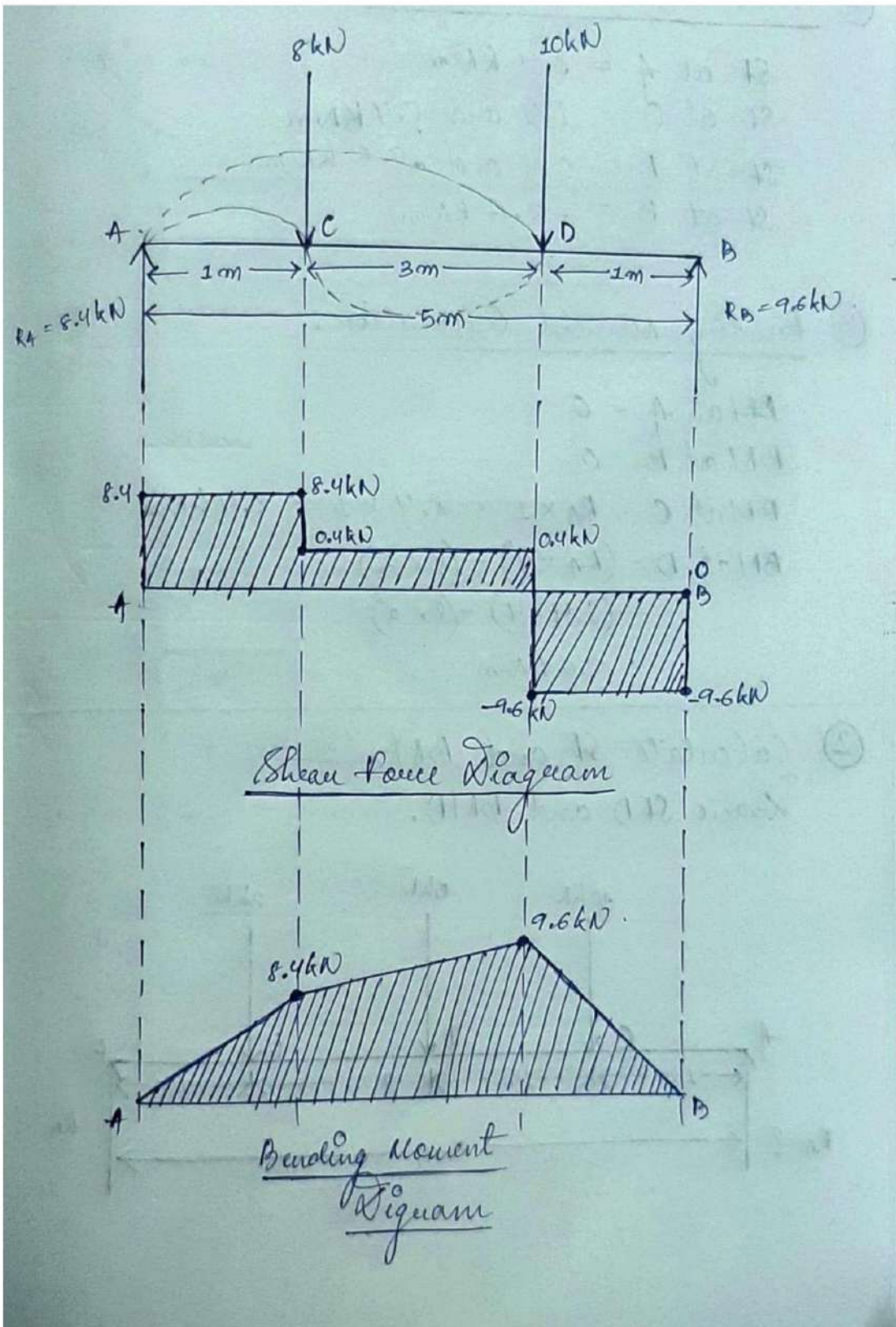
② Shear force Calculation:

SF at A = 8.4 kNm
SF at C = 8.4 and 0.4 kNm
SF at D = 0.4 and -9.6 kNm
SF at B = -9.6 kNm

③ Bending Moment Calculation:

BM at A = 0
BM at B = 0
BM at C = $R_A \times 1$ = 8.4 × 1 = 8.4 kNm
BM at D = $(R_A \times 4) - (8 \times 3)$
= $(8.4 \times 4) - (8 \times 3)$
= 9.6 kNm

② Calculate SF and BM.
Draw SFD and BMD.
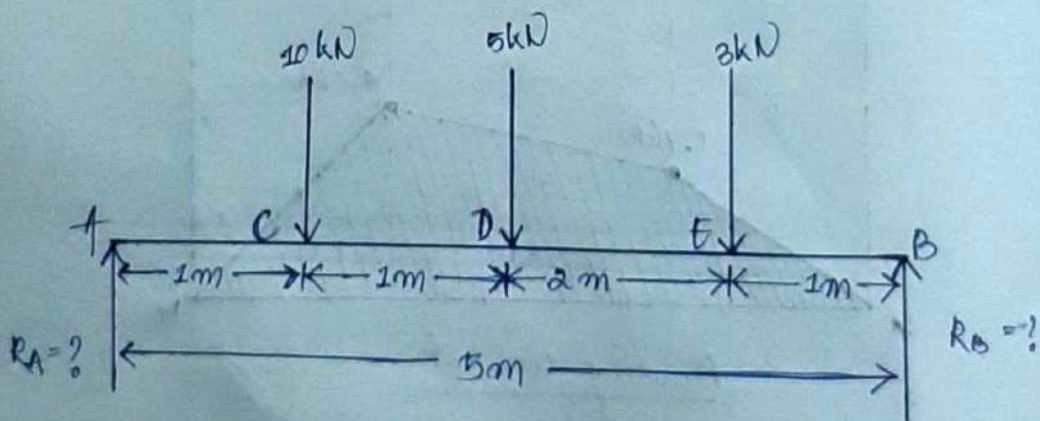
$R_A = ?$

10 kN   5kN   3kN

A   C↓   D↓   E↓   B

←—1m—→←—1m—→*—2m—*—1m—→

$R_B = ?$

←————————— 5m —————————→

Solu": ① Reaction Calculation :

Taking moment about $A = 0$

All the clockwise moment $=$ All the anticlockwise moment

$\Rightarrow$ $(10 \times 1) + (5 \times 2) +$ $= R_B \times 5$
$\qquad (3 \times 4)$

$\Rightarrow$ $\quad 32 = R_B \times 5$

$\Rightarrow$ $R_B = \dfrac{32}{5}$

$\Rightarrow$ $\boxed{R_B = 6.4 \, kN}$

$\therefore$ Total load $= 10 + 5 + 3$

$\Rightarrow R_A + R_B = 18 \, kN$

$\Rightarrow R_A + 6.4 = 18 \, kN$

$\Rightarrow R_A = 18 - 6.4 \, kN$

$\Rightarrow \boxed{R_A = 11.6 \, kN}$

96

182

Shear force Diagram

97

183
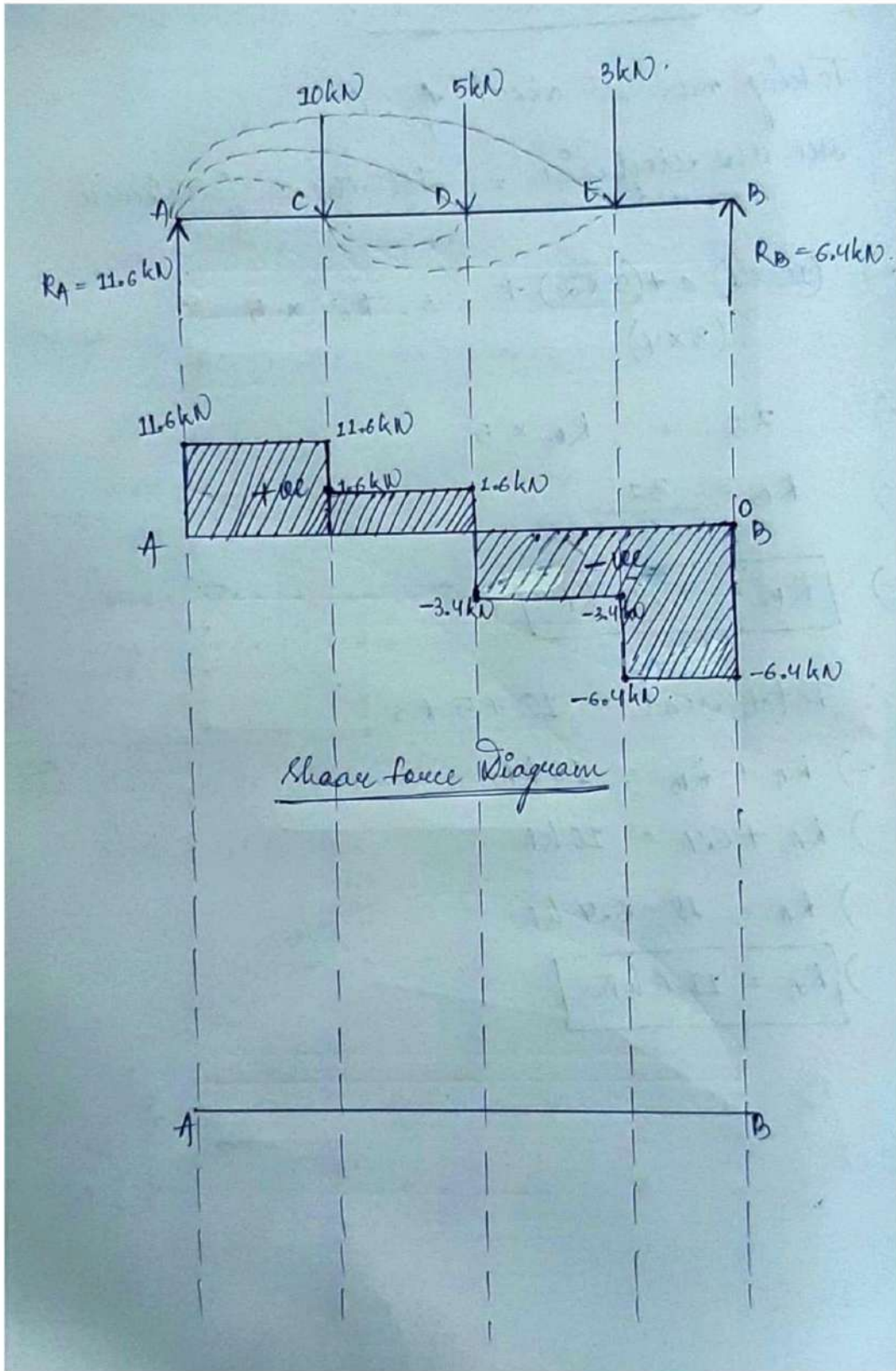
② Shear Force Calculation:

SF at A = 11.6 kNm
SF at BC = 11.6 and 1.6 kNm
SF at D = 1.6 and -3.4 kNm
SF at E = -3.4 and -6.4 kNm
SF at B = -6.4 kN

③ Calculate SF and BM.
Draw SFD and BMD.

(5×10)

A ————————— 5kN ————————— B

$R_A = ?$ |←——————— 10 m ———————→| $R_B = ?$
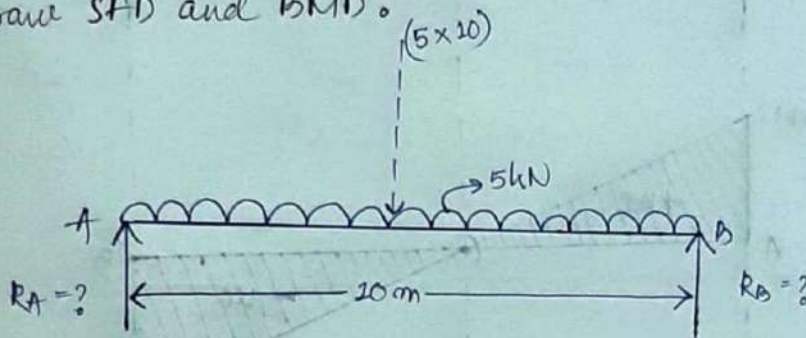
Soln?) ① Reaction Calculation:

Taking moment about A = 0

All the clockwise   =   All the anticlockwise
    moments   moments.

$\Rightarrow (5 \times 10) \times \dfrac{10}{2} = R_B \times 10$

$\Rightarrow R_B = \dfrac{5 \times 10 \times 10}{2 \times 10}$ $\Rightarrow \boxed{R_B = 25 kN}$

∴ Total load = 5 × 10 kN
$\Rightarrow R_A + R_B = 50 kN$ $\Rightarrow R_B = 50 - 25$
$\Rightarrow R_A + 25 kN - 50 kN$ $\Rightarrow \boxed{R_B = 25 kN}$

98

184

when SF = 0, BM is maximum

(5×10)

→5kN

A _____ 10/2 _____ C _____ B

10 m

$R_A = 25kN$ $R_B = 25kN$

25kN

+ve

A

+ve

B
o

-25kN

Shear force
Diagram

62.5 kN.

A o   o B

Bending Moment
Diagram

99
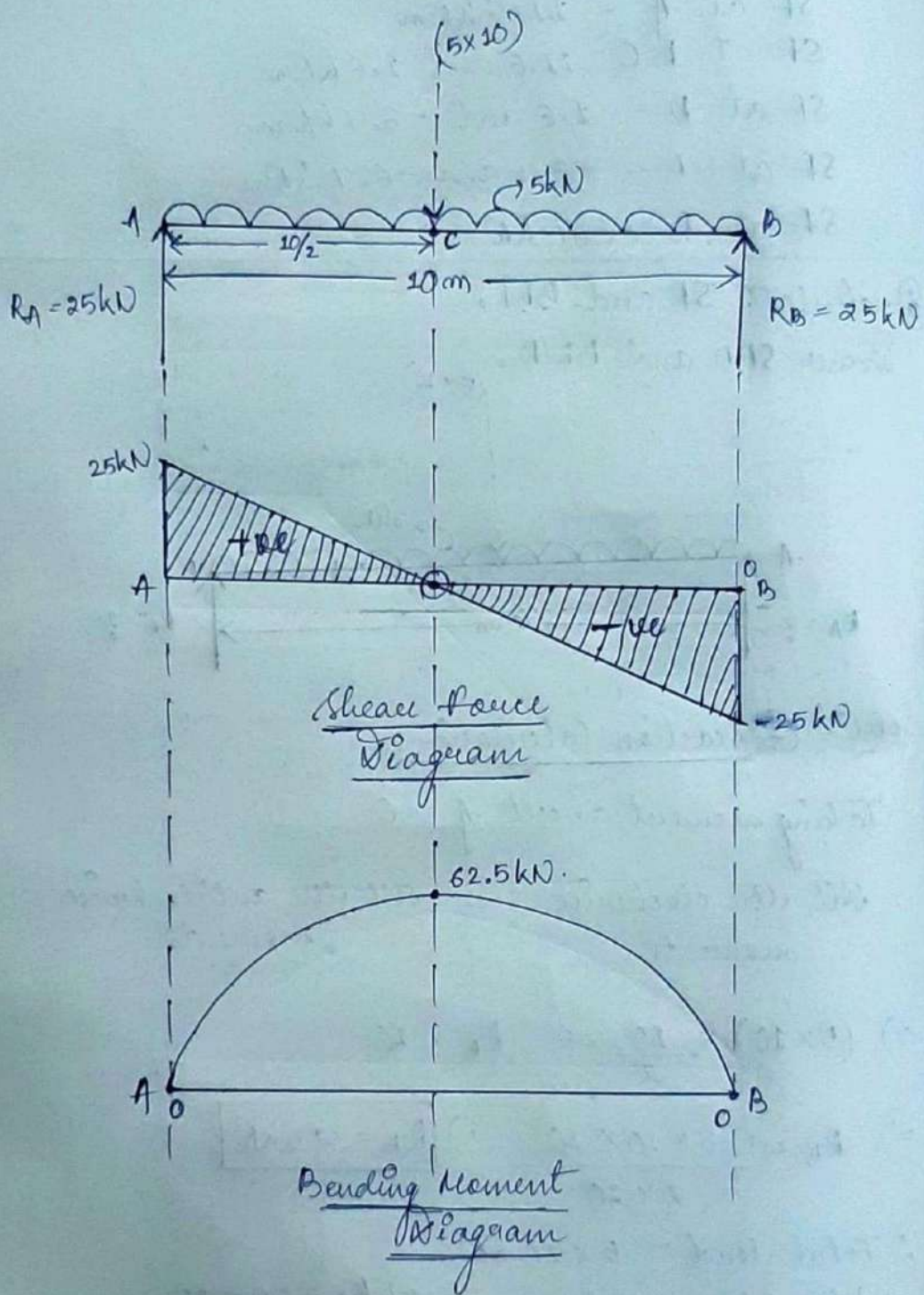
② Shear force Calculation:

SF at A = 25 kN
SF at C = 0 kN
SF at B = -25kN and 0kN.

③ Bending Moment Calculation:

BM at A = 0 kN-m
BM at B = 0 kN-m
BM at C = (BM$_{max}$)
$$= (R_A \times 5) - (5 \times 5 \times 2.5)$$
$$= (25 \times 5) - (5 \times 5 \times 2.5)$$
$$= 62.5 \, kNm$$

④ Calculate SF and BM.
   Draw SFD and BMD.

$R_A = ?$ $\overset{A}{\uparrow}$ ... 1800 N/m ... C ... 4m ... D ... 2m ... $\overset{B}{\uparrow}$ $R_B = ?$

2m — 4m — 2m
8m

Shear Force Diagram

Bending Moment Diagram
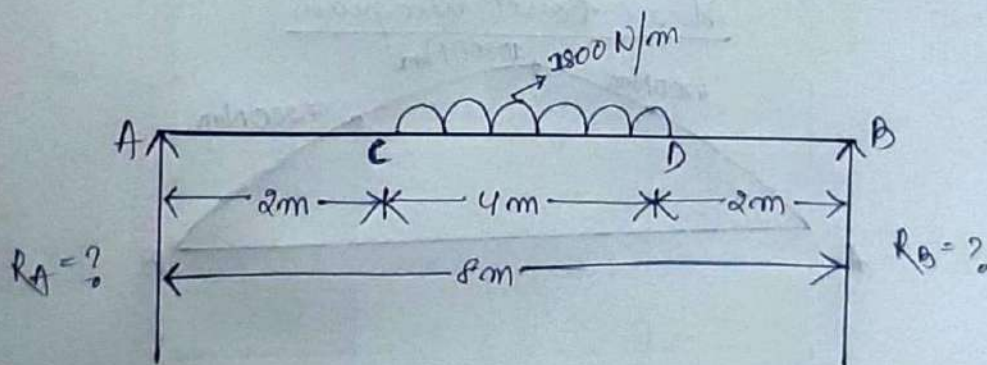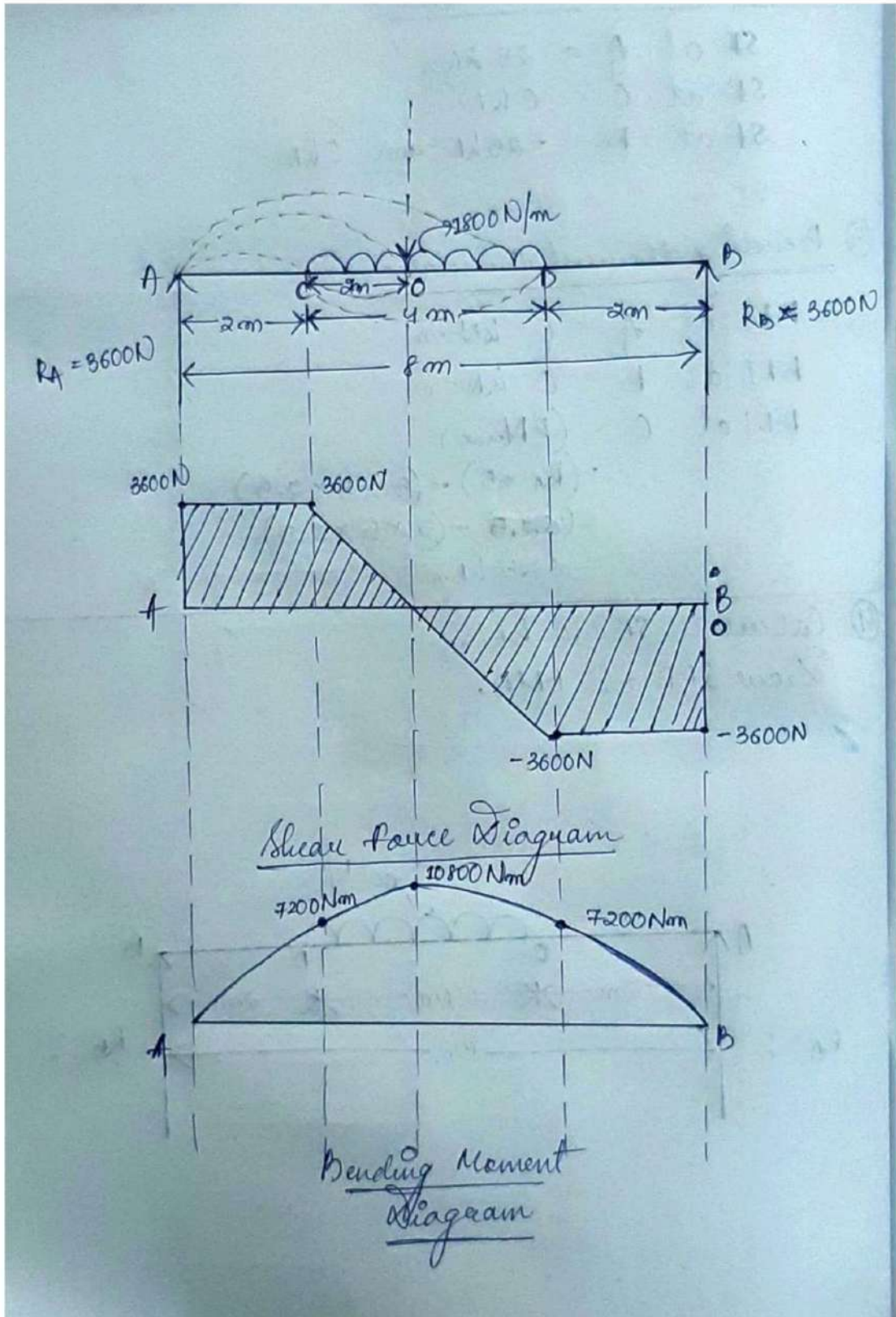
101

① <u>Reaction Calculation:</u>

Total load = $1800 \dfrac{N}{m} \times 4\,m$

$= 7200\,N$

Taking moment about $A = 0$

All the clockwise = All the anticlockwise
moment   moment

$\Rightarrow (1800 \times 4) \times (2 + 2) = R_B \times 8$

$\Rightarrow 28,800 = R_B \times 8$

$\Rightarrow R_B = \dfrac{28,800}{8}$

$\Rightarrow \boxed{R_B = 3600\,N}$

$\therefore$ Total load $= 1800 \times 4\ N$

$\Rightarrow R_A \ast R_B = 7200\,N$

$\Rightarrow R_A + 3600\,N = 7200\,N$

$\Rightarrow R_A = 7200\,N - 3600\,N$

$\Rightarrow \boxed{R_A = 3600\,N}$

② Shear force Calculation:

SF at A = 3600 N
SF at C = 3600 N
SF at D = - 3600 N $\left(\because \begin{array}{c} 3600 - 7200 \\ = - 3600 \end{array} \right)$
and ON

③ Bending Moment Diagram:

BM at A = 0
BM at B = 0
BM at C = $(R_A \times 2)$ = (3600×2) = 7200 &N m
BM at O = $(R_A \times 4) - (1800 \times 2)$

   $= (3600 \times 4) - (1800 \times 2)$

   $= 10800 \ N m$

BM at D = $(R_A \times 6) - (1800 \times 4 \times 2)$

   $= (3600 \times 6) - (1800 \times 4 \times 2)$

   $= 7200 \ N m$

103

189

⑤ Calculate SF and BM.
Draw SFD and BMD.
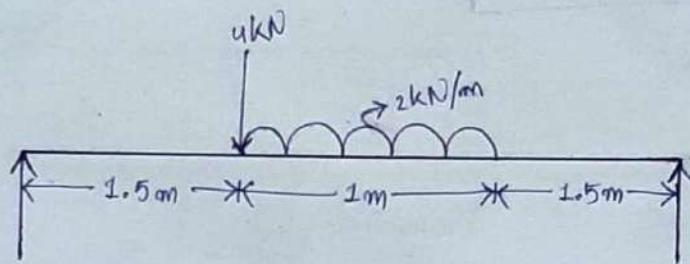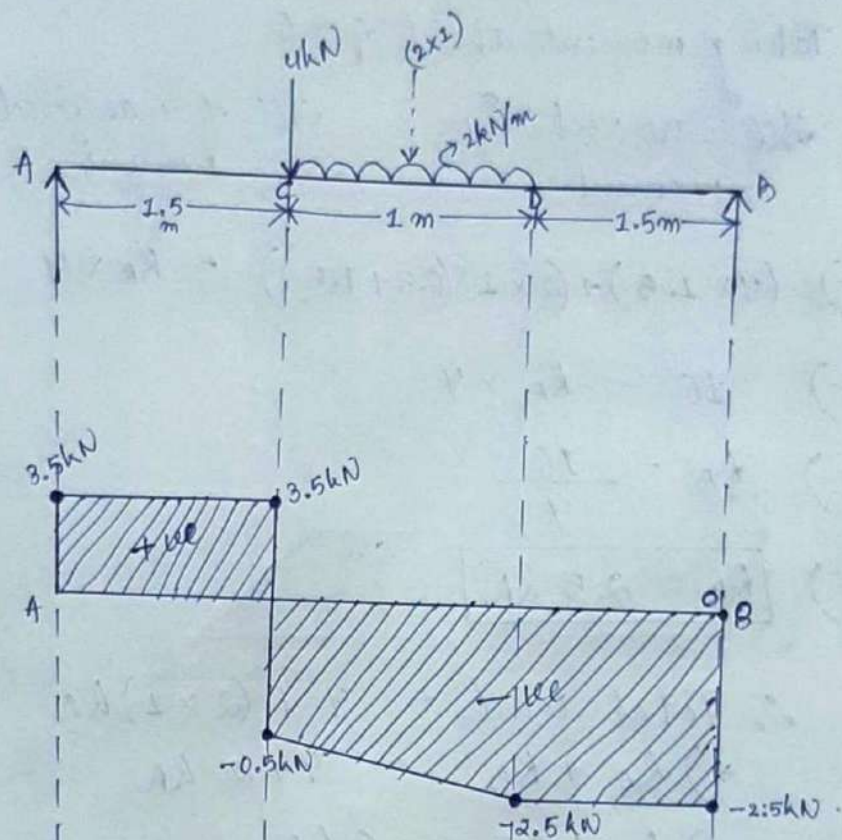
4kN

2kN/m

1.5 m   1m   1.5m

Shear force Diagram

Bending Moment Diagram.

105

(i) Reaction Calculation:

Taking moment about $A = 0$

All the clockwise = All the anticlockwise
moment   moment

$\Rightarrow (4 \times 1.5) + (2 \times 1 \times (0.5 + 1.5)) = R_B \times 4$

$\Rightarrow 10 = R_B \times 4$

$\Rightarrow R_B = \dfrac{10}{4}$

$\Rightarrow \boxed{R_B = 2.5 \text{ kN}}$

$\therefore$ Total load $= 4 + (2 \times 1) \text{ kN}$
$\Rightarrow R_A + R_B = 4 + 2 \text{ kN}$
$\Rightarrow R_A + 2.5 = 6 \text{ kN}$
$\Rightarrow R_A = 6 - 2.5 \text{ kN}$
$\Rightarrow \boxed{R_A = 3.5 \text{ kN}}$

(ii) Shear force Calculation:

SF at $A = 3.5 \text{ kN}$
SF at $C = 3.5 \text{ kN}$
SF at $D = -2.5 \text{ kN}$ $\left[\because -0.5 - (2 \times 1) = -2.5 \right]$
SF at $B = -2.5 \text{ kN}$ and 0 kN

106

192

(iii) Bending Moment Calculation:

BM at A = 0

BM at B = 0

BM at C = $(R_A \times 1.5) = (3.5 \times 1.5)$

$= 5.25 \, kNm$

BM at D = $(R_A \times (1.5 + 1)) - (4 \times 1) - (2 \times 1 \times 0.5)$

$= (3.5 \times (1.5 + 1)) - (4 \times 1) - (2 \times 1 \times 0.5)$

$= 3.75 \, kNm.$

107

193

⑥ Calculate Sf and BM.
Draw SFD and BMD.
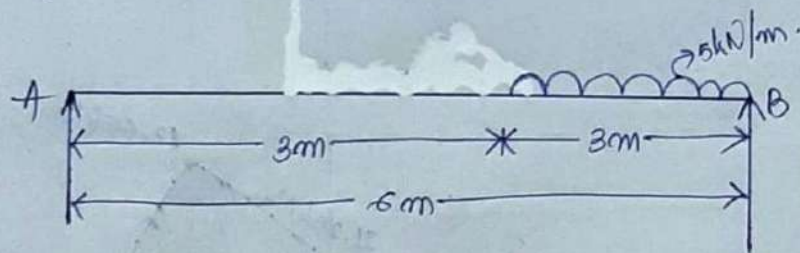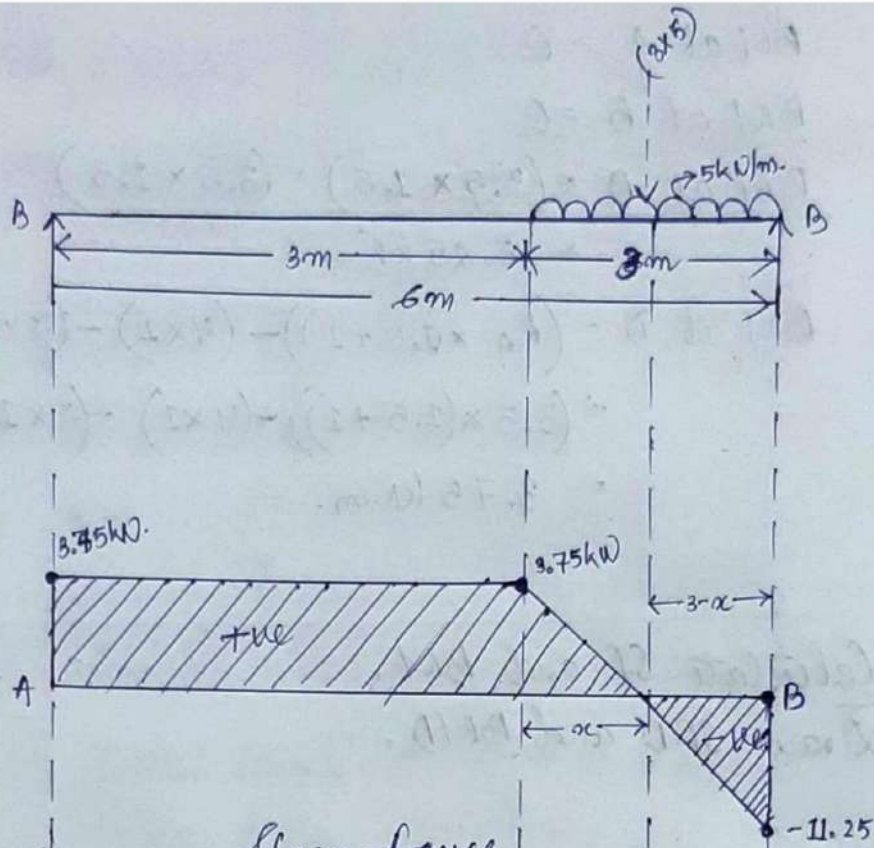
Shear force Diagram
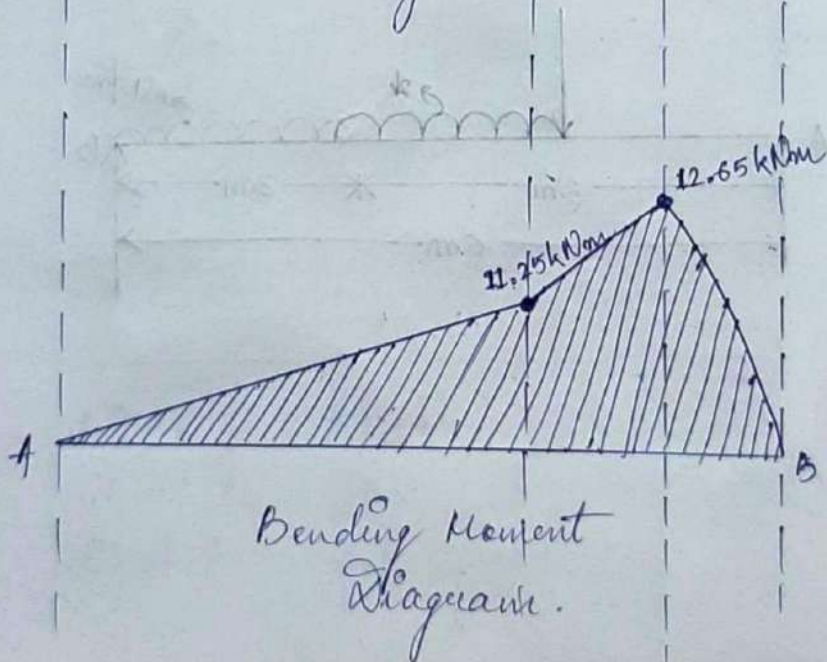
Bending Moment Diagram.

109

① Reaction Calculation:

Taking moment about $A = 0$

All the clockwise — All the anticlockwise
moments moments

$\Rightarrow (5 \times 3 \times 1.5) + 3 = R_B \times 6$

$\Rightarrow 67.5 = R_B \times 6$

$\Rightarrow R_B = \dfrac{67.5}{6}$

$\Rightarrow \boxed{R_B = 11.25\,kN}$

$\therefore$ Total load $= 5 \times 3\,kN$

$\Rightarrow R_A + R_B = 15\,kN$

$\Rightarrow R_A + 11.25 = 15\,kN$

$\Rightarrow R_A = 15 - 11.25$

$\Rightarrow \boxed{R_A = 3.75\,kN}$

② Shear force Calculation:

SF at $A = 3.75\,kN$
SF at $C = 3.75\,kN$.
SF at $B = -11.25\,kN$ $\left[\because 3.75 - 15 = -11\,kN\right]$
and
$0\,kN$

110

196

Now apply similar triangle rules,

$$\frac{3.75}{11.25} = \frac{x}{3-x}$$

$$\Rightarrow \quad 3.75(3-x) = 11.25x$$

$$\Rightarrow \quad 11.25 - 3.75x = 11.25x$$

$$\Rightarrow \quad 11.25 = 11.25x + 3.75x$$

$$\Rightarrow \quad 11.25 = 15x$$

$$\Rightarrow \quad x = \frac{11.25}{15}$$

$$\Rightarrow \quad \boxed{x = 0.75\,m}$$

So, $3-x = 3 - 0.75$
$$\qquad = 2.25\,m$$

(iii) <u>Bending Moment Calculation:</u>

BM at A = 0 , BM at B = 0

BM at C = $(R_A \times 3) - (3.75 \times 3) = 11.25\,kNm$

BM at O = $(R_A \times 3 + 0.75) - \left(5 \times 0.75 \times \dfrac{0.75}{2}\right)$

$$\qquad = (3.75 \times 3.75) - \left(5 \times 0.75 \times \dfrac{0.75}{2}\right)$$

$$\qquad = 12.65\,kNm$$

111

197

⑦ Calculate SF and BM. Draw SFD and BM.
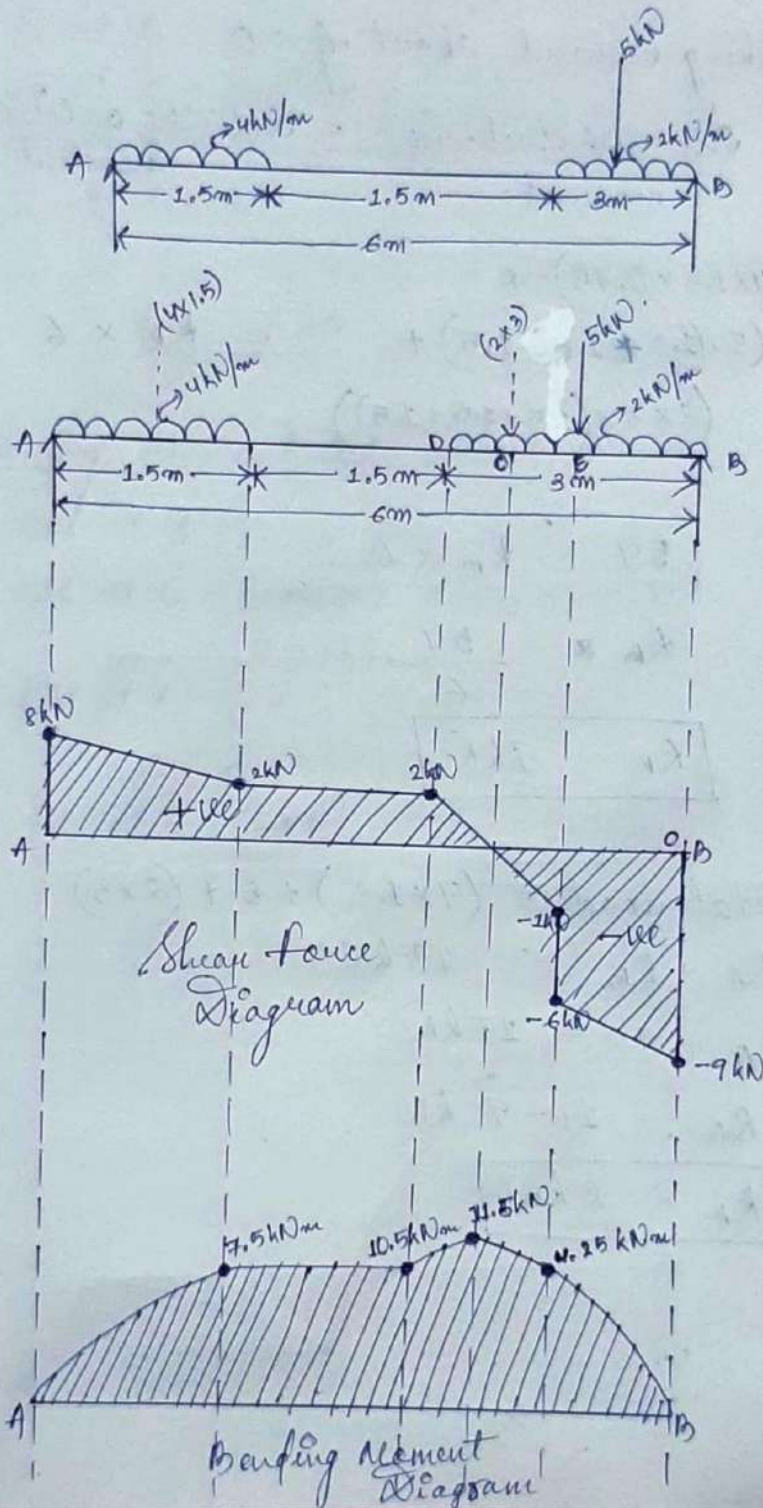
Shear Force Diagram

Bending Moment Diagram

112

(i) Reaction Calculation :

Taking moment about $q = 0$

All the clockwise = All the anticlockwise
   moment   moment

$$\Rightarrow (4 \times 1.5 \times 0.75) +$$
$$(5 \times (1.5 + 1.5 + 1.5)) + \qquad = R_B \times 6$$
$$(2 \times 3 \times (1.5 + 1.5 + 1.5))$$

$$\Rightarrow \quad 54 = R_B \times 6$$

$$\Rightarrow \quad R_B = \frac{54}{6}$$

$$\Rightarrow \quad \boxed{R_B = 9 kN}$$

$\therefore$ Total load $= (4 \times 1.5) + 5 + (2 \times 3)$
$$\Rightarrow R_A + R_B = 17 kN$$
$$\Rightarrow R_A + 9 = 17 kN$$
$$\Rightarrow R_A = 17 - 9 \ kN$$
$$\Rightarrow \boxed{R_A = 8 kN}$$

113

199

GANDHI ACADEMY OF TECHONOLOGY AND ENGINEERING BERHAMPUR

LECTURE NOTE
ON
HYDRAULICS & IRRIGATION ENGG.

PREPARED BY:
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LECTURER IN CIVIL

Hydrostatics :—

It is the branch of science which deals with study of fluids at rest.

Fluid :—

Any substance which is capable of flowing is called as a fluid.

Ex : liquid, gas

Properties of Fluid :—

Density or mass density :—

(i) Density or mass density of a fluid is defined as the ratio of mass of fluid to its volume.

(ii) It is denoted as ρ (rho).

Mathematically $\rho = \dfrac{m}{v}$

The value of density of water is 1000 kg/m³.

Specific weight or weight density :—

(i) It is the ratio of weight of fluid to its volume.

(ii) It is denoted as γ (gamma).

Mathematically $\gamma = \dfrac{w}{v}$

$$\gamma = \dfrac{m}{v} = \dfrac{mg}{v} = \rho g$$

Specific Volume :—

It is the ratio of volume of liquid to the mass of liquid.

Mathematically, Specific volume $= \dfrac{v}{m} = \dfrac{1}{\frac{m}{v}} = \dfrac{1}{\rho}$

Specific Gravity :—

Specific gravity is defined as the ratio of weight density or density of a fluid to the weight density or density of a standard fluid. For liquid standard fluid is water. For gas standard fluid is air. Specific gravity is also known as relative density.

$S_{mercury} = \dfrac{\text{Spe weight of mercury}}{\text{weight of water}}$

$\Rightarrow 13.6 = \dfrac{\text{density of mercury}}{1000}$

⇒ Density of mercury $= 13600 \, kg/m^3$

$w = mg$
$1 \times 9.81 = 9.81 \, N$
$1 \, kg = 9.81 \, N$

♠ Density of air $1.22 \, kg/m^3$

Q. Calculate the Specific weight, density and Specific gravity of 1 lit of a liquid which weighs 7N.

Ans. Specific weight $= \gamma = \dfrac{w}{V}$

$$= \dfrac{7}{\frac{1}{1000}} = 7000 \, N/m^3$$

Given data :

$V = 1 \, lit = \dfrac{1}{1000} \, m^3$

$w = 7N$

$\gamma = \rho g$

$\rho = \dfrac{\gamma}{g} = \dfrac{7000}{9.81} = 713.55 \, kg/m^3$

$G = \dfrac{\rho_l}{\rho_w} = \dfrac{713.55}{1000} = 0.713 \, (unitless)$

Viscosity :—

Viscosity is defined as the property of fluid which offers resistance to the movement of one layer of fluid over another adjacent layer of fluid.

The top layer causes a shear stress (τ) on the adjacent bottom layer.

Newton's law of Viscosity :—

According to this law, shear stress is directly proportional to velocity gradient $\left(\dfrac{dv}{dy}\right)$

Mathematically

$$\tau \text{ (Tau)} \propto \frac{dv}{dy}$$

$$\Rightarrow \tau = \mu \frac{dv}{dy}$$

Where τ (Tau) = shear stress in N/mm²

μ (mu) = proportionality constant called coefficient of dynamic viscosity or simply viscosity

du = change velocity

dy = distance between two layers of fluid

$$\Rightarrow \mu = \frac{\tau}{\left(\frac{dv}{dy}\right)}$$

Unit of viscosity (μ) :—

$$\mu = \frac{\tau}{(dv/dy)} = \frac{N/mm^2}{\left(\frac{mm/sec}{m}\right)} = \frac{N \cdot sec}{mm^2}$$

Kinematic Viscosity :—

→ It is defined as the ratio between the dynamic viscosity (μ) and density of fluid (ρ).

→ It is denoted by ν (nu).

Mathematically

$$\boxed{\nu = \mu / \rho}$$

Unit of ν :—

$$\nu = \mu/\rho = \frac{\frac{N \cdot sec}{m^2}}{kg/m^3} \qquad \left[\begin{array}{l}\text{unit of density}\\ \text{is kg/m}^3\\ \text{as } 1N = 1 kg \cdot \frac{m}{s^2}\end{array}\right]$$

$$= \frac{kg \cdot m}{s^2} \times \frac{sec}{m^2} \times \frac{m^3}{kg}$$

$$= m^2/sec$$

1 poise = 1 cm²/sec

Problem-1

If the velocity distribution over a plate is given by $u = \frac{2}{3}y - y^2$ in which v is the velocity in m/sec at a distance y meter above the plate. Determine the shear stress at $y=0$ and $y=0.15m$. Take dynamic viscosity of fluid as 8.63 poise.

Sol^n Given data

$$u = \frac{2}{3}y - y^2 \qquad \left(\frac{d}{dy}x^n = nx^{n-1}\right)$$

$$\frac{du}{dy} = \frac{2}{3}\frac{d(y)}{dy} - \frac{d}{dy}(y^2)$$

$$= \frac{2}{3}\times 1 - 2y'$$

$$\mu = 8.63 \text{ poise} = 8.63 \text{ cm}^2/\text{sec}$$

$$= 8.63 \times 10^{-4} \text{ m}^2/\text{sec}$$

We know from Newton's law of viscosity

$$\tau = \mu\frac{du}{dy}$$

$$\tau_{y=0} = ? \quad \& \quad \tau_{y=0.15} = ?$$

$$\tau_{y=0} = \mu\left(\frac{du}{dy}\right)_{y=0} = 8.63\times10^{-4}\times\left(\frac{2}{3}-2y\right)$$

$$= 0.5752 \text{ N/m}^2$$

$$\tau_{y=0.15} = \mu\left(\frac{du}{dy}\right)_{y=0.15} = 8.63\times10^{-4}\times\left(\frac{2}{3}-2y\right)$$

$$= 0.3147 \text{ N/m}^2$$

Problem-2

A plate 0.025 mm distant from a fixed plate moves at 60 cm/sec and requires a force of 2 N/m² to maintain speed. Determine the fluid viscosity between the plates?

Sol^n Given data

$$dy = 0.025 \text{ mm}$$

$$= 0.025 \times 10^{-3} \text{ m}$$

$$du = 60 \text{ cm/sec} = 0.6 \text{ m/sec}$$

we know from newton's
law of viscosity

$$\tau = \mu \cdot \frac{du}{dy} \quad\text{---(1)}$$

τ given = 2 N/m²

putting the given values in eqn (1)

$$2 = \mu \cdot \frac{0.2}{0.02 \times 10^{-3}}$$

$$\Rightarrow \mu = 2.22 \times 10^{-5} \ N\text{-}s/m^2$$

which is the required answer.

Problem - 3

Calculate the dynamic viscosity of an oil, which is used for lubrication between a square plate of size 0.8m × 0.8m and an inclined plane with angle of inclination 30°. As shown in fig. the weight on the square plate is 300N and it slides down and inclined plate with a uniform velocity of 0.3 m/sec. Thickness of oil film is 1.5mm.

Soln Given data

Area of plate (A) = 0.8 × 0.8 = 0.64 m²

$$\theta = 30°$$
$$w = 300 N$$
$$du = 0.3 \ m/sec$$
$$dy = 1.5mm = 1.5 \times 10^{-3} \ m$$

Shear force (T) = w sin θ

$$= 300 \sin 30°$$
$$= 150 N$$

τ = shear stress

$$= \frac{shear\ force}{shear\ area} = \frac{150}{0.64}$$

we know from newton's law of viscosity

$$\tau = \mu \cdot \frac{du}{dy}$$

$$\Rightarrow \frac{150}{0.64} = \mu \times \frac{0.3}{1.5 \times 10^{-3}}$$
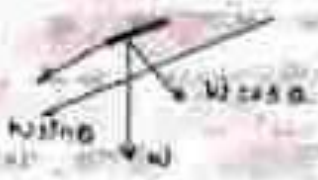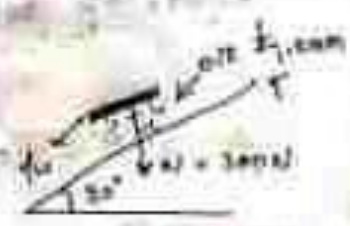
$\Rightarrow b = 1.17 \ N\text{-sec}/m^2$

Surface tension :—

→ Surface tension is defined as the tensile force acting on the surface of a liquid in contact with a gas or on the surface between two immiscible (unable to mix) liquids such that the contact surface behaves like a membrane under tension.

→ It is denoted as σ (sigma).

→ Unit is N/m

Surface tension of a liquid droplet :—

Let p = pressure intensity inside the droplet

σ = surface tension of liquid

d = dia of droplet

Pressure force on area

= stress or pressure × Area

$= p \times \dfrac{\pi}{4}(d^2)$

Surface tension = Force = Surface tension × perimeter

$\sigma \ N/m$

$= \sigma \times \pi d$

Equating both

$$p \times \pi/4 \ (d)^2 = \sigma \times \pi d$$

$$\Rightarrow p = \dfrac{4\sigma}{d}$$

Problem -2

The surface tension of water in contact with air at 30°C is 0.0725 N/m. The pressure inside a droplet of water is to be 0.02 N/cm^2. Calculate the diameter of the droplet of water.

Sol^n Given data

$\sigma = 0.0725 \ N/m$

$p = 0.02 \ N/cm^2 = 0.02 \times 10^4 \ N/m^2$

$p = \dfrac{4\sigma}{d}$

$\Rightarrow 0.02 \times 10^4 = \dfrac{4 \times 0.0725}{d}$

$\Rightarrow d = 0.00145 \ m = 1.45 \ mm$

Surface tension of hollow soap bubble :-

A hollow bubble has two surfaces in contact with air

pressure force $= P \times \frac{\pi}{4} \langle d^2$ (Same as previous case)

Surface tension force $= 2 \times \sigma \times \pi d$

Equating both we have

$$P \times \frac{\pi}{4}(d^2) = 2 \times \sigma \times \pi d$$

$$\Rightarrow P = \frac{8 \sigma}{d}$$

Problem - 7

Find the surface tension in a soap bubble of 40mm diameter when the inside pressure is 2.5 N/m².

Solⁿ : Given data

$d = 40mm = 0.04 m$

$P = 2.5 N/m²$

$P = \frac{8 \sigma}{d} \Rightarrow 2.5 = \frac{8 \times \sigma}{0.04}$

$\Rightarrow \sigma = 0.0125 N/m$ **Ans**

Capillarity :-

Capillarity is defined as a phenomenon of rise or fall of a liquid surface in a small tube relative to the adjacent general level of liquid when the tube is held vertically in the liquid.

The rise of liquid surface is known as capillary depression.

Hence in this figure

h is capillary rise

Units

→ cm, m, mm etc.
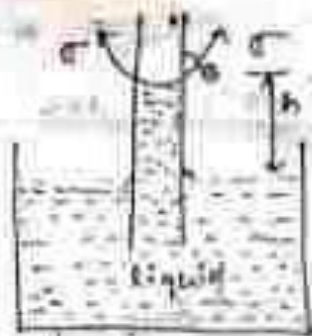
Expression for finding capillary rise :-

$$h = \frac{4\sigma \cos\theta}{\rho g d}$$

where :

σ = surface tension in N/m

θ = Angle of contact

ρ = mass density of fluid

d = dia of tube

Problem-4

Calculate the capillary rise in a glass tube of 2.5 mm diameter when immersed vertically in (a) water (b) Mercury.

Take surface tension σ = 0.725 N/m for water and σ = 0.52 N/m for mercury. Angle of contact for mercury = 130°. Specific gravity for mercury is 13.6 and for water is 1.0

Sol^n

Case (a) For water :

d = 2.5 mm = 2.5 × 10⁻³ m

σ = 0.0725 N/m

ρ = 1000 kg/m³

g = 9.81 m/sec²

$$h = \frac{4\sigma \cos\theta}{\rho g d}$$

$$= \frac{4 \times 0.0725 \times \cos 0°}{1000 \times 9.81 \times 2.5 \times 10^{-3}}$$

$$\therefore = 0.0118 \, m = 1.18 \, cm$$

Case (b) For mercury

$$h = \frac{4\sigma \cos\theta}{\rho g d}$$

$\sigma_{mercury}$ = 0.52 N/m

θ = 130°

ρ = 13.6 × 1000

g = 9.81 m/sec²

(If not given in question, then you take 0.12 in case of mercury)

$$d = 2.5 \ mm = 2.5 \times 10^{-3} \ m$$

$$h = \frac{4 \times 0.52 \times \cos 130^\circ}{13.6 \times 1000 \times 9.81 \times 2.5 \times 10^{-3}} = -0.004 \ m$$

(-ve sign indicate capillary depression)

Problem-5

Find out the minimum size of glass tube that can be used for measure water level if the capillary rise in the tube is restricted to 2 mm. Take σ of water in contact with air as 0.0735 N/m.

Sol^n   Given data

$$h = 2 \times 10^{-3} \ m$$
$$\sigma = 0.0735 \ N/m$$
$$\theta = 0^\circ \ (\text{for water})$$

$$h = \frac{4 \sigma \cos \theta}{\rho g d}$$

$$\Rightarrow 2 \times 10^{-3} = \frac{4 \times 0.0735 \times \cos 0^\circ}{1000 \times 9.81 \times d}$$

$$\Rightarrow d = 0.015 \ m \Rightarrow 1.5 \ cm$$

Chapter - 2

— : Pressure & its measurement :

Pressure intensity : —

Consider a small area dA in large mass fluid. If the fluid is stationary, then the force exerted by the surrounding fluid on the area will always be perpendicular to the surface dA. Let dF is the force acting on the area dA, then the ratio of $\frac{dF}{dA}$ is known as pressure intensity.

→ It is represented by p.

Mathematically $p = \frac{dF}{dA} = \frac{Force}{Area}$

Unit : —

Force is in N, KN, Kg etc.
Area is in cm^2, m^2, mm^2.

So p has a unit of N/mm^2 or kN/m^2 or kg/cm^2 etc.

Pressure variation in fluid at rest : —

The pressure at any point in fluid at rest is obtained by the hydrostatic law which states that rate of change of pressure in vertically downward direction must be equal to specific weight of the fluid at that point.

As → hydrostatic law.

$$\frac{dp}{dz} = \gamma = \rho g$$

$$\Rightarrow dp = \rho g \cdot dz$$

Integrating both sides,

$$\int dp = \int \rho g \cdot dz = \rho g \int dz = \rho g z$$

$$\Rightarrow \boxed{p = \rho g z}$$

Problem

Q. Calculate the pressure due to a column of 0.3 of (a) water, (b) an oil of sp. gr. 0.8, and (c) mercury of sp. gr. 13.6. Take density of water $\rho = 1000 \ kg/m^3$

Sol^n Given data,

Height of liquid column, $z = 0.3 m$

The pressure at any point in a liquid is given by

$$P = \rho g z$$

$$\rho = 1000 \, kg/m^3$$

$$P = \rho g z = 1000 \times 9.81 \times 0.3 = 2943 \, N/m^2$$

$$= \frac{2943}{10^4} \, N/cm^2 = 0.2943 \, N/cm^2$$

(b) For oil of Sp. gr = 0.8

We know that the density of a fluid is equal to specific gravity of fluid multiplied by density of water.

∴ Density of oil,

$$\rho_0 = Sp. \, gr. \, of \, oil \times density \, of \, water.$$

$$= 0.8 \times 10^3 = 0.8 \times 1000$$

$$= 800 \, kg/m^3$$

(ρ_0 = density of oil)

$$P = \rho_0 \times g \times z$$

$$= 800 \times 9.81 \times 0.3$$

$$= 2354.4 \, N/m^2 = \frac{2354.4}{10^4} \, \frac{N}{cm^2}$$

$$= 0.2354 \, \frac{N}{cm^2} \qquad \underline{Ans}$$

(c) For mercury, Sp. gr. = 13.6

We know that the density of a fluid is equal to specific gravity of fluid multiplied by density of water.

∴ Density of mercury,

$$\rho_s = Specific \, gravity \, of \, mercury \times density \, of \, water$$

$$= 13.6 \times 1000 = 13600 \, kg/m^3$$

$$P = \rho_s \times g \times z$$

$$= 13600 \times 9.81 \times 0.3$$

$$P = 40025 \, N/m^2$$

$$= \frac{40025}{10^4} = 4.002 \, \frac{N}{cm^2} \qquad \underline{Ans}$$

Q.2 The pressure intensity at a point in a fluid
is given 3.924 N/cm². Find the corresponding height
of fluid when the fluid is (a) water, and
(b) oil of sp. gr. 0.9.

Solⁿ   Given data,

pressure intensity, $P = 3.924 \frac{N}{cm^2}$

$= 3.924 \times 10^4 \frac{N}{m^2}$

The corresponding height (z) of the fluid is given

$$Z = \frac{P}{\rho \times g}$$

(a) For water, $\rho = 1000 \, kg/m^3$

$$Z = \frac{P}{\rho \times g} = \frac{3.924 \times 10^4}{1000 \times 9.81} = 4 \, m \text{ of water}$$ Ans

(b) For oil, sp. gr. = 0.9

Density of oil $\rho_0 = 0.9 \times 1000 = 900 \, kg/m^3$

$$Z = \frac{P}{\rho_0 \times g} = \frac{3.924 \times 10^4}{900 \times 9.81} = 4.44 \, m \text{ of oil}$$ Ans

Q.3 An oil of sp. gr. 0.9 is contained in a vessel.
At a point the height of oil is 40m. Find the
corresponding height of water at the point.

Solⁿ   Given data,

Sp. gr. of oil, $S_0 = 0.9$

Height of oil, $Z_0 = 40m$

Density of oil, $\rho_0 = $ sp. gr. of oil × density of
water

$= 0.9 \times 1000 = 900 \, kg/m^3$

Intensity of pressure

$P = \rho_0 \times g \times Z_0$

$= 900 \times 9.81 \times 40 \frac{N}{m^2}$

Corresponding height of water $= \frac{P}{\text{Density of water} \times g}$

$= \frac{900 \times 9.81 \times 40}{1000 \times 9.81}$

$= 0.9 \times 40 = 36 \, m \text{ of water}$ Ans

Q.4. An open tank contains water upto a depth of 2m and above it an oil of sp.gr. 0.9 for a depth of 1m. Find the pressure intensity (i) at the interface of the two liquids, and (ii) at the bottom of the tank.

Soln. Given data,

Height of water, $Z_1 = 2.0$m

Height of oil, $Z_2 = 1$m

Sp.gr. of oil, $S_0 = 0.9$

Density of water, $\rho_1 = 1000$ kg/m³

Density of oil, $\rho_2 = $ sp.gr. of oil $\times$

Density of water =

$= 0.9 \times 1000 = 900$ kg/m³

Pressure intensity at any point

$$p = \rho g Z$$

(i) At interface, i.e., at A

$$p = \rho_2 \times g \times 1.0$$
$$= 900 \times 9.81 \times 1.0$$
$$= 8829 \frac{N}{m^2} = \frac{8829}{10^4}$$
$$= 0.8829 \; N/cm^2 \quad Ans$$

(ii) At the bottom, i.e., at B

$$p = \rho_2 \times g Z_2 + \rho_1 + g \times Z_1$$
$$= 900 \times 9.81 \times 1.0 + 1000 + 9.81 \times 2.0$$
$$= 8829 + 19620$$
$$= 28449 \; N/m^2 = \frac{28449}{10^4} \; N/cm^2$$
$$= 2.8449 \; N/cm^2 \quad Ans$$

Absolute, Gauge, Atmospheric and vacuum pressure :-

The pressure on a fluid is measured in two different systems. In one system, it is measured above the absolute zero or complete vacuum and it is called the absolute pressure and in other system, pressure is measured above the atmospheric pressure and it is called gauge pressure.

1. Absolute pressure :—

→ Absolute pressure is defined as the [pressure] which is measured with reference to absolute vacuum pressure.

2. Gauge pressure :—

→ It is defined as the pressure which is measured with the help of a pressure measuring instrument, in which the atmospheric pressure is taken as datum. The atmospheric pressure scale is marked as zero.

3. Vacuum pressure :—

→ It is defined as the pressure below the atmospheric pressure.

→ The relationship between the absolute pressure, gauge pressure and vacuum pressure are shown in figure.

Absolute zero pressure
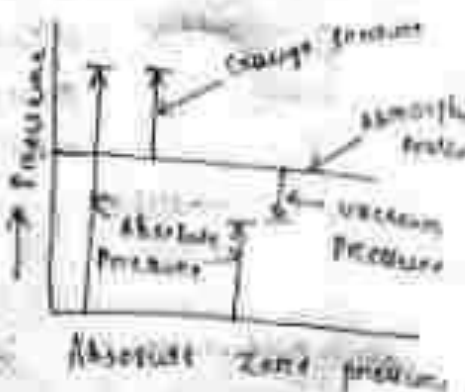
Mathematically :

(1) Absolute pressure = Atmospheric pressure + Gauge pressure

$$P_{ab} = P_{atm} + P_{gauge}$$

(2) Vacuum pressure = Atmospheric pressure − Absolute pressure

Problems :—

Q.1 — What are the gauge pressure and absolute pressure at a point 3m below the free surface of a liquid having a density of $1.53 \times 10^3 \, kg/m^3$ if the atmospheric pressure is equivalent to 750mm of mercury? the specific gravity of mercury is 13.6 and density of water = 1000 kg/m³.

Sol. Given data :-

depth of liquid, h = 3m

Density of liquid, ρ = $1.53 \times 10^3 \, kg/m^3$

Atmospheric pressure head, H_a = 750 mm of Hg

$$= \frac{750}{1000} = 0.750 \text{ of Hg}$$

Atmospheric pressure, $P_{atm} = \rho_0 \times g \times z_0$

where ρ_0 = density of Hg = sp. gr. of mercury
$\times$ density of water

$= 13.6 \times 1000 \text{ kg/m}^3$ ($\because z_0 = 0.760 \text{ m}$)

$= 100043 \text{ N/m}^2$

Pressure at a point, which is at a depth of
3m from the free surface of the liquid is
given by, $p = \rho_0 \times g \times z_1$

$= (1.53 \times 1000) \times 9 \times z_1$

$= 45028 \text{ N/m}^2$

Gauge pressure, $p = 45028 \text{ N/m}^2$ __Ans__

Now absolute pressure = Gauge pressure + Atmospheric pressure

$= 45028 + 100043$

$= 145090 \text{ N/m}^2$ __Ans__

Measurement of pressure :—

The pressure of a fluid is measured by
the following devices :
1. Manometers   2. Mechanical gauges

Manometers :—

Manometers are defined as the devices used for
measuring the pressure at a point in a fluid
by balancing the column of the fluid by the
same or another column of the fluid.

⟶ They are classified as
(i) Simple manometers
(ii) Differential manometers.

(i) Simple manometers :—

A simple manometer consists of a glass tube
having one of its ends connected to a point where
pressure is to be measured and other end
remains open to atmosphere.

⟶ common types of simple manometer are :
1. Piezometer
2. U-tube manometer

Piezometer :—

→ It is the simplest from of manometer.

→ The rise of liquid gives the pressure head at that point.

→ If at a point A, the height of liquid say water is h in piezometer tube, then

$$\text{pressure at A} = \rho \times g \times h \ \frac{N}{m^2}$$

U-tube manometer :—

It consists of glass tube bent in U-shape, one end of which is connected to a point at which pressure is to be measured and other end remains open to the atmosphere.

(Piezometer)

(a) For gauge pressure   (b) For vaccum pressure

(a) For gauge pressure :—

Let A is the point at which pressure is to be measured, whose value is P.

→ The datum is A-A

h₁ = Height of light liquid above the datum line.

h₂ = Height of heavy liquid above the datum line.

S₁ = density of light liquid = 1000 × S₁

S₂ = sp. gr. of light liquid

S₂ = sp. gr. of heavy liquid

S₂ = density of heavy liquid = 1000 × S₂

As the pressure is the same for the horizontal surface. Hence pressure above the horizontal datum line A-A in the left column and in the right

column of U-tube manometer should be same

pressure above A-A in the left column

$$= P + P_1 \times g \times h_1$$

pressure above A-A in the right column

$$= P_2 \times g \times h_2$$

Hence equating the two pressure $P + P_1 g h_1 = P_2 g h_2$

$$P = (P_2 g h_2 - P_1 \times g \times h_1)$$

(b) For vacuum pressure :-

For measuring vacuum pressure, the level of the heavy liquid in the manometer will be. Then

pressure above A-A in the left column :

$$P_2 g h_2 + P_1 g h_1 + P$$

Pressure head in the right column above A-A = 0

$$P_2 g h_2 + P_1 g h_1 + P = 0$$

$$P = - (P_2 g h_2 + P_1 g h_1)$$

Q. The right limb of a simple U-tube manometer containing mercury is open to the atmosphere while the left limb is connected to a pipe in which a fluid of sp.gr. 0.9 is flowing. The centre of the pipe is 12 cm below the level of mercury in the right limb. Find the pressure of fluid in the pipe if the difference of mercury level in the two limbs is 20cm.

Solⁿ   Given data,

sp. gr. of fluid, $s_1 = 0.9$

Density of fluid, $P_1 = s_1 \times 1000$

$$= 0.9 \times 1000$$

$$= 900 \, kg/m^3$$

Sp. gr. of mercury, $s_2 = 13.6$

Density of mercury,

$$P_2 = 13.6 \times 1000 \, kg/m^3$$

Difference of mercury level, $h_2 = 20cm = 0.20$

Height of fluid from A-A, $h_1 = 20 - 12$

$$= 8cm = 0.08 \, m$$

Let P = Pressure of fluid in pipe

Equating the pressure above A-A, we get

$$p + \rho_1 g h_1 = \rho_2 g h_2$$

$$p + 900 \times 9.81 \times 0.05 = 13.6 \times 1000 \times 9.81 \times 0.5$$

$$p = 13.6 \times 1000 \times 9.81 \times 0.2 - 900 \times 9.81 \times 0.05$$

$$= 8466.2 - 706 = 25977 \ N/m^2$$

$$= 2.597 \ N/cm^2 \ \underline{Ans}$$

Q. A simple U-tube manometer containing mercury is connected to a pipe in which a fluid of sp. gr. 0.8 and having vacuum pressure is flowing. The other end of the manometer is open to atmosphere. Find the vacuum pressure in pipe, if the difference of mercury level in the two limbs is 40 cm and the height of fluid in the left limb from the centre of pipe is 15 cm below.

Sol^n Given data,

Sp. gr. of Fluid , $S_1 = 0.8$

Sp. gr. of mercury , $S_2 = 13.6$

Density of Fluid , $\rho_1 = 800$

Density of mercury , $\rho_2 = 13.6 \times 1000$

Difference of mercury level, $h_2 = 40$ cm $= 0.4$ m

Height of fluid in left limb, $h_1 = 15$ cm $= 0.15$ m.

Let the pressure in pipe $= p$. Equating pressure above datum line A-A, we get.

$$S_2 g h_2 + \rho_1 g h_1 + p = 0$$

$$p = - [\rho_2 g h_2 + \rho_1 g h_1]$$

$$= - [13.6 \times 1000 \times 9.81 \times 0.4 + 800 \times 9.81 \times 0.15]$$

$$= - [53366.4 + 1177.2]$$

$$= - 54543.6 \ N/m^2$$

$$= - 5.454 \ N/cm^2 \quad \underline{Ans}$$
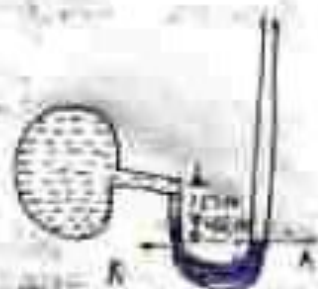
Differential Manometers :-

Differential manometers are the devices used for measuring the difference of pressures between two points in a pipe, or in two different pipes. A differential manometer consists of a U-tube, containing a heavy liquid , whose two ends are connected to the points , whose difference of pressures is to be measured. Most common types of differential manometers are :

2. U-tube differential manometer and
Inverted U-tube differential manometer.

U-tube differential manometer:

(a) Two points at different levels

(b) A and B are at the same level
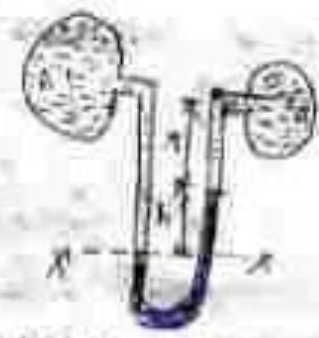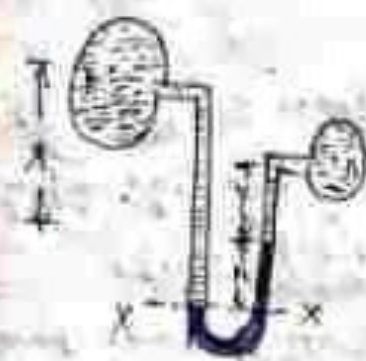
(a) The two points A and B are at different level and also contains liquids of different sp. gr. These points are connected to the U-tube differential manometer. Let the pressure at A and B are P_A and P_B.

Let h = difference of mercury level in the U-tube.

y = distance of the centre of B from the mercury level in the right limb.

x = distance of the centre of A from the mercury level in the right limb.

ρ_1 = density of liquid at A.

ρ_2 = density of liquid at B.

ρ_g = density of heavy liquid or mercury.

Taking datum line at X-X.

Pressure above X-X in the left limb $= \rho_1 g(h+x) + P_A$

where P_A = pressure at A.

Equating the two pressure, we have

$$\rho_1 g(h+x) + P_A = \rho_g \cdot g \cdot h + \rho_2 g y + P_B$$

$$P_A - P_B = \rho_g \cdot g \times h + \rho_2 \cdot g \cdot y - \rho_1 g(h+x)$$

$$= h \times g (\rho_g - \rho_1) + \rho_2 g y - \rho_1 g x$$

Difference of pressure at A and B

$$= h \times g (\rho_g - \rho_1) + \rho_2 g y - \rho_1 g x$$

(b) The points A and B are at the same level

and contains the same liquid of density ρ_1.

Pressure above $X-X$ in right limb

$$= \rho_g \times g \times h \pm \rho_1 \times g \times x + P_B$$

Pressure above $x-x$ in left limb $= \rho_1 \times g \times (h+x) + P_A$

Equating the two pressure

$$\rho_g \times g \times h + \rho_1 \times g \times x + P_B = \rho_1 \times g \times (h+x) + P_A$$

$$P_A - P_B = \rho_g \times g \times h + \rho_1 g x - \rho_1 g (h+x)$$

$$= g \times h (\rho_g - \rho_1).$$

Q. A pipe contains an oil of sp. gr. 0.9. A differential manometer connected at the two points A and B shows a difference in mercury level as 15cm. Find the difference of pressure at the two points.

Ans. Given data

Sp. gr. of oil , $S_1 = 0.9$

Density , $\rho_1 = 0.9 \times 1000 = 900 \, kg/m^3$

Difference in mercury level, $h = 15 cm = 0.15 m$

Sp. gr. of mercury , $S_g = 13.6$

∴ Density , $\rho_g = 13.6 \times 1000 \, kg/m^3$

The difference of pressure is given

$$P_A - P_B = g \times h (\rho_g - \rho_1)$$

$$= 9.81 \times 0.15 (13600 - 900)$$

$$= 18688 \, N/m^2 \quad \underline{Ans}$$

Q. A differential manometer is connected at the two points A and B of two pipes as shown in fig. The pipe A contains a liquid of sp. gr. 1.5 while pipe B contains a liquid of sp. gr. 0.9. The pressure at A and B are 1 kgf/cm² and 1.80 kgf/cm² respectively. Find the difference in mercury level in the differential manometer.

Sol. Given data

Sp. gr. of liquid at A, $S_1 = 1.5$

∴ $\rho_1 = 1500$

Sp. gr. of liquid at B, $S_2 = 0.9$

∴ $\rho_2 = 900$

Pressure at A,
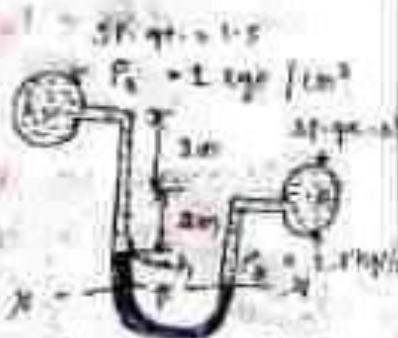
$$P_A = 1 \, kgf/cm^2$$
$$= 1 \times 10^4 \, kgf/m^2$$
$$= 10^4 \times 9.81 \, N/m^2 \quad (\because 1\,kgf = 9.81)$$

pressure at B:
$$P_B = 1.8 \, kgf/cm^2$$
$$= 1.8 \times 10^4 \, kgf/m^2$$
$$= 1.8 \times 10^4 \times 9.81 \, N/m^2$$

Density of mercury $= 13.6 \times 1000 \, kg/m^3$

Taking X–X as datum line.

pressure above X–X in the left limb
$$= 13.6 \times 1000 \times 9.81 \times h + 1500 \times 9.81 \times (2+3) + P_A$$
$$= 13.6 \times 1000 \times 9.81 \times h + 1500 \times 9.81 + 9.81 \times 10^4$$

pressure above X–X in the right limb =
$$= 900 \times 9.81 \times (h+3) + P_B$$
$$= 900 \times 9.81 \times (h+3) + 1.8 \times 10^4 \times 9.81$$

Equating the two pressures, we get.
$$13.6 \times 1000 \times 9.81\,h + 1500 \times 9.81 + 9.81 \times 10^4 =$$
$$900 \times 9.81 \times (h+3) + 1.8 \times 10^4 \times 9.81$$

Dividing by 1000×9.81, we get
$$13.6h + 7.5 + 10 = (h+3.0) \times 0.9 + 18$$
$$\Rightarrow 13.6h + 17.5 = 0.9h + 1.8 + 18$$
$$\Rightarrow 13.6h + 17.5 = 0.9h + 19.8$$
$$\Rightarrow (13.6 - 0.9)h = 19.8 - 17.5 \quad \text{or} \quad 12.7h = 0.3$$
$$\Rightarrow h = \frac{2.3}{12.7} = 0.181\,m = 18.1 \, cm \quad \underline{Ans}$$

Q. A differential manometer is connected at the two points A and B as shown in Fig. At B air pressure is $9.81 \, N/cm^2$ (abs), find the absolute pressure at A.

Sol. Given data

Air pressure at $B = 9.81 \, N/cm^2$
or $P_B = 9.81 \times 10^4 \, N/m^2$
Density of oil $= 0.9 \times 1000 = 900 \, kg/m^3$
Density of mercury $= 13.6 \times 1000 \, kg/m^3$
Let the pressure at A is P_A
Taking datum line at X–X

Pressure above Y-Y in the right limb:

$$= 1000 \times 9.81 \times 0.4 \times P_B$$
$$= 9886 + 9.806 \, 9 \times 100$$
$$= 103986$$

Pressure above X-X in the left limb:

$$= 13.6 \times 1000 \times 9.81 \times 0.1 8 + 900 \times 9.81 \times 0.2$$
$$+ P_A$$

$$= 13341.6 \pm 1765 h 8 + P_A$$

Equating the two pressure heads:

$$103986 = 13341.6 + 1765 \times 8 + P_A$$
$$P_A = 103986 - 15107.4 = 88878.8$$
$$P_A = 88878.8 \, N/m^2$$

$$= \frac{88878.8 \, N}{10000 \, cm^2} = 8.888 \, \frac{N}{cm^2}$$

∴ Absolute pressure at A = ? ? N/cm² **Ans**
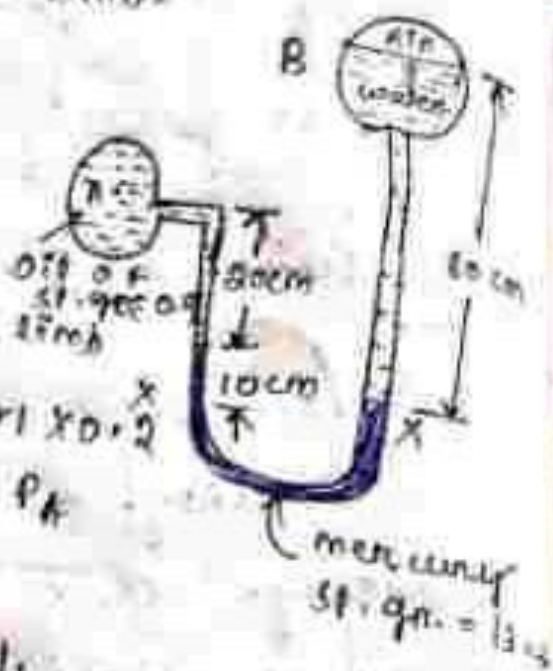
—: Pressure exerted on immersed surfaces :

Total pressure :—

Total pressure is defined as the force exerted by a static fluid on a surface either plane or curved when the fluid comes in contact with the surface.

Center of pressure (C.P) :—

Centre of pressure is defined as the point of application of the total pressure on the surface.

Vertical plane surface submerged in liquid :—

consider a plane vertical surface immersed in liquid as shown below

Let A = Total area of the surface

$\bar{h}$ = Distance of C.G from the free Surface of liquid

G = Center of gravity

P = Centre of pressure

h^* = Distance of centre of pressure (C.P) from free liquid surface.
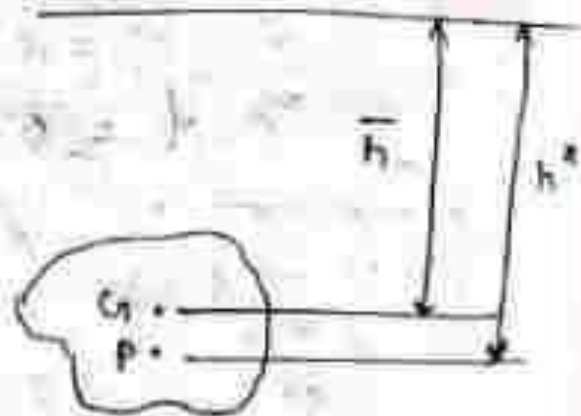
Therefore total pressure $(P) = \rho g A \bar{h}$

centre of pressure $(C.P) = h^* = \dfrac{I_G}{A\bar{h}} + \bar{h}$

Plane surface	C.G from the base	Area	Moment of Inertia about an axis passing through C.G. and parallel to base (I_G)	Moment of Inertia about base
1. Rectangle	$x = \dfrac{d}{2}$	$b d$	$\dfrac{bd^3}{12}$	$\dfrac{bd^3}{3}$
2. Triangle	$x = \dfrac{h}{3}$	$\dfrac{bh}{2}$	$\dfrac{bh^3}{36}$	$\dfrac{bh^3}{12}$
3. Circle	$x = \dfrac{d}{2}$	$\dfrac{\pi d^2}{4}$	$\dfrac{\pi d^4}{64}$	—
4. Trapezium	$x = \left[\dfrac{2a+b}{a+b}\right]\dfrac{h}{3}$	$\dfrac{a+b}{2} \times h$	$\left[\dfrac{a^2+4ab+b^2}{36(a+b)}\right] \times h^3$	
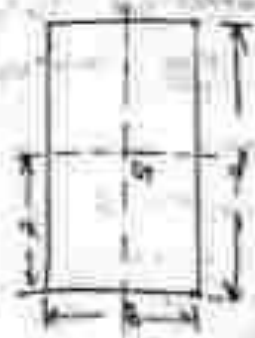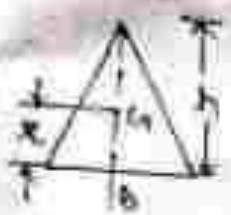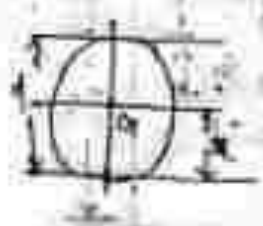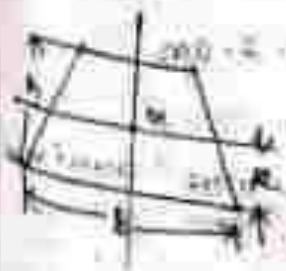
Problem

A rectangular plane surface is 2m wide and 3m deep. It lies in vertical plane in water. Determine the total pressure and position of centre of pressure on the plane surface when the upper edge is horizontal and (a) coincides with water surface (b) 2.5m below the free water surface.

Sol:

Width of plane surface $= b = 2m$

Depth of plane surface $= d = 3m$

(a) Upper edge coincides with water surface

$$F = \rho g A \bar{h}$$

Where $\rho = 1000 \, kg/m^3$

$g = 9.81 \, m/s^2$

$A = 3 \times 2 = 6 \, m^2$

$\bar{h} = \frac{1}{2}(3) = 1.5m$

$F = 1000 \times 9.81 \times 6 \times 1.5$

$= 88290 \, N$

Depth of centre of pressure is given by equation

$$h^* = \frac{I_G}{A\bar{h}} + \bar{h}$$

Where $I_G = $ M.O.I. about C.G of the area of surface

$= \frac{bd^3}{12} = \frac{2 \times 3^3}{12} = 4.5 \, m^4$

$h^* = \frac{4.5}{6 \times 1.5} + 1.5 = 0.5 + 1.5 = 2.0m$

(b) Upper edge is 2.5m below water surface

Total pressure (F) is given

$$F = \rho g A \bar{h}$$

Where $\bar{h} = $ Distance of C.G from free surface of water $= 2.5 + \frac{3}{2} = 4.0m$

$$F = 1000 \times 9.91 \times 6 \times 40$$
$$= 9 \, 3 \, \text{c} \, 044 \, \text{N} \quad \text{An}$$

centre of pressure is given

by $h^* = \dfrac{I_n}{A\bar{h}} + \bar{h}$

where $I_G = 4.5$, $A = 6.0$,

$$\bar{h} = 4.0$$

$$h^* = \dfrac{4.5}{6.0 \times 4.0} + 4.0$$

$$= 0.1875 + 4.0 = 4.1875 = 4.1875 \, m$$
$$\underline{An}$$
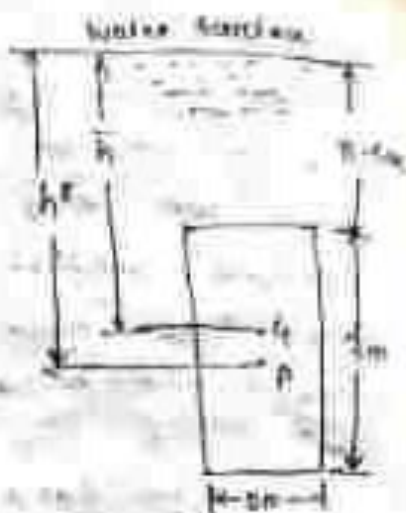
Problem:

Determine the total pressure on a circular plate of diameter 1.5 m which is placed vertically in water in such a way that the centre of the plate is 3 m below the free surface of water. Find the position of centre of pressure also.

Sol^n

Given data

Dia. of plate $= d = 1.5 \, m$

Area $= A = \dfrac{\pi}{4}(1.5)^2 = 1.767 \, m^2$

$\bar{h} = 3.0 \, m$

Total pressure is given

$$F = \rho g A \bar{h}$$

$$= 1000 \times 9.51 \times 1.767 \times 3 \times 10^3$$

$$= 52002.11 \, N \quad \underline{An}$$

Position of centre of pressure (h^*)

$$h^* = \dfrac{I_G}{A\bar{h}} + \bar{h}$$

where $I_G = \dfrac{\pi d^4}{64} = \dfrac{\pi \times 1.5^4}{64} = 0.2485 \, m^4$

$$h^* = \dfrac{0.2485}{1.767 \times 3.0} + 3.0 = 0.0466 + 3.0$$

$$= 3.0466 \, m \quad \underline{An}$$
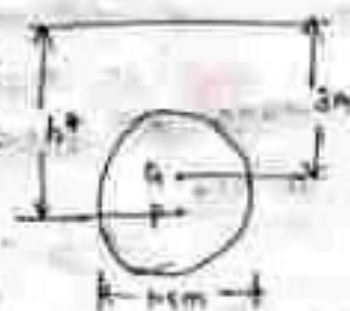
Problem

Determine the total pressure and centre of pressure on an isoscales triangular plate of base 4m and altitude 4m when it is immersed vertically in an oil of specific gravity 0.9. The base of the plate coincides with the free surface of oil.

Sol^n

Given data

Base of plate = b = 4m

Height of plate = h = 4m

$\therefore$ Area, $A = \dfrac{b \times h}{2} = \dfrac{4 \times 4}{2} = 8 m^2$

Specific gravity of oil

$$S = 0.9$$

Density of oil $\rho = 900 \, kg/m^3$

The distance of c.g from free surface of oil,

$$\bar{h} = \frac{1}{3} \times h = \frac{1}{3} \times 4 = 1.33 \, m$$

Total pressure (F) $= \rho \, g \, A \bar{h}$

$$= 900 \times 9.81 \times 8.0 \times 1.33 \, N$$

$$= 93977.64 \, N \quad \underline{\underline{Ans}}$$

centre of pressure (h^*) from the free surface of oil,

$$h^* = \frac{I_G}{A \bar{h}} + \bar{h}$$

Where I_G = m.o.i of triangular section about its c.g is $\dfrac{bh^3}{36} = \dfrac{4 \times 4^3}{36} = 7.11 \, m^4$

$$h^* = \frac{7.11}{8.0 \times 1.33} + 1.33$$

$$= 0.66667 + 1.33 = 1.99 \, m \quad \underline{\underline{Ans}}$$
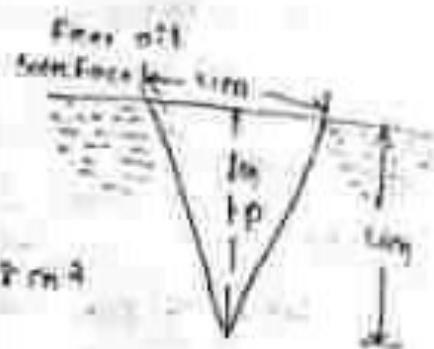
Horizontal plane surface submerged in liquid:-

Consider a plane horizontal surface immersed in a static liquid. As every point of the surface is at the same depth from the free surface of the liquid, the pressure intensity will be equal on the entire surface and equal to $P = \rho g h$, where h is depth of surface.

Let,

A = Total area of surface

Then total force, F, on the surface

Free surface

$$= P \times Area = \rho g \times h \times A$$
$$= \rho g A \bar{h}$$

Where $\bar{h}$ = Depth of c.g from free surface of liquid.
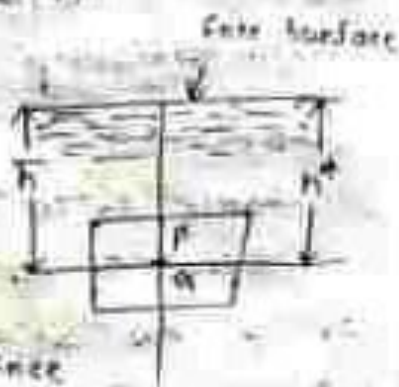
Problem

Figure shows a tank full of water. Find (i) Total pressure on the bottom of tank.

Soln: Given data

Depth of water on bottom of tank $h_1 = 3 + 0.6 = 3.6\,m$

Width of tank = 2m

Length of tank at bottom = 4m

(ii) Area at the bottom

$$A = 4 \times 2 = 8\,m^2$$

(ii) Total pressure F on the bottom is

$$F = \rho g A \bar{h} = 1000 \times 9.81 \times 8 \times 3.6$$
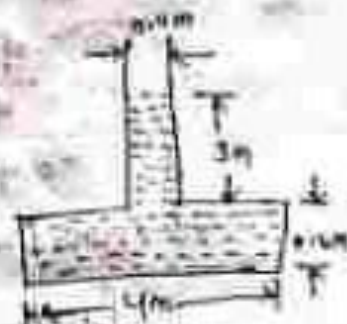
-: Kinematics of fluid flow :-
RATE OF FLOW OR DISCHARGE (Q):-

It is defined as the quantity of a fluid flowing per second through a section of a pipe or a channel. For an incompressible fluid (or liquid) the rate of flow or discharge is expressed as the volume of fluid flowing across the section per second. For compressible fluids the rate of flow is usually expressed as the weight of fluid flowing across the section. Thus

(i) For liquids the units of Q are m^3/sec or lit/sec.

ii) For gases the unit of Q is kgf/s or Newton/s.

Consider a liquid flowing through a pipe in which.

A = Cross-Sectional area of pipe.

V = Average velocity of fluid across the Section.

The discharge $Q = A \times V$

COMNTINUITY EQUATION :-

The equation based on the principle of conservation of mass is called continuity equation. Thus for a fluid flowing through the pipe at all the cross-section the quantity of fluid per second is constant. Consider two cross-section of a pipe as shown.

Let V_1 = Average velocity at cross-section 1-1.

P_1 = Density at Section 1-1

A_1 = Area of pipe at section 1-1.

and V_2, P_2, A_2 are corresponding values at section 2-2

Then rate of flow at section 1-1 = $P_1 A_1 V_1$

Rate of flow at Section 2-2 = $P_2 A_2 V_2$

According to law of conservation of mass

Rate of flow at Section 1-1 = Rate of flow at Section 2-2

Or $P_1 A_1 V = P_2 V_2 A_2$

Equation (5.2) is applicable to the compressible as well as incompressible fluid and is called continuity eqn.

If the fluid is incompressible, then $p_1 = p_2$ and continuity equation (5.2) reduces to

$$A_1 V_1 = A_2 V_2$$

Problem :-

The diameters of a pipe at the section 1 and 2 are 10cm and 15cm respectively. Find the discharge through the pipe if the velocity of water flowing through the pipe at section 1 is 5 m/s. Determine also the velocity at section 2.

Solution :-

At section 1. $D_1 = 10cm = 0.1m$

$$A_1 = \frac{\pi}{4} (D^2) = \frac{\pi}{4} (.1)^2 = 0.007854 m^2$$

$$V_1 = 5 m/s$$

At section 2. $D_2 = 15cm = 0.15m$

$$A_2 = \frac{\pi}{4} (.15)^2 = 0.01767 m^2$$

ii) Discharge through pipe is given by equation (5.1)

i.e $Q = A_1 \times V_1$

$$= 0.007854 \times 5 = 0.03927 \ m^3/s$$

using eq (5.2), We have $A_1 V_1 = A_2 V_2$

$$= 0.007854 \times$$

$$V_2 = \frac{A_1 V_1}{A_2} = \frac{0.007854 \times 5}{0.01767} = 2.22 \ m/s$$

problem :-   A 30cm diameter pipe, conveying water, branches into two pipes of diameter 20cm and 15cm respectively. If the average velocity in 30cm diameter pipe is 2.5m/s. Find the discharge in this pipe. Also determine the velocity in pipe if the average velocity in 20cm diameter pipe is 2 m/s.

Solution :-
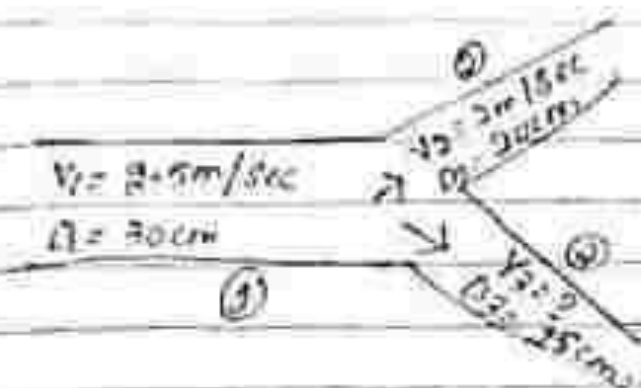
$D = 30 \text{cm} = 0.30m.$

$A_1 = \frac{\pi}{4} D_1^2 = \frac{\pi}{4} \times \cdot 3^2 = 0.0706 \text{ m}^2.$

$V_1 = 2.5 \text{ m/s}.$

$D_2 = 20 \text{cm} = 0.20m$

$A_2 = \frac{\pi}{4} (\cdot 2)^2 = \frac{\pi}{4} \times \cdot 4 = 0.0314 \text{ m}^2.$

$V_2 = 2 \text{ m/s}.$

$D_3 = 15\text{cm} = 0.15 m.$

$A_3 = \frac{\pi}{4} (\cdot 15)^2 = \frac{\pi}{4} \times 0.225 = 0.01767 \text{ m}^2$

Find i) Discharge in pipe 1 or Q_1
      ii) Velocity in pipe of dia 15cm or V_3.

Let Q_1, Q_2, $\& $ Q_3 are discharge in pipe 1,2 and 3 respectively.

Then according to continuity eqn.
$$Q_1 = Q_2 + Q_3.$$

i) The discharge Q_1 in pipe 1 is give by
$Q_1 = A_1 V_1 = 0.0706 \times 2.5 \text{ m}^3/s = 0.1767 \text{ m}^3/s \underline{\text{ Ans}}.$

ii) Value of V_3
$$Q_2 = A_2 V_2 = 0.0314 \times 2.0 = 0.0628 \text{ m}^3/s.$$

Substituting the values of Q_1 and Q_2 in eq (1).
$$0.1767 = 0.0628 + Q_3.$$

$Q_3 = 0.1767 - 0.0628 = 0.1139 \text{ m}^3/s$

But $Q_3 = A_3 V_3 = 0.01767 \times V_3$ or $0.1139 = 0.01767 \times V_3.$

$$V_2 = \frac{0.1139}{0.01767} = 6.514 \text{ m/s}. \quad \underline{Ans}$$

Problem :-

Water flows through a pipe AB 1·2 m diameter at 3 m/s and then passes through a pipe BC 1·5 diameter. At C, the pipe branches. Branch CD is 0.8m diameter and carries one third of the flow in AB. The flow velocity in branch CE is 2·5 m/s. Find the volume rate of flow in AB, the velocity in CD and the diameter of CE.

Solution

Diameter of pipe	$D_{AB} = 1·2m$.
Velocity of flow through AB	$V_{AB} = 3·0 m/s$.
Dia of pipe BC	$D_{BC} = 1·5 cm$
Dia of branched pipe CD	$D_{CD} = 0·8 m/s$
Velocity of flow in pipe CE	$V_{CE} = 2·5 m/s$
Let the flow rate in pipe AB	$= Q m/s$
Velocity of flow in pipe. BC	$V_{BC} m/s$
Velocity of flow in pipe CD	$= V_{CD} m/s$
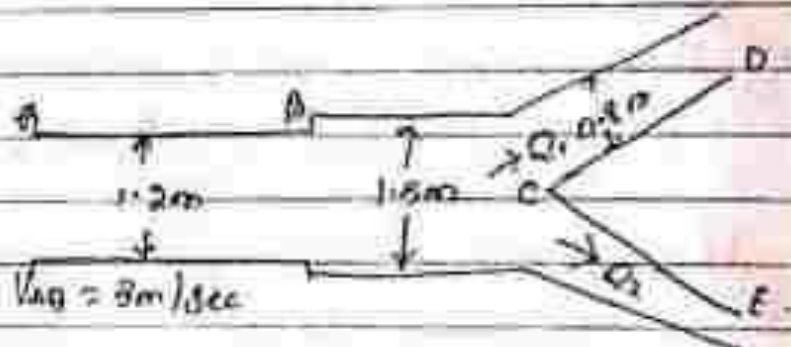
$$V_{AB} = 3m/sec$$
$$V_{CE} = 2·5 m/sec.$$

Diameter of pipe. $CE = D_{CE}$.
Then flow rate through $CD = Q/3$.
and flow rate through $CE = Q - Q/3 = \frac{2Q}{3}$.

i) New Volume flow rate through AB = $Q = V_{AB} \times$ Area of AB
$$= 3.0 \times \frac{\pi}{4}(D_{AB})^2 = 3.0 \times \frac{\pi}{4}(1.2)^2$$
$$= 3.393 \, m^3/s \quad \text{Ans}$$

ii) Applying continuity equation to pipe AB and pipe BC:
$V_{AB} \times$ Area of pipe AB $= V_{BC} \times$ Area of pipe BC

or $\qquad 3.0 \times \frac{\pi}{4}(D_{AB})^2 = V_{BC} \times \frac{\pi}{4}(D_{BC})^2$

or $\qquad 3.0 \times (1.2)^2 = V_{BC} \times (1.5)^2$

or $\qquad V_{BC} = \dfrac{3 \times 1.2^2}{1.5^2} = 1.92 \, m/s \quad \text{Ans}$

iii) The flow rate through pipe
$$CD = Q_1 = \frac{Q}{2} = \frac{3.393}{2} = 1.131 \, m^3/s$$

$$Q_1 = V_{CD} \times \text{Area of pipe } CD \times \frac{\pi}{4}(D_{CD})^2$$
$$1.131 = V_{CD} \times \frac{\pi}{4} \times 0.8^2 = 0.5026 \, V_{CD}$$

$$V_{CD} = \frac{1.131}{0.5026} = 2.25 \, m/s \quad \text{Ans}$$

iv) Flow rate through CE.
$$Q_2 = Q - Q_1 = 3.393 - 1.131 = 2.262 \, m^3/s$$
$$Q_2 = V_{CE} \times \text{Area of pipe } CE = V_{CE} \frac{\pi}{4}(D_{CE})^2$$
$$2.262 = 2.5 \times \frac{\pi}{4} \times (D_{CE})^2$$

$$D_{CE} = \sqrt{\frac{2.262 \times 4}{2.5 \times \pi}} = \sqrt{1.152} = 1.0735 \, m$$

$\therefore \qquad$ Diameter of pipe $CE = 1.0735 \, m \quad \text{Ans}$

BERNOULLI'S EQUATION FROM EULER'S EQUATION :—
Bernoulli's equation is obtained by integrating the euler's equation of motion (6.2) as.

$$\int \frac{dp}{\rho} + \int g\,dz + \int V\,dv = \text{constant}$$

If flow is incompressible, ρ is constant and

$$\therefore \quad \frac{p}{\rho} + gz + \frac{V^2}{2} = \text{constant}$$

or

$$\frac{p}{\rho g} + z + \frac{V^2}{2g} = \text{constant}$$

or

$$\frac{p}{\rho g} + \frac{V^2}{2g} + z = \text{constant}$$

Equation (vi) is a Bernoulli's equation in which

$\frac{p}{\rho g} = $ Pressure energy per unit weight of fluid or pressure head.

$V^2/2g = $ kinetic energy per unit weight or kinetic head.

$z = $ potential energy per unit weight or potential head.

Assumptions:— The following are the assumptions made in the derivation of Bernoulli's eqⁿ.

1) The fluid is Ideal i.e. Viscosity is zero.
ii) The flow is Steady.
iii) The flow is Incompressible.
(iv) The flow is Irrotational.

Problem:— Water is flowing through a pipe of 5cm diameter under a pressure of 29.43 N/cm² and with mean Velocity of 2.0 m/s. Find the total head or total energy per unit weight of the water at cross-section which is 5m above the datum line.

Solution:—

Diameter of pipe = 5cm = 0.5 m

Pressure, $P = 29.43 \ N/cm^2 = 29.43 \times 10^4 \ N/m^2$

Velocity $\qquad$ $V = 2\cdot 0\ m/s$.

Datum head $\qquad$ $Z = 5m$

Total head $\qquad$ = pressure head + kinetic head + datum head

pressure head $= \dfrac{P}{\rho g} = \dfrac{29.43 \times 10^{3}}{1000 \times 9.81} = 30m$ $\left\{\begin{array}{l}\text{for water} \\ = 1000\ \dfrac{Kg}{m^3}\end{array}\right.$

Kinetic head $= \dfrac{V^2}{2g} = \dfrac{2 \times 2}{2 \times 9.81} = 0.204\,m$

$\therefore$ Total head $= \dfrac{P}{\rho g} + \dfrac{V^2}{2g} + Z = 30 + 0.204 + 5 = 35.204\,m$ **Ans**

Problem :— A pipe, through which water is flowing, is having diameter 20cm and 10cm of at the cross-section 1 and 2 respectively. The Velocity of Water at section 1 is given 4.0m. Find the Velocity head at section 1 and 2 also rate of discharge.

Solution :—

$D_1 = 20cm = 0.2m$

$\therefore$ Area $A_1 = \dfrac{\pi}{4} D_1^2 = \dfrac{\pi}{4}(0.2)^2 = 0.0314\,m^2$

$V_1 = 4.0\ m/s$

$D_2 = 0.1m$

$\therefore A_2 = \dfrac{\pi}{4}(0.1)^2 = 0.00785\,m^2$

i) Velocity head at section 1 —

$= \dfrac{V_1^2}{2g} = \dfrac{4.0 \times 4.0}{2 \times 9.81} = 0.815\,m$ **Ans**

ii) Velocity head at Section 2 $= V_2^2/2g$.

To find V_2 apply continuity equation at 1 or 2 :—

$A_1 V_1 = A_2 V_2$ &c. $V_2 = \dfrac{A_1 V_1}{A_2} = \dfrac{.0314 \times 4.0}{.00785} = 16.0\,m/s$.

$\therefore$ Velocity head at section 2 $= \dfrac{V_2^2}{2g} = \dfrac{16.0 \times 16.0}{2 \times 9.81}$

$= 13.047\,m$ **Ans**

iii) Rate of discharge $= A_1 V_1$ or $A_2 V_2$
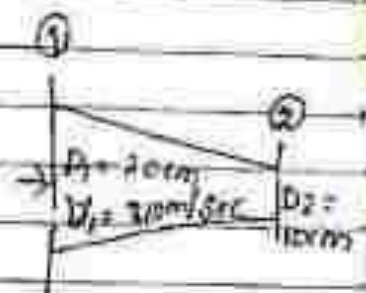
$= 0.0314 \times 4.0 = 0.1256 \, m^3/s$

$= 125.6 \, liters/s$ __Ans.__

__Problem :-__ The Water is flowing through a pipe having diameter 20 cm and 10 cm at Section 1 and 2 respectively. The rate of flow through pipe is 35 litres. The Section 1 is 6 m above datum and Section 2 is 4 cm above datum. If the pressure at Section 1 is 39.24 N/cm², find the intensity of pressure at Section 2.

__Solution :-__

$D_1 = 20 \, cm = 0.2 \, m$

At Section 1 $A_1 = \dfrac{\pi}{4}(2)^2 = 0.0314 \, m^2$

$\quad P_1 = 39.24 \, N/cm^2$

$\quad = 39.24 \times 10^4 \, N/m^2$

$\quad Z_1 = 6.0 \, m$
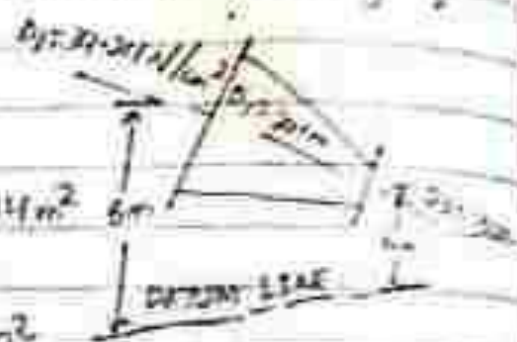
At Section 2 $\quad D_2 = 0.10 \, m$

$\quad A_2 = \dfrac{\pi}{4}(0.1)^2 = 0.00785 \, m^2$

$\quad Z_2 = 4 \, m$

$\quad P_2 = ?$

Rate of flow : $\quad Q = 35 lit/s = \dfrac{35}{1000} = 0.035 \, m^3/s$

Now : $\quad Q = A_1 V_1 = A_2 V_2$

$\therefore \quad V_1 = \dfrac{Q}{A_1} = \dfrac{0.035}{0.0314} = 1.114 \, m/s$

and $\quad V_2 = \dfrac{Q}{A_2} = \dfrac{0.035}{0.00785} = 4.456 \, m/s$

applying Bernoulli's eqⁿ at Section 1 & 2 we get

$$\dfrac{P_1}{\rho g} + \dfrac{V_1^2}{2g} + Z_1 = \dfrac{P_2}{\rho g} + \dfrac{V_2^2}{2g} + Z_2$$

or $\quad \dfrac{39.24 \times 10^4}{1000 \times 9.81} + \dfrac{(1.114)^2}{2 \times 9.81} + 6.0 = \dfrac{P_2}{1000 \times 9.81} + \dfrac{(4.456)^2}{2 \times 9.81}$

Oir $40 + 0.062 + 6.0 \quad \frac{P_2}{9810} + 1.012 + 4.0$

or $46.063 = \frac{P_2}{9810} + 5.012$

∴ $\frac{P_2}{9810} = 46.063 - 5.012 = 41.051$

$P_2 = 41.051 \times 9810 \text{ N/m}^2$

$= \frac{41.051 \times 9810}{10^4} \text{ N/cm}^2 = 40.27 \text{ N/cm}^2 \underline{Ans.}$

problem :- A pipe of diameter 400mm carries water at a velocity of 25 m/s. The pressure at the points A & B are given as 29.43 N/cm² and 22.563 N/cm² respectively while the datum head at A and B are 28 m and 30m. Find the loss of head between A & B.

Solution :-

Dia of pipe $D = 400mm = 0.4$

Velocity $V = 25 m/s$

At point A : $P_A = 29.43 \text{ N/cm}^2 = 29.43 \times 10^4 \text{ N/m}^2$

$Z_A = 28m$

$V_A = V = 25 m/s$

∴ Total energy at A

$E_A = \frac{P_A}{\rho g} + \frac{V_A^2}{2g} + Z_A$

$= \frac{29.43 \times 10^4}{1000 \times 9.81} + \frac{25^2}{2 \times 9.81} + 28$

$= 30 + 31.85 + 28 = 89.85 m.$

At point B : $P_B = 22.563 \text{ N/cm}^2 = 22.563 \times 10^4 \text{ N/m}^2$

$Z_B = 30m$

$V_B = V = V_A = 25 m/s$

Total energy at B : $E_B = \frac{P_B}{\rho g} + \frac{V_B^2}{2g} + Z_B$
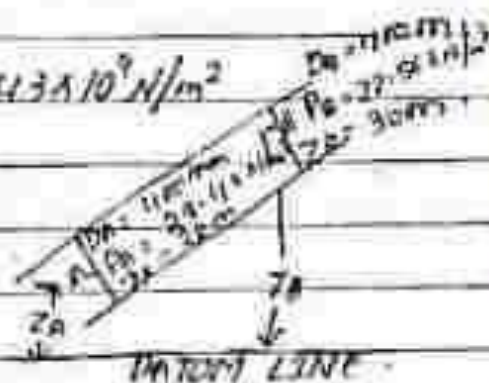
$$= \frac{22.563 \times 10^4}{1000 \times 9.81} + \frac{25^2}{2 \times 9.81} + 30 = 23 + 31 + 25 + 30$$

$$= 89.85 m$$

Loss of energy $= E_A - E_B = 89.85 - 84.85 = 5 m$ kg

PRACTICAL APPLICATIONS OF BERNOULLI'S EQUATION

Bernoulli's eqn is applied in all problems of incompressible fluid flow where energy consideration are involved. Rest we shall consider its application to the following measuring devices:

1. Venturimeter
2. Orifice meter
3. Pitot - tube

Venturimeter :— A venturimeter is a device used for measuring the rate of a flow of a fluid flowing through a pipe. It consist of three parts

i) A short converging part. ii) Throat
iii) Diverging part. It is based on the principle of Bernoulli eqn.

Expression for rate of flow through Venturimeter.
Consider a Venturimeter fitted in a horizontal pipe through which a fluid is flowing as shown in

Let $d_1 =$ diameter at Inlet or at Section
$P_1 =$ pressure at Section
$V_1 =$ Velocity of fluid at Section.
$a =$ area at Section $(1) = \frac{\pi}{4} d_1^2$

d_2, P_2, V_2, a_2 are corresponding values at Section 2.

Applying Bernoulli eqn at Section (1) & (2) we get.

$$\frac{P_1}{Pg} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{Pg} + \frac{V_2^2}{2g} + Z_2$$

As pipe is horizontal hence $z_1 = z_2$.

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} \quad \text{or} \quad \frac{P_1 - P_2}{\rho g} = \frac{V_2^2}{2g} - \frac{V_1^2}{2g}$$

But $\frac{P_1 - P_2}{\rho g}$ is the difference of pressure heads at section 1 and 2 and it is equal to h or $\frac{P_1 - P_2}{\rho g} = h$

Substituting the value of $\frac{P_1 - P_2}{\rho g}$ in the above eq^n.

$$h = \frac{V_2^2}{2g} - \frac{V_1^2}{2g}$$

Now applying continuity equation at section 1 & 2.

$$a_1 V_1 = a_2 V_2 \quad \text{or} \quad V_1 = \frac{a_2 V_2}{a_1}$$

Substituting the value of V_1 in eq^n.

$$h = \frac{V_2^2}{2g} - \frac{\left(\frac{a_2 V_2}{a_1}\right)^2}{2g} = \frac{V_2^2}{2g}\left[1 - \frac{a_2^2}{a_1^2}\right] = \frac{V_2^2}{2g}\left[\frac{a_1^2 - a_2^2}{a_1^2}\right]$$

$$= V_2^2 = 2gh \cdot \frac{a_1^2}{a_1^2 - a_2^2}$$

$$V_2 = \sqrt{2gh \cdot \frac{a_1^2}{a_1^2 - a_2^2}} = \frac{a_1}{\sqrt{a_1^2 - a_2^2}} \sqrt{2gh}$$

∴ Discharge $Q = a_2 V_2$.

$$= a_2 \cdot \frac{a_1}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh} = \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}$$

Equation gives the discharge under ideal conditions and is called , theoretical discharge Actual

$$Q_{act} = Cd \times \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}$$

Where Cd = Co-efficient of Venturimeter & its value is less than 1.

Value of h given by differential U-tube manometer

Case-1:- Let the differential manometer contains a liquid which is heavier than the liquid flowing through the pipe

S_h = Sp. gravity of the heavier liquid

S_o = sp. gravity of the liquid flowing through pipe

h = Difference of the heavier liquid column in u-tube

Then

$$h = x \left[\frac{S_h}{S_o} - 1 \right].$$

Case II :- If the differential manometer contains a liquid which is lighter than the liquid flowing through the pipe, the value of h is given by

$$h = x \left[1 - \frac{S_l}{S_o} \right].$$

Where

S_l = Sp. gz. of lighter liquid in u-tube

S_o = Sp. gz. of fluid flowing through pipe

x = Difference of the lighter liquid column in u-tube

Problem :- A horizontal venturimeter with inlet and throat diameter 30cm and 15cm respectively is used to measure the flow of water. The reading of differential manometer connected to the inlet and the throat is 20cm of mercury. Determine the rate of flow. Take $C_d = 0.98$.

Solution :-

Dia at inlet $d_1 = 30cm$

∴ Area at inlet $a_1 = \frac{\pi}{4} d_1^2 = \frac{\pi}{4} (30)^2 = 706.85 cm^2$

Dia at throat $d_2 = 15cm$

$a_2 = \frac{\pi}{4} \times 15^2 = 176.7 cm^2$

$C_d = 0.98$.

Reading of differential manometer = x = 20cm of

Mercury

∴ Difference of pressure head is given by.

$$h = x \left[\frac{S_h}{S_L} - 1 \right].$$

Where S_h = Sp. gravity of mercury = 13.6, S_L = Sp. gravity of Water = 1

$$= 20 \left[\frac{13.6}{1} - 1 \right] = 20 \times 12.6 \, cm = 252.0 \, cm \, of \, Water$$

The discharge through Venturimeter is given by eqn.

$$Q = Cd \; \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}.$$

$$= 0.98 \times \frac{706.85 \times 176.7}{\sqrt{(706.85)^2 - (176.7)^2}} \times \sqrt{2 \times 9.81 \times 252}$$

$$= \frac{36067992 \times 36}{\sqrt{499576.9 - 31223.7}} = \frac{3604593.36}{684.4}.$$

$$= 125756 \, cm^3/s = \frac{125756}{1000} \, lit/s = 125.756 \, lit/s$$

Problem :- An oil of Sp. gr. 0.8 is flowing through a Venturimeter having inlet diameter 20cm and throat diameter 10cm. The oil-mercury differential manometer shows a reading of 25cm. Calculate the discharge of oil through the horizontal Venturimeter. Take $C_d = 0.98$.

Solution :

Sp. gr. of oil, $S_o = 0.8$ Reading of differential

Sp. gr. of mercury $S_h = 13.6$ manometer = x = 25cm

Reading of pressure head $h = x \left[\frac{S_h}{S_o} - 1 \right]$.

$$= 25 \left[\frac{13.6}{0.8} - 1 \right] cm \, of \, oil = 25 \left[17 - 1 \right] = cm \, cm \, of \, oil$$

Dia at inlet : $d_1 = 20cm$

$$a_1 = \frac{\pi}{4} d_1^2 = \frac{\pi}{4} \times 20^2 = 314.16 \, cm^2.$$

$d_2 = 10cm$.

$$a_2 = \frac{\pi}{4} \times 10^2 = 78.54 \, cm^2.$$

$Cd = 0.98$.

∴ The discharge Q is given by

$$Q = Cd \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}$$

$$= 0.98 \times \frac{314.16 \times 78.54}{\sqrt{(314.16)^2 - (78.54)^2}} \times \sqrt{2 \times 981 \times 40}$$

$$= \frac{21421375.68}{\sqrt{98696 - 6168}} = \frac{21421375.68}{304} \, cm^3/s$$

$$= 70465 \, cm^3/s = 70465 \text{ liters } Ans.$$

Problem :— A horizontal venturimeter with inlet diameter 20cm and throat diameter 10cm is used to measure the flow of oil sp.gr. 0.8. The discharge of oil through venturimeter is 60 liters/s. Find the reading of the oil—mercury differential manometer. Take $Cd = 0.98$.

Solution :

$d_1 = 20cm$

$$a_1 = \frac{\pi}{4} 20^2 = 314.16 \, cm^2.$$

$d_2 = 10cm$

$$a_2 = \frac{\pi}{4} \times 10^2 = 78.54 \, cm^2$$

$$Cd = 0.98.$$

$$Q = 60 \text{ liters/s} = 60 \times 1000 \, cm^3/s$$

using the equation $Q = Cd \cdot \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}$

Or $\qquad 60 \times 1000 = 9.81 \times \dfrac{314.16 \times 78.54}{\sqrt{(314.16)^2 - (78.54)^2}} \times \sqrt{2 \times 981 \, h}$

$$\dfrac{12106.8 \cdot 18 \sqrt{h}}{981}$$

Or $\sqrt{h} = \dfrac{60 \times 1000}{12.71.0.58 \cdot 18} = 17.029.$

$h = (17.029)^2 = 289.98 \, cm \, of \, oil.$

But $\qquad h = x\left[\dfrac{S_h}{S_o} - 1\right].$

Where $\qquad S_h = Sp. \, gr. \, of \, mercury = 13.6.$

$\qquad S_o = Sp. \, gr. \, of \, oil = 0.8.$

$\qquad x = Reading \, of \, manometer.$

$\therefore \qquad 289.98 = x\left[\dfrac{13.6}{0.8} - 1\right] = 16x.$

$$x = \dfrac{289.98}{16} = 18.12 \, cm.$$

$\therefore$ Reading of oil - mercury differential manometer

$$= 18.12 \, cm \qquad Ans$$

problem :- A horizontal venturimeter with inlet diameter 20cm and throat diameter 10cm is used to measure the flow of water. The pressure at inlet is 17.658 N/cm² and the vaccum pressure at the throat is 30 cm of mercury. Find the discharge of water through venturimeter. Take Cd = 0.98.

Solution :-

Dia - at inlet $\qquad d_1 = 20 cm.$

$\therefore \qquad a_1 = \dfrac{\pi}{4} \times (20)^2 = 314.16 cm^2.$

Dia, at throat $\qquad d_2 = 10cm$

$\qquad a_2 = \dfrac{\pi}{4} \times 10^2 = 78.71 cm^2.$

$\qquad P_1 = 17.658 N/cm^2. = 17.658 \times 10^4 \, N/m^2$

ρ for Water $= 1000 \frac{Kg}{m^3}$ and $\therefore \frac{P_1}{\rho g} = \frac{17.658 \times 10^4}{9.81 \times 10^3}$

$= 18 m$ of water

$\frac{P_2}{\rho g} = -30 cm$ of mercury

$= -0.30 m$ of mercury $= -0.30 \times 13.6 = -4.08 m$ of water

$\therefore$ Differential head

$= h = \frac{P_1}{\rho g} - \frac{P_2}{\rho g} = 18 - (-4.08)$

$= 18 + 4.08 = 22.08 m$ of Water $= 2208 cm$ of water

The discharge Q is given by equation

$$Q = Cd \cdot \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}$$

$$= 0.98 \times \frac{314.16 \times 78.54}{\sqrt{(314.16)^2 - (78.74)^2}} \times \sqrt{2 \times 981 \times 2208}$$

$$= \frac{50.228827.41}{304} \times 165555 cm^3/s = 65.55 lit/s$$

Orifice meter or orifice plate :- It is a device used for measuring the rate of flow of a fluid through a pipe. It is a cheaper device as compared to venturimeter. It also works on the same principle as that of venturimeter. It consist of a flat circular plate which has a circular sharp edged hole called orifice, which is concentric with the pipe. The orifice diameter is kept generally 0.5 times the diameter of the pipe, though it may vary from 0.4 to 0.8 times the pipe diameter.

$\qquad$ Let $P_1 =$ pressure at section (1)

$\qquad V_1 =$ velocity at section (1)

$\qquad a_1 =$ area of pipe at section (1)

Discharge $(Q) = \dfrac{cd \cdot ao \sqrt{2gh}}{\sqrt{1-\left(\frac{ao}{a_1}\right)^2}} = \dfrac{cd \cdot a_0 \cdot a_1 \sqrt{2gh}}{\sqrt{a_1^2 - ao^2}}$

$ao = $ area of orifice :-
$h = $ head :

Where $cd = $ Co-efficient of discharge for orifice meter
The Co-efficient of discharge for orifice meter
is mexets mxch smaller that for a venturimeter.

problem :- An orifice meter with orifice diameter
10cm is inserted in a pipe of 20cm diameter. The
pressure gauges fitted upstream and downstream
of the orifice meter give reading of 19.62
N/cm^2 and 9.81 N/cm^2 respectively. Co-efficient
of discharge for the orifice meter is given as 0.6.
Find the discharge of water through pipe

Solution :—

Dia of orifice $d_0 = 10cm$.

∴ Area, $a_0 = \dfrac{\pi}{9}(10)^2 = 78.54\ cm^2$

Dia of pipe = $d_1 = 20cm$.

∴ Area $a_1 = \dfrac{\pi}{4}(20)^2 = 314.16\ cm^2$.

$P_1 = 19.62\ N/cm^2 = 19.62 \times 10^4\ N/m^2$.

$\dfrac{P_1}{Pg} = \dfrac{19.62 \times 10^4}{1000 \times 9.81} = 10m\ of\ water$.

$h = \dfrac{P_1}{Pg} - \dfrac{P_2}{Pg} = 20.0 - 10.0 = 10m\ of\ water$
$= 1000\ cm\ of\ water$.

$Cd = 0.6$

The discharge Q is given by equation

$$Q = Cd \cdot \frac{a_0 a_1}{\sqrt{a_1^2 - a_0^2}} \times \sqrt{2gh}$$

$$= 0.6 \times \frac{70 - 50 \times 314.16}{\sqrt{(314.16)^2 - (70.50)^2}} \times \sqrt{2 \times 9.81 \times 1000}$$

$$= \frac{2073668.02}{304} = 68219.28 \ cm^3/s$$

$$= 68.21 \ litres/s$$

Pitot - tube :— It is a device used for measuring the velocity of flow at any point in a pipe or a channel. It is based on the principle that if the velocity of flow at a point becomes zero the pressure there is increased due to the conversion of the kinetic energy into pressure energy. The liquid rises up in the tube due to the conversion of kinetic energy into pressure energy. The velocity is determined by measuring the rise of liquid in the tube.

Consider two points (1) and (2) at the same level in such a way that point (2) is just as the inlet of the pitot - tube and point (1) is far away from the tube.

$P_1 =$ Intensity of pressure at point (1)
$V_1 =$ Velocity of flow at (1)
$P_2 =$ Pressure at point (2)
$V_2 =$ Velocity at point (2) Which is zero
$H =$ depth of tube in the liquid
$h =$ rise of liquid in the tube above the free surface.

Applying Bernoulli's eq^n at point (1) & (2)

$$\frac{P_1}{\rho g} + \frac{V_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{V_2^2}{2g} + z_2$$

Put $z_1 = z_2$ at points —(1) &(2) are on the same line and
$V_2 = 0$.

$\frac{f_1}{g}$ = pressure head at (1) = H.

$\frac{f_2}{g}$ = pressure head at (2) = (h+H).

Substituting their values, we get

$H + \frac{V_1^2}{2g} = (h+H)$ $\therefore h = \frac{V_1^2}{2g}$ or $V_1 = \sqrt{2gh}$.

This is theoretical velocity. Actual velocity is given by

$(V)_{act} = C_v \sqrt{2gh}$

Where C_v = co-efficient of pitot-tube.

$\therefore$ Velocity at any point

$V = C_v \sqrt{2gh}$

problem :- A pitot-static tube placed in the centre of a 200 mm pipe line has one orifice pointing upstream and other perpendicular to it. The mean velocity in the pipe is 0.80 of the central velocity. Find the discharge through through the pipe if the pressure difference between the two orifices is 60mm of water. Take the co-efficient of pitot tube as $C_v = 0.98$.

Solution :-

Dia of pipe $d = 200mm = 0.30m$.

Diff. of pressure head $h = 60mm$ of water = 6cm of water.

                                            $C_v = 0.98$

Mean velocity. $\overline{V} = 0.80 \times$ central velocity.

Central velocity is given by eq^n.

                  $= C_v \sqrt{2gh} = 0.98 \sqrt{2 \times 9.81 \times .06} = 1.063 m/s$

          $\overline{V} = 0.80 \times 1.063 = 0.8504 \, m/s$.

Discharge $Q =$ Area of pipe $\times \overline{V}$

$$= \frac{\pi}{4} d^2 \times \bar{v} = \frac{\pi}{4} (.30)^2 \times 0.6504 = 0.04 \, m^3/s$$

Problem :- Find the velocity of the flow of an oil through a pipe, when the difference of mercury level in a differential U-tube manometer connected to the two tappings of the pipe. take he is 100mm. take co-efficient of pitot-tube 0.98 and sp. gr. of oil = 0.8.

Solution :

Diff. of mercury level	$x = 100mm = 0.1m$
Sp. gr. of oil	$S_o = 0.8$
Sp. gr. of mercury	$S_g = 13.6$
	$C_v = 0.98$

Diff. of pressure head $h = x \left[\dfrac{S_g}{S_o} - 1 \right] = 1 \left[\dfrac{13.6}{0.8} - 1 \right]$

$$= 1.6 \, m \text{ of oil}.$$

∴ Velocity of flow $C_v \sqrt{2gh} = 0.98 \sqrt{2 \times 9.81 \times 1.6}$

$$= 5.49 \, m/s \quad \text{Ans}$$

Module - 2 :-   Chapter 2 :-

-: Flow over Notched and Weirs :-

Notch :-

A notch is a device used for measuring the rate of flow of a liquid through a small channel or a tank.

Weir :-

A weir is a concrete or masonary structure placed in an open channel over which the flow occurs. It is generally in the form of vertical wall, with a sharp edge at the top.

Types of Notches :-

According to the shape of the opening notches are classified into.

(a) Rectangular notch
(b) Triangular notch
(c) Trapzoidal notch
(d) Stepped notch

Types of weires :-

(1) According to shape of the opening

(a) Rectangular weir
(b) Triangular weir
(c) Trapzoidal weir

(2) According to the shape of crest

(a) Sharp - crested weir
(b) Broad - crested weir
(c) Narrow - crested weir
(d) Ogee - shaped weir

Discharge over a Rectangular notch or weir :-

Discharge $(Q) = \frac{2}{3} c_d \cdot L \sqrt{2g} \cdot H^{3/2}$

where, c_d = co-efficient of discharge

L = length of notch or weir

H = Head of water over crest

Q.1 Find the discharge of water flowing over a rectangular notch of 2m length when the measured head over the notch is 300mm. Take c_d = 0.60

Soln Given data :—

$$L = 2.0\ m$$
$$H = 300\ mm = 0.3\ m$$
$$Cd = 0.62$$

$$Q = \frac{2}{3} \times Cd \cdot L \cdot \sqrt{2g} \cdot H^{3/2}$$

$$= \frac{2}{3} \times 0.62 \times 2 \times \sqrt{2 \times 9.81} \times 0.3^{3/2}$$

$$= 0.592\ m^3/sec$$

Q.2 Determine the height of a rectangular weir of length 6m to be built across a rectangular channel. The maximum depth of water on the upstream side of the weir is 1.8m and discharge is 2000 lit/sec. Take Cd = 0.6.

Soln Given data :—

$$L = 6\ m$$
$$H_1 = 1.8\ m$$

Height of weir = H_2

Head of water = H

$$H_1 = H_2 + H$$

$$H = ?$$

Putting the given values in below formula

$$Q = \frac{2}{3} \cdot Cd \cdot L \cdot \sqrt{2g} \cdot H^{3/2}$$

$$\Rightarrow 2 = \frac{2}{3} \times 0.6 \times 6 \times \sqrt{2 \times 9.81} \times H^{3/2}$$

$$\Rightarrow H = 0.328\ m$$

Height of weir = $H_2 = H_1 - H$

$$= 1.8 - 0.328$$

$$= 1.472\ m$$

Discharge over a triangular notch or weir :—

Discharge $(Q) = \frac{8}{15} \times Cd \times \tan\frac{\theta}{2} \times \sqrt{2g} \times H^{5/2}$

θ = angle of notch

H = Head of water above V-notch

Cd = Co-efficient of discharge

Q.3 Find the discharge over a triangular notch of angle 60° when the head over the V-notch is 0.3 m. Take Cd = 0.6.

Soln Given data : Angle of notch (θ) = 60°

$H = 0.3\,m$

$C_d = 0.6$

Discharge $(Q) = \frac{8}{15} \times C_d \times \tan\frac{\theta}{2} \times \sqrt{2g} \times H^{5/2}$

$= \frac{8}{15} \times 0.6 \times \tan\frac{60^\circ}{2} \times \sqrt{2 \times 9.81} \times 0.3^{5/2}$

$= 0.0649\,m^3/sec$

Discharge over a trapezoidal notch or weir:-

Trapezoidal weir or notch is a combination of a rectangular and triangular notch or weir.
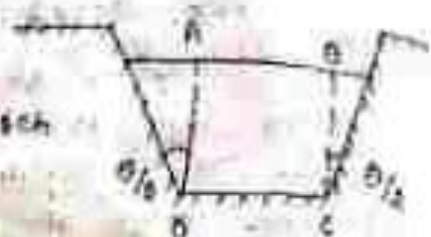
$Q_{trapezoidal} = Q_{rectangular} + Q_{triangular}$

Discharge $(Q) = \frac{2}{3} C_{d_1} \cdot L\sqrt{2g} \cdot H^{3/2} + \frac{8}{15} C_{d_2} \cdot \tan\frac{\theta}{2} \cdot \sqrt{2g} \cdot H^{5/2}$

Q.4 Find the discharge through a trapezoidal notch which is 1m wide at the top and 0.4m at the bottom and is 30cm in height. The head of water on the notch is 20 cm. Assume C_d for rectangular portion = 0.62 while for triangular portion = 0.60.

Given data:
Top width, $AE = 1\,m$
Base width, $CD = L = 0.4\,m$
Head of water, $H = 0.30\,m$
For rectangular portion, $C_{d_1} = 0.62$
For triangular portion, $C_{d_2} = 0.60$

From $\triangle ABC$, we have

$\tan\frac{\theta}{2} = \frac{AB}{BC} = \frac{(AE - CD)/2}{H}$

$= \frac{(1.0 - 0.4)/2}{0.3} = \frac{0.6/2}{0.3} = \frac{0.3}{0.3} = 1$

Discharge through trapezoidal notch is given

$Q = \frac{2}{3} C_{d_1} \times L \times \sqrt{2g} \times H^{3/2} + \frac{8}{15} C_{d_2} \times \tan\frac{\theta}{2} \times \sqrt{2g} \times H^{5/2}$

$= \frac{2}{3} \times 0.62 \times 0.4 \times \sqrt{2 \times 9.81} \times (0.3)^{3/2} + \frac{8}{15} \times 0.60 \times 1 \times \sqrt{2 \times 9.81} \times (0.3)^{5/2}$

$= 0.06549 + 0.02535$

$= 0.09084\,m^3/s = 90.84\,\text{lit/s}$ Ans

Discharge over a stepped notch :-

A stepped notch is a combination of rectangular notches. The discharge through stepped notch is equal to the sum of the discharges through the different rectangular notches.

Consider a stepped as shown in figure.

Let H_1 = Height of water above the crest of notch 1.

L_1 = Length of notch 1,

H_2, L_2 and H_3, L_3 are corresponding values for notches 2 and 3 respectively.

C_d = co-efficient of discharge for all notches.

Total discharge $Q = Q_1 + Q_2 + Q_3$

$$Q = \frac{2}{3} \times C_d \times L_1 \sqrt{2g} \left(H_1^{3/2} - H_2^{3/2} \right) + \frac{2}{3} C_d \times L_2 \sqrt{2g} \left(H_2^{3/2} - H_3^{3/2} \right) + \frac{2}{3} C_d \times L_3 \sqrt{2g} \times H_3^{3/2}$$

Q.5 Figure shows a stepped notch. Find the discharge through the notch if C_d for all sections 0.62

Sol. Given data,

L_1 = 40 cm
L_2 = 80 cm
L_3 = 120 cm

H_1 = 50+30+15 = 95 cm
H_2 = 30+15 = 45 cm ; H_3 = 15 cm

C_d = 0.62
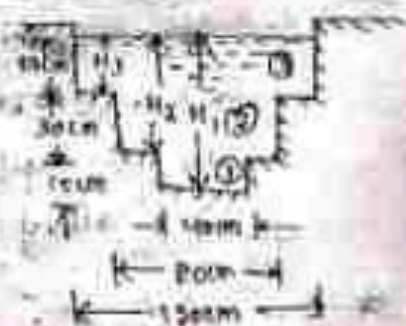
Total discharge $Q = Q_1 + Q_2 + Q_3$

$$Q_1 = \frac{2}{3} \times C_d \times L_1 \times \sqrt{2g} \left(H_1^{3/2} - H_2^{3/2} \right)$$

$$= \frac{2}{3} \times 0.62 \times 40 \times \sqrt{2 \times 981} \times \left[95^{3/2} - 40^{3/2} \right]$$

$$= 732.26 \left(925.94 - 410.54 \right)$$

$$= 154067 \, cm^3/s = 154.067 \, lit/s$$

$$Q_2 = \frac{2}{3} \times C_d \times L_2 \times \sqrt{2g} \times \left(H_2^{3/2} - H_3^{3/2} \right)$$

$$= \frac{2}{3} \times 0.62 \times 80 \times \sqrt{2 \times 981} \times \left(40^{3/2} - 15^{3/2} \right)$$

$$= 1464 \cdot 52 (715 \cdot 94 - 353 \cdot 55) \, cm^3/s$$

$$= 530141 \, cm^3/s = 530 \cdot 144 \, lit/s$$

and $Q_3 = \dfrac{2}{3} \times c_d \times L_3 \times \sqrt{2g} \times H_3^{3/2}$

$$= \dfrac{2}{3} \times 0 \cdot 62 \times 120 \times \sqrt{2 \times 9 \cdot 81} \times 50^{3/2}$$

$$= 776771 \, cm^3/s = 776 \cdot 771 \, lit/s$$

$\therefore \ Q = Q_1 + Q_2 + Q_3$

$$= 154 \cdot 067 + 530 \cdot 144 + 776 \cdot 771$$

$$= 1460 \cdot 98 \, lit/s \qquad \underline{Ans}$$

Types of Flow through the pipes :—

Steady Flow :—

Steady Flow is defined as that type of flow in which the fluid characteristics like velocity, pressure, density etc at a point does not change with time.

$$\frac{\partial v}{\partial t} = 0 \ , \ \frac{\partial p}{\partial t} = 0$$

Unsteady Flow :—

Unsteady Flow is that type of flow in which velocity, pressure and density at a point change with respect to time.

$$\frac{\partial v}{\partial t} \neq 0 \ , \ \frac{\partial p}{\partial t} \neq 0$$

Uniform Flow :—

Uniform Flow is defined as that type of flow in which the velocity at any given time does not change with respect to space i.e. length of direction of flow.

$$\frac{\partial v}{\partial s} = 0$$

Non - Uniform Flow :—

Non uniform flow is that type of flow in which the velocity at any given time change with respect to space.

$$\frac{\partial v}{\partial s} \neq 0$$

Reynold's Number :—

→ For a pipe flow the type of flow is determined by a non-dimensional number called Reynold's Number.

→ It is denoted by Re.

→ Mathematically $Re = \dfrac{\rho v D}{\mu}$

where ρ = Density of fluid

v = velocity of fluid flow

D = Dia of pipe

μ = dynamic viscosity of fluid

Laminar flow :—

→ Laminar flow is defined as that type of flow in which the fluid particles move along a well defined path.

→ For a laminar flow $Re < 2000$

Turbulent flow :—

→ Turbulent flow is that type of flow in which the fluid particles move in zig-zag way. Due to zig-zag way movement of fluid a heavy energy loss occurs.

→ For turbulent flow is be possible $Re > 4000$

Kinematic viscosity (ν) :—

It is defined as the ratio of dynamic viscosity (μ) to the density of fluid (ρ) flowing.

Mathematically, $\nu = \mu/\rho$

Q. Water flowing in a pipe of diameter 300 mm at a velocity of 3m/sec. Find the type of flow whether it is laminar or turbulent. Take $\nu = 0.01$ stoke.

Solⁿ   Given data,

Dia of pipe $(D) = 300\,mm = 0.3\,m$

velocity of pipe $(v) = 3m/sec$

kinematic viscosity $(\nu) = 0.01$ stoke

$= 0.01\ cm^2/sec$

$= 0.01 \times 10^{-4}\ m^2/sec$

$$Re = \frac{\rho v D}{\mu}$$

$\nu = \mu/\rho$

$= \frac{1000 \times 3 \times 0.3}{0.001}$ $\Rightarrow \mu = \nu \rho = \frac{0.01 \times 10^{-4}}{1000}$

$= 900000 / 9000$ $= 0.001\ \frac{N\cdot s}{m^2}$

∴ As $Re > 4000$ Flow is turbulent.

Module - 2 :-
    Chapter - 4 :-

Loss of energy in pipes :-

When a fluid is flowing through a pipe, the fluid experiences some resistance due to which some of the energy of fluid is lost. This loss of energy is classified as :

Energy losses

1- Major energy losses
This is due to friction and is calculated by the following formulas :-
(a) Darcy - Weisbach formula
(b) chezy's formula.

2- minor energy losses
This is due to
(a) Sudden expansion of pipe
(b) sudden contraction of pipe
(c) bend in pipe
(d) pipe fittings etc.
(e) An obstruction in pipe.

Loss of energy (or head) due to friction :-
(a) Darcy - Weisbach formula :-
    The loss of head (or energy) in pipes due to friction is calculated from Darcy-weisbach equation which has been derived in chapter 10 and is given by

$$ h_f = \frac{4 \cdot f \cdot L \cdot v^2}{d \times 2g} $$

h_f = loss of head due to friction
f = co-efficient of friction which is a function of Reynold's number

$= \frac{16}{Re}$ for $Re < 2000$ (viscous flow)

$= \frac{0.079}{Re^{1/4}}$ for Re varying from 4000 to 10^6

L = length of pipe
V = mean velocity of flow

d = diameter of pipe

Chezy's formula for finding head loss due

friction in pipe :—

Velocity $(V) = c\sqrt{mi}$

where, c = chezy's constant

m = Hydraulic radius

i = head loss per unit length

$m = \dfrac{A}{P}$ A = Area of flow

P = perimeter

$i = \dfrac{h_f}{L}$ h_f = Head loss

L = Length of flow

for pipe carrying full flow

$$m = \dfrac{A}{P} = \dfrac{\frac{\pi}{4}d^2}{\pi d} = \dfrac{d}{4}$$

Power Requirement (P) :—

$$P = \dfrac{\rho g Q h_f}{1000} \text{ in } k \cdot w$$

ρ = density of fluid in kg/m^3

g = Acceleration due to gravity m/sec²

Q = Rate of flow m^3/sec

h_f = head loss in m.

Q.1 Find the head loss due to friction in a pipe of diameter 300 mm and length 50m, through which water is flowing at a velocity of 3m/s using (i) Darcy formula (ii) chezy's formula. For which $c = 60$. Take V for water = 0.01 stoke.

Sol" Given data :

Dia of pipe ; $d = 300mm = 0.3m$

Length of pipe ; $L = 50m$

velocity of flow = $V = 3m/s$

chezy's constant , $c = 60$

kinematic viscosity ; $V = 0.01 stoke = 0.01 cm^2/s$

$= 0.01 \times 10^{-4} m^2/s$

(ii) Darcy formula is given by —

$$h_f = \frac{4f \cdot L \cdot v^2}{d \times 2g}$$

Where 'f' = co-efficient of friction is a function of Reynold's number, R_e

But R_e is given by $R_e = \frac{V \times d}{\nu} = \frac{3.975 \times .30}{0.01 \times 10^{-4}}$

$$= 9 \times 10^5$$

Value of $f = \frac{0.079}{R_e^{1/4}} = \frac{0.079}{(9 \times 10^5)^{1/4}} = 0.00256$

∴ Head loss, $h_f = 4 \times 0.00256 \times 90 \times 23$

$$= \frac{}{0.3 \times 2.0 \times 9.81} = 2.7921 \, m$$

Ans

(iii) Chezy's formula:

$$V = C \sqrt{mi}$$

where $C = 60$, $m = \frac{d}{4} = \frac{0.30}{4} = 0.075 \, m$

∴ $3 = 60 \sqrt{0.075 \times i}$ or $9 = \left(\frac{3}{60}\right)^2 \times \frac{1}{0.075}$

But $\frac{h_f}{90} = 0.0333$

Equating the two values of i, we have $\frac{h_f}{90} = 0.0333$

∴ $h_f = 90 \times 0.0333 = 1.4995 \, m$. Ans

Q.2 An oil of sp. gr. 0.7 is flowing through a pipe of diameter 300mm at the rate of 500 lit/s. Find the head loss due to friction and power required to maintain the flow for a length of 1000 m. Take $\nu = .29$ stokes.

Soln. Given data,

Sp. gr. = 0.7, $S = 0.7$

Dia of pipe, $d = 300mm = 0.30$

Discharge, $Q = 500 \, lit/s = 0.5 \, m^3/s$

Length of pipe, $L = 1000m$

Velocity, $V = \frac{Q}{Area} = \frac{0.5}{\frac{\pi}{4} d^2} = \frac{0.5 \times 4}{\pi \times 0.3^2}$

$$= 7.075 \, m/s$$

∴ Reynolds number, $R_e = \frac{V \times d}{\nu} = \frac{7.075 \times .3}{0.29 \times 10^{-4}}$

$\therefore$ co-efficient of friction, $f = \dfrac{.079}{R_e^{1/4}}$

$\qquad = \dfrac{0.79}{(7.56 \times 10^4)^{1/4}} = 0.0048$

$\therefore$ Head loss due to friction, $h_f = \dfrac{4 \cdot f \times L \times v^2}{d \times 2g}$

$\qquad = \dfrac{4 \times 0.0048 \times 1000 \times 7.073^2}{0.12 \times 2 \times 9.81} = 163.1\,m$

Power required $= \dfrac{\rho g \cdot Q \cdot h_f}{1000} \times KW$

where $\rho = $ density of oil $= 0.7 \times 1000 = 700\,kg/m^3$

power required $= \dfrac{700 \times 9.81 \times 0.5 \times 163.1}{1000}$

$\qquad = 560.27\,KW.$ Ans

Minor Energy (Head) Losses :—

The loss of head or energy due to friction in a pipe is known as major loss while the loss of energy due to change of velocity of the flowing fluid in magnitude or direction is called minor loss of energy. The minor loss of energy includes the following cases :

1. Loss of head due to sudden enlargement,
2. Loss of head due to sudden contraction,
3. Loss of head at the entrance of a pipe,
4. Loss of head at the exit of a pipe,
5. Loss of head due to an obstruction in a pipe,
6. Loss of head due to bend in the pipe,
7. Loss of head in various pipe fittings.

Q-1. An crude oil of kinematic viscosity 0.4 stoke is flowing through a pipe of diameter 300 mm at the rate of 300 litre per sec. Find the head lost due to friction for a length of 50 m of the pipe.

Soln: Given data,

Kinematic Viscosity $= v = 0.4$ stoke
$\qquad = 0.4\,cm^2/s = 0.4 \times 10^{-4}\,m^2/s$

Dia of pipe $= d = 300\,mm = 0.30\,m$

Discharge $= Q = 300$ lit/s $= 0.3\,m^3/s$

Length of pipe, $l = 50m$

velocity of flow, $V = \dfrac{Q}{Area} = \dfrac{0.3}{\frac{\pi}{4}(0.3)^2} = 4.24\ m/s$

∴ Reynolds number, $Re = \dfrac{V \times d}{\nu} = \dfrac{4.24 \times 0.30}{0.4 \times 10^{-4}}$

$$= 3.18 \times 10^4$$

As Re lies between 4000 and 100000, the value

of $f = \dfrac{0.079}{(Re)^{1/4}} = \dfrac{0.079}{(3.18 \times 10^4)^{1/4}} = 0.00591$

$h_f = \dfrac{4f \cdot L \cdot V^2}{d \times 2g} = \dfrac{4 \times 0.00591 \times 50 \times 4.24^2}{0.3 \times 2 \times 9.81}$

$$= 3.61\ m$$

Hydraulics and total energy line :—

The concept of hydraulic gradient line and total energy line is very useful in the study of flow of fluids through pipes. They are defined as ;

Hydraulic Gradient Line :—

It is defined as the line which gives the sum of pressure head $\left(\dfrac{p}{w}\right)$ and datum head (z) of the

→ showing the pressure head (p/w) of a flowing fluid in a pipe from the centre of the pipe.

Total Energy line :—

It is defined as the line which gives the sum of pressure head, datum head and kinetic head of a flowing fluid in a pipe with respect to some reference line.

Module - 9 :—   chapter - 5 :—

Flow through open channel :—

Flow in open channels is defined as the
Flow of a liquid with a free surface i.e. a
surface is a surface having constant pressure
such as atmospheric pressure. Thus a liquid
Flowing at atmospheric pressure through a
passage is known as flow in open channels.

Discharge through open channel by chezy's
formula :—

$$Q = A \cdot V$$

$$V = C\sqrt{mi}$$

C = chezy's constant

m = hydraulic mean radius

i = Head loss per meter length of flow

Q.1. find the velocity of flow and rate of
Flow of water through a rectangular channel of
6m wide and 3m deep, when it is running full.
The channel is having bed slope as 1 in 2000.
Take chezy's constant C = 55.

Soln   Given data

width of rectangular channel, b = 6m

Depth of channel, d = 3m

∴ Area , A = 6×3 = 18 m²

bed slope, $i = 1$ in 2000 $= \dfrac{1}{2000}$

chezy's constant, $c = 55$

perimeter, $P = b + 2d = 9 + 2 \times 3 = 15 m$

∴ Hydraulic mean depth, $m = \dfrac{A}{P} = \dfrac{12}{15} = 1.15 m$

velocity of flow is

$$V = c\sqrt{mi} = 55\sqrt{1.15 \times \dfrac{1}{2000}} = 1.506 m/s$$

Rate of flow, $Q = V \times Area = V \times A$

$$= 1.506 \times 18 = 27.12 m^3/s$$

Q.2 Find the slope of the bed of a rectangular channel of width 5m when depth of water is 2m and rate of flow is given as 20 m³/s. Take chezy's constant, $c = 50$.

Soln Given data,

width of channel, $b = 5m$

Depth of water, $d = 2m$

Rate of flow, $Q = 20 m^3/s$

chezy's constant, $c = 50$

Let the bed slope $= i$

Using equation, we have $Q = A c \sqrt{mi}$

where $A = Area = b \times d = 5 \times 2 = 10 m^2$

$m = \dfrac{A}{P} = \dfrac{10}{b + 2d} = \dfrac{10}{5 + 2 \times 2} = \dfrac{10}{5+4} = \dfrac{10}{9} m$

$20 = 10 \times 50 \times \sqrt{\dfrac{10}{9} \, i}$ or $\sqrt{\dfrac{10}{9} i} = \dfrac{20}{500} = \dfrac{2}{50}$

Squaring bothside, we have $\dfrac{10}{9} i = \dfrac{4}{2500}$

$i = \dfrac{4}{2500} \times \dfrac{9}{10} = \dfrac{36}{25000} = \dfrac{1}{694.44}$

∴ Bed slope is 1 in 694.44.

Q.3 A flow of water of 100 lits per second flows down in a rectangular flume of width 600mm and having adjustable bottom slope. If chezy's constant c is 56 find the bottom slope necessary for uniform flow with a depth of flow of 300mm. Also find the conveyance k of the flume.

Soln Given data,

Discharge, $Q = 100 lits/s = \dfrac{100}{1000} = 0.10 \, m^3/s$

$b = 600\,mm = 0.60\,m$

$d = 300\,mm = 0.30$

Area of Flow, $A = b \times d = 0.6 \times 0.3 = 0.18\,m^2$

Chezy's constant, $c = 55$

Let the slope of bed $= i$

Hydraulic mean depth, $m = \dfrac{A}{P} = \dfrac{b \cdot d}{b + 2d}$

$$= \frac{0.18}{0.6 + 2 \times 0.3} = \frac{0.18}{1.2} = 0.15\,m$$

We have, $Q = A\,c\sqrt{mi}$

$0.10 = 0.18 \times 55 \times \sqrt{0.15 \cdot i}$

Squaring both sides, we have $0.15\,i = \left(\dfrac{0.10}{0.1795\ldots}\right)^2$

$$= 0.00009\ldots i$$

$i = \dfrac{0.00009\ldots}{0.15} = 0.0000612 = \dfrac{1}{\dfrac{1}{0.0000612}}$

$= \dfrac{1}{1629}$ **Ans.**

Q-4 Find the discharge through a trapezoidal channel of width 8m and side slope of 1 horizontal to 2 vertical. The depth of flow of water is 2.4m and value of Chezy's constant c = 50. The slope of the bed of the channel is given 1 in 4000.

Sol. Given data,

$b = 8m$

Side slope = 1 horizontal to 2 vertical

$d = 2.4m$

$c = 50$

Bed slope, $i = \dfrac{1}{4000}$

depth, $d = 2.4$
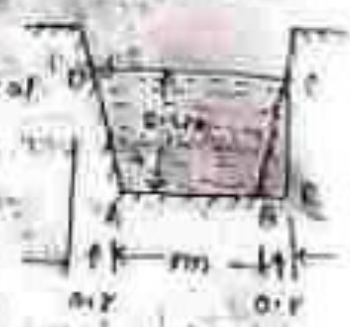
Horizontal distance, $BE = \dfrac{2.4}{2} = 0.4\,m$

Top width of the channel,

$CD = AB + 2 \times BE$

$= 8.0 + 2 \times 0.4 \times 2 = 9.6m$

Area of trapezoidal channel, $ABCD = A$

$A = (AB + CD) \times \dfrac{d}{2} = (8 + 9.6) \times \dfrac{2.4}{2}$

$= 17.6 \times 1.2 = 21.12\,m^2$

wetted perimeter, $P = AB + BC + AD = AB + 2BC$

$$BC = \sqrt{BE^2 + CE^2} = \sqrt{(0.5)^2 + (1.4)^2} = 0.529 m \qquad (\because BC = AD)$$

$$P = 7.0 + 2 \times 2.529 = 13.058 m$$

Hydraulic mean depth, $m = \dfrac{A}{P} = \dfrac{12.49}{13.058} = 1.47 m$

the discharge Q is given

$$Q = AC\sqrt{mi}$$

$$= 21.18 \times 5.0 \sqrt{1.417 \times \dfrac{1}{4000}} = 21.23 \, m^3/s \quad \text{Ans}$$

Discharge of open channel using manning's Formula :—

$$Q = A \cdot V$$

$V = $ velocity

$A = $ Area of flow

$V = C\sqrt{mi}$

$C = \dfrac{1}{N} m^{1/6}$

where $N = $ Manning's constant

$m = $ hydraulic radius

$m = \dfrac{A}{P}$ $\quad A = $ Area of flow

$P = $ wetted perimeter

Q. F Find the discharge through a rectangular channel 3.5 m wide, having depth of water 1.5m and bed slope as 1 in 2000. Take the value of K = 2.36 in Bazin's formula.

Ans : Given data

$b = 3.5 m$

$d = 1.5 m$

Area, $A = b \times d = 3.5 \times 1.5 = 5.25 \, m^2$

wetted perimeter, $P = d + b + d$

$= 1.5 + 3.5 + 1.5 = 6.5 m$

Hydraulic mean depth, $m = \dfrac{A}{P} = \dfrac{5.25}{6.50} = 0.808$

Bed slope, $i = \dfrac{1}{2000}$

Bazin's constant = K = 2.36

Using Bazin's formula $Q = \dfrac{157.6}{1.81 + \dfrac{k}{\sqrt{m}}} = \dfrac{157.6}{1.81 + \dfrac{2.36}{\sqrt{0.808}}}$

= 20.76

Discharge, $Q = Ac\sqrt{mi}$

$= 3.76 \times 20.76 \times \sqrt{0.576 \times \dfrac{1}{2000}} = 2.337$

Q. Find the discharge through a rectangular channel 4m wide, having depth of water 3m and bed slope 1 in 1500. Take the value of N = 0.03 in the Kutter's formula.

Soln. Given data

$b = 4m$

$d = 3m$

Bed slope, $i = \dfrac{1}{1500} = 0.000667$

Kutter's constant, $N = 0.03$

Area of flow = b × d = 4×3 = 12 m²

Wetted perimeter, $P = d + b + d = 3+4+3 = 10m$

Hydraulic mean depth, $m = \dfrac{A}{P} = \dfrac{12}{10} = 1.2 m$

Using Kutter's formula;

$c = \dfrac{23 + \dfrac{0.00155}{i} + \dfrac{1}{N}}{1 + \left(23 + \dfrac{0.00155}{i}\right) \times \dfrac{N}{\sqrt{m}}} = \dfrac{23 + \dfrac{0.00155}{0.000667} + \dfrac{1}{N}}{1 + 23 + \dfrac{0.00155}{0.000667} \cdot \dfrac{N}{\sqrt{m}}}$

$= \dfrac{23 + 2.3238 + 33.33}{1 + (23 + 2.3238) \times 0.0275} = \dfrac{58.653}{1.698} = 32.41$

Discharge, $Q = Ac\sqrt{mi} = \sqrt{12 \times 0.000667} = 10.9550$

Q. Find the discharge through a rectangular channel of width 5m, having a bed slope of 4 in 8000. The depth of flow is 1.5m and take the value of N in manning's formula at 0.012.

Soln. Given data,

$b = 5m$

$d = 1.5m$

Area of flow, $A = b \times d = 5 \times 1.5 = 7.5 m²$

Wetted perimeter, $P = b + d + d = 5 + 1.5 + 1.5 = 8$

Hydraulic mean depth, $m = \dfrac{A}{P} = \dfrac{7.5}{8} = 0.9$

Bed slope, $i = 4$ in 8000 $= \dfrac{4}{8000} = \dfrac{1}{2000}$

Value of N = 0.012

Using Manning's formula;

$$\text{Discharge}, Q = AC\sqrt{mi}$$

... = 3.477 m³/s

Most economical section of channel :—

A section is said to be most economical when the cost of construction of the channel is minimum. Most economical section is also called the best section or most efficient section.

The conditions to be most economical for the following shapes of the channels are

1. Rectangular section
2. Trapezoidal section
3. Circular section

Rectangular section :—

The condition for most economical section is

$$\boxed{b = 2d}$$ which implies

$$m = \frac{A}{P} = \frac{b \cdot d}{b + 2d} = \frac{2d \times d}{2d + 2d} = \frac{d}{2}$$

$$\boxed{m = \frac{d}{2}}$$

Q. A rectangular channel of width 4m is having a bed slope of 1 in 1500. Find the maximum discharge through the channel. Take value of C = 50.

Given data,

$$b = 4m$$

Bed slope, $i = \dfrac{1}{1500}$

Chezy's constant, C = 50

Discharge will be maximum, when the channel is most economical. The conditions for most economical Rectangular channel are:

(i) $b = 2d$ or $d = \dfrac{b}{2} = \dfrac{4}{2} = 2m$

(ii) $m = \dfrac{d}{2} = \dfrac{2}{2} = 1m$

∴ Area of most economical rectangular channel,

$$A = b \times d = 4 \times 2 = 8 m²$$

$$\text{Discharge}, Q = AC\sqrt{mi} = 8 \times 50 \times \sqrt{1 \times \frac{1}{1500}} = 10.32 \text{ m}^3/s$$

Q.9 A rectangular channel carries water at the ... of 400 lit/s when bed slope is 1 in 2000. Find most economical dimensions of the channel if ...

Sol. Given data:

Discharge, $Q = 400$ lit/s $= 0.4 \ m^3/s$

Bed slope is $i = \dfrac{1}{2000}$

Chezy's constant, $c = 50$

For the rectangular channel to be most economical

(i) width, $b = 2d$

(ii) Hydraulic mean depth, $m = \dfrac{d}{2}$

∴ Area of flow, $A = b \times d = 2d \times d = 2d^2$

Discharge, $Q = Ac\sqrt{mi}$

$0.4 = 2d^2 \times 50 \times \sqrt{\dfrac{d}{2} \times \dfrac{1}{2000}}$

$= 2 \times 50 \times \sqrt{\dfrac{1}{2 \times 2000}} \ d^{5/2}$

$d^{5/2} = \dfrac{0.4}{1.581} = 0.253$

$d = (0.253)^{2/5} = 0.577 \ m$

$b = 2d = 2 \times 0.577 = 1.154 \ m$ **Ans**

Trapezoidal section :—

The condition for most economical trapezoidal section is

$$\boxed{\dfrac{b + 2nd}{2} = d\sqrt{n^2 + 1} \ - \ d}$$

where $b = $ width of channel
        $n = $ side slope
        $d = $ depth of water

$m = \dfrac{A}{P} = $ ...

$A = \left(\dfrac{\text{bottom width} + \text{top width}}{2}\right) \text{depth}$

$= \left(\dfrac{b + b + 2nd}{2}\right)d = (b + nd)d$

$P = b + 2d\sqrt{n^2 + 1}$

$m = \dfrac{(b + nd)d}{b + 2d\sqrt{n^2 + 1}}$

Using eqn (1), $2d\sqrt{n^2 + 1} = b + 2nd$

$$m = \frac{(b+nd)d}{b+2\sqrt{b+2nd}} = \frac{(b+nd)d}{2(b+nd)} = \frac{d}{2}$$

$$\boxed{m = \frac{d}{2}}$$

A trapezoidal channel has side slopes of 1 horizontal to 2 vertical, and the slope of the bed is 1 in 1500. The area of the section is 40 m². Find the dimensions of the section if it is most economical. Determine the discharge of the most economical section if C = 80.

Solⁿ Given data,

Side slope, $n = \frac{Horizontal}{Vertical} = \frac{1}{2}$

Bed slope, $i = \frac{1}{1500}$

Area of section, $A = 40\,m^2$

Chezy's constant, $C = 80$

For the most economical section,

$$\frac{b+2nd}{2} = d\sqrt{n^2+1} \quad or \quad \frac{b+2\times\frac{1}{2}d}{2} = d\sqrt{(1/2)^2+1}$$

or $\frac{b+d}{2} = d\sqrt{\frac{1}{4}+1} = 1.118d$

or $b = 2\times1.118d - d = 1.236d$

But area of horizontal section,

$A = \frac{b+(b+2nd)}{2}\times d = (b+nd)d$

$A = (1.236d + \frac{1}{2}d)d$ ($\because b = 1.236d$ and $n = \frac{1}{2}$)

$= 1.736\,d^2$

But $A = 40\,m^2$

$40 = 1.736\,d^2$

$d = \sqrt{\frac{40}{1.736}} = 4.80\,m$ **Ans**

Substituting the value of d,

$b = 1.236 \times 4.80 = 5.932\,m$

Discharge for most economical section, hydraulic mean depth for most economical section is

$m = \frac{d}{2} = \frac{4.80}{2} = 2.40\,m$

Centrifugal Pumps :—

Introduction :—

→ The hydraulic machine which convert the mechanical energy into hydraulic energy are called pumps.

→ The hydraulic energy is in the form of pressure energy.

→ If the mechanical energy is converted into pressure energy by means of centrifugal force acting on the fluid, the hydraulic machine is called centrifugal pump.

Main parts of a centrifugal pump :—

The following are the main parts of a centrifugal pump :

1. Impeller
2. Casing
3. Suction pipe with a foot valve and a strainer.
4. Delivery pipe

1. Impeller :—

→ The rotating part of a centrifugal pump is called 'impeller'.

→ It consists of a series of backward curved vanes.

→ The impeller is mounted on a shaft which is connected to the shaft of an electric motor.

2. Casing :—

→ The casing of a centrifugal pump is similar to the casing of a reaction turbine.

→ It is an air tight passage surrounding the impeller and is designed in such a way that the kinetic energy of the water discharged at the outlet of the impeller is converted into pressure energy before the water leaves the casing and enters the delivery pipe. The following three types of the casings are commonly adopted.

a) Volute Casing :-

Figure shows the volute casing, which surrounds the impeller. It is of spiral type in which area of flow increases gradually. The increase in area of flow decreases the velocity of flow. The decrease in velocity increases the pressure of the water resulting through the casing. It has been observed that in the case of volute casing, the efficiency of the pump increases slightly as a large amount of energy is lost due to the formation of eddies in this type of casing.
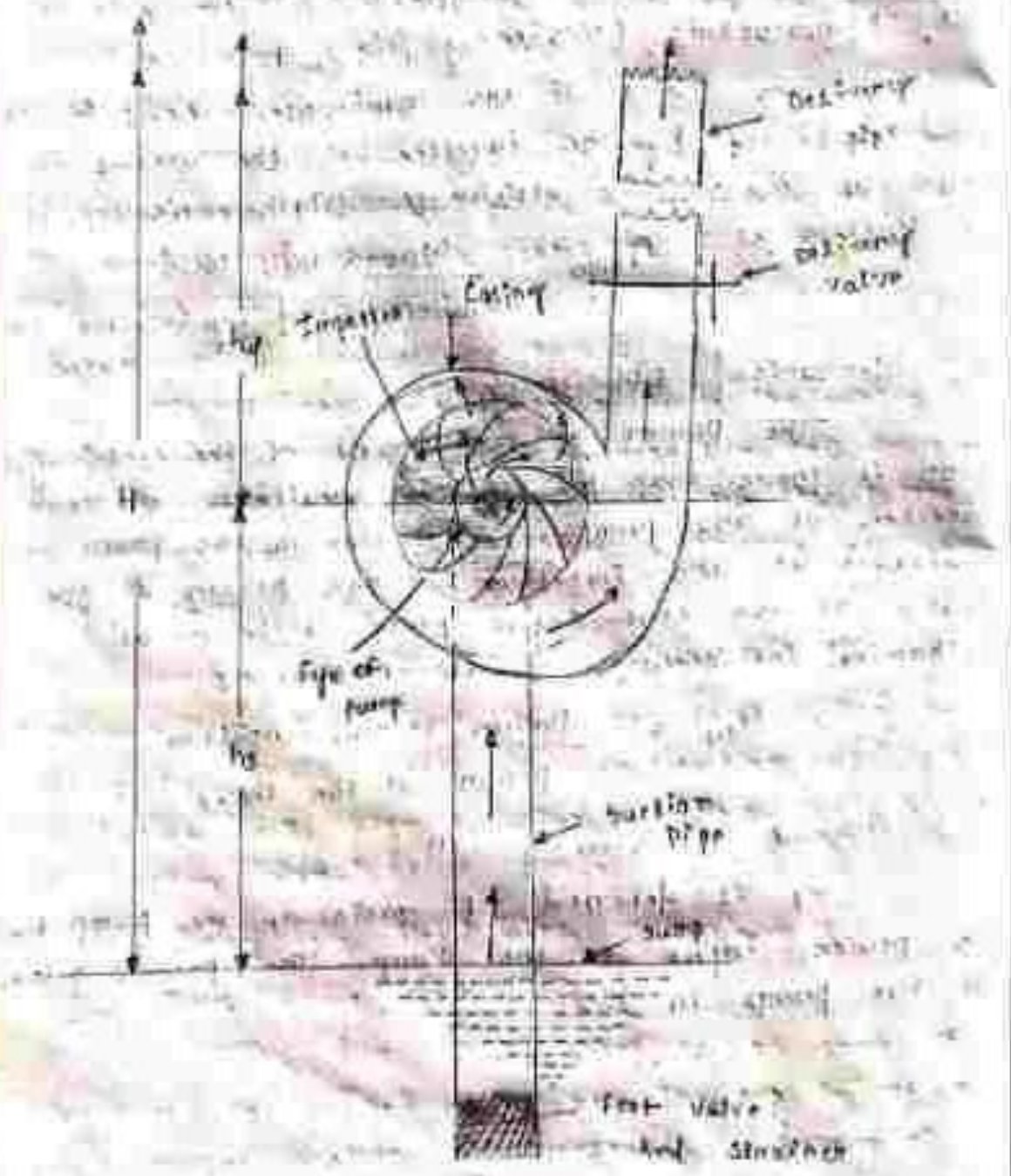
Main parts of a centrifugal pump

3. Suction pipe with a foot valve and a strainer :—

→ A pipe whose one end is connected to the inlet of the pump and other end dips into water of a sump is known as Suction pipe.

→ A foot valve which is a non-return valve are one-way type of valve is fitted at the lowest end of the suction pipe.

→ The foot valve opens only in the upward direction.

→ A strainer is also fitted at the lower end of the suction pipe.

4. Delivery pipe :—

A pipe whose one end is connected to the outlet of the pump and other end delivers the water at a required height is known as delivery pipe.

Definitions of heads and efficiencies of a centrifugal pump :—

1. Suction Head (h_s) :—

It is the vertical height of the centre line of the centrifugal pump above the water surface in the tank, the pump from which water is to be lifted as shown of figure. This height is also called suction lift and is denoted by h_s.

2. Delivery Head (h_d) :—

The vertical distance between the centre line of the pump and the outside surface in the tank to which water is delivered is known as delivery head.

→ this is denoted by h_d.

3. Static Head (H_s) :—

The sum of suction head and delivery head is known as static head.

→ This is represented by H_s and is written as $H_s = h_s + h_d$.

6. **Efficiencies of a centrifugal pump :—**

In case of a centrifugal pump, the power is transmitted from the shaft of the electric motor to the shaft of the pump and from to the impeller. From the impeller, the power is given to the water. Thus power is decreasing from the shaft of the pump to the impeller and then to the water. The following are the important efficiencies of a centrifugal pump : (a) Manometric efficiency (η_{man})

(b) Mechanical efficiency (η_m)

(c) Overall efficiency (η_o).

(a) **Manometric Efficiency (η_{man}) :—**

The ratio of the manometric head to the head imparted by the impeller to the water is known as manometric efficiency. Mathematically, is is written as

$$\eta_{man} = \frac{\text{Manometric head}}{\text{Head imparted by impeller to water}}$$

(b) **Mechanical Efficiency (η_m) :—**

The power at the shaft of the centrifugal pump is more than the power available at the impeller of the pump. The ratio of the power available at the impeller to the power at the shaft of the centrifugal pump is known as mechanical efficiency. It is written as

$$\eta_m = \frac{\text{Power at the impeller}}{\text{Power at the shaft}}$$

(c) **Overall Efficiency (η_o) :—**

It is defined as ratio of the pump to the power input to the pump. The power output of the pump in KW.

Irrigation :-

The process of artificial application of water to the soil for the growth of agricultural crops is termed as Irrigation. It is practically a science of planning and designing a water supply system for the agricultural land to protect the crops from bad effects of low rainfall. It includes the construction weir, dam, barrage, canal system for regular supply of water to culturable cultivated land.

Necessity of Irrigation :-

Followings are the factors which govern the necessity of Irrigation :

(1) Insufficient rainfall

(2) Uneven distribution of rainfall

(3) Improvement of perennial crop throughout the year.

(4) Development of agriculture in desert area.

Benefits of Irrigation :-

Followings are the important benefits of irrigation

(1) Yield of crops

(2) Improvement of cash crops like vegetable, fruits ett.

(3) Nevegation i.e. communication and transportation of agricultural goods.

(4) Hydroelectric power generation

(5) Water supply

(6) Prosperity of farmers i.e farmers may or money and improve their living standard by raising two or more crops on the same land.

(7) Source of revenue i.e cultivator may give some taxes by taking water from the canal.

(8) General communication i.e the inspection road along the canal bank may surve as communication road.

Gross command Area :—

The whole area enclosed between an imaginary boundary land which can be included in an irrigation project for supplying water to the agricultural land by network of canal is known as gross command area (G.C.A)

G.C.A = cultivable area + uncultivable area

Uncultivable area :—

The area where agriculture can be done and crops can be grown satisfactorily is called as cultivable area.

(C.C.A) Cultivable Command area :—

The total area within an irrigation project where the cultivation can be done and crops can be grown is known as cultivable command area. Again C.C.A may be of two categories:

(1) cultivable cultivated area
(2) Cultivable uncultivated area

Cultivable Uncultivated area :—

It is the area within the C.C.A where cultivation is possible but it is not being cultivated at present due to non-availability fund etc.

Cultivable cultivated area :—

It is the area within the C.C.A where cultivation is possible but it is not being cultivated at present due to non-availability fuel etc.

Intensity of Irrigation :—

Intensity of irrigation may be defined as a ratio of cultivated land for a particular crop to the total cultivable command area. It is expressed as a percentage of C.C.A

Qt: If total culturable command area is 100 Hectare, where wheat is cultivated in 30 Hectare, Intensity of irrigation for wheat

$$= \frac{30}{100} \times 100 = 30\%$$

Q. The gross command area of an irrigation is 1 lakh hectare. The culturable command area is 75% of gross command area. The intensity of irrigation for rice and wheat are 50% and 55% respectively. Find the area of each crop and the particular crop has been grown.

Ans. Given data:-

$$G.C.A = 1 \text{ lakh hectare}$$
$$= 100000 \text{ hectare}$$

$$C.C.A = 100000 \times \frac{75}{100}$$

$$= 75000 \text{ hectare}$$

Intensity of irrigation

Rice

$$\frac{\text{Area of rice}}{C.C.A} \times 100 = 50$$

$$\frac{\text{Area of rice}}{75000} \times 100 = 50$$

$$\Rightarrow \text{Area of rice} = 50 \times 750 = 37500 \text{ hectare}$$

Wheat

$$\frac{\text{Area of wheat}}{75000} \times 100 = 55$$

$$\Rightarrow \text{Area of wheat} = 55 \times 750 = 41250 \text{ hectare}$$

Crop Season :-

The period during which same particular type of crop grown every year on the same land is known as crop season. Following are the major crop seasons (a) Kharif
(b) Rabi

Khariff Season :—

This Season ranges from June to July. The crops are shown in the very beginning of monsoon and harvested at the end of the autumn.

The major khariff crops are rice, millet, maize, Jute, groundnut.

Rabi Season :—

This Season ranges from October to March. The crops are shown in the very beginning of winter and harvested at the end of the Spring Season.

The major rabi crops are wheat, gram, mustard, Pulses, onion.

Again their are several crops which are not included in either khariff or in rabi Season, but they require more time and they cover both the main Season (Khariff & rabi).

Ex: Cotton is a 8 months crop; Sugarcane is a 12 months crop (perennial)

Crop Ratio :—

It is defined as the ratio of the areas of two main Season.

Ex: If the area under khariff crop is 5000 hectare and area under Rabi crop is 10000 hectare then Crop ratio is 1:2

Cash Crop :—

The crops which are cultivated by the farmers to sell in the open market to meet their financial requirements are known as Cash crops.

are vegetables, fruits etc.

Crop Period :—

The crop period is defined as the total period from the time of sowing a crop to the time of harvesting. In other words it is the period in which crop remains in the field.

Number of watering :—

The total depth of water required by a crop is not supplied at a time, but it is supplied over the base period by stages depending upon the requirement.

Ex: Rice requires 190 cm of water. If watering depth applied at each stage is 10 cm, then number of watering $\frac{190}{10} = 19$ times.

Paleo Irrigation :—

The initial watering which is done to provide moisture to the soil just before sowing any particular crop is known as paleo irrigation.

Kor Watering :—

The first watering is done when the crop has grown to about 3 cm. This watering is known as kor watering.

The number of watering depends on type of soil (moisture retaining property), soil condition, climate condition.

Base Period :—

(i) Base period is defined as the period from first watering to last watering of a crop.
It is also known as base.
(ii) It is denoted by 'B' and expressed number of days.

Ex:

Crop	Base
(i) Rice	120 days
(ii) Wheat	120 days
(iii) cotton	200 days
(iv) Sugarcane	320 days

Delta (Δ) :—

Each crop requires certain amount of water for its growth and development. If the total amount of water supplied to the crop (1st watering to last watering) is stored on the land without any loss there will be a thick layer of water standing on that land. This depth of water layer is known as delta for that crop. It is denoted as 'Δ', and expressed in cm.

Ex :

Crop	Delta
1. Rice	125 cm
2. Wheat	40 cm
3. Ground nut	30 cm

Duty (D) :—

(i) The duty of water is defined as number of hectares that can be irrigated by constant supply of water at the rate 1 cumec ($1 m^3/sec$) throughout the base period.

(ii) It is expressed in hectare/cumec and is denoted by 'D'.

Ex :

Crop	Duty
1. Rice	$B = 120, \Delta = 125 cm = 1.25 m$
	$D = \dfrac{8.64 \times 120}{1.25} = 829.44$ hec/cumec
	(≈ 900 hec/cum)
2. Wheat	$B = 120, \Delta = 40 cm = 0.4 m$
	$D = \dfrac{8.64 \times 120}{0.4} = 2592$
	(≈ 2600 hec/cum)

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Relation between base (B), Delta (Δ) & Duty (D) :—

Let D = Duty in hectare/cumec
Δ = Delta in metre
B = Base period in days

From definition of duty 1 cumec of water flowing continuous for 'B' days gives depth of water 'Δ' over 'D' hectare

$$1 m^3/sec \times B \times 24 \times 3600 \, sec$$
$$= 86400 \, B \, m^3 \quad (water\ applied)$$

$$D \times 10000 \times Δ \quad (water\ stored)$$

Equating water applied = water stored, we have

$$86400 \, B \, m^3 = D \times 10000 \times Δ$$

$$\Rightarrow \frac{8.64 \times B}{Δ} = D$$

Discharge required of a canal :—

Discharge can be calculated for a crop by dividing area of that particular crop with its duty.

$$Q = \frac{A}{D}$$

Where, A = Area of crop in hectare
D = Duty of crop in hec/cum
Q = Discharge

Q. A canal channel is to be design for irrigating 5000 hectare in Kharif crop and 4000 hecs in rabi crop. The water requirement for Kharif and rabi are 60 cm and 25 cm respectively. The crop period / Kor period for Kharif is 3 weeks and rabi is 4 week. Determine the discharge of the channel for which it is to be design.

Given data -

Khariff cropis

Δ_{rice} = 60 cm = 0.6m

B_{rice} = 277 = 21 days

A_{rice} = 5000 hectre

$Duty (D) = \dfrac{8.64 \times B}{\Delta}$

$= \dfrac{8.64 \times 21}{0.6}$

= 302.4 hec/cumec

$Q_R = \dfrac{A}{D} = \dfrac{5000}{302.4}$ = 16.53 cumec

Rabi crop

Δ_{wheat} = 25 cm = 0.25m

B_{wheat} = 447 = 27 days

A_{wheat} = 4000 hectre

$Duty (D) = \dfrac{8.64 \times B}{\Delta}$

$= \dfrac{8.64 \times 27}{0.25}$

= 967.68 hec/cumec

$Q_R = \dfrac{A}{D} = \dfrac{4000}{967.68}$ = 4.13 cumec

So the channel is to be design for the maximum discharge of 16.53 cumec.

Q. find the delta for a crop when the duty is 864 hec/cumec the base period of this crop is 120 days?

Ans Given data

$\Delta = ?$

$D = 764$ hec/cumec

$B = 120$ days

$$D = \frac{8.64 \times B}{\Delta}$$

$$\Rightarrow 764 = \frac{8.64 \times 120}{\Delta}$$

$$\Rightarrow \Delta = \frac{8.64 \times 120}{764} \approx 1.2\,m$$

Q. Find the delta for sugarcane when duty is 760 hec/cumec on the field and the base period of the crop being 140 days?

Ans Given data

$\quad\quad D = 760$ hec/cumec

$\quad\quad B = 140$ days

$\quad\quad \Delta = ?$

$$D = \frac{8.64 \times B}{\Delta}$$

$$\Rightarrow 760 = \frac{8.64 \times 140}{\Delta}$$

$$\Rightarrow \Delta = \frac{8.64 \times 140}{760} \approx 1.55\,m$$

Q. The gross command area of an irrigation project is 1.2 lakh hectare, the culturable command area is 75% of G.C.A. the intensities of irrigation for Kharif and rabi respectively. If the duties are 50% and 55% respectively. If the duties are 1200 hec/cumec and 1400 hec/cum for Kharif and rabi are 1200 hec/cumec and 1400 hec/cum respectively. Determine the discharge at the head of the canal considering 20% provisions for transmission loss & overlap allowance / evaporation loss etc.

Ans Given data

$\quad\quad G.C.A = 1.2$ lakh hectare

$\quad\quad = 1,20,000$ hectare

$$C.C.A = 100000 \times \frac{75}{100}$$

$$= 75000 \text{ hectare}$$

Intensities of irrigation

Kharif

$$\frac{\text{Area for Kharif}}{C.C.A} \times 100 = 50$$

$$\Rightarrow A_K = \frac{C.C.A \times 50}{100} \quad ①$$

$$= 75000 \times \frac{50}{100}$$

$$= 37500 \text{ hectare}$$

Rabi crop

$$\frac{A_{rabi}}{C.C.A} \times 100 = 55$$

$$\Rightarrow A_{Rabi} = \frac{55}{100} \times C.C.A$$

$$= \frac{55}{100} \times 75000$$

$$= 41250 \text{ hectare}$$

Duties for Kharif = 1500 hec/cum

$$Q_K = \frac{A_K}{D_K} = \frac{37500}{1500} = 31.25$$

Duties for rabi = 1400 hec/cum

$$Q_R = \frac{A_R}{D_R} = \frac{41250}{1400} = 29.46 \text{ cumec}$$

considering losses 20% extra amount of water is need

$$31.25 \times \frac{20}{100} = 6.25 \text{ cumec}$$

$$\therefore Q \text{ required} = 31.25 + 6.25 = 37.5 \text{ cumec}$$

Determine the discharge of a canal from the following data:

Crop	Bas period In days	Area in Hectre	Duty in hectare
Rice	120 days	4000	1500
wheat	150 days	3500	2000
Sugarcane	310 days	3000	1200

Ans. Given data

For rice

$B = 120$ days

$A = 4000$ hectare

$D = 1500$ hec/cumec

$$Q_R = \frac{A}{D} = \frac{4000}{1500} = 2.66 \text{ cumec}$$

For wheat

$B = 150$ days

$A = 3500$ hectre

$D = 2000$ hec/cum

$$Q_w = \frac{A}{D} = \frac{3500}{2000} = 1.75 \text{ cumec}$$

For Sugarcane

$B = 310$ days

$A = 3000$ hec

$D = 1200$ hec/cumec

$$Q_s = \frac{A}{D} = \frac{3000}{1200} = 2.5 \text{ cumec}$$

$$Q_{kharif} = Q_R + Q_s$$

$$= 2.66 + 2.5 = 5.16 \text{ cumec}$$

$$Q_{Rabi} = Q_w + Q_s$$

$$= 1.75 + 2.5 = 4.25 \text{ cumec}$$

Therefore required canal discharge =

maximum (5.16, 4.25)

$= 5.16$ cumec

Overlap allowance :—

Sometimes a crop of one season may overlap the next crop season by a few days more which it requires to mature. During this period of overlaping the irrigation water is to be supplied simultaneously to the crops both the season. Due to this extra demand discharge of the canal is to be increased. This provision of increasing demand is adjusted by adopting appropriate overlap allowance. This is expressed in percentage.

Water logging and Drainage :—

Introduction :—

(1) In agricultural land when the pores within the root zone are get saturated due to subsoil water or ground water the air circulation within the soil pores are get stop. This phenomena is called as water logging.

Due to water logging fertility of the land is reduced. Yield of crop is stop.

Causes of water logging :—

Following are the main causes of water logging.

(1) Over Irrigation :—

Inundation Irrigation System, since there is no control over water supply it may cause over irrigation.

(2) Seepage from canals :—

In unlined canal system water percolates through the bank of canals. Thus ground water table gets rise.

(3) Nature of soil :—

The soil having low permeability like black cotton soil (more plasticity) does not allow water to percolate water through it. Thus it lead to water logging.

(4) Excessive rainfall :—

If the rainfall is excessive water gets no time to drained of completely. There by a pool of stagnant water is formed which to water logging.

(5) Topography of land :—

If the agricultural land is flat with depression and undulations this leads also water logging.

(VI) Poor Irrigation Management :—

If the main canal is kept open for a long period unnecessarily without computing the crops water requirement, leads to over irrigation which results into water logging.

(VII) Seepage from reservoir :—

If the reservoir basin consist of permeable zones, cracks which were not detected during the construction of dam these may cause seepage of water.

(VIII) Flood :—

If an area gets affected by flood and their is no proper drainage system the water table gets raised these causes water logging.

(IX) Obstruction in natural water course :—

If the bridges or culverts are constructed across a water course with the opening with insufficient discharge capacity the upstream area gets flooded and this cause water logging.

Obstruction to Sub Soil Drainage :—

If some impermeable zone exist at a lower depth below the ground surface, then the movement of sub soil water gets obstructed and causes water logging in the area.

Effects of Water logging :—

Following are the effects of water logging

(1) Salinization of Soil :—

Due to waterlogging the dissolved salts like sodium carbonate, sodium chloride and sodium sulphate come to the surface of the soil. When the water evaporates from the surface, the salts are deposited there. This process is known as salinization of soil. Excessive concentration of salts makes the land alkaline. It does not allow the plants to thrive and thus the yield

of crop is reduced. This process is also known as salt efflorescence.

(ii) Lack of aeration :—

The crops require some nutrients for their growth which are supplied by some bacteria or micro-organisms by breaking the complex homogeneous compounds into simple nitrogen compounds which are consumed by the plants for their growth. But the bacteria require oxygen for their life and activity. When the aeration in the soil is stopped by water logging, these bacteria cannot survive without oxygen and the fertility of the land is lost which results in reduction of yield.

(3) Fall of soil temperature :—

Due to water logging, the agricultural land is considerably lowered. At low temperature of the soil the activity of the bacteria becomes very slow and consequently the plants do not get the requisite amount of food in time. Thus, growth of the plants is hampered and the yield also is reduced.

(4) Growth of weeds and aquatic plants :—

Due to water logging, the agricultural land is converted to marshy land and the weeds and aquatic plants are grown in plenty. These plants consume the soil foods in advance and thus the crops are destroyed.

(5) Diseases of crops :—

Due to low temperature and poor aeration, the crops get some diseases which may destroy the crops or reduce the yield.

(6) Difficulty in cultivation :—

In water logged area it is very difficult to carry out the operation of cultivation such as tilling, ploughing, etc.

Restriction of Root growth :—

When the water table rises near to the root Zone the soil gets saturated. The growth of the roots is confined only to the top layer of the soil. So, the crops cannot be matured properly and the yield is reduced.

Control of Water logging (i.e. anti Water logging measures) :—

The following measures may be taken to control water logging :—

(1) Prevention of percolation from canals :—

The irrigation canals should be lined with impervious lining to prevent the percolation of water through the bed and banks of the canals. Thus, the water logging may be prevented.

Intercepting drains may be provided along the course of the irrigation canals in places where the percolation of water is detected. The percolating water is intercepted by the drains and the water is carried to other natural water course.

(2) Prevention of percolation from reservoirs :—

During the construction of dam, the geological survey should be conducted on the reservoir basin to detect the zone of permeable formations through which water may percolate. These zones should be treated to prevent seepage. If afterwards it is found that there is still leakage of water through some zone, then Sheet Piling should be done to prevent the leakage.

(3) Control of intensity of irrigation :—

The intensity of irrigation may cause water logging so, it should be controlled in a planned way

that there is no possibility of water logging in a particular area.

(ii) **Economical use of water :—**

If the water is used economically, then it may control the water logging and the yield of crops may be high. Special training is required to be given to the cultivators to realise the benefits of economical use of water. It helps them to get more crops by eliminating the possibility of water logging.

(iii) **Fixing of crop pattern :—**

Soil survey should be conducted to fix the crop pattern. The crops having high rate of evapotranspiration should recommended for the area susceptible to water logging.

(iv) **Providing drainage system :—**

Suitable drainage system should be provided in the water logging areas so that the rain water does not stand for long days. A net work of sub-surface drains are provided which are connected to the surface drains. The surface drains discharge the water to the river or any water course.

(v) **Improvement of natural drainage :—**

Sometimes, the natural drainage may be completely licked up or obstructed by weeds, aquatic plants, etc. The affected section of the drainage should be improved by excavating and cleaning the obstructions.

(vi) **Pumping of ground water :—**

A number of open wells or tube wells are constructed in the water logged area and the ground water is pumped out until the water table goes down to a safe level. This itself ground water may be utilized for irrigation or may be discharged to the river or any water course.

(9) Construction of Sump wells :-

Sump wells may be constructed within the water logged area and they help to collect the surface water. The water from the sump wells may be pumped to the navigable lands or may be discharged to any river.

: Hydrology :—

The science of study the different form of water available above the earth surface or below the earth surface is known as hydrology.

Hydrologic cycle or water cycle :—

The water of the universe always change from one state to other under the effect of sun. The water from the surface sources like lakes, rivers, ocean, etc., converts to vapour by evaporation due to solar heat. The vapour goes on accumulating continuously in the atmosphere. This vapour is again condensed due to the sudden fall of temperature and pressure. Thus, clouds are formed. These clouds again causes the precipitation (i.e. rainfall). Some of the vapour is converted to ice at the peak of the mountains. The ice again melts in summer and flows as rivers to meet the sea or ocean. These processes of evaporation, precipitation and melting devices go on continuously like an endless chain and thus a balance is maintained in the atmosphere. This phenomenon is known as hydrologic cycle.
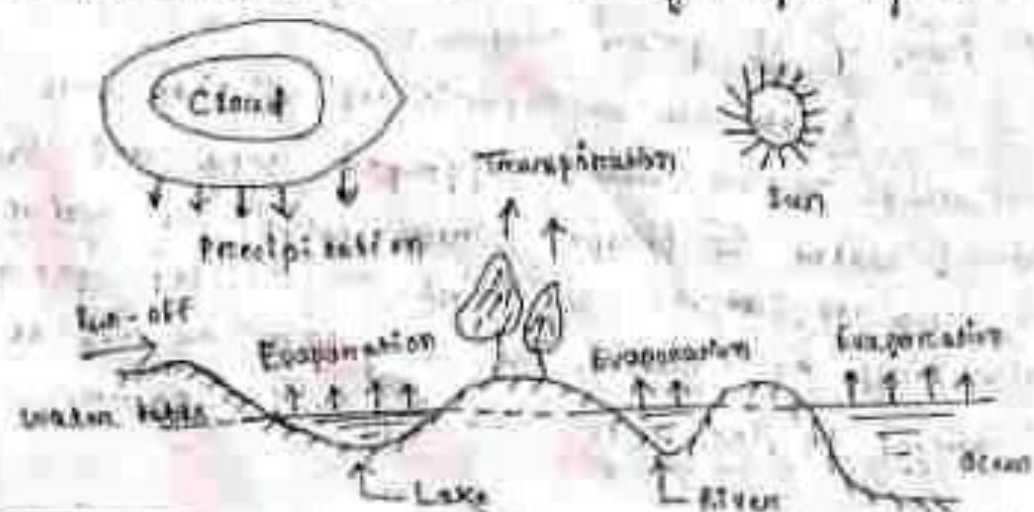
. The water vapour goes on collecting in the atmosphere up to a certain limit. When this limit is reached, and the atmospheric temperature and pressure fall to a certain value, the water vapour gets condensed and thereby cloud is formed. Eventually droplets are formed and deposited to earth in the form of rain, snowfall, hail, etc. This is known as precipitation.

Types of Precipitation or Rainfall :–

Depending upon the various atmospheric conditions, the precipitation may be of the following types :–

1. Convective precipitation :–

In tropical countries, when on a particular day the ground surface gets heated unequally, the warm air is lifted to high altitude and the cooler air takes its place with high velocity. Thus, the warm moist air alone is condensed at the high altitude causing heavy rainfall. This is known as convective precipitation.
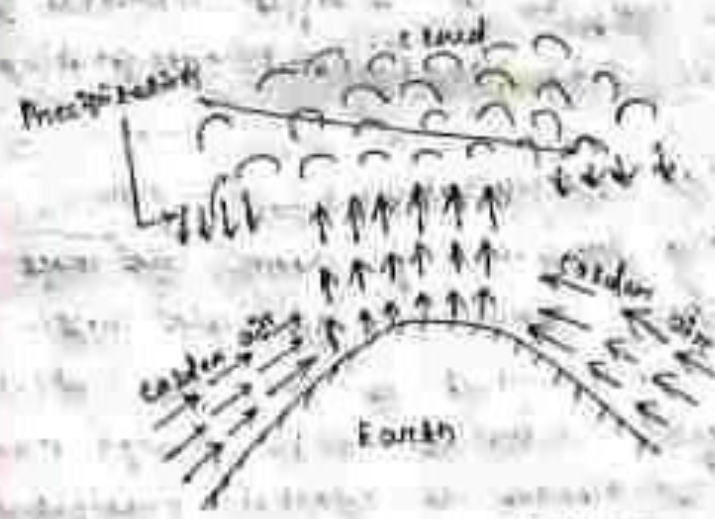
(Convective precipitation)

2. Orographic precipitation :–

The moving warm moist air when obstructed by some mountain rises up to a high altitude. It thus gets condensed and precipitation occurs. This is known as orographic precipitation.
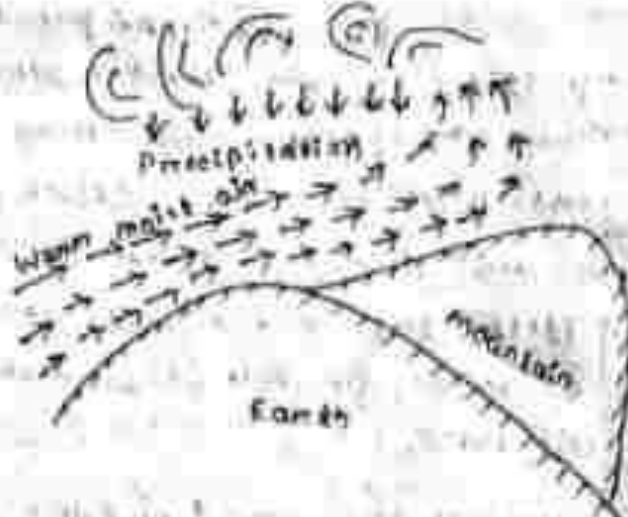
(Orographic precipitation)

Cyclonic precipitation :—

This type of precipitation caused by difference of pressure within the air mass on the surface of earth. If low pressure is generated at some place the warm air from the surrounding area rushes to the zone of low pressure & the warm air rises up with cooling mass and gets condensed at higher altitude and ultimately heavy rainfall occurs. This may be of 2 types (1) Frontal precipita (2) Non-Frontal precipitation

Frontal precipitation :—

When the moving warm air mass is obstructed by zone of cold air mass, the warm moist air rises up to higher altitude where it get condensed and heavy rainfall occurs this is known as Frontal precipitation.

Frontal precipitation is of 2 types

(Hot air Frontal precipitation)

(cold air frontal precipitation)
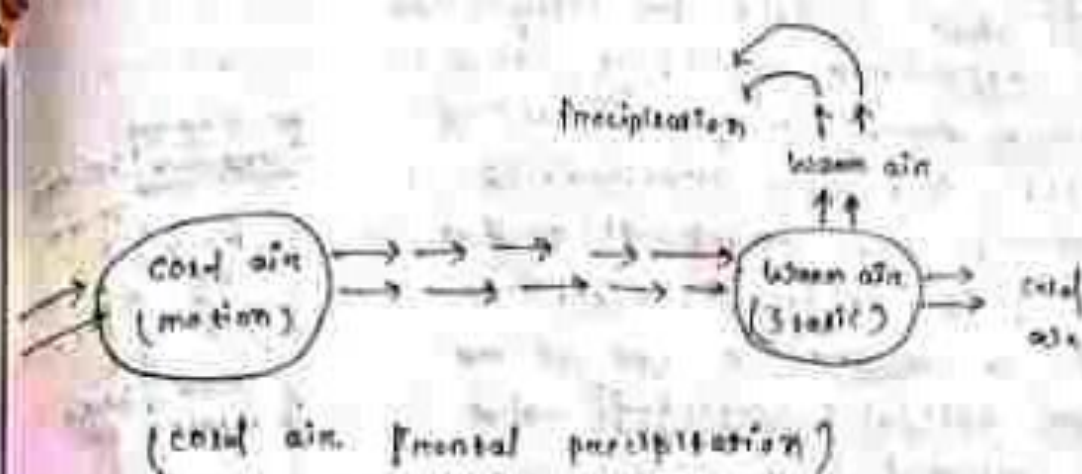
Non-frontal precipitation :—

When the warm air reaches to the zone of low pressure air pocket its form and the warm air rises up like a chimney. At higher altitude this air may get condensed and heavy rainfall occurs. This is known as non-frontal precipitation.

Measurement of Rainfall (i.e. Precipitation) :—

This measurement which is used to measure the amount of rainfall is known as raingauge. The principal of raingauge is the amount of rainfall in a small area will represent the amount of rainfall in a large area provided the meteorological characteristics for both small and large area are similar. The raingauges are of the following types.

A. Non-Recording type raingauge :—

Simon's raingauge is a non-recording type of raingauge which is most commonly used. It consist of metal casing of diameter 127mm which is set on a concrete foundation. A glass bottle of capacity about 100mm of rainfall is placed within the casing. A funnel with brass rim is fitted on the top of the bottle. The rainfall is recorded at every 24 hours. Generally, the measurement is taken at 8.30 a.m.

everyday. In case of heavy rainfall the measurements should be taken 2 or 3 times daily so that the bottle does not overflow. To measure the amount of rainfall the glass bottle is taken off and the collected water is measured in a measuring glass, and recorded in the raingauge record book. When the glass bottle is taken off it is immediately re-placed with a new bottle of same capacity.
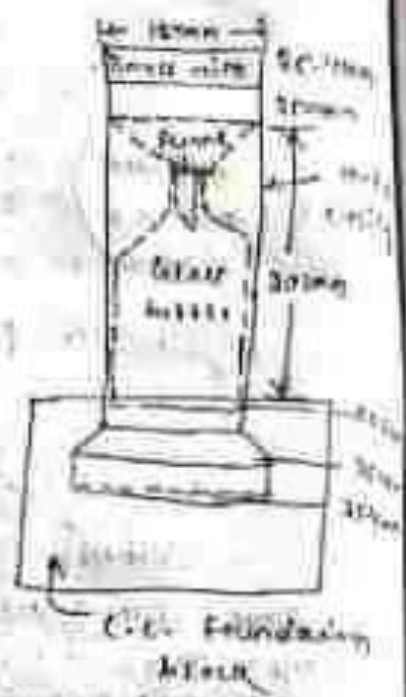
2. Recording type Raingauge :— [Siemens raingauge]

In this type of raingauge, the amount of rainfall is automatically recorded on a graph paper by some mechanical device. Here, no person is required for measuring the amount of rainfall from the container in which the rain water is collected. The recording type raingauge may be of 3 types :

(a). Weighing Bucket Raingauge :—

This type of raingauge consists of a receiving bucket which is placed on pan. The pan is again fitted with same weighing mechanism. A pencil arm is pivoted with the weighing mechanism in such a way that the movement of the bucket can be traced by a pencil on a moving recording drum. So when the water is collected in the bucket the increasing weight of water is transmitted through the pencil which traces a curve on the recording drum. The raingauge produces a graph of cumulative rainfall versus time and hence it is some times called integrating raingauge. The graph is known as the mass curve of rainfall.

Receiving bucket

Pan

Weighting Mechanism

Recording drum

Pencil arm

Rainfall in cm

→ Time in hours
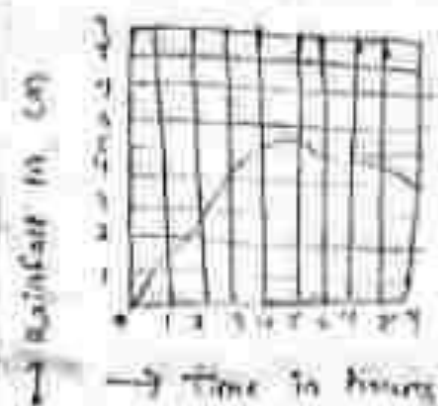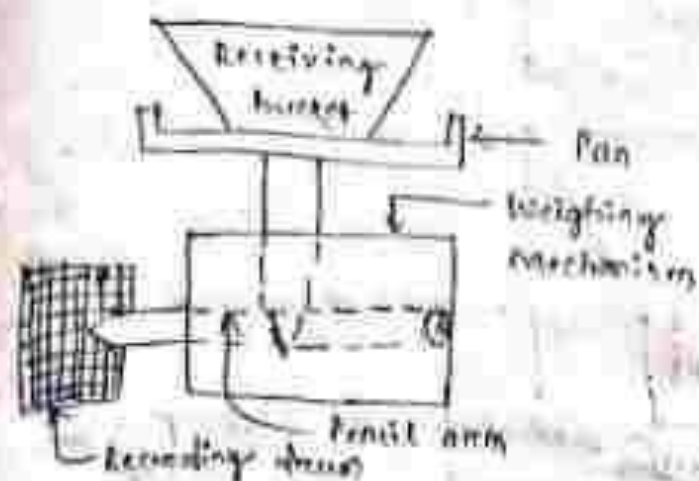
(Weighing bucket raingauge)

Tipping Bucket Raingauge :—

It consist of a circular collector of diameter 30 cm in which the rain water is initially collected. The rain water then passes through a funnel fitted to the circular collector and gets collected in two compartment tipping buckets pivoted below the funnel.

when 0.25 mm rain water is collected in one bucket then it tips and discharge the water in a reservoir kept below the buckets. At the same time the other bucket comes below the funnel and the rain water goes on collecting in it. When the requisite amount of rain water is collected it also tips and discharge the water in the reservoir. In this way, a circular motion is generated by the buckets. This circular motion is transmitted to a pen or pencil which traces a wave like curve on the sheet mounted on a revolving drum. The total rainfall may be ascertained from the graph. There is an opening with stopcock at the bottom of the reservoir for discharging the collected rainwater. Sometimes a measuring glass is provided to verify the result shown by the graph.
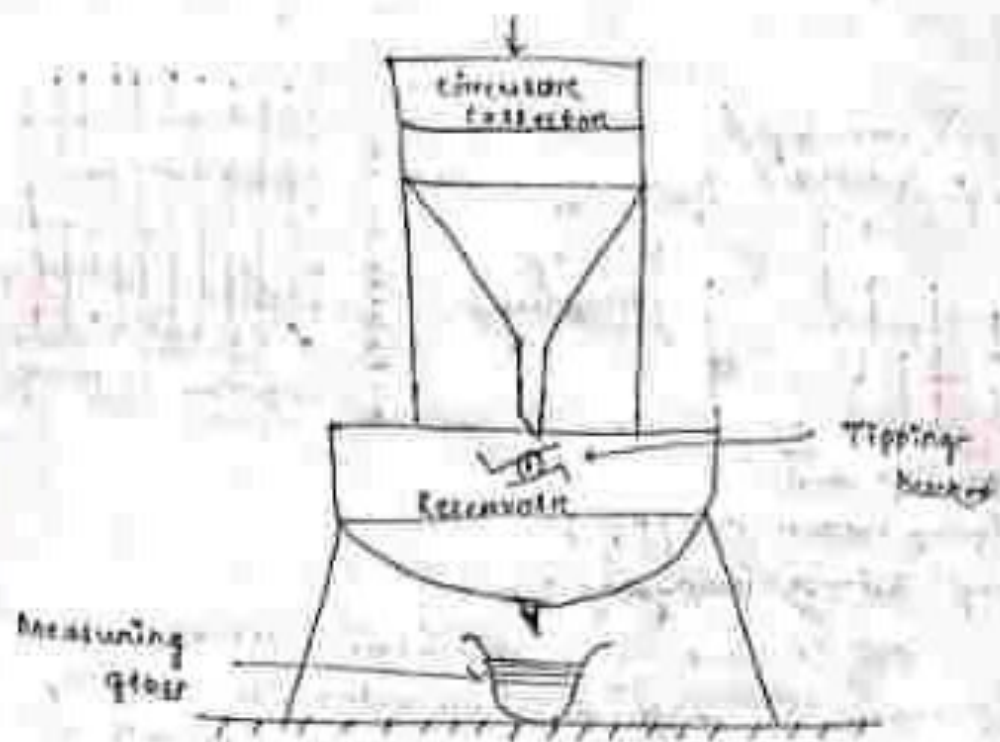
(c) **Float type**

(Tipping Bucket raingauge)

Float type raingauge:

In this type of raingauge, a funnel is provided as one end of a rectangular container and a rotating recording drum is provided at the other end. The rain water enters the container through the funnel. A float is provided within the container which rises up a rain water gets collected there. The float consists of a rod which contains a pen also for recording the amount of rainfall on the graph paper wrapped on the recording drum. It consists of a syphon which starts functioning when the float rises to some definite height and the container goes on emptying gradually.
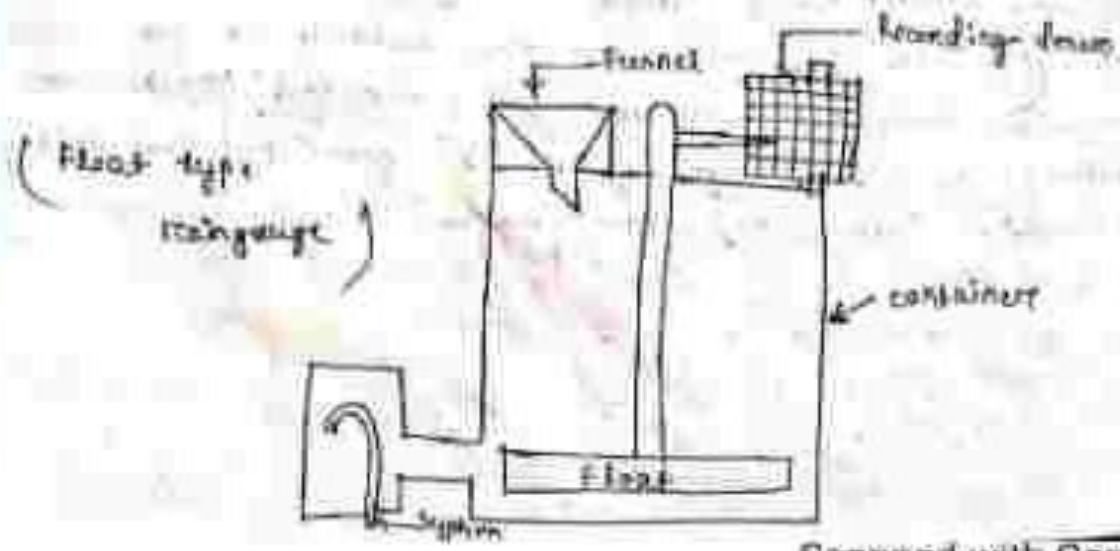
(Float type raingauge)

Average depth of precipitation :—

One raingauge station cannot represent a large basin. So a basin is always composed of many raingauge stations which are evenly distributed throughout the whole basin. Again, the amount of rainfall may not be equal in all raingauge stations. Since the average rainfall of the basin is required for estimating the runoff from the basin, it is customary to apply any suitable method to determine the average depth of precipitation. The following three methods are generally adopted to calculate the average depth of precipitation.

1. Arithmetic Mean Method :—

This method is very simple. In this method the rainfall values obtained from all the raingauge stations are added and divided by the number of stations to get the average value. Suppose, N is the number of stations and $c_1, c_2, c_3, \ldots$ etc. are the rainfall values obtained from the stations. Then, average depth of precipitation =

$$= \frac{c_1 + c_2 + c_3 + \cdots}{N}$$

2. Thiessen Polygon Method :—

This method is highly suitable for large areas. It is based on the assumption that each raingauge station has its own domain within the basin area. That domain may be defined by geometrical construction as follows.

(i) Suppose A, B, C, D, E, F are the raingauge stations. All the stations are joined by dotted lines to form a number of triangles.

(ii) The ⊥ar bisectors are drawn to each sides of the triangles. Thus a closed polygon abcde is formed which indicates the domain of the raingauge station.

(iii) Similarly, all the other raingauge stations within

the basin are joined to form triangles. The ⊥ bisectors are drawn as above. Thus, a number of polygons are plotted.

(iv) Geometrically, it can be proved that each polygon represents the domain of each raingauge station.

(v) The area of each polygon is measured by graph paper or by planimetre.

(vi) The result is then tabulated as follows.
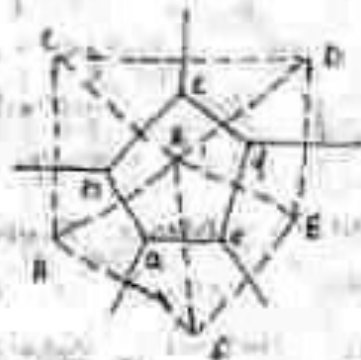
Station	Rainfall in cm	Area of polygon in sq km	Product of rainfall and area
(1)	(2)	(3)	(4) × (3)×(2)
A	10·0	120	1200·0
B	12·0	95	1140·0
C	9·5	105	997·5
D	8·0	125	1000·0
E	8·7	85	127·0
F	7·8	100	780·0
G	6·0	105	630·0
		Σ 745·0	Σ 6904·5

Average depth of precipitation = $\dfrac{6904·5}{745·0}$ = 9·27 cm

(3) Iso-hyetal Method :——

A iso-hyetal line represents a line joining the points of equal depth of precipitation. So, it is just like a contour line. In this method all the raingauge stations are located within the map of the required basin. Then depth of precipitation of all stations are noted at the respective station point

Using the Iso-hyetal lines are drawn at a time interval by the method of interpolation. The area enclosed between the two successive Iso-hyet lines in found out by graph paper are by planimeter. The result is tabulated as shown to get the average depth of precipitation.

Iso-hyetal Interval	Average depth	Area b/n two successive Iso-hyet lines	Average depth × area
(1)	(2)	(3)	(4) = (2)×(3)
10 – 9	9.5	a	9.5 × a
9 – 8	8.5	b	8.5 × b
8 – 7	7.5	c	7.5 × c
7 – 6	6.5	d	6.5 × d
		ΣA (say)	ΣB (say)

average depth of precipitation =

$$\frac{\Sigma B}{\Sigma A} = x \text{ cm (say)}$$

Rainfall Hyetograph :—

It is the graphical representation of Rainfall Intensity in y-axis (ordinate) Versus time in hours in abscissa axis (x-axis).

$$\left(\text{Intensity} = \frac{\text{Rainfall}}{\text{Time}} \right)$$

For example :—
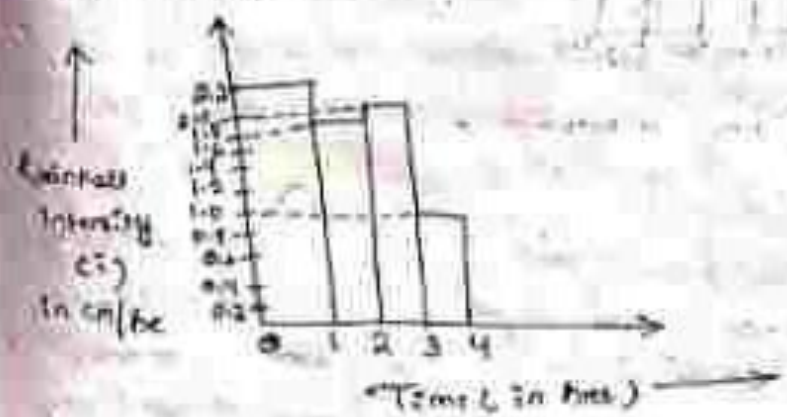
Total Rainfall = 5.5×1 + 1.5×1 + 2×1 + 2×1
= 7 cm Rainfall

Q.2 The following are the rates of rainfall for successive 30 mins period for a Storm direction of 210 mins. (5.5, 6.0, 12.5, 7.0, 3.25, 6.0 cm/hr)

Find the total rainfall.
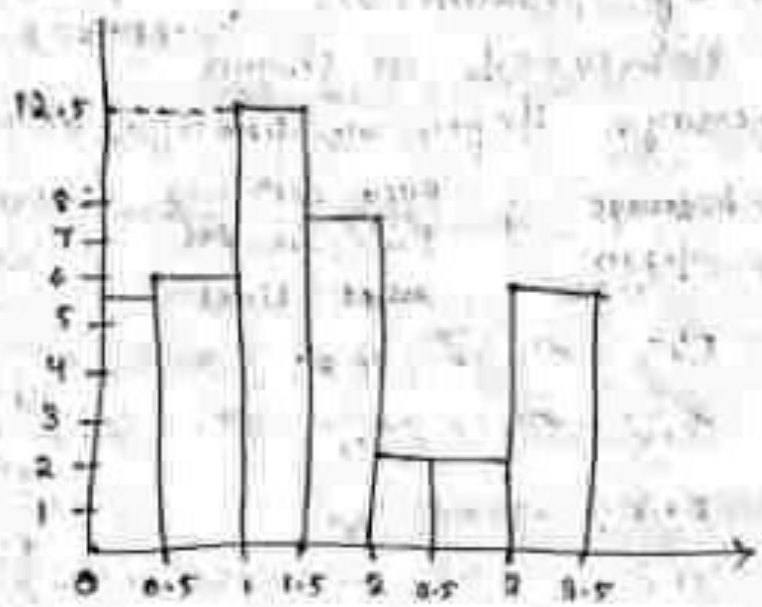
Total rainfall = 5.5 × 0.5 + 6.0 × 0.5 + 12.5 × 0.5 + 7.0 × 0.5 + 3.25 × 1 + 6.5 × 0.5

= 22.5 cm/hre

Q.3 Draw the rainfall hyetograph from the Rainfall mass curve given below.
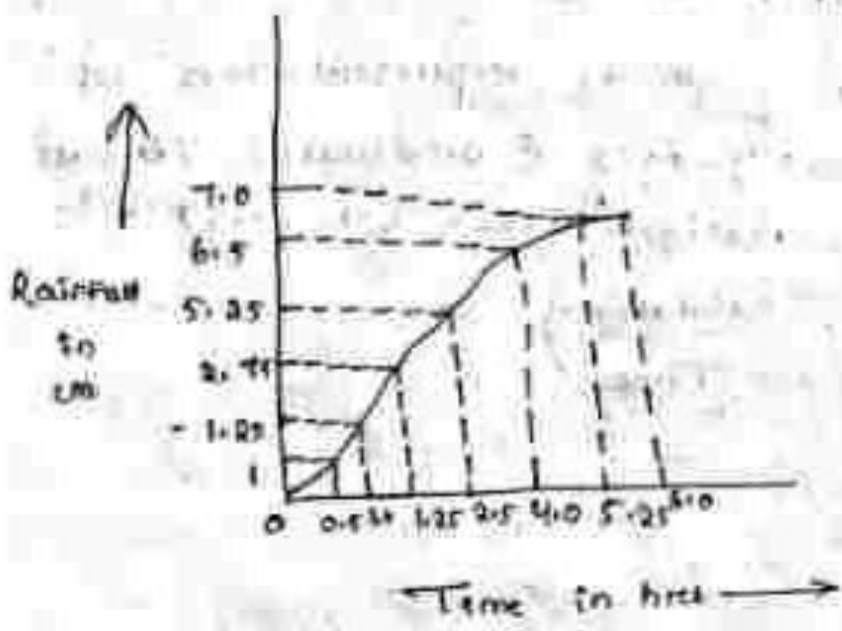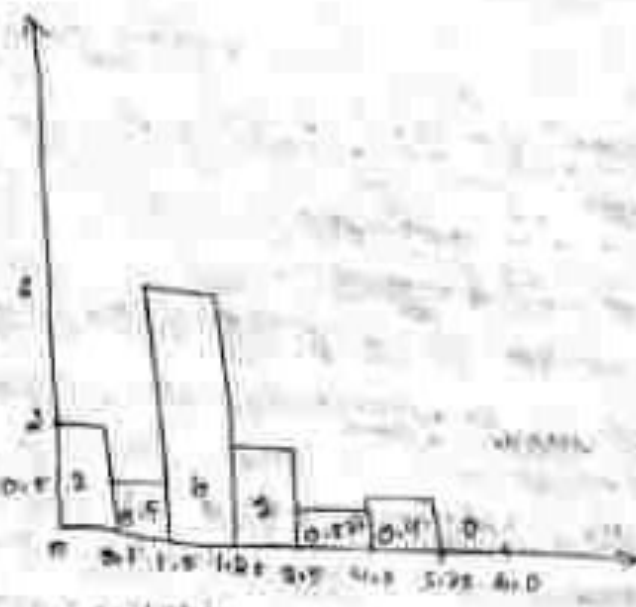
← Time in hrs →

$$i_1 = \frac{1}{0.5} = 2\,cm/hr$$

$$i_2 = \frac{0.25}{0.5} = 0.5\,cm/hr$$

$$i_3 = \frac{1.0}{0.5} = 2\,cm/hr$$

$$i_4 = \frac{2.5}{1.25} = 2\,cm/hr$$

$$i_5 = \frac{1.25}{1.25} = 0.833\,cm/hr$$

$$i_6 = \frac{0.5}{1.25} = 0.03\,cm/hr$$

$$i_7 = \frac{0}{0.25} = 0\,cm/hr$$

$$\text{Total rainfall} = 2 \times 0.5 + 0.5 \times 0.5 + 0.25 \times 2 + 2 \times 1.25$$
$$+ 0.833 \times 1.25 + 0.4 \times 1.25 + 0 \times 0.25$$
$$= 6.4415 \simeq 7\,cm$$

Run off :—

Whenever it rain part of the rainwater goes deep below the ground and finally reaches to ground water, some part loses due to natural process like evaporation, transpiration, interception and remaining water after constituting the interception Storage, over and under the ground Surface. This water is known as runoff.

Runoff can by classified as

1) Surface Runoff
2) Sub Surface Runoff

surface runoff

sub surface runoff
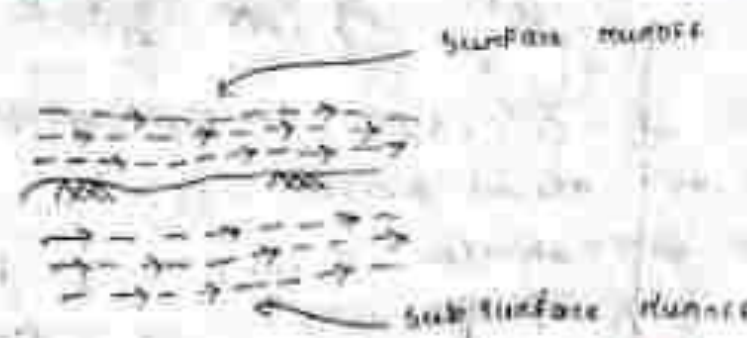
Catchment area :—

The catchment area of a given stream is the area from where the rain water run off to the river through the streams, springs, tributaries etc. Catchment area is also known as watershed.

Estimation of Flood discharge :—

Flood discharge can be extracted by

(a) Dicken's Formula :—

$$Q = C \times A^{3/4}$$

where Q = Discharge max cumec (m^3/sec)

A = catchment area in sq.km

C = A constant depending upon the factors affecting the flood.

* Dicken's formula is used for catchments in North India and central India.

(b) Ryve's Formula :—

$$Q = C \times A^{2/3}$$

where Q = flood discharge in cumec

A = Catchment area in sq.km

C = A constant depending upon the factors affecting the flood.

Q. Find the flood discharge of an area using Dicken's formula and Ryve's formula. The constant C in Ryve's formula and Dicken's formula may be taken as 6.8, 11.5 respectively. The area of catchment may be taken as

Ans   Given data

$$A = 100000 \text{ hec}$$

$$1 \text{ hectare} = 10000 m^2$$

$$100000 = 100000 \times 10000 m^2$$

$$= 10^9 m^2$$

$$1 km^2 = 1 km \times 1 km$$

$$= 1000 m \times 1000 m$$

$$1 km^2 = 10^6 m^2$$

$$\frac{1}{10^6} km^2 = 1 m^2$$

$$10^3 km^2 = 10^9 m^2$$

$$C_D = 11.5$$

$$C_R = 6.8$$

According to Dicken's formula

$$Q = C \cdot A^{3/4}$$

$$= 11.5 \times (10^3)^{3/4}$$

$$= 2045.02 \text{ cumec}$$

According to Ryve's formula

$$Q = C \cdot A^{2/3}$$

$$= 6.8 \times (10^3)^{2/3}$$

$$= 680 \text{ cumec}$$

Cross drainage Works :—

Necessity of Cross drainage work :—

In an irrigation project canals may have to cross natural drainages like rivers, streams etc. The crossing of the canals with such obstacles cannot be avoided. So suitable structure must be constructed at the crossing point to ensure the easy flow of water in the canal and drainage in their respective direction. These structures are known as cross drainage works.

Necessity of cross drainage works :—

The following factors justify the necessity of cross drainage work :

(i) At the crossing point the water of the canal and drainage will get intermixed. For smooth running of the canal and drainage water the some is required.

(ii) The site condition of the crossing point may be such that without any suitable structure the water of the canal and drainage can be diverted to their natural direction. So cross drainage works must be provided to maintain their natural direction of flow.

Types of cross-drainage works :—

According to the relative bed levels, maximum water levels and relative discharge of the canals and drainages the cross drainage works may be of the following types.

Type-1 Irrigation canal passes over the drainage :—

This condition involves the construction of following :

(a) Aqueduct :—

The hydraulic structure in which the irrigation canal is taken over the drainage (such, river, stream etc) is known as aqueduct. This structure is suitable when

bed level of canal is above the highest flood level of drainage. In this case, the drainage water passes cleanly below the canal.
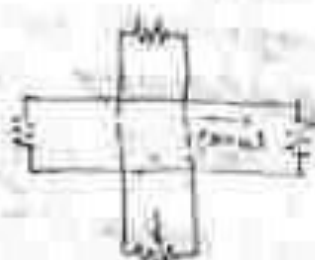
(b) Siphon Aqueduct :—

In a hydraulic structure where the canal is taken over the drainage, but the drainage water cannot pass cleanly below the canal. It flows under siphonic action, so it is known as siphon aqueduct. This structure is suitable when the level of canal is below the highest flood level of the drainage.

Type - II   Drainage passes over the irrigation canal :

This condition involves the construction of the following :

(a) Super passage :—

The hydraulic structure in which the drainage is taken over the irrigation canal is known as super passage. The structure is suitable when the bed level of the drainage is above the full supply level of the canal. The water of the canal flows cleanly below the drainage.
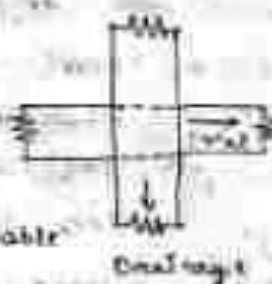
(b) Siphon Super passage :—

The hydraulic structure in which the drainage is taken over the irrigation canal, but the canal water passes below the drainage under siphonic action is known as siphon super passage. This structure is suitable when the bed level of drainage is below the full supply level of the canal.

Type - III   Drainage and canal intersection each other at the same level :—

This condition involves the construction of the following :

(a) Level crossing :—

When the beds of the drainage and canal are practically at the same level, then a hydraulic

Structure is constructed which is known as a crossing. This is suitable for the crossing of large drainage with maximum channel.

(b) Inlet and outlet :—

In the crossing of small drainage with small channel no hydraulic structure is constructed. Simple openings are provided for the flow of water in their respective dimensions. This arrangement is known as inlet and outlet.

Aqueduct :—

The aqueduct is just like a bridge where a canal is taken over the deck supported by piers instead of a road or a railway. Generally, the canal is in the shape of a rectangular trough which is constructed with reinforced cement concrete. Sometimes, the trough may be of trapezoidal section. An inspection road is provided along the side of the trough. The bed and banks of the drainage below the trough is protected by boulder pitching with cement grouting. The section of the trough is designed according to the full supply discharge of the canal. A free board of about 0.50m should be provided. The height and location of piers are designed according to the highest flood level and velocity of flow of the drainage. The piers may be of brick masonry, stone masonry or reinforced cement concrete. Here, deep foundation (pile well foundation) is most necessary for the pier. The concrete foundation may be done by provided

and depth of foundation according to the availability of hard soil.

Siphon Aqueduct :—

The Siphon aqueduct, the bed of the drainage is depressed below the bottom level of the canal trough by providing sloping apron on both sides of the crossing. The sloping apron may be constructed by stone pitching or cement concrete. The bottom of the drainage below the canal trough is constructed with cement concrete in the form of tunnel. This tunnel acts as a siphon. Cut off walls are provided on both sides of the apron to prevent scouring. Boulder pitching should be provided on the upstream and downstream of the cut off walls. The other components like canal trough, piers, inspection road etc. should be designed according to the methods adopted in case of aqueduct.
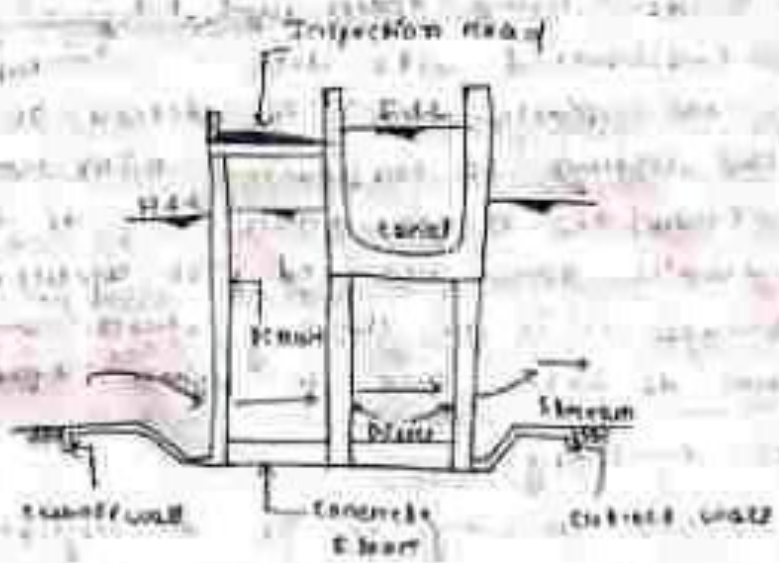
Siphon aqueduct

Super passage :—

The supper passage is just opposite of the aqueduct. In this case, the bed level of the drainage is above the fully supply level of the canal. The drainage is taken through a rectangular
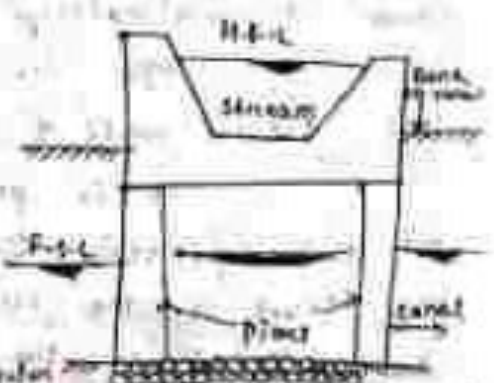
On trapezoidal through of channel which is supported on the deck supported by piers. The section of the drainage through depends on the high flood discharge. A free board of about 1.5m should be provided for safety. The through should be constructed of mini cement concrete. The bed and banks of the canal below the drainage through should be protected by boulder pitching or lining with concrete slab. The foundation of the piers will be same as in the case of aqueduct.

Siphon Super passage :—

It is just opposite syphon aqueduct. In this case, the canal passes below the drainage through. The section of the through is designed according to high flood discharge. The bed of the canal is depressed below the bottlevel of the drainage through by providing sloping apron on both sides of the crossing. The sloping apron may be constructed with stone pitching or concrete slab. The section of the canal below the through is constructed often concrete in the form of tunnel which acts as syphon. Cut-off walls are provided on upstream and down stream side of sloping apron. Other components are same as in the case of syphon aqueduct.

Level crossing :—

Level crossing is an arrangement provided to regulate the flow of water through the drainage and the canal when they cross each other approximately at the same bed level. The level crossing consists of the following components.

(i) Crest wall :—

It is provided across the drainage just at the upstream side of the crossing point. The top level of the crest wall is kept at full supply level of the canal.

(c) Drainage Regulator :—

It is provided across the drainage just on the down-stream side of the crossing point. The regulator consists of adjustable shutters of different items.

(3) Canal Regulators :—

It is provided across the canal just at the downstream side of the crossing point. This regulator also consists of adjustable shutters at different items.

Operation :—

In dry season when the discharge of the drainage is very high, the drainage regulator is kept closed and the canal water is allowed to flow at count. In rainy season, when the discharge of the drainage is very high, the drainage regulator is kept completely open and the canal regulator is adjusted according to requirements. The level crossing is recommended for the crossing of main canal with large drainage.
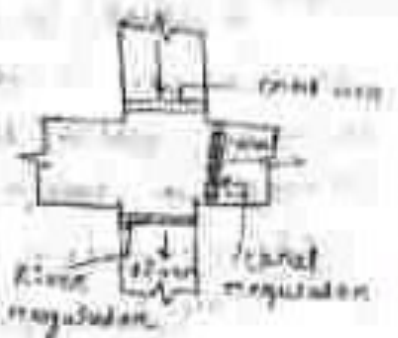
Inlet and outlet :—

In case of crossing of a small irrigation channel with a small drainage, no hydraulic structure is constructed, because, the discharges of the drainage and the channel are practically low and these can be easily located by using system like inlet and outlet arrangement. In this system an inlet is provided in the channel bank simply by open cut and the drainage water is allowed to join the channel. Then at a suitable point on the down stream side of the channel an outlet is provided by open cut-

and the water from the irrigation channel is allowed to flow through a leading channel leaving the original route of the drainage are protected by stone pitching. The bed and banks of the irrigation channel between inlet and outlet points should be also be protected by stone pitching.
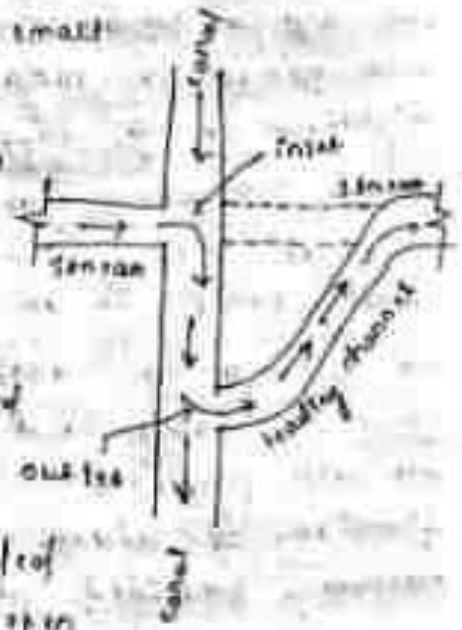
Flow Irrigation :—

Canals :—

A canal is an artificial channel, generally trapizoidal in shape constructed on the ground, to carry water to the fields either from the river or from a tank or reservoir.

Classification of canals :—

Canals can be classified in following ways

(a) Classification based on the nature of source of Supply :

(1) Permanent canal
(2) Inundation canal

• A canal is said to be permanent when it is fed by a permanent source of supply. It has also permanent masonry works for regulation and distribution of supplies.

A permanent canal is also sometimes known as perennial canal when the sources from which canal takes it an its bed perennial river.

→ Inundation canals usually draw their rivers whenever there is a high stage in the river. They are not provided with any head works for diversion of river water to the canal. They are however, provided with a canal head regulators

(b) Classification based on financial out put :

(1) Productive canal
(2) Protective canal

productive canals are those which yield a revenue to the nation after fully developing irrigation in the area.

A protective canal is a sort of relief work constructed with the idea of protecting a particular area from famine.

a) Classification based on the function of the canals:

(1) Irrigation canal
(2) Carrier canal
(3) Feeder canal
(4) Power canal

1. A irrigation canal carries water to the agricultural fields.

2. A carrier canal besides doing irrigation, carries water for another canal.

3. A feeder canal is constructed with the idea of feeding one or more canals.

b) Classification based on boundary surface of the canal:

(1) Alluvial canal:

An alluvial canal is the one which is excavated in alluvial soils, such as silt.

(2) Non-alluvial canals:

A non-alluvial canal is the one which is excavated in non-alluvial soils, such as loam, clay, hard soil (murrum).

(3) Rigid boundary canals:

Rigid boundary canals are those which have rigid sides and rigid base, such as lined canals.

c) Classification based on the discharge and its relative importance in a given network of canals:

(1) Main canal:

Main canal generally carries water directly from the river or reservoir. Such a canal

carries heavy supplies and is not used for direct irrigation except in exceptional circumstances. Main canals are at relatively low command to hand supplies to branches canals, and major distributaries.

(2) Branch canal :—

Branch canals are the branches of the main canal in either direction, taking off at required intervals. In general, branch canals also do not carry out any direct irrigation, but sometimes direct outlets may be provided. Branch canals are usually feeder channels for major and minor distributaries. They usually carry a discharge of over 5 cumecs.

(3) Major distributary :—

Main distributaries usually called Rajbaha, take off from a branch canal. They may also sometimes take off from the main canal, but their discharge is generally lesser than branch canals. They are real irrigation channels, in the sense that they supply water for irrigation to the field through outlets provided along them.

→ Their discharge varies from 1/4 to 5 cumecs.

(4) Minor distributary :—

Minor distributaries are minor takes off from branch canals or from distributaries. Their discharge is usually less than 1/4 cumecs. They supply water to the water courses through outlets provided along them.

(5) Water course :—

A water course or field channel is a small channel which ultimately feeds the water to irrigation fields. Depending upon the size and extent of the irrigation scheme, a field

channel may take off from a distributary or sometimes, it may even take off from the branch channel, for the field situated very near the branch canal.

Classification based on canal alignment:

According to the alignment a canal may be classified such follow:

1) Contour canal :—

The canal which is aligned approximately parallel to the contour lines is known as contour canal. This canal can irrigate the area on one side only. This canal may cross natural drainage and hence cross drainage works are necessary.
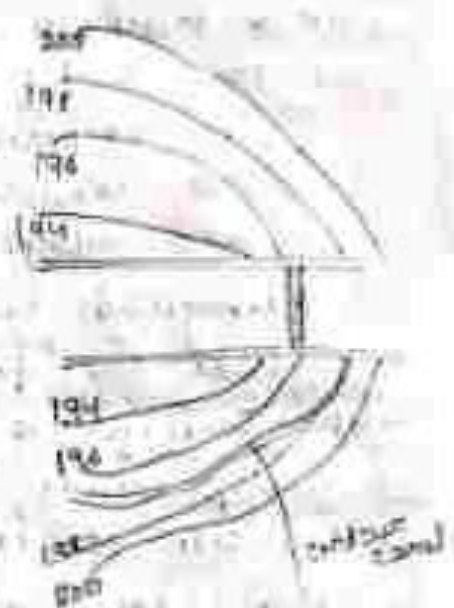
2) Ridge canal or Watershed canal :—

The canal which is aligned along the ridge line (watershed line) is known as ridge canal or watershed canal. The advantages of this type of canal is that it can irrigate the areas on both sides again there is no possibility of crossing any natural drainage and hence no cross drainage work is necessary.
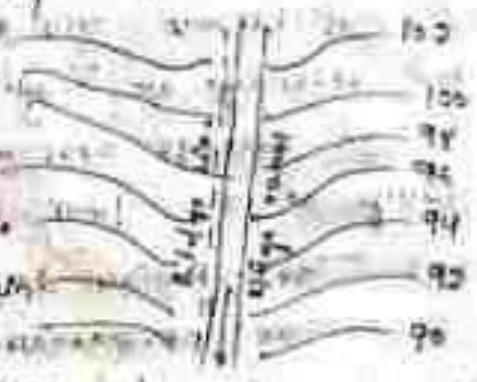
3) Side - slope canal :—

The canal which is aligned approximately at right angle to the contour lines is known as side slope canal. This again irrigate the areas on one side only. Again, it does not cross any natural drainage and hence, cross drainage works are not necessary.
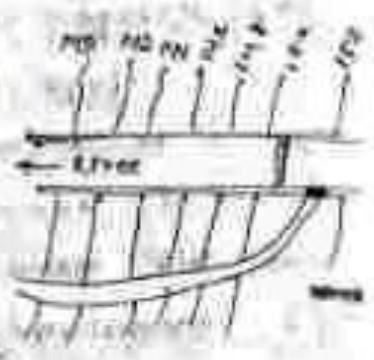
Losses in canals :—

When water continuously flows through a canal losses take place due to seepage, deep percolation and evaporation. These losses are some times known as transmission losses. These losses should be properly accounted for, estimating lesser quantities water will be available for cultivation at the tail end.

→ Water losses in canals can be broadly classified under three head :

(1) Evaporation losses
(2) Transpiration losses
(3) Seepage losses

(1) Evaporation losses :—

The loss due to evaporation is generally a small percentage of the total loss. In unlined canal it hardly exceeds 1 to 2 percent of the total water entering into the canal. The evaporation depend upon
(i) climate factor such as temperature, humidity and wind velocity.
(ii) canal factor such as surface, hidden depth and velocity of flow.

The average evaporation loss per day may vary between 4mm to 10mm.

(2) Transpiration losses :—

The transpiration loss take place through the of vegetation and weed growth along the bank of canal. However, this form a extremely small part of total loss.

(3) Seepage losses :—

Seepage losses constitute major portion of loss in an unlined canal. The seepage losses are due to (i) absorption of water in the upper layer of soil below the canal bed.
(ii) percolation of water into the water table, thus raising the water table.

the however water table is much lower, seepage losses are only due to absorption; percolation losses are always much more than the absorption losses.

Different components of irrigation canal and their function : —
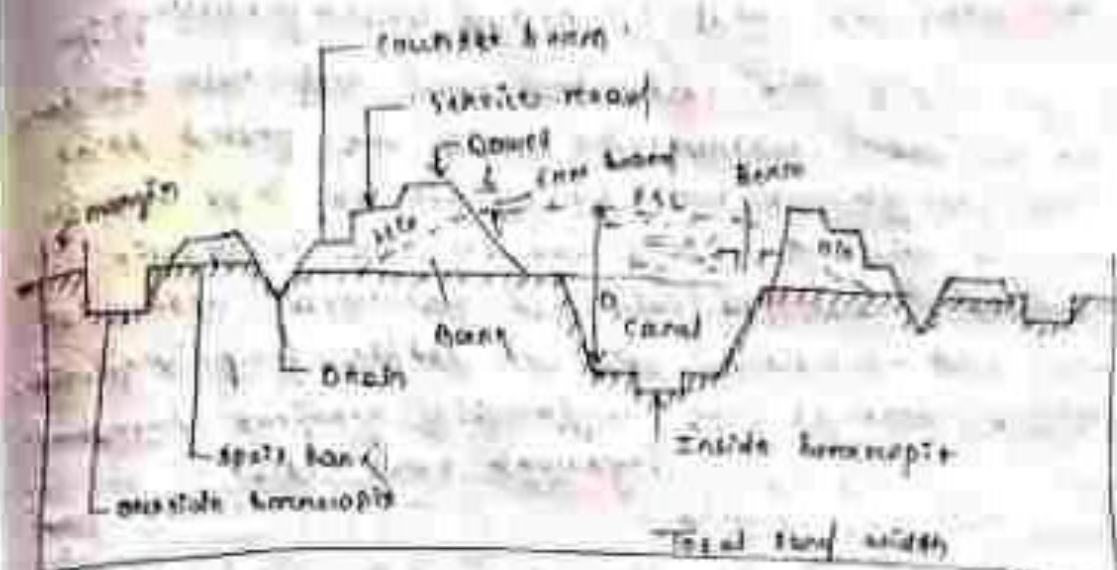
FSL = Full Supply Level
D = Full Supply depth

Canal Bank : —

The canal bank is necessary to maintain water in the canal to the full supply level. But the section of the canal bank is different for different site condition.

(i) when the canal fully in cutting : —

In this case the banks are constructed on both sides of the canal to provide only a inspection road. Here, the hydraulic gradient has no function. So the height of the bank will be low and the top width will be minimum just to provide the road way. The side slope will be 1.5 : 1 on 2 : 1 according to the nature of the soil.

(A) when the canal in partial cuttings and banking (filling) :—

In this case the banks are constructed on side of the canal to retain water. The height of the banks depends on the full supply level of the canal. Again, the section of the canal depends on the hydraulic gradient. The top width and its side slope of the bank should be such that hydraulic gradient should have a minimum _____ cover as 0.5m.

(B) When the canal in cutting banking (filling) :—

In this case, the canal and both the ____ banks are constructed above the ground level. The height of the bank will be high and its section will be large due to the hydraulic gradient. But to minimise the cross section of the bank a core wall or puddle clay is provided, which detains the hydraulic gradient downwards.

((A) canal section in full cutting)
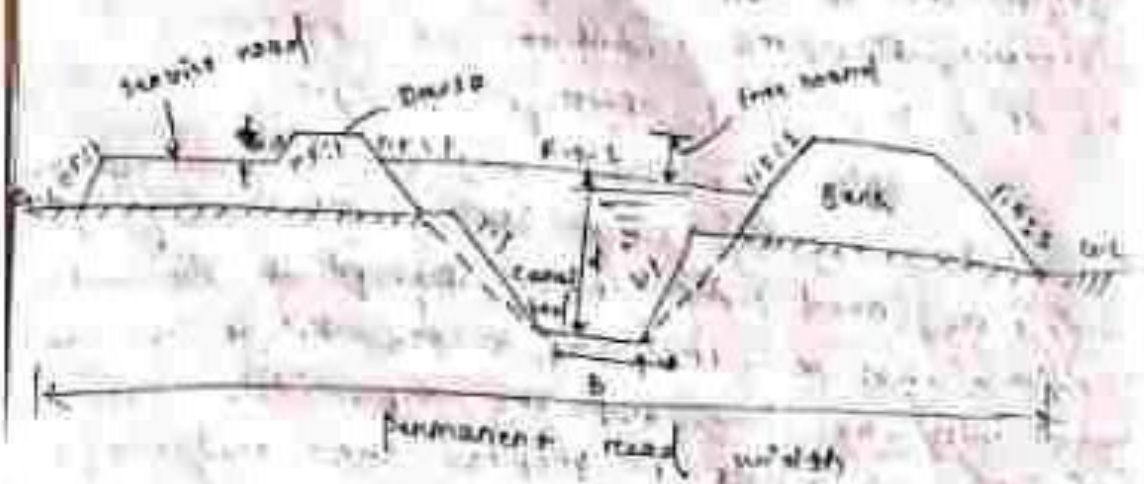
((B) canal section partially cuttings and filling)
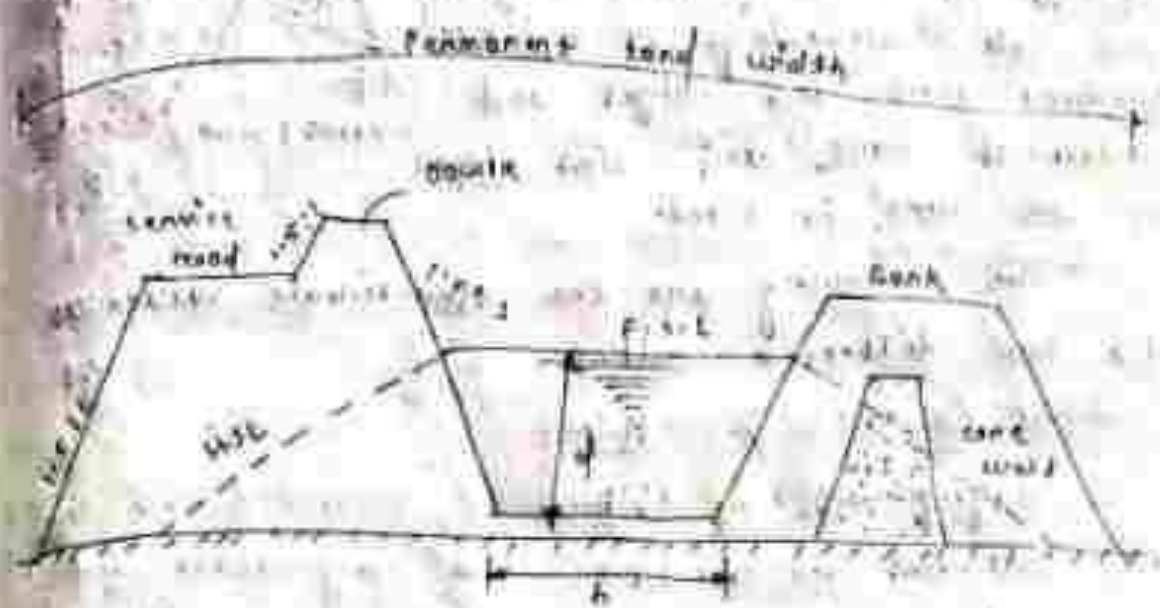
Berm :-

The distance between the toe of the bank and the top edge of cutting is termed as berm. The berm is provided for the following reasons:

(1) To protect the bank from erosion.

(2) To provide a space for widening the canal section in future, if necessary.

(3) To prevent the bank from sliding down towards the canal section.

(4) The silt deposited on the berm makes an impervious lining.

(5) The necessary material can be excavated on the berm.

The width of berm varies from D to 2D where D is the full supply depth of the canal.

Hydraulic Gradient :-

When the water is termed by the canal bank, the seepage occurs through the body of the bank. Due to the resistance of the soil, the saturation line forms a sloping line which may pass through country side of the bank. This sloping line is known as the hydraulic gradient on

saturation gradient.

The hydraulic gradient depends on the permeability of the soil. According to hydraulic gradient line the height and width of bank and side slope as the bank is fixed.

The following are the approximate values of HG for different soils.

Soil	H.G
Clayey soil	1:4
Alluvial soil	1:5
Sandy soil	1:2

Counter Berm :—

When the water is retained by a canal the hydraulic gradient line passes through the body of the bank. For stability of the bank, this gradient should not intersect the outer side of the bank. It should pass through the soil and a minimum cover of 0.5m should always be maintained. Sometimes it may occur that the hydraulic gradient line intersect the outer side of the bank. In that case, a projection is provided over the bank to obtain minimum cover. This projection is known as counter berm.

Free board :—

It is the distance between the full supply level and top of the bank. The amount of free board varies from 0.5m to 1m.

It is provided for the following reasons.

1. To keep a sufficient margin so that the canal does not overlap the bank in case of heavy rainfall or fluctuation in water supply.

2. To keep the saturation gradient much below the top of the bank.

Side Slope :—

The side slopes of the canal bank and canal section depend on the angle of repose of the soil existing on the site. So, to determine the side slopes at different sections, the soil sampled should be collected from the sites and should be tested in the soil testing laboratory. The necessity of such test is that if the permissible slope (as per angle of repose) is not provided in an embankment or cutting, then the soil in that place will go on sliding gradually until the angle of repose for that particular soil is attained.

For instance, suppose an embankment was constructed with side slope 1:1 but, according to the nature of the soil, the side slope should be $1\frac{1}{2}:1$. Then the initial shape ABCD will automatically take the final shape A_1, B_1, C_1, D_1 after slide in the due course.

Again, an opposite incident may occur, suppose an embankment was constructed with side slope 2:1, but later it was found that the side slope of 1:1 was sufficient to maintain the angle of repose for that soil. In this case, an unnecessary earthwork was done.

Sliding of bank

Extra earth filling

final earthwork
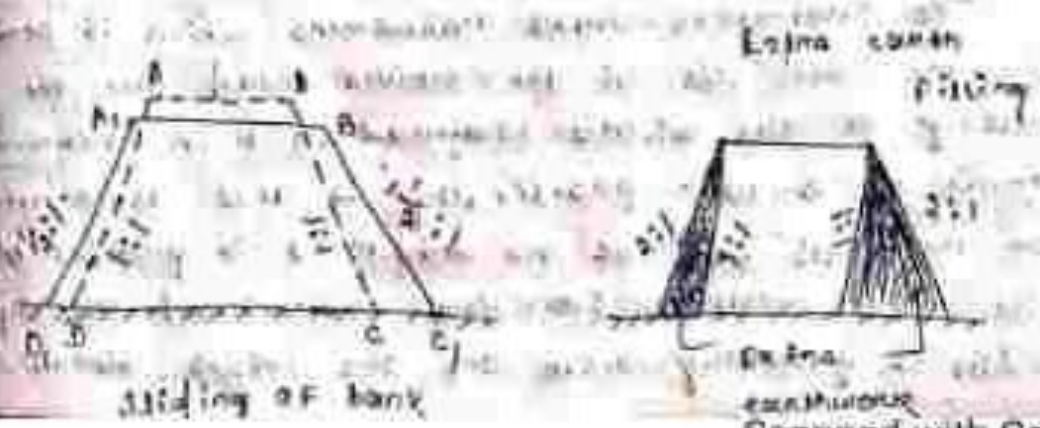
The permissible side slopes for some soil are given in the following table:

Type of soil	Side slope in cutting	Side slope in banking
Clayey soil	1:1	1½:1
Alluvial soil	1:1	2:1
Sandy loam	1½:1	2:1
Sandy soil	2:1	3:1

Service Road :—

The roadway which is provided on the top of the canal bank for inspection and maintenance works is known as service road or Inspection road. For main canal, the service roads are provided on both the banks. But for branch canals, the road is provided on one bank only. The width of the service roads for main canal varies from 4 to 5m. The width of the road for the branch canal varies from 3 to 4m.

The initial purpose of the service road is to conduct the inspection and maintenance works. But usually these roads serve the purpose of communication between the different villages and for transporting agricultural goods. Therefore it becomes necessary to construct metalled service road and dust road to serve these purposes.

Dowla or Dowka :—
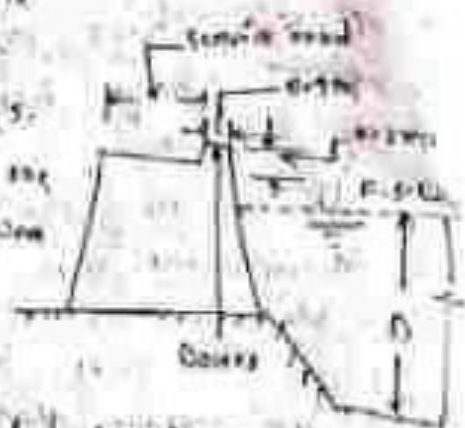
The protective small embankment which is provided on the canal side of the service road for the safety of the whthe playing which is known as dowla or dowka. Practically it acts as a curb on the canal side of the road. It is provided above the F.S.L. with a provision of freeboard. The top width is generally 0.5m and the height above the

level is about 0.5 m. The side slope is similar to the side slope of the bank.

Spoil Bank :-

When the canal is constructed in huge cutting, excavated earth may not be completely required for forming the bank. In such a case, the earth is deposited in the form of small banks which are known as spoil banks. The spoil banks are provided on one side or both sides of the canal bank depending on the quality of earth and the available space. The spoil banks run parallel to the main bank. But are not continuous. Sufficient spaces are left between the adjacent spoil banks for proper drainage.

Borrowpit :-

When the canal is constructed in partial cutting and partial banking, the excavated earth may not be sufficient for forming the required bank. In such a case, the extra earth required for the construction of banks is taken from some pits which are known as borrowpits. The borrowpits may be inside or outside the canal.

The inside borrowpits may be located at the centre of the canal. The width of the borrowpit should be half of the base width of canal. The minimum depth should be 1 m. The excavation is done in a number of borrowpits leaving a gap between them. The gap is generally half of the length of each borrowpit. The idea behind this is that the borrowpit will act as water pockets where the silt will be deposited and ultimately the canal bed will get levelled up.

The outer borrowpits may be adjacent to the heel of the bank with a clearance of 5 m between the heel and edge of borrowpit. But the outer borrowpit may create some inconvenience. So it is better to take the earth from the borrow lands as far away from the canal.

(Borrowpit)
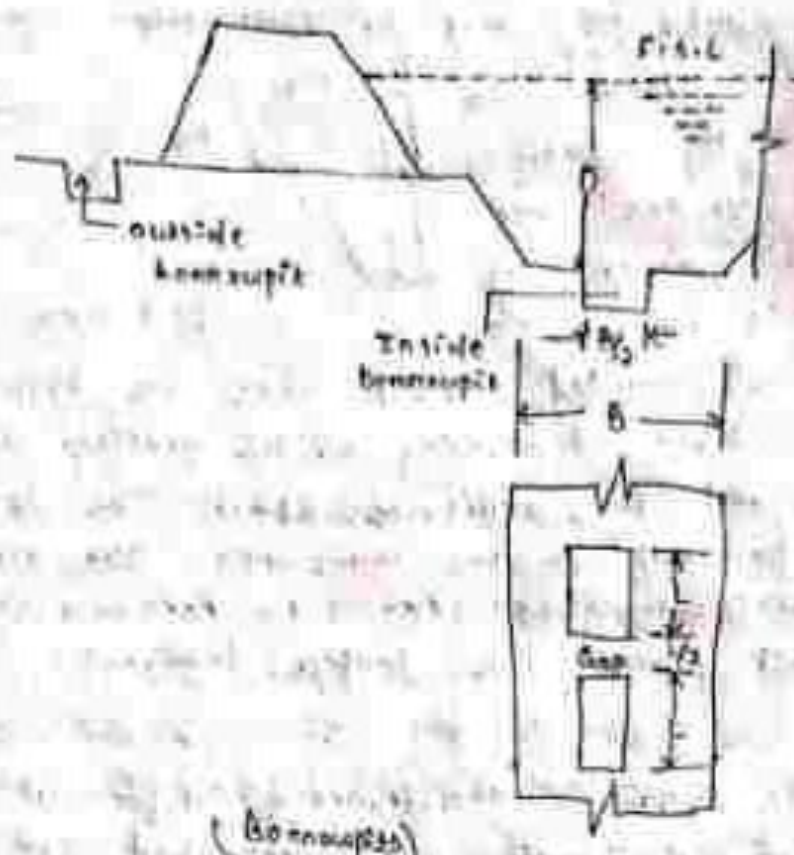
Land Width :—

The total land width required for the construction of a canal depends on the nature of the site condition, such as fully in cutting or fully in banking or partly in cutting and partly in banking. These conditions arise according as in designed bed level of the canal and the natural ground surface. So, total land width differs with the site condition. However, to determine the total land width, the following dimensions should be added.

1. Top width of the canal.
2. Twice the berm width.
3. Twice the bottom width of banks.
4. A margin of one metre from the heel of the bank on both sides.
5. Width of external borrowpit if any.
6. A margin of 0.5m from the outer edge of borrowpit on both sides, if external borrowpit becomes necessary.

Types of lining :—

The following are the different types of lining which are generally recommended according to the various site conditions:

1. Cement concrete lining
2. Pre-cast concrete lining
3. Cement mortar lining
4. Lime concrete lining
5. Brick lining
6. Boulder lining
7. Shot crete linings
8. Asphalt lining
9. Bentonite and clay lining
10. Soil - cement lining

Cement concrete lining :—

This lining is recommended for the canal in full banking. The cement concrete lining is widely accepted as the best impervious lining. It can reduce the effect of scouring and ensure very efficiency. The velocity of flow may be kept above 2.5 m/sec. It can eliminate completely growth of weeds. The lining is done by the following steps:

(a) Preparation of Sub-grade :—

The sub-grade is prepared by ramming the surface properly with a layer of sand. Then a slurry of cement and sand (1:3) is spread uniformly over the prepared bed.

(b) Laying of concrete :—

The cement concrete of grade M15 is spread uniformly according to the determined thickness (generally the thickness varies from 100 to 150 mm). After laying the concrete is tapped gently until the slurry comes on the top. The curing is done for two weeks. As the concrete is liable to get damaged by the change of temperature, the expansion joints are provided at appropriate place. Normally no reinforcement is required for this cement concrete but in special cases, a network of 6 mm diameter mild steel may be provided with spacing from centre to centre.
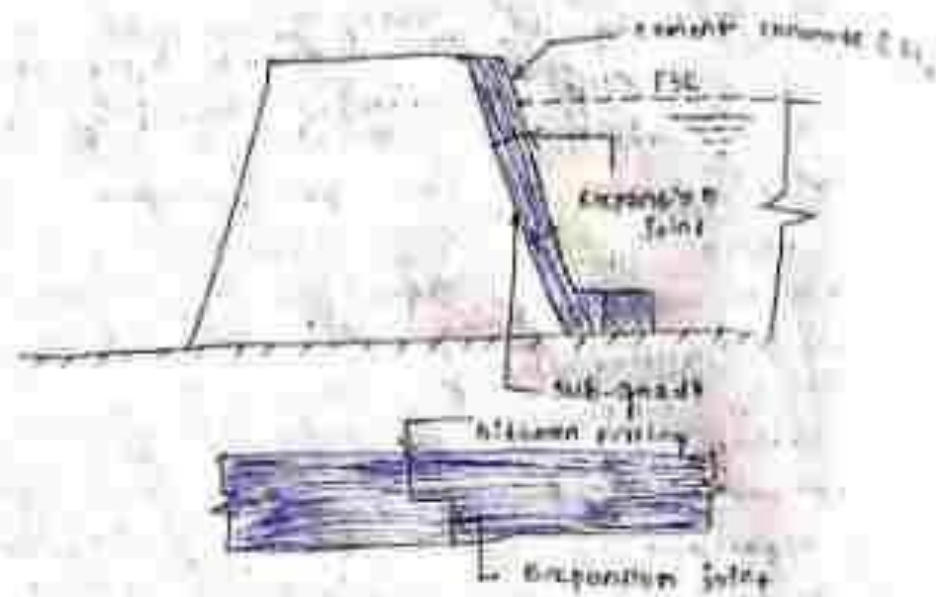
(Cement concrete lining)

Pre - cast concrete linings :-

This type is recommended for the canal full lining. It consists of pre-cast concrete slabs of size 600mm x 40cm x 5cm which are set along the canal bank and laid with cement mortar (1:3). A minimum of 6mm diameter nail is provided in the slab with spacing 10cm apart of center. The proportion of the concrete is recommended as 1:2:4. Rebates are provided on all the four sides of the slab so that proper joints may be obtained when they are placed side by side. The joints are finished with cement mortar (1:3). Expansion joints are provided at a suitable interval. The slabs are set in the following figure.

(a) The sub-grade is prepared by properly ramming the soil with a layer of sand. The bed is levelled so that the slabs can be placed easily.

(b) The slabs are stacked at approximate along the course of the canal. The slabs are placed with cement mortar (1:3) by seating them carefully properly. The joints are finished with cement mortar (1:3).
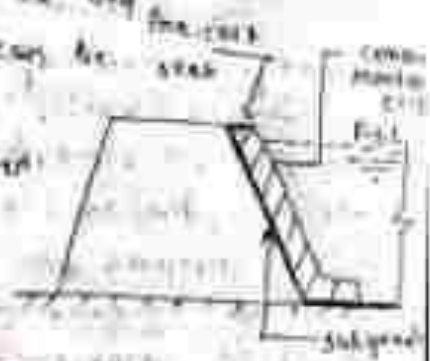
(c) The curing is done for a week.

Cement Mortar Lining :—

This type of lining is recommended for the canal laying in cutting where hard soil or rocky soil is available. This thickness of the cement mortar lining is generally 2 cm. The sub-grade is prepared by ramming the soil after cutting. Then over the compacted sub-grade, the cement mortar is laid uniformly and the surface is finished with neat cement paste. This lining is impervious but is not durable. The curing should be done properly.

cement mortar
lining

soil

Brick Concrete Lining :—

The lining is prepared by the double layer brick lining laid with cement mortar (1:6) over the compacted sub-grade. The first class bricks should be recommended for the work. The surface of the lining is finished with cement plaster. The curing should be done perfectly.
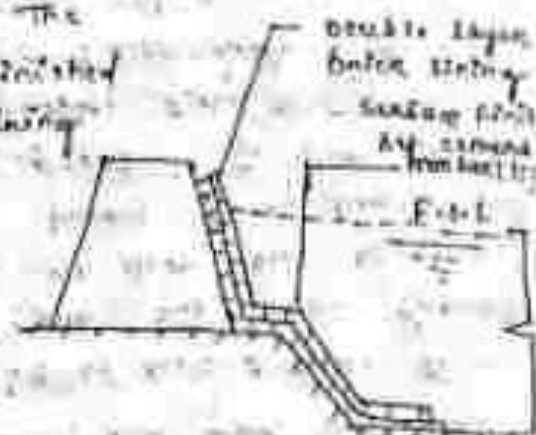
Brick lining is always preferred for the following reasons :

a) This lining is economical.

b) Work can be done very rapidly.

c) Expansion joints are not required.

d) Repair work can be done easily.

e) Bricks can be manufactured from the excavated earth near the site. However brick lining has certain disadvantages.

a) It is not completely impervious.

b) It has low resistance against erosion.

c) It is not so much durable.

double layer
brick lining
surface finished
by cement
plastering

soil

(Brick lining)

Boulder Lining :—

In hilly areas where the boulders are available in plenty, this type of lining is generally recommended. The boulders are used for lining on the slopes, maintaining the slope as the banks and the bed level of the canal. The joints of the boulders are packed with cement mortar (1:4). The surface is finished with cement mortar (1:4).

Curing is necessary in this lining. This lining is very durable and impervious. But the cost of the material is very high. So it cannot be recommended for all cases.

Shot crete lining :—

In this system, the cement mortar (1:4) is directly applied on the sub-grade by an equipment known as cement gun. The mortar is termed as shot crete and the lining is known as shot crete lining. The process is also known as guniting, as a gun is used for laying the mortar. Sometimes, this lining is known as gunited lining. The lining is done in two ways.

(a) **By dry mix :—**

In this method, a mixture of cement and moist sand is prepared and loaded to the cement gun. Then it is forced through the nozzle of the gun with the help of compressed air. The mortar spreads over the sub-grade to a thickness which varies from 2.5 cm to 5 cm.

(b) **By wet mix :—**

In this process, the mixture of cement, sand and water is prepared according to the approved consistency. The mixture is loaded in the gun and forced on the sub-grade.

This type of lining is very costly and it is not durable. It is suitable for resurfacing and old lining concrete lining.

Asphalt lining :—

This lining is prepared by spraying (i.e. bitumen) at a very high temperature (about 150°C) on the subgrade to a thickness ranges from 3 mm to 6 mm. The hot asphalt when becomes cool forms a loosen prime membrane over the subgrade. This membrane is covered with a layer of earth and gravel. The lining is very cheap and can control the seepage of water very effectively but it cannot control the growth of weeds.

Bentonite and clay lining :—

In this lining a mixture of bentonite and clay are mixed thoroughly to form a sticky mud. This mud is spread over the sub-grade to form an impervious membrane which is effective in controlling the seepage of water, but it cannot control the growth of weeds. This lining is generally recommended for small channels.

Soil-cement lining :—

This line is prepared with a mixture of soil and cement. The usual quantity of cement is 10% of the weight of dry soil. The soil and cement are thoroughly mixed to get an uniform mixture. The mixture is laid on the subgrade and it is made thoroughly compact. The lining is effective to control the seepage of water, but it cannot control the growth of weeds. So, this is recommended for small channels only.

Advantages and Disadvantages of canal lining :—

Advantages :—

1. It reduces the loss of water due to seepage and hence the duty is enhanced.
2. It controls the water logging and hence the bad effects of water-logging are eliminated.
3. It provides smooth surface and hence the velocity of flow can be increased.
4. Due to the increased velocity the discharge capacity of a canal is also increased.
5. Due to the increased velocity, the evaporation loss

also be reduced.

6. It eliminates the effect of scouring in the canal bed.

7. The increased velocity eliminates the possibility of silting in the canal bed.

8. It controls the growth of weeds along the canal sides and bed.

9. It provides the stable section of the canal.

10. It reduces the requirement of land width of the canal, because smaller section of the canal can produce greater discharge.

11. It prevents the sub-soil salt to come in contact with the canal water.

12. It reduces the maintenance cost for the canal.

Disadvantages :—

1. The initial cost of the canal linings is very high. So, it makes the project very expensive with respect to the output.

2. It involves much difficulties for repairing the damaged section of lining.

3. It takes too much time to complete the project work.

4. It becomes difficult, if the outlets are required to be shifted or new outlets are required to be provided, because the dismantling of the lined section is difficult.

— : DAM : —

Introduction :—

An impervious high barrier which is constructed across a river valley to form a deep storage reservoir is known as dam. It is suitable in hilly region where a deep gorge section is available for the storage reservoir. The dam is meant for serving multipurpose functions such as storage.

(b) Hydroelectric power generation ;

(c) Flood control ;

(d) Water supply ;

(e) Fishery ;

(f) Recreation.

Based on materials of construction :—

1. Rigid dam :—

It is constructed with rigid materials like masonry, concrete, steel or timber. It is designated as, (a) masonry dam ; (b) concrete dam, (c) steel dam ; (d) timber dam.

2. Non-rigid dam :—

It is constructed with non-rigid materials such as earth, clay, rock materials, etc. It is designated as, (a) earthen dam ; (b) rock-fill dam, (c) composite dam.

Based on structural behaviour :—

1. Solid gravity dam :—

It is constructed with masonry or concrete. It resists the forces acting on it by its own weight. It is approximately triangular in section.

2. Arch dam :—

It is a curved masonry or concrete dam which resists the forces acting on it by the principle of arch action.

3. Buttress dam :—

It behaves like a retaining wall. It consists of sloping deck on the upstream side which is supported by a number of buttress in the form of reinforced concrete wall or counterfort

4. **Embankment dam :—**

It is non-rigid dam constructed simply by earth work in trapezoidal section. Sometimes it may be of earth work with clay core or rock fill. It resists forces acting on it by its shear strength.

C. Based on Functions :—

1. Storage dam :—

It is constructed to form a reservoir in which the water is closed during the opening of rainy season or flood and utilised for irrigation in the period of draught. The water is also utilised for the generation of hydro power, water supply, etc.

2. Detention dam :—

It is mainly constructed to detain the flood water temporarily in a reservoir and then released gradually so that the downstream area may not be damaged due to sudden flood water.

3. Diversion dam :—

It is constructed to divert the water from a perennial river to a channel for the purpose of irrigation or to a conduit for the purpose of generation of hydroelectric power.

4. Coffer dam :—

When an area in the river bed is enclosed temporarily by sheet pilings for excluding water for the sake of construction of well foundation (i.e. pier foundation) then it is known as coffer dam.

D. Based on Hydraulic Behaviour :—

1. Over Flow dam :—

The dam which consists of crest or shuttery of waste weirs on the top to allow the surplus water to overflow is known as overflow dam.

2. Non overflow dam :-

The dam in which spill ways are provided to discharge the surplus water is not allowed to flow over the crest, is known as non-overflow dam.

Solid gravity Dam :-

The solid gravity dam may be constructed with rubble masonry or concrete. The rubble masonry is done according to the shape of the dam with rich cement mortar. The upstream and downstream faces are finished with rich cement mortar. The upstream and downstream faces are provided with rich cement mortar.

Nowadays, concrete gravity dams are preferred, because they can be easily constructed by laying concrete layer by layer with construction joints.
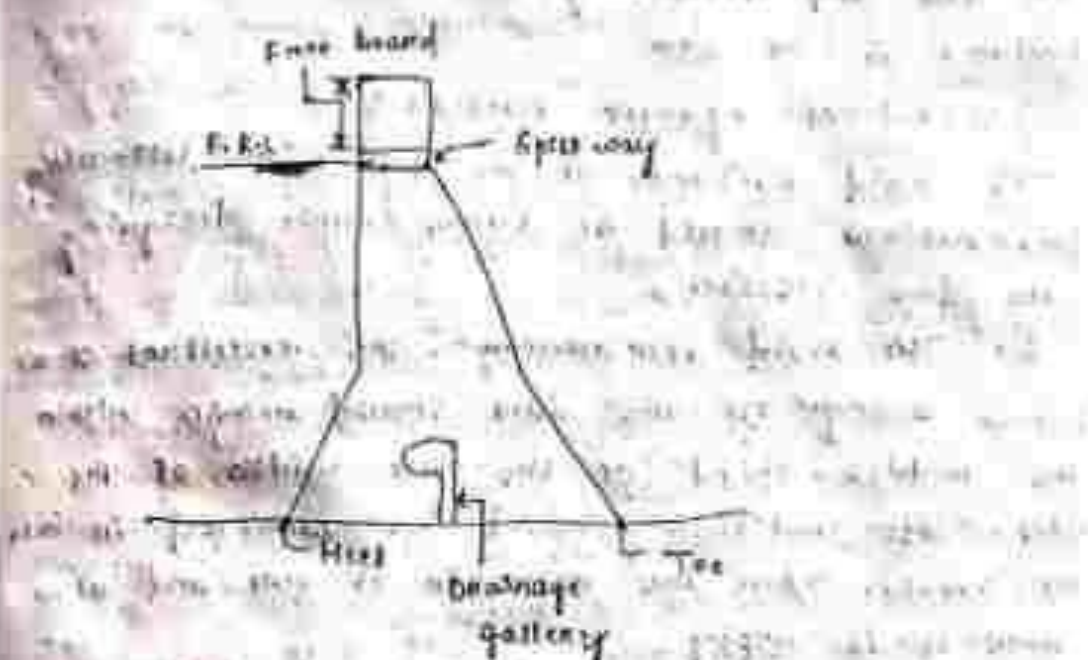
(Solid gravity dam)

Causes of failure of gravity dam :-

1. By over turning :-

The solid gravity dam may fail by over turning at its toe, when the total horizontal forces acting on the dam are greater than the total vertical force. In such a case, the resultant force passes through a point outside the middle third of the base of the dam.

2. By sliding :—

The total horizontal forces acting on a dam tend to slide the entire dam at the base on any horizontal section of the dam.

3. By over stressing :—

If the permissible working compressive stress of concrete or masonry exceeds due to some conditions, then the dam may fail by crushing due to overstressing of the concrete or masonry.

4. By cracking :—

The tensile stresses should not be allowed to develop on the upstream faces of the dam. If due to some reasons, the tension is developed in the dam section, crack will form in the body of the dam and ultimately this will cause the failure of the dam.

Precautions against Failure :—

To avoid failure of the dam, the following precautions should be taken while designing the dam section ,

1. To avoid overturning, the resultant of all forces acting on the dam should remain within the middle - third of the base width of the dam. This condition should be achieved in both the cases, when the reservoir is full and also when it is empty.

2. In this dam, the sliding should be fully resisted when the condition for no sliding exists in the dam section.

The condition for no sliding is given by

$$\tan \theta = \frac{\Sigma P}{\Sigma W}$$

and $\tan \theta < \mu$

where, ΣP = sum of horizontal forces, ΣW = sum of vertical forces, μ = co-efficient of

position of the materials of dam.

3. In the dam section, the compressive stresses of concrete or masonry should not exceed the permissible working stresses to avoid failure due to crushing.

4. There should be no tension in the dam section to avoid the formation of cracks. This condition may be achieved by maintaining the middle - third rule.

5. The factor of safety should be taken as 1.5.

Earthin dam :—

Earthin dams are constructed purely by earth cover in trapezoidal section. These are most economical and suitable for weak foundation. Earthin dams are classified as follows :

Based on method of construction :—

Rolled fill Dam :—

In this method, the dam is constructed in successive layers of earth by mechanical compaction. The selected soil is transported from borrowpits and laid on the dam section, to layers of about 15 cm. The layers are thoroughly compacted by rollers of recommended weight and type.

Hydraulic fill dam :—

In this method, the dam section is constructed with the help of water. Sufficient water is poured in the borrowpit and by pumping thoroughly, slurry is formed. This slurry is transported to the dam site by pipe line and discharged mean the upstream and downstream faces of the dam. The coarser material get deposited mean the face and the fine material move towards the centre and get deposited there.

Semi - hydraulic fill dam :—

In this method, the selected earth is transported from the borrowpit and dumped within the section of the dam, as done in the case of rolled fill dam. While dumping no water is used.

Homogeneous type Dam :—

This type of dam is constructed purely of earth in trapezoidal section having the side slopes according to the angle of repose of the soil. The top width and height depends on the depth of water to be retained and the position of the seepage line.

Zoned type Dam :—

This type of dam consists of several materials, the impervious core is made of puddle clay and the other previous shell is constructed with the mixture of earth, sand, gravel, etc. The core is trapezoidal in section and its width depends on the seepage characteristics of the soil mixture on the upstream side.

Homogeneous type dam

Zoned type dam

Diaphragm type dam :—

In this type of dam, a thin impervious core of diaphragm is provided which may consist of puddle clay or cement concrete or bituminous concrete. The upstream and downstream body of the dam is constructed with pervious clays which consists of the mixture of soil, sand, gravel, etc. The thickness of the core is generally less than 3m.
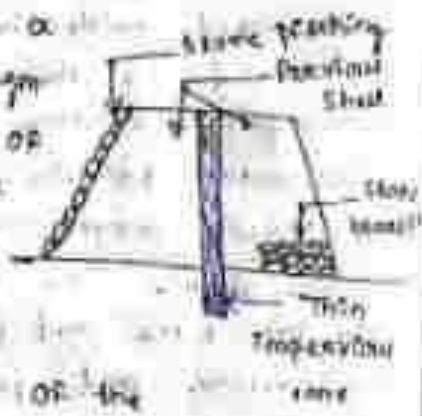
Causes of failure of earthen dam :—

(1) **Hydraulic failure :—**

This type of failure caused due:

(aa) Overtopping :—

If the actual flood discharge is much more than the estimated flood discharge on the basis of which free board is kept insufficient, then this results in the overtopping of the dam.

(b) Erosion :—

If the stone protection of the upstream side is insufficient, then the upstream face may be damaged by erosion due to wave action.

(2) Seepage Failure :—

This type of failure may be caused by:

(a) Piping or undermining :—

Due to the continuous seepage flow through the body of the dam and through the sub-soil below the dam, the downstream side gets eroded or washed out and a hollow pipe like groove is formed which extends gradually towards the upstream through the base of the dam.

(b) Sloughing :—

The crumbling of the toe of the dam is known as sloughing.

(3) Structural Failure :—

This type of failure may be caused by

(a) Sliding of the side slopes :—

Sometimes, it is found that the side slope of the dam slides down to form some steeper slope.

(b) Damage by burrowing animals :—

Some burrowing animals like crawfish, snakes, squirrels, rats etc could damage to the dam by digging holes through the foundation and body of the dam.

(c) Damage by earthquake :—

Due to earthquake cracks may develop on the body of the dam and the dam may eventually collapse.

Spill ways :—

Necessity of spill ways :—

The Spill ways are openings provided at the body of the dam to discharge safely the excess water flood water level rises above the normal pool level.

The Spill ways are provided on the dam for the following reasons.

(a) The height of the dam is always fixed according to the maximum reservoir capacity. The normal pool level indicates the maximum capacity of the reservoir.

(b) The top of the dam is generally utilised by making road. The surplus water in not to be allowed to over top the dam, so to stop the over over topping by the surplus water, the spill ways become extremely essential.

(c) To protect the downstream base and floor of the dam from the effect of souring and erosion, the spill ways are provided so that the excess water flows smoothly.

Types of Spill ways

The following are the common types of spill ways.

Drop spill way

In drop spill way, the over flowing water falls freely and almost vertically on the downstream side of the hydraulic structure. This type of spill way is suitable for weirs or low dams. The crest of the spill way is provided with nose so that the water jet may not strike the downstream base of the structure. To protect the structure from the effect of sourcing horizontal impervious apron should be provided on the downstream side.

Sometimes a basin is constructed on the downstream side to form a small artificial pool, which is known as water Cushion. This Cushion serves the purpose of energy dissipation.
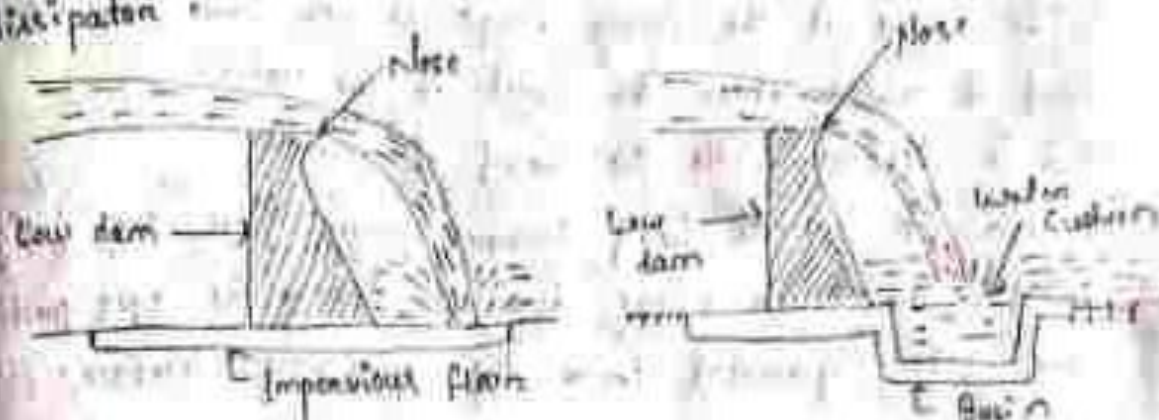
(a) Drop Spillway with impervious apron
(b) Drop Spillway with water Cushion

The drop spill way is not suitable for a high dam, because the downstream apron will be subjected to high impact force for which massive protection works will be necessary.

→ Again, high impact on the downstream apron may cause vibration in the structure which may create cracks in the foundation.

→ Thus, the stability of the structure will be in danger due to undermining.

Ogee Spillway :-

→ The ogee spillway is a modified form of drop spill way.

→ Here, the down stream profile of the spill way is made to coincide with the shape of the lower nappe of the free falling water jet from a sharp crested weir.

(Ogee Spillway)
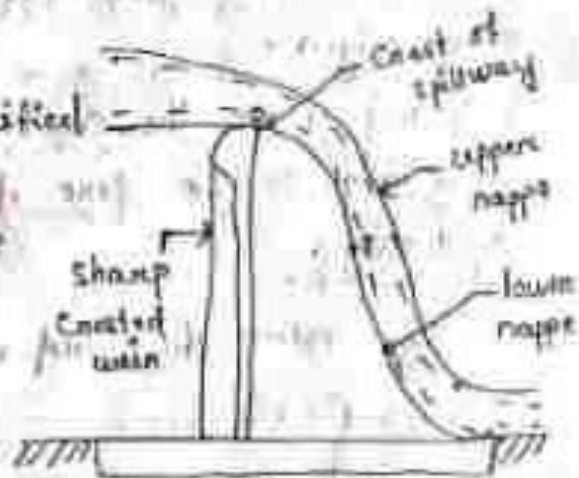
→ In this case, the shape of the lower nappe is similar to a projectile and hence downstream surface of the ogee spill way will follow the parabolic path where 'o' is the origin of the parabola.

→ The downstream face of the spill way forms a concave curve from a point "T" and meets with the downstream

from:
→ This point 'T' is known as point of tangency. Thus the spill way takes the shape of the letter 's' (i.e. elongated from). Hence, this spill way is termed as ogee Spill way.

→ The shape of the lower nappe is not same for all the head of water above the Crest of the weir.

→ It is differ with the head of water.

→ When the actual head becomes more than the designed head, the lower nappe does not follow the ogee profile and gets Separated from the spill way Surface.

The shape of the ogee Spill way has been developed by U.S. Army Corps Engineers which is known as Water way experimental station Spill way Shape.

→ The equation given by them is:

$$x^n = K \cdot H^{n-1} \cdot y$$

where, x and y are the Coordinates of a point P on the ogee profile taking O as origin.

→ K and n are the Constants according to the slope of the upstream face of the spill way.

$n_1 = 0.5 H$ $a = 0.1 7 5 H$

$n_2 = 0.21 H$ $b = 0.237$

(Shape of ogee Spill way)

The values of K and n are given as follows:-

Shape of U/s face of spill way	K	n
vertical	2.0	1.85
1:3 (H:Y)	1.936	1.836
1:1/2 (H:Y)	1.939	1.810
1:1 (H:Y)	1.873	1.776

Siphon Spill way :-

The spill way which acts on the principle of Siphon is known as Siphon Spillway.

→ The Siphon Spill way may be of 2 types :

(a) Saddle Siphon spill way :-

→ It consists of a reinforced concrete hollow pipe in the shape of an inverted 'U'.

→ The upper limb is short and consists of a bell mouth inlet.

→ The lower limb is longer and consists of a bell mouth exit.

→ The inlet mouth is kept submerged below the Full Supply level (F.S.L) to control the entry of floating debris into the siphon duct which may disturb the siphonic action.

→ The exit mouth is also kept submerged below the water level of the stilling basin.

The functioning of the spillway is described as follows :-

1. When the water is just on the full reservoir level (i.e. on the crest of the spill way), there is no flow of water.

2. As the water starts rising above the F.R.L, the flow of water also starts over the crest.

3. When the inlet of the depressor gets submerged, the entry of air to the siphon duct through the air vent is stopped.

4. The entrapped air in the top portion of the siphon duct is then sucked by the flowing water. Thus the inside pressure is dropped below the outside atmospheric Pressure. Due to this pressure difference a suction pressure is created which draws more and more water over the
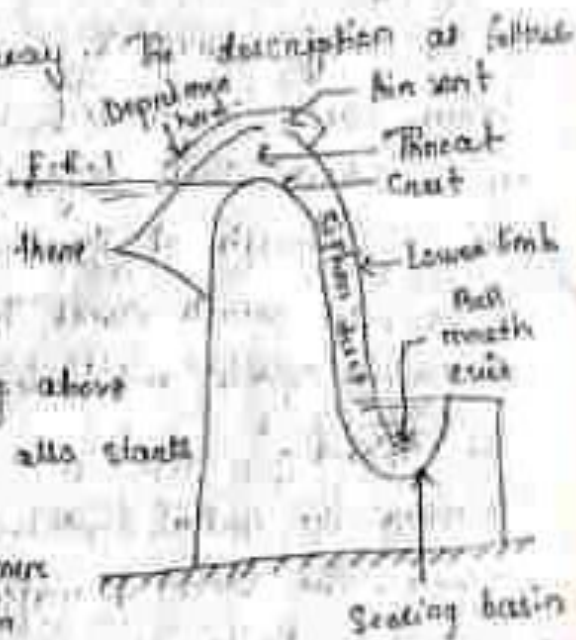
(Saddle Siphon Spill way)

Crest of the Siphon. This suction press goes on increasing gradually.

5. Due to the gradual increase of suction press, a time comes when the Siphonic action starts and the Siphon duct goes on running full. This phenomenon is known as priming. until the entrapped air is forced out completely through the Sealing basin, the condition of priming does not arise. However, this condition arises when the water level rises to a considerable height but it takes too much time. So, to expedite the priming of Siphon at times some priming devices are used.

Priming Devices :-

These are several priming devices, but the priming by baby Siphon is generally adopted.

→ The baby Siphon is a small Siphon which is located below the main Siphon but slightly above the inlet mouth of the main Siphon.
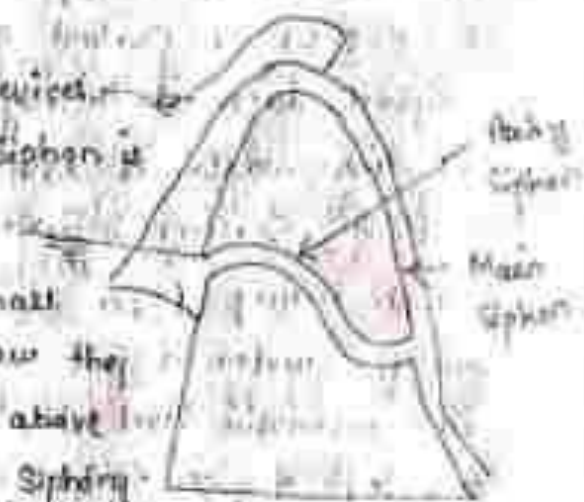
→ The inlet mouth covers the passage (Baby Siphon) of baby Siphon and hence the floating debris cannot enter the Siphon.

→ When the water level rises above the crest of the baby Siphon, the sheet of water starts flowing and the sheet of water starts flowing and the lower limb of the main Siphon is sealed.

→ Thus, when the water rises above the full reservoir level, the main siphon gets primed very quickly.

(b) Volute Siphon spill way :—

It consists of a vertical shaft having a funnel at the top and at the bottom end is connected to a bend

pipe. The bend pipe again is connected to a horizontal pipe which carries the flowing water away from the base of the dam.

→ The top level of the funnel is a kept just at the full reservoir level.

→ The funnel consists of several volutes (curved vane or blades).

→ Thus the water has a spiral motion while passing through the funnel.

→ A circular drum is placed over the funnel. The drum is supported on pillars.
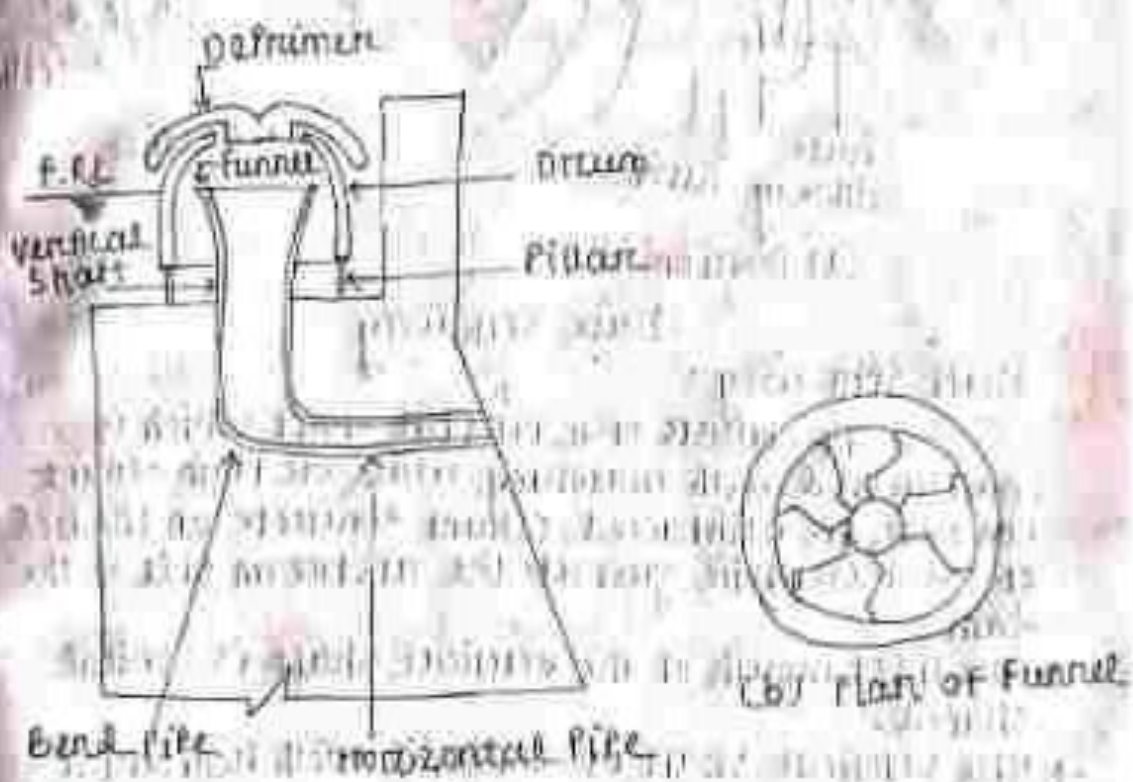
(a) Section of siphon

(b) Plan of funnel

Volute siphon spillway

Chute or Trough Spill way :-

This spill way is simply a rectangular open channel or trough (known as chute) provided on the dam to discharge the surplus water from the reservoir to the same river on the downstream side.

→ The spill way may be provided along the abutment of the dam or along the edge of the reservoir at the full supply level.

→ The chute is constructed by joining pre-cast R.C.C.

→ channels in a longitudinal slope of 1 in 4 or 1 in 6
→ The channels are supported on pillars
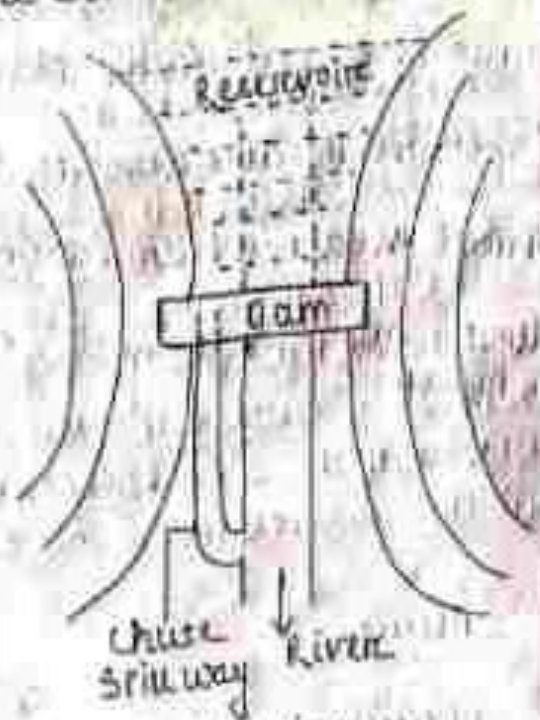
(a) Position

(b) Section

Chute spillway

Shaft Spillway :-

It consists of a vertical shaft which is constructed with masonry work or plain cement concrete or reinforced cement concrete on the bed of the reservoir just at the upstream side of the dam.

→ The inlet mouth of the vertical shaft is conical shaped.

→ The vertical shaft is connected with horizontal shaft.

→ The horizontal shaft again may be taken through the body of the dam (in case of gravity dam) or through the base of the dam (in case of earthen dam) or, may be connected to a tunnel outside the dam.

→ The inlet mouth is kept at the normal pool level of the reservoir.

(b) Section of shaft

(a) Position of shaft
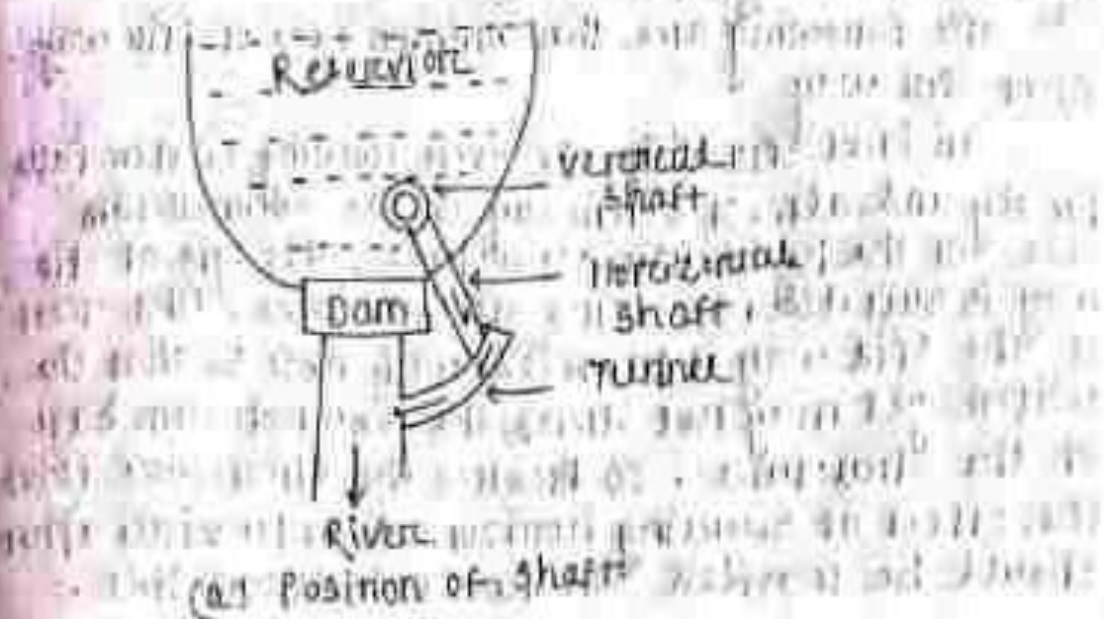
Side channel spill way :-
 The side channel spill way is completely separate from the main body of the dam.

→ The spill way is constructed at right angle to the dam and at any side according to the site condition

→ The crest of the spill way is kept at the normal pool level of the reservoir. when the water rises above the N.P.L. it spill over the crest of the spill way and flows through the side channel and ultimately meets the same river on the downstream side.

⇒ This type of spill way is recommended for the sites where other types of spill ways are found unsuitable.

→ The side walls of the channel may be constructed with brick masonry are stone masonry.

side channel spill way
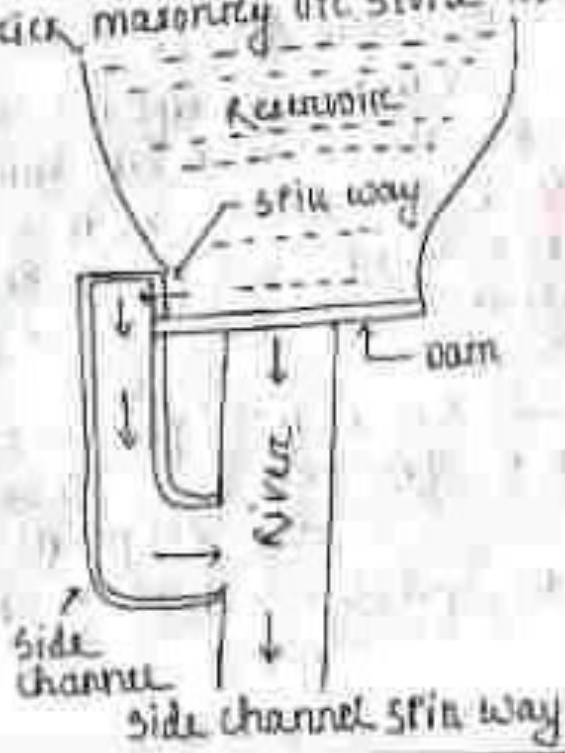
GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING
GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

Highway Engineering

For 4TH Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)

Prepared By
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UNIT 1 - INTRODUCTION

Importance of Highway transportation:

Highway transportation is the means of detail distribution between homes, shops, factories, etc. It is only the roads which can carry goods from and to aerodromes, harbours and railway stations. Considering the utility of roads anywhere in the different parts of a country, they can be rightly compared to arteries in a human body just as arteries maintain man's health by providing circulation of blood; similarly roads promote nation's wealth by keeping its people and goods moving. Thus, we see that progress and well-being of a nation depends much on roads. In fact, roads are the life lines of nation's economy.

The importance or necessity of highway transportation can be easily judged from the following purposes or advantages of roads:

They facilitate conveyance of people, goods, raw-materials, manufactured articles, etc. speedily and easily in the different parts of a country.

1. They act as the only source of communication in regions of high altitude *i.e* in mountainous regions.

2. They help in growth of trade and other economy activities in and outside the villages and towns by establishing contact between towns and villages.

3. They help in providing efficient distribution of agricultural products and natural resources all over the country.

4. They help in price stabilization of commodities due to mobility of products all over the country.

5. They help in social and cultural advancement of people and making the villagers active and alert members of the community.

6. They help in promoting the cultural and social ties among people living in different part of a country and thus strengthen the national unity.

7. They help in providing improved medical facilities quickly to human beings, especially to those who live in rural areas.

8. They provide more employment opportunities.

9. They enhance land value and thus bring better revenue.

10. They serve as feeders for Airways, Waterways and Railways.

11. They help in reducing distress among the people, caused due to famine, by supplying them food and clothing quickly.

Lastly, it can be sad that roads are the symbol of country's progress and thus development made by any country can be judged by the quality and network of it's road system.

Indian roads congress:

Indian Roads Congress (IRC) was set up by the Government of India in December, 1934 on the recommendations of Jayakar Committee with the objective of promoting and encouraging the science for building and maintenance of roads. It also provides a national forum for sharing of knowledge and pooling of experience on the entire range of subjects dealing with the construction and maintenance of roads and bridges. IRC has now about13,500 members comprising of engineers of all ranks from Central and State Governments, Engineering Services of Army, Border Roads Organization, Road Research Institutes, Engineering Colleges, Local Bodies, and private enterprises.

Functions of Indian Roads Congress (IRC):

IRC a body of professional highway engineers provides the following services:

I. It provides a forum for expression of collective opinion of its members for all matters affecting the construction and maintenance of roads in India.
II. It promotes the use of the standard specifications and practices.
III. It provided with the suggestions for the better methods of planning, designing, construction, administration and maintenance of roads.
IV. It conducts periodical meetings to discuss technical problems regarding roads.
V. It makes the laws for the development, improvement and protection of the roads.
VI. It furnishes and maintains libraries and museums for encouraging the science of road making.

Functions of Central Road Research Institute (CRRI):

CRRI was started by the Central Government in 1950, for the research work in the highway engineering. CRRI is a series of laboratories under the council of scientific and industrial research in India. It offers the following services:

1. Carries basic and applied research for the design, construction and maintenance of the highways.
2. Carries research on traffic safety and transport economics.
3. Carries research on economical utilization of locally available materials for construction and maintenance of roads.
4. Research for the development of the new machinery, tools equipment and instruments for highway engineering.
5. To provide technical advice and consultancy services to various organizations.
6. To provide library and documentation services.

Ministry of Surface Transport:

The **Ministry of Road Transport and Highways** is a ministry of the Government of India, that is the apex body for formulation and administration of the rules,

regulations and laws relating to road transport, transport research and also to increase the mobility and efficiency of the road transport system in India. Through its officers of Central Engineering Services (Roads) cadre it is responsible for the development of National Highways of the country. Road transport is a critical infrastructure for economic development of the country. It influences the pace, structure and pattern of development. In India, roads are used to transport over 60 percent of the total goods and 85 percent of the passenger traffic. Hence, development of this sector is of paramount importance for India and accounts for a significant part in the budget.

Roads wing of ministry of surface transport:

The roads wing of the ministry of Surface Transport handles the road mattersof the Central Govt. It is headed by a Director General.

The Director General is assisted by two additional Director Generals(one for roads and one for bridges), a numbers of Chief Engineers, Superintending Engineers, Executive Engineers and Asst. Executive Engineers. The roads winghas a chief Engineer for the North-East region posted at Guwahati and a Liaison-cum-Inspectorate organization consisting of S.E's and E.E's in the various states. The functions of the roads wing of Surface Transport are:

a. To control funds approved by Central Government for the developmentof National Highways.
b. To control the central road fund.
c. To prepare plans for development and maintenance of National Highwaysin consultation with state PWD's.
d. To oversee technically the quality of works executed by the agencies.
e. To administer matters regarding road research.
f. To examine technically the projects of roads and bridges prepared by the PWD's.
g. To administer the central road program other than National Highways in the Union Territories.

IRC classification of roads :

IRC(Indian Roads Congress) has classified the roads in the India in the following 5 categories:

(a) National Highways
(b) State Highways
(c) Major District Roads
(d) Other District Roads
(e) Village Roads

National Highways(NH): National highways are the major arterial roads spanning in the length and breadth of the country and connects the Capital to the various state capitals of the country or with the neighboring countries.

They also connect the famous tourism places of the country. National highways are numbered and written as NH-1, NH-2 etc. They have the highest design specifications.

Example : NH -1 Delhi-Ambala-Amritsar, NH-21 Chandigarh- Mandi- Manali.

State Highways(SH): State highways are the roads which connect the state capital to other states and to the district headquarters in the state. They have design specifications similar to those of the National Highways because they carry enough traffic.

Major District Roads(MDR): These roads connect the district headquarters to the main town centres in the district, and to the headquarters of the other districts also. They also connect these major town centres to the other state highways of importance. They have lower design specifications as compared to the NH and SH.

Other district roads(ODR): These roads connect the rural areas town centres to the major district roads of higher importance. They provide the facilities for the transportation of the raw materials or the goods mainly of agriculturalproducts from the rural towns to the higher markets and vice-versa.

Village Roads (VR): These roads connect the rural villages with one another and to the nearest higher level road or to the nearest town centre. They have lower design specifications and many of them are not even metalled.

Organisation of state highway department

Responsibility for new construction and maintenance works on the National Highways is under the control of the Chief Engineer National Highways (CE (NH)). The CE (NH) reports to MOST for works carried out on the National Highway network.

This wing has been set up in keeping with the requirements of MOST to:

➢ Reduce the line of communication between the GOI and State Authorities

➢ achieve efficiencies in implementation by avoiding the cumbersome and outdated delegations for administrative and technical sanctionwhich limit the ability of the Odisha Works Department to respond quickly

➢ Achieve uniform maintenance and construction standards on NHs.

At present, 16 Nos. of National Highways measuring 3592.932 km in length traversed through the state of Odisha. Out of 3592.932 km of total length of National Highways in Odisha, 3071.722 km is under the control of NH wing of State PWD, and remaining 521.210 km have been transferred to National Highways Authority of India for improvement under NHDP and Port connectivity.

UNIT 2- ROAD GEOMETRICS

Glossary of terms used in geometric and their importance:

Right of way

Right of way (ROW) or land width is the width of land acquired for the road, along its alignment. It should be adequate to accommodate all the cross- sectional elements of the highway and may reasonably provide for future development. To prevent ribbon development along highways, control lines and building lines may be provided. Control line is a line which represents the nearest limits of future uncontrolled building activity in relation to a road. Building line represents a line on either side of the road, between which and the road no building activity is permitted at all. The right of way width is governed by:

1. Width of formation: It depends on the category of the highway and widthof roadway and road margins.
2. Height of embankment or depth of cutting: It is governed by the topography and the vertical alignment.
3. Side slopes of embankment or cutting: It depends on the height of the slope, soil type etc.
4. Drainage system and their size which depends on rainfall, topography etc.
5. Sight distance considerations: On curves etc. there is restriction to the visibility on the inner side of the curve due to the presence of some obstructions like building structures etc.
6. Reserve land for future widening: Some land has to be acquired in advance anticipating future developments like widening of the road.

Table : Normal right of way for open area

Road classification	Roadway width in m	
	Plain and rolling terrain	Mountainous and steep terrain
Open areas		
NH/SH	45	24
MDR	25	18
ODR	15	15
VR	12	9
Built-up areas		
NH/SH	30	20
MDR	20	15
ODR	15	12
VR	10	9

S– shoulder
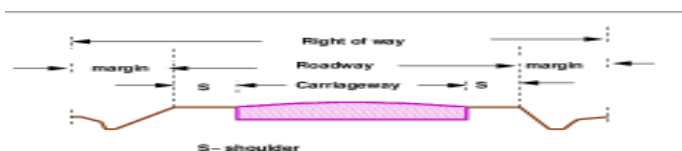
- Width of formation
- Height of embankment
- Side slopes
- Drainage system
- Sight distances consideration on horizontal curves
- Future extension

Formation width

Width of formation or roadway width is the sum of the widths of pavements or carriage way including separators and shoulders. This does not include the extra land in formation/cutting. The values suggested by IRC are given in Table:

Road classification	Roadway width in m	
	Plain and rolling terrain	Mountainous and steep terrain
NH/SH	12	6.25-8.8
MDR	9	4.75
ODR	7.5-9.0	4.75
VR	7.5	4.0

Table : Width of formation for various classed of roads

Road margin

The portion of the road beyond the carriageway and on the roadway can be generally called road margin. Various elements that form the road margins are given below.

1. Shoulders

Shoulders are provided along the road edge and is intended for accommodation of stopped vehicles, serve as an emergency lane for vehiclesand provide lateral support for base and surface courses. The shoulder should bestrong enough to bear the weight of a fully loaded truck even in wet conditions. The shoulder width should be adequate for giving working space around a stopped vehicle. It is desirable to have a width of 4.6 m for the shoulders. A minimum width of 2.5 m is recommended for 2-lane rural highways in India.

2. Parking lanes

Parking lanes are provided in urban lanes for side parking. Parallel parking is preferred because it is safe for the vehicles moving on the road. The parking lane should have a minimum of 3.0 m width in the case of parallel parking.

3 .Bus-bays

Bus bays are provided by recessing the kerbs for bus stops. They are provided so that they do not obstruct the movement of vehicles in the carriage way. They should be at least 75 meters away from the intersection so that the traffic near the intersections is not affected by the bus-bay.

4. Service roads

Service roads or frontage roads give access to access controlled highways like freeways and expressways. They run parallel to the highway and will be usually isolated by a separator and access to the highway will be provided only at selected points. These roads are provided to avoid congestion in theexpressways and also the speed of the traffic in those lanes is not reduced.

5 .Cycle track

Cycle tracks are provided in urban areas when the volume of cycle traffic is high Minimum width of 2 meter is required, which may be increased by 1 meter for every additional track.

6 .Footpath

Footpaths are exclusive right of way to pedestrians, especially in urban areas. They are provided for the safety of the pedestrians when both the pedestrian traffic and vehicular traffic is high. Minimum width is 1.5 meter and may be increased based on the traffic. The footpath should be either as smooth as the pavement or more smoother than that to induce the pedestrian to use the footpath.

7 .Guard rails

They are provided at the edge of the shoulder usually when the road is on an embankment. They serve to prevent the vehicles from running off the embankment, especially when the height of the fill exceeds 3 m. Various designs of guard rails are there. Guard stones painted in alternate black and white are usually used. They also give better visibility of curves at night under headlights of vehicles.

8. Carriage Way

It is the width of the road which is used by the traffic for moving on it. It is generally central portion of the total land width and is paved and surfaced with the bituminous concrete for service to the road users. Width of the carriage way depends on the number of the lanes in the road which again depends on theclass of the highway. If it is higher level road like NH then it will need more numbers of lanes and therefore the carriageway width will be more.
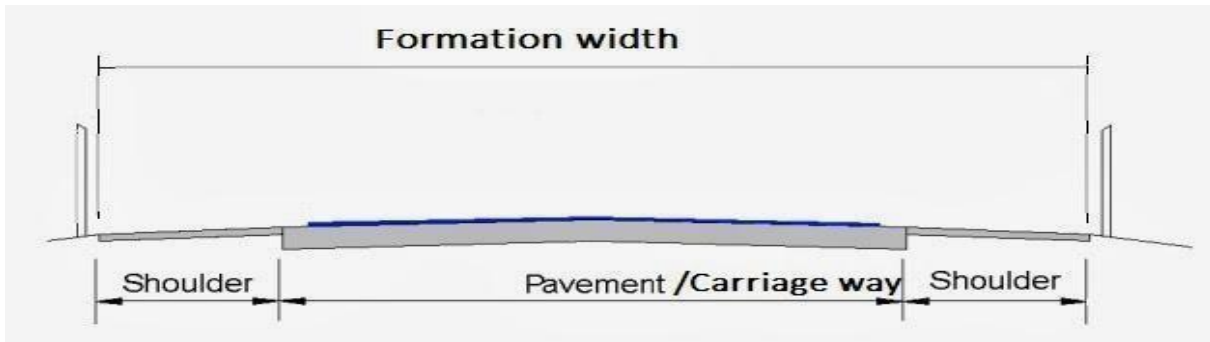
Side slopes

Side slopes are important in maintaining the stability of the roadbed and pavement structure as well as providing an area for the safety of errant vehicles. Side slopes are constructed in both fill (embankment) areas (those falling above the natural ground level) and cut areas (those falling below the natural ground level). As a general reference, slopes in embankment areas are commonly referred to as fill slopes or front slopes. When it is determined that no parallel ditch section is needed the front slope is graded to meet natural ground. In cut areas, side slopes are referred to as front slopes and back slopes, the back slope being necessary to bring the roadway cross section back up to meet the natural ground level. Ditch sections included as part of either fill or cut sections have a front slope, a ditch bottom with a defined shape and width, and a back slope. Criteria for rates of these slopes (by road classes) are shown in Fig.

FILL SLOPES

CUT SLOPES
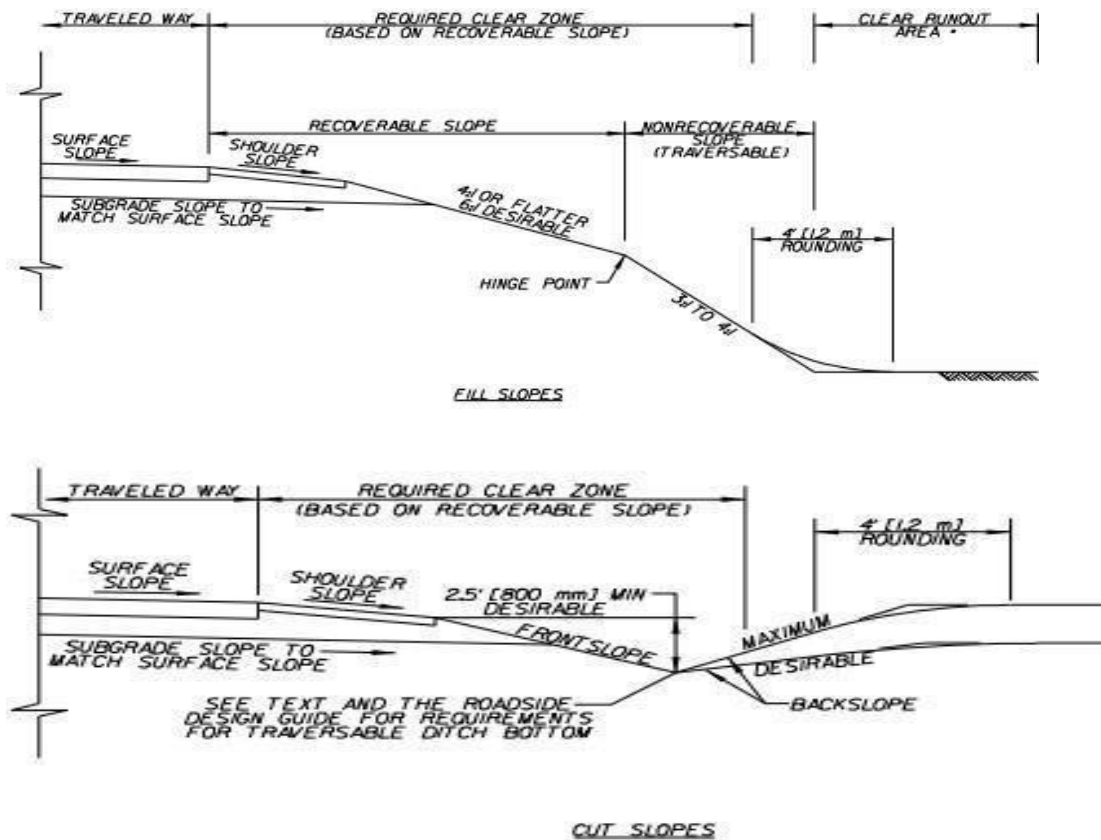
Kerbs

Kerbs indicate the boundary between the carriage way and the shoulder footpaths. Different types of kerbs are (Figure):

1. Low or mountable kerbs : This type of kerbs are provided such that they encourage the traffic to remain in the through traffic lanes and also allow the driver to enter the shoulder area with little difficulty. The height of this kerb is about 10 cm above the pavement edge with a slope which allows the vehicle to climb easily. This is usually provided at medians and channelization schemes and also helps in longitudinal drainage.

   Semi-barrier type kerbs : When the pedestrian traffic is high, these kerbs are provided. Their height is 15 cm above the pavement edge. This type of kerb prevents encroachment of parking vehicles, but at acute emergency it is possible to drive over this kerb with some difficulty.

2. Barrier type kerbs : They are designed to discourage vehicles from leaving the pavement. They are provided when there is considerableamount of pedestrian traffic. They are placed at a height of 20 cm above the pavement edge with a steep batter.

3. Submerged kerbs : They are used in rural roads. The kerbs are provided at pavement edges between the pavement edge and shoulders. They provide lateral confinement and stability to the pavement.
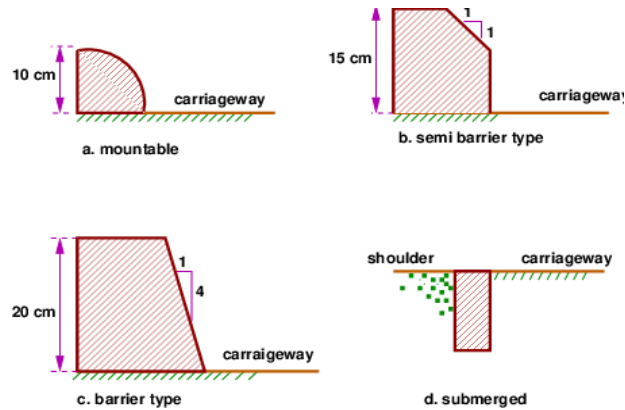
Figure : Different types of kerbs

Formation level

The Formation Level is the level at which excavation ceases and construction commences. It is the lowest point of the path structure. It is the prepared groundon which the sub base layer is laid.

Camber

Camber or cant is the cross slope provided to raise middle of the road surface in the transverse direction to drain off rain water from road surface. The objectives of providing camber are:

1. Surface protection especially for gravel and bituminous roads
2. Sub-grade protection by proper drainage
3. Quick drying of pavement which in turn increases safety

Too steep slope is undesirable for it will erode the surface. Camber is measured in *1 in n* or *n%* (Eg. 1 in 50 or 2%) and the value depends on the type of pavement surface. The values suggested by IRC for various categories of pavement is given in Table 1. The common types of camber are parabolic, straight, or combination of them (Figure)
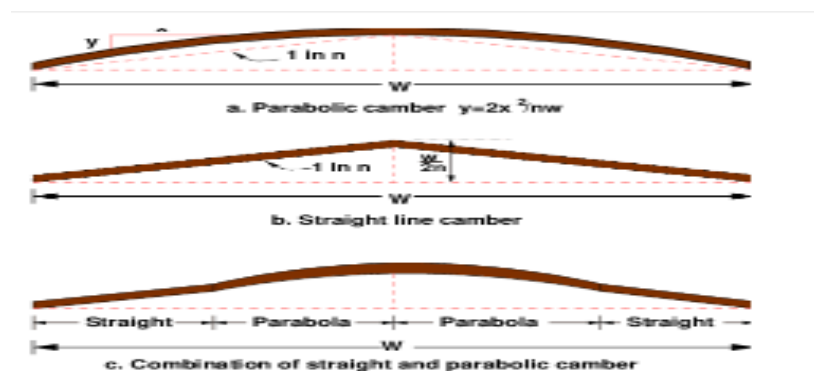
Figure : Different types of camber

Table : IRC Values for camber

Surface type	Heavy rain	Light rain
Concrete/Bituminous	2 %	1.7 %
Gravel/WBM	3 %	2.5 %
Earthen	4 %	3.0 %

Gradient

It is the slope provided to the surface of the road in the longitudinal direction for the vertical alignment of the road. There are three kinds of gradients:

    a) Ruling Gradient
    b) Limiting Gradient
    c) Exceptional Gradient
    d) Minimum Gradient.
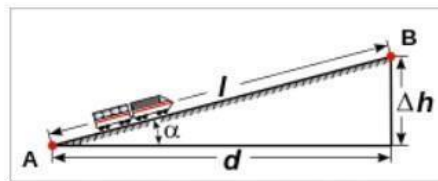
A vehicle on ascending gradient

➤ **Ruling gradient** is the design gradient, so it is used to design the road in the vertical alignment.

➤ **Limiting and exceptional gradients** are provided in the limited stretch of the roads where necessary and in case of the emergencies or exceptional cases when such need arises respectively.

➤ **Minimum gradient** is the gradient which is required as the minimum from the drainage point of view in case of the plane areas.

Design speed:

The design speed, as noted earlier, is the single most important f actor in the design of horizontal alignment. The design speed also depends on the type of the road. For e.g, the design speed expected from a National highway will bemuch higher than a village road, and hence the curve geometry will vary significantly.

The design speed also depends on the type of terrain. A plain terrain can afford to have any geometry, but for the same standard in a hilly terrain requiressubstantial cutting and filling implying exorbitant costs as well as safety concern due to unstable slopes.

Therefore, the design speed is normally reduced for terrains with steep slopes. For instance, Indian Road Congress (IRC) has classified the terrains into four categories, namely plain, rolling, mountainous, and steep based on the cross slope as given in table. Based on the type of road and typeof terrain the

design speed varies. The IRC has suggested desirable or ruling speed as well as minimum suggested design speed and is tabulated in table :

Table : Terrain classification

Table : Terrain classification

Terrain classification	Cross slope (%)
Plain	0-10
Rolling	10-25
Mountainous	25-60
Steep	60

The recommended design speed is given in Table .

Table : Design speed in   as per IRC (ruling and minimum)

Type	Plain	Rolling	Hilly	Steep
NS&SH	100-80	80-65	50-40	40-30
MDR	80-65	65-50	40-30	30-20
ODR	65-50	50-40	30-25	25-20
VR	50-40	40-35	25-20	25-20

Terrain classification Cross slope (%)

Terrain classification	Cross slope (%)
Plain	0-10
Rolling	10-25
Mounta inous	25-60
Steep	60

The recommended design speed is given in Table .

Table : Design speed in as per IRC (ruling and minimum)

Type	Plain	Rolling	Hilly	Steep
NS&SH	100-8 0	80-65	50-40	40-30
MDR	80-65	65-50	40-30	30-20
ODR	65-50	50-40	30-25	25-20
VR	50-40	40-35	25-20	25-20

1. Topography:

The next important factor that affects the geometric design is the topography. It is easier to construct roads with required standards for a plain terrain. However, for a given design speed, the construction cost increases multiform with the gradient and the terrain. Therefore, geometric design standards are different for different terrain to keep the cost of construction and time of construction under control. This is characterized by sharper curves and steeper gradients.

2. Other factors :

In addition to design speed and topography, there are various other factors that affect the geometric design and they are brie y discussed below:

Vehicle: The dimensions, weight of the axle and operating characteristics of a vehicle influence the design aspects such as width of the pavement, radii of the curve, clearances, parking geometrics etc. A design vehicle which has standard weight, dimensions and operating characteristics are used to establish highway design controls to accommodate vehicles of a designated type.

Human: The important human factors that influence geometric design are the physical, mental and psychological characteristics of the driver and pedestrians like the reaction time.

Traffic: It will be uneconomical to design the road for peak traffic flow.Therefore a reasonable value of traffic volume is selected as the design hourly volume which is determined from the various traffic data collected. The geometric design is thus based on this design volume, capacity etc.

Environmental: Factors like air pollution, noise pollution etc. should be given due consideration in the geometric design of roads.

Economy: The design adopted should be economical as far as possible. It should match with the funds allotted for capital cost and maintenance cost.

Others: Geometric design should be such that the aesthetics of the region isnot affected.

Average running speed

Running speed is the average speed maintained over a particular course whilethe vehicle is moving and is found by dividing the length of the course by the time duration the vehicle was in motion. i.e. this speed doesn't consider the timeduring which the vehicle is brought to a stop, or has to wait till it has a clearroad ahead. The running speed will always be more than or equal to the journey speed, as delays are not considered in calculating the running speed.

Sight Distances:

The safe and efficient operation of vehicles on the road depends very much on the visibility of the road ahead of the driver. Thus the geometric design of the road should be done such that any obstruction on the road length could be visible to the driver from some distance ahead . This distance is said to be the sight distance.

Types of sight distance

Sight distance available from a point is the actual distance along the road surface, over which a driver from a specified height above the carriage way has visibility of stationary or moving objects. Three sight distance situations are considered for design:

1. Stopping sight distance (SSD) or the absolute minimum sight distance
2. Intermediate sight distance (ISD) is defined as twice SSD
3. Overtaking sight distance (OSD) for safe overtaking operation
4. Head light sight distance is the distance visible to a driver during night driving under the illumination of head lights
5. Safe sight distance to enter into an intersection.

The most important consideration in all these is that at all times the driver travelling at the design speed of the highway must have sufficient carriageway distance within his line of vision to allow him to stop his vehicle before colliding with a slowly moving or stationary object appearing suddenly in his own traffic lane.

The computation of sight distance depends on:

1. **Reaction time of the driver**

   Reaction time of a driver is the time taken from the instant the object is visible to the driver to the instant when the brakes are applied. The total reaction time may be split up into four components based on PIEV theory. In practice, all these times are usually combined into a total perception-reaction time suitable for design purposes as well as for easy measurement. Many of the studies shows that drivers require about 1.5 to 2 secs under normal conditions. However, taking into consideration the variability of driver characteristics, a higher value is normally used in design. For example, IRC suggests a reaction time of 2.5 secs.

2. **Speed of the vehicle**

   The speed of the vehicle very much affects the sight distance. Higher the

speed, more time will be required to stop the vehicle. Hence it is evident that, as the speed increases, sight distance also increases.

3. Efficiency of brakes

The efficiency of the brakes depends upon the age of the vehicle, vehicle characteristics etc. If the brake efficiency is 100%, the vehiclewill stop the moment the brakes are applied. But practically, it is not possible to achieve 100% brake efficiency. Therefore the sight distance required will be more when the efficiency of brakes are less. Also for safegeometric design, we assume that the vehicles have only 50% brake efficiency.

4. Frictional resistance between the tyre and the road

The frictional resistance between the tyre and road plays an important role to bring the vehicle to stop. When the frictional resistance is more, the vehicles stop immediately. Thus sight required will be less. No separate provision for brake efficiency is provided while computing the sight distance. This is taken into account along with the factor of longitudinal friction. IRC has specified the value of longitudinal frictionin between 0.35 to 0.4.

5. Gradient of the road.

Gradient of the road also affects the sight distance. While climbing up a gradient, the vehicle can stop immediately. Therefore sight distance required is less. While descending a gradient, gravity also comes into action and more time will be required to stop the vehicle. Sight distance required will be more in this case.

Stopping sight distance (SSD)

Stopping sight distance (SSD) is the minimum sight distance available on a highway at any spot having sufficient length to enable the driver to stop a vehicle travelling at design speed, safely without collision with any other obstruction.

There is a term called *safe stopping distance* and is one of the important measures in traffic engineering. It is the distance a vehicle travels from the pointat which a situation is first perceived to the time the deceleration is complete. Drivers must have adequate time if they are to suddenly respond to a situation. Thus in highway design, sight distance at least equal to the safe stopping distance should be provided. The stopping sight distance is the sum of lag distance and the braking distance. Lag distance is the distance the vehicletravelled during the reaction time *t* and is given by *vt*, where *v* is the velocityin *m/sec²*. Braking distance is the distance travelled by the vehicle during braking operation. For a level road this is obtained by equating the work done in stopping the vehicle and the kinetic energy of the vehicle. If *F* is the maximum frictional force developed and the braking distance is *l*, then work done against friction in stopping the vehicle is *Fl = fWl* where *W* is the total weight of the vehicle. The kinetic energy at the design speed is

$$\frac{1}{2}mv^2 = \frac{1}{2}\frac{Wv^2}{g}$$

$$fWl = \frac{Wv^2}{2g}$$

$$l = \frac{v^2}{2gf}$$

Therefore, the SSD = lag distance + braking distance and given by:

$$SSD = vt + \frac{v^2}{2gf}$$

where v is the design speed in *m/sec²*, *t* is the reaction time in *sec*, *g* is the acceleration due to gravity and *f* is the coefficient of friction. The coefficient of friction *f* is given below for various design speed.

Table 1: Coefficient of longitudinal friction

Speed, kmph	<30	40	50	60	>80
f		0.40	0.38	0.37	0.36	0.35

When there is an ascending gradient of say +*n*%, the component of gravity adds to braking action and hence braking distance is decreased. The component of gravity acting parallel to the surface which adds to the the braking force is equal to $W \sin \alpha \approx W \tan \alpha = Wn/100$. Equating kinetic energy and work done:

$$\left(fW + \frac{Wn}{100}\right)l = \frac{Wv^2}{2g}$$

$$l = \frac{v^2}{2g\left(f + \frac{n}{100}\right)}$$

Overtaking sight distance:

The overtaking sight distance is the minimum distance open to the vision of the driver of a vehicle intending to overtake the slow vehicle ahead safely against the traffic in the opposite direction. The overtaking sight distance or passing sight distance is measured along the centre line of the road over which a driver with his eye level 1.2 m above the road surface can see the top of an object 1.2 m above the road surface.

The factors that affect the OSD are:

1. Velocities of the overtaking vehicle, overtaken vehicle and of the vehicle

coming in the opposite direction.

2. Spacing between vehicles, which in-turn depends on the speed
3. Skill and reaction time of the driver
4. Rate of acceleration of overtaking vehicle
5. Gradient of the road

The dynamics of the overtaking operation is given in the figure which is a time-space diagram. The x-axis denotes the time and y-axis shows the distance travelled by the vehicles. The trajectory of the slow moving vehicle (B) is shown as a straight line which indicates that it is travelling at a constant speed.A fast moving vehicle (A) is travelling behind the vehicle B. The trajectory of the vehicle is shown initially with a steeper slope. The dotted line indicates the path of the vehicle A if B was absent. The vehicle A slows down to follow the vehicle B as shown in the figure with same slope from t_0 to t_1. Then it overtakes the vehicle B and occupies the left lane at time t_3. The time duration $T = t_3 - t_1$ is the actual duration of the overtaking operation. The snapshots of the road at time t_0, t_1, and t_3 are shown on the left side of the figure. From the Figure 1, the overtaking sight distance consists of three parts.
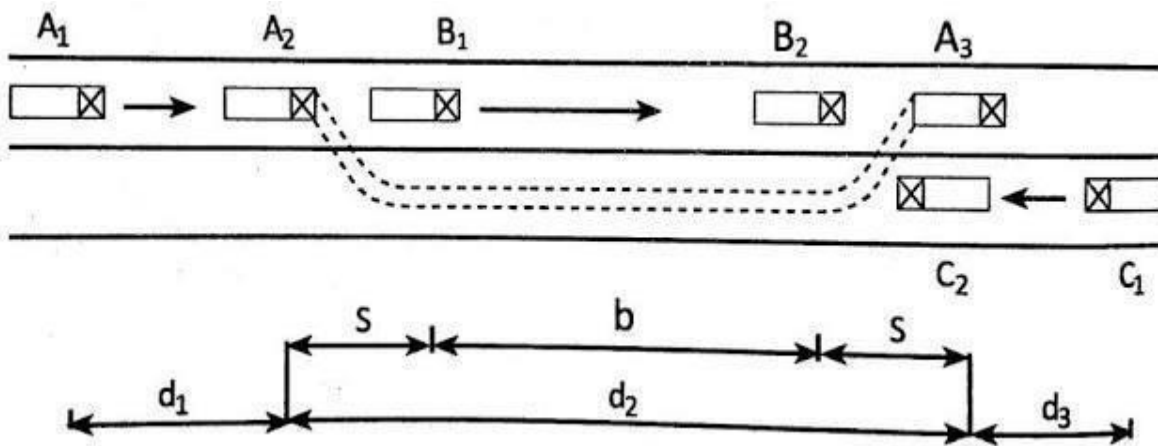
1. d_1 the distance travelled by overtaking vehicle A during the reactiontime $t = t_1 - t_0$

2. d_2 the distance travelled by the vehicle during the actual overtaking operation $T = t3-t1$

3. d_3 is the distance travelled by on-coming vehicle C during the overtaking operation (T).

Therefore:

$$OSD = d_1 + d_2 + d_3 \tag{3}$$

It is assumed that the vehicle A is forced to reduce its speed to v_b, the speedof the slow moving vehicle B and travels behind it during the reaction time t the driver. So d_1 is given by:

$$d_1 = v_b t \tag{4}$$

Then the vehicle A starts to accelerate, shifts the lane, overtake and shift back to the

original lane. The vehicle A maintains the spacing s before and afterovertaking. The spacing s in m is given by:

$$s = 0.7v_b + 6 \tag{5}$$

Let T be the duration of actual overtaking. The distance travelled by B during the overtaking operation is $2s + v_bT$. Also, during this time, vehicle A accelerated from initial velocity v_b and overtaking is completed while reaching final velocity v. Hence the distance travelled is given by:

$$
\begin{aligned}
d_2 &= v_bT + \frac{1}{2}aT^2 \\
2s + v_bT &= v_bT + \frac{1}{2}aT^2 \\
2s &= \frac{1}{2}aT^2 \\
T &= \sqrt{\frac{4s}{a}} \\
d_2 &= 2s + v_b\sqrt{\frac{4s}{a}}
\end{aligned}
\tag{6}
$$

The distance travelled by the vehicle C moving at design speed v m/sec during overtaking operation is given by:

$$d_3 = vT \tag{7}$$

The overtaking sight distance is:

$$OSD = v_bt + 2s + v_b\sqrt{\frac{4s}{a}} + vT$$

where v_b is the velocity of the slow moving vehicle in m/sec^2, t the reaction time of the driver in sec, s is the spacing between the two vehicle in m given by equation 5 and a is the overtaking vehicles acceleration in m/sec^2. In case the speed of the overtaken vehicle is not given, it can be assumed that it moves 16 kmph slower the design speed.
The acceleration values of the fast vehicle depends on its speed and given in Table .

Table : Maximum overtaking acceleration at different speeds:

Speed (kmph)	Maximum overtaking acceleration (m/sec²)
25	1.41
30	1.30
40	1.24
50	1.11

Note that:

1. On divided highways, d_3 need not be considered
2. On divided highways with four or more lanes, IRC suggests that it is not necessary to provide the OSD, but only SSD is sufficient.

Overtaking zones

Overtaking zones are provided when OSD cannot be provided throughout the length of the highway. These are zones dedicated for overtaking operation, marked with wide roads. The desirable length of overtaking zones is 5 time OSD and the minimum is three times OSD (Figure).
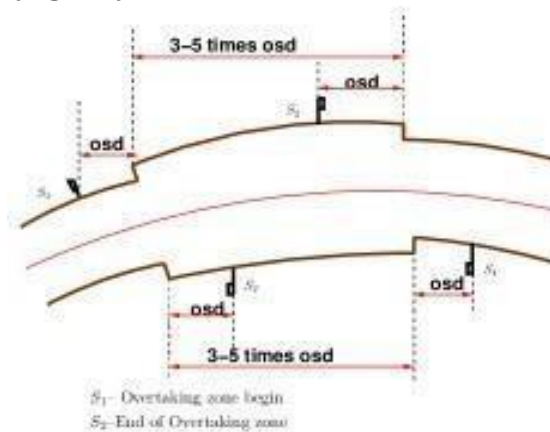
Figure : Overtaking zones

Sight distance at intersections

At intersections where two or more roads meet, visibility should be provided for the drivers approaching the intersection from either sides. They should be able to
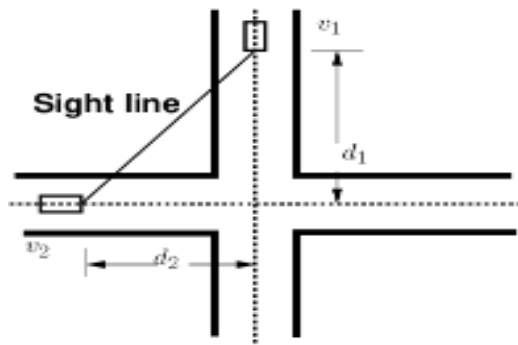
perceive a hazard and stop the vehicle if required. Stopping sight distance for each road can be computed from the design speed. The sightdistance should be provided such that the drivers on either side should be able to see each other. This is illustrated in the figure . **Figure :** Sight distance at intersections

Design of sight distance at intersections may be used on three possibleconditions:
1. Enabling approaching vehicle to change the speed
2. Enabling approaching vehicle to stop
3. Enabling stopped vehicle to cross a main road

CURVES

Definition of Curves:

Curves are regular bends provided in the lines of communication like roads,railways etc. and also in canals to bring about the gradual change of direction.

They are also used in the vertical plane at all changes of grade to avoid theabrupt change of grade at the apex.

Curves provided in the horizontal plane to have the gradual change in direction are known as Horizontal curves, whereas those provided in the verticalplane to obtain the gradual change in grade are known as vertical curves. Curvesare laid out on the ground along the centre line of the work. They may be circular or parabolic.

Types of Curves:

The amount of rise seen on a given cross-section of a turning road, it is otherwise known as slope.

Vertical Curves

Vertical curves are provided to change the slope in the road and may or may not be symmetrical. They are parabolic and not circular like horizontal curves. Identifying the proper grade and the safe passing sight distance is the main design criterion of the vertical curve, crest vertical curve the length shouldbe enough to provide safe stopping sight distance and in sag vertical curve the length is important as it influences the factors such as headlight sight distance, rider comfort and drainage requirements.

Types of Vertical Curve:
Sag Curve

Sag Curves are those which change the alignment of the road from uphill to downhill,
Crest Curve/Summit Curve

Crest Curves are those which change the alignment of the road from downhill to uphill. In designing crest vertical curves it is important that the grades be not] too high which makes it difficult for the motorists to travel upon it.

Super elevation :

When a vehicle travels in a circular path or curved path, it is subjected to an outward force

which makes a vehicle to overturn and skid due to Centrifugal force. To overcome this force and for safe travel of a vehicle, the outer edge of the road is raised above the inner edge. This is known as super elevation.
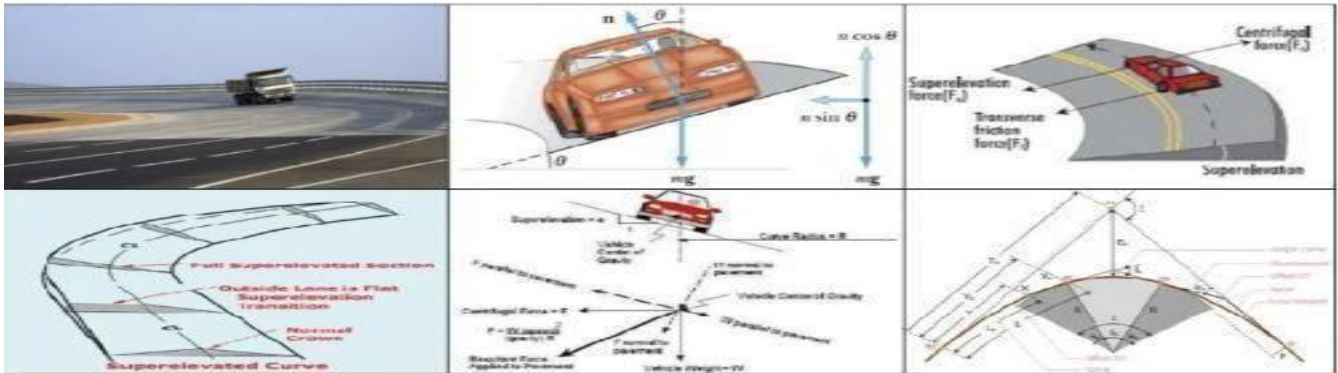
Super-Elevation / Banking of road reduces the effect of centrifugal force onthe running wheels. If super-elevation is not provided with the entire centripetal.

force is produced by the friction between the vehicle's tires and the roadway, thus results in reducing the speed of a vehicle.

Advantages of providing Super elevation:-

1. Super elevation is provided to achieve the higher speed of vehicles.It increases the stability of fast-moving vehicles when they pass through a horizontal curve, and it also decreases the stresses on the foundation.
2. In the absence of super elevation on the road along curves, potholes are likely to occur at the outer edge of the road.
3. The Indian road congress(IRC) has prescribed the max value of Super Elevation is 1 in 15.

Derivation of Super Elevation:

When a vehicle passes from a straight to a curved path or in other words when a vehicle negotiates horizontal curve following two forces act on vehicle.

1. Centrifugal Force
2. Weight of the Vehicle

Thus super-elevation e is the ratio of the height of the outer edge with respect to the horizontal width.

Centrifugal Force - The centrifugal force is a function of the speed of the moving

vehicle. It always acts at the centre of gravity of the vehicle. It's direction always tends to outside, i.e., it always tends to push the vehicle out of the track. to counteract this tendency, the outer edge of the road is raised above the inner edge. **This rise of the outer edge is called super-elevation or cant or banking.**

$$e = \tan \theta$$

In practice, the value of θ is kept as 4° or a slope of 1 in 15 with horizontal.
The total height of the outer edge with respect to the inner edge:

$$E = e \times \text{width of road}$$
$$= e B$$

The centrifugal force P $= Wv^2/gR$

Where,

W = weight of the vehicle
v = velocity of the vehicle
R = radius of circular curve
P = centrifugal force
g = acceleration due to gravity

Super Elevation
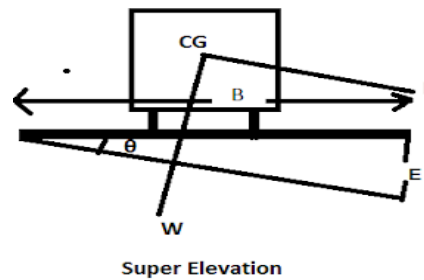
1. **Effect of Centrifugal Force**
    1. The tendency to overturn the vehicle.
    2. The tendency to skid the vehicle laterally.

Stability Condition Against Overturning

Overturning due to Centifugal force
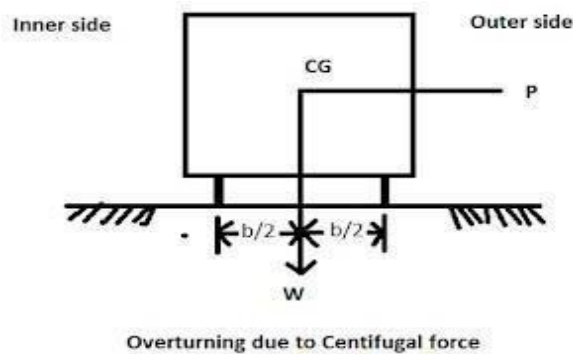
The figure shows a vehicle moving on horizontal a curve. Forces acting onthe vehicle are

a.) Centrifugal force P acting outward at C.G.

b.) Weight W acting downward at C.G.

Let h be the height of C.G. of the vehicle above the road level.The overturning moment due to centrifugal force.

$$= P \times h$$

The restoring moment $\qquad = W \times b/2$

where b is the centre to centre distance of wheels of the vehicle.

in limiting equilibrium

$$Ph = Wb/2$$
$$P/W = b/2h$$

When the centrifugal ratio, P/W is equal to B/2h there is a danger ofoverturning. Thus to avoid overturning, the centrifugal ratio should always be less than b/2h.

$$P/W = v^2/Gr$$
$$v^2/gR < b/2h$$

Thus to avoid overturning h should be as small as possible. Only due to thisreason modern passenger cars have a low centre of gravity.

Stability Condition Against Skidding

Skidding due to Centifugal force
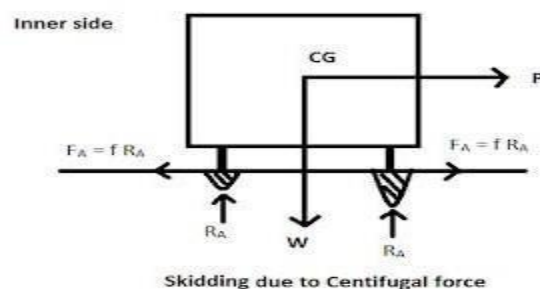
The lateral thrust $P = Wv^2/gR$ is resisted by the frictional force between the tyre and pavement surface. If the lateral resisting friction is less than the centrifugal force P, then skid will occur. Thus in limiting equilibrium, P = Maximum lateralfriction developed as shown in the figure.

$$P = F_a + F_b$$

or $\qquad P = f(R_A + R_B)$

= fW

Or $\qquad$ P/W = f

Thus when the centrifugal ratio attains the value equal to the lateral coefficientof friction, there is a danger of lateral skidding.

If f < b/2h skidding would occur.

If f > B/2h overturning at the outer edge would occur.

Methods o f providing super – elevation :

Super-elevation is designed for the particular vehicle called design vehicle which has some standard weight and dimensions. But in the actual case, theroad has mixed traffic conditions. Different vehicles require different values of super-elevation. For example Heavily loaded trucks require the small value of super-elevation otherwise toppling may occur, fast moving vehicles may be provided with high super-elevation while slow moving ones require small super-elevation. The design procedure for super-elevation is as follows:

Step 1 Find value of super-elevation taking 75% of design speed neglecting f,
$\qquad$ Hence, e = (0.75v) 2 / (g*R)

Step 2 If value of e is less than 0.07 then it is taken for design otherwise valueof e is taken as 0.07.

Step 3 Find value of frictional coefficient (f) with full design speed regarding maximum super-elevation.

$$\text{Hence, f = v2 / (g*R) – e = v2 / (g*R) – 0.07}$$

Step 4 If value for f is less than 0.15 then it is taken for design otherwise valuefor f is taken as 0.15.

Step 5 The allowable speed for maximum value of e = 0.07 and f = 0.15 is calculated
$\qquad$ Hence, Allowable speed (V$_a$) = √ (0.22g*R)
If the allowable velocity is greater than or equal to v then the design is adequate otherwise other speed control measures are adopted.

Different guidelines are given in NRS for the design of horizontal curvature.In terms of velocity in kmph it is calculated as,
$\qquad$ Or, V² / (126.5*R) = e + f
And, $\qquad$ V$_a$ = √ [126.5*R*(e + f)]

Methods of introducing superelevation:

Superelevation is introduced in two ways usually

Method 1: Ellimination of Crown

Method 2: Rotation of outer Surface

Methods of providing Superelevation
Stage 1: Minimum Superelevation
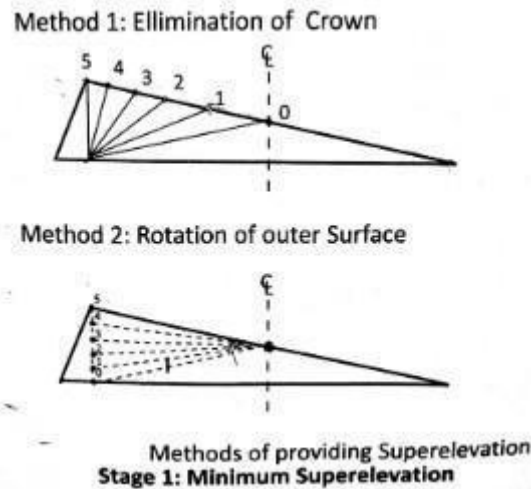
1. **Elimination of the crown of the cambered section**
   - ➤ The outer edge half of the cross slopes is rotated about the crown at the desired rate such that its surface falls on the same plane as the inner half.
   - ➤ The position of the crown is progressively shifted outwards which increases the width of the inner half of the cross-section progressively. It is also called as the diagonal crown method.

2. **Rotation of the pavement cross section to attain full superelevation**
   - ➤ Rotation about the center line: It is mostly preferred by the majority of designers. This method involves progressively revolving the straight road surface about the center line depressing the inner edge and raising the outer edge simultaneously by an amount of half the total amount of superelevation. Thus the earthwork is the balanced i.e. volume of cutting and filling required in this method will be equal. Due to depressing inner edge below general level, the disadvantage of this method is drainage.
   - ➤ Rotation about the inner edge: This method involves progressively revolving the straight road surface about the inner edge thereby raising the center line and the outer edge proportionately to the desired slope. Here the outer edge is raised by the full amount of superelevation. This method is preferred in very flat terrain in high rainfall areas in order to avoid drainage problem. The rise of the center line is considered a disadvantage in this method since the vertical alignment of the road is altered.
   - ➤ Rotation about the outer edge: This method involves progressively revolving the straight road surface about the outer edge thereby depressing the center line and the outer edge proportionately to the desired slope. Here the inner edge is depressed by the full amount of superelevation with respect to the outer edge. This method is similar to the rotation about the inner edge.

Rotation about ₵   Rotation about inner edge   Rotation about outer edge
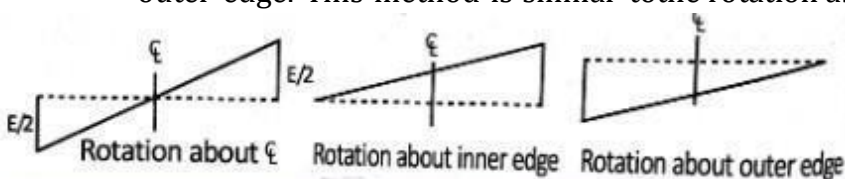
EXTRA WIDENING

On a horizontal curve especially when there are not many very large radius, it is common to widen the pavement or carriageway slightly more than the normal width. This additional increase in the width of the pavement is termed as extra widening. The reasons of providing extra widening are given below:

1. Due to the rigidity of rear wheel base of the vehicle moving along the curve they don't trace the same path as taken by the front steering wheels.This is called off tracking. During this case, mechanical widening is required.
2. Due to the psychological tendency of drivers, drivers tend to take the outer lane for greater visibility and easy gradient at the beginning of the curves. So extra width is provided.
3. Drivers have the tendency to keep greater clearance between opposite vehicles on curves when road visibility is inadequate.
4. Drivers have the tendency to keep away from the edge of the carriageway while driving on curves.
5. When traveling in a curved path more space is occupied by the vehicles during turning.
6. Trailer units require even larger extra width at curves.
7. At speeds higher than the design speed outward slipping of the rear wheel may occur due to centrifugal force and thus more width of the road is covered. Even in lower speeds, the front tires tend to go out of the pavement and thus extra widening is required.
8. The extra width is provided to ensure safe and efficient overtaking operations.
9. The amount of extra widening required depends on the following factors:
    1. Length of the wheel base of the vehicle
    2. Psychological factor
    3. Radius of the curve

   The analysis of extra widening is done in two parts namely:
    1. Mechanical widening
    2. Psychological widening

(a) Off-tracking

(b) Preferential use of outer lane
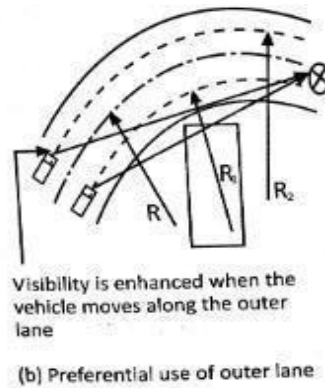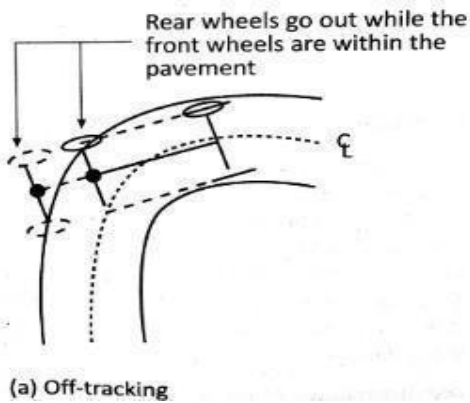
Mechanical widening
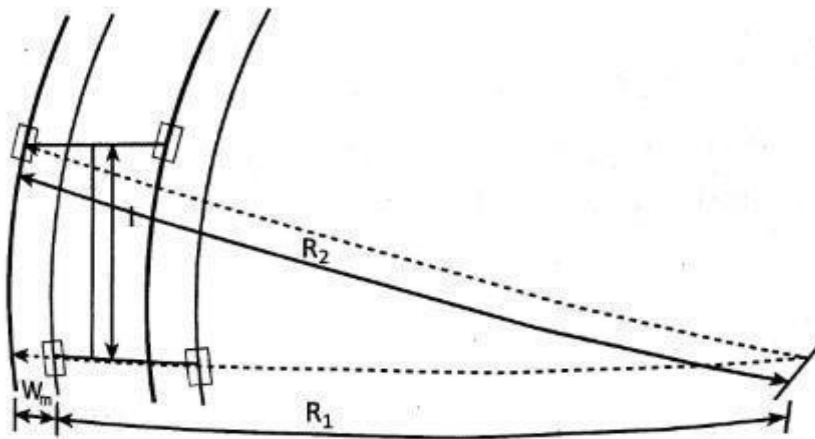
It is the amount of the extra width required to account for the off tracking of the vehicle due to the rigidity of the rear wheel base. Due to off tracking, the rear wheels follow a path of shorter radius than the front ones and thus it increases the effective width of road space required by the vehicle. So we provide extra width to provide same clearance between vehicles, to provide safety against transverse skidding during high speeds and also to provide stability for vehicles like trailer trucks. Thus it is an important factor during operation of high proportions of vehicles.

Let R1 be the radius of the path travelled by the outer track line of the rear wheel, R2 be the radius of the path travelled by the outer track line of the front wheel and l be the distance between the front and rear wheels. Then the mechanical widening is given by:

$W_m = R_2 - R_1$

Or, $R_1 = R_2 - W_m$

From the triangle, we can say that

$R_2^2 = R_1^2 + l^2$

Or, $R_2^2 = (R_2 - W_m)^2 + l^2$

Or, $R_2^2 = R_2^2 - 2*R_2*W_m + W_m^2 + l^2$

Or, $l^2 = 2*R_2*W_m - W_m^2$

Hence the widening required will be,

Wm = l2 / (2*R2 – Wm)

For large radius R1≈R2 which is the mean radius of the curve, Wm becomes very small in comparison to R then the mechanical widening is given by:

Wm = l2 / (2*R) For n number of lanes,

Wm = (n*l2) / (2*R)

1. Psychological widening

It is the amount of additional width required due to various psychological reasons i.e. for greater maneuverability, for confidence, for the tendency to maintain greater clearance between vehicles, for the tendency of drivers to drive close to the edges of the pavement on curves, etc. which the driver will be needing is termed as psychological widening. It is very difficult to assess so IRC has proposed an empirical relation for the psychological widening.

Wpsy = v / (2.64*√R)

Where, Wpsy is the psychological widening, v is the velocity in m/s and R isthe radius in meter.

In terms of kmph,

Wpsy = v / (9.5*√R)

Therefore the total widening required at a horizontal curve is,

We = Wm + Wpsy = (n*l2) / (2*R) + v / (9.5*√R)

Where, v is in kmph.

Definition and Types of Vertical Curves:

The curve in a vertical alignment which is produced when two different gradients meet is known as vertical curves. It is provided to secure safety, safety, appearance and visibility. The most common practice has been to use parabolic curves in summit curves. This is because of the ease of setting it out on the field and the comfortable transition from one gradient to another. Furthermore, the use of parabolic curves gives excellent riding comfort. In the case of valley curves, the use of cubic parabola is preferred as it closely approximates the ideal transition requirements.

Types of vertical curves

Depending upon the shape of profile, a vertical curve may be divided into:

1. **Summit curve:** When two grades meet at the summit and the curve will have convexity upwards, the curve is simply referred as summit curve.

2. **Valley (Sag) curve:** When two grades meet at the valley (sag) and the curve will have convexity downwards, the curve is simply referred as the valley (sag) curve.

Elements of vertical curve

The different elements of vertical curve are:

1. Deviation angle (N) = n1 – n2
2. Tangent length (T) = L/2
3. Length of the curve (L) = 2*T = N*R
4. Apex distance (E) = ±T2 / (2*R) = ±L2 / (8*R)
5. Mid-ordinate (M) = R [1 – cos {(180*N) / (2*π)}]
6. Where, n is the side gradient and R is the radius of the curve.

<u>Design of Vertical Summit Curve</u>

Summit curves are those curves which have convexity upwards. They are formed under the four following conditions:

1. When a positive gradient meets another mild positive gradient
2. When a positive gradient meets a level zero gradient
3. When a positive gradient meets with a negative gradient
4. When a negative gradient meets another steeper negative gradient

During the design of vertical summit curve the comfort, appearance, and security of the driver should be considered. The sight distance must be considered in the design. All the types of sight distance should be considered during design as far as possible. During movement in a summit curve, there is less discomfort to the passengers because the centrifugal force developed by the movement of the vehicle on a summit curve act upwards which is opposite to the direction in which its weight acts. This relieves the load on the springs of the vehicle so stress developed will be less.

A simple parabolic curve is preferred in summit curve due to its easy implementation in the field, good riding comfort during driving and easy computation. The important part in summit curve design is the computation of the length of the summit curve which is done considering the sight distance parameters as shown below:

Two cases may arise during the design,

1.<u>LENGTH OF SUMMIT CURVE IS GREATER THAN THE SIGHT DISTANCE</u>

The equation of the parabolic curve is,

Or, y = a*x2

Where, a = N / (2*L)

From figure,

Heights (h1) = as12 and h2 = as22
Then, s1 + s2 = √ (h1 / a) + √ (h2 / a)
Or, s2 = (1/a) (√h1 + √h2)2
Or, s2 = (2*L) * (√h1 + √h2)2 / N
Therefore, L = (NS2) / [2 * (√h1 + √h2)2]
Where L is the length of the summit curve, s is the sight distance, N is the deviation angle, h1 is the height of the driver's eye from the road surface, h2 is the height of obstruction from the road surface.
In terms of stopping sight distance (SSD),
Height h1 = 1.2m and h2 = 0.15m
Substituting we get,

L = Ns2 / 4.4
  o In terms of overtaking sight distance (OSD) and intermediate sight distance (ISD),
Height h1 = 1.2m and h2 = 1.2m
Substituting we get,

L = Ns2 / 9.6

2. LENGTH OF SUMMIT CURVE IS LESS THAN THE SIGHT DISTANCE

From basic geometry, one can write,

Or, s = L/2 + h1/n1 + h2/n2
Or, s = L/2 + h1/n1 + h2 / (N – n1)
For minimum value of s we differentiate the equation with respect to n1 and set it to zero,
Or, ds/dh1 = -h1/n12 + h2 / (N – n1)2 = 0
Or, h2n12 = h1 * (N2 + n12 – 2*N*n1)
Solving the equation we get,

Therefore, n1 = (N * √ (h1h2) – Nh1) / (h2 – h1)
Substituting value of n1 in s we get,
Or, s = L/2 + h1 / (N * √ (h1h2) – Nh1) / (h2 – h1) + h2 / [N – (N * √ (h1h2) – Nh1) / (h2 – h1)]
Solving for L we get,

L = 2s – [√ (2h1) + √ (2h2)] 2 / N
  o   In terms of stopping sight distance (SSD),
Height h1 = 1.2m and h2 = 0.15m
Substituting we get,

L = 2s – 4.4/N

In terms of overtaking sight distance (OSD) and intermediate sight distance (ISD),
Height h1 = 1.2m and h2 = 1.2m
Substituting we get,

L = 2s – 9.6/N

It should be known that these values are the minimum lengths and greater sight distances should be used where there is economically and technically feasible.

Design of vertical valley curve
1. When a negative gradient meets another mild negative gradient
2. When a negative gradient meets a level zero gradient
3. When a negative gradient meets with a positive gradient
4. When a positive gradient meets another steeper positive gradient

As compared to the design of summit curve, valley curve requires more consideration. During day time the visibility in valley curves are not hindered but during night time the only source of visibility becomes headlight in the absence of street lights. And in valley curves, the centrifugal force generated by the vehicle moving along a valley curve acts downwards along with the weight of the vehicle and this adds to the stress induced in the spring of the vehicle which causes jerking of the vehicle and discomfort to the passengers. Thus, the most important things to consider during valley curve design are:

o Impact and jerking free movement of vehicles at design speed
o Availability of stopping sight distance under headlight of vehicles during night driving

The best shape for a valley curve is transition curve. Some prefer to use the circular curve or quadratic parabola or combined circular spiral curve but mostly cubic parabola is generally preferred in vertical valley curves. Valley curve is made fully translational by providing two similar transition curves of equal length. It is set by cubic parabola $y = bx^3$ where $b = 2N / 3L^2$. The length of the valley transition curve is designed to the following two criteria:

COMFORT CRITERIA
This is used in design to provide impact-free movement of vehicles at design speed. In this criterion, the allowable rate of change of centrifugal acceleration (c) is limited to a comfortable level of 0.6m/s³. The rate of change of centrifugal acceleration is given by,

Or, $c = [(v^2 / R) - 0] / t = (v^2 / R) / (L_s / v)$

Then, $L_s = v^3 / (c \cdot R)$

For a cubic parabola, the value of R is given by,

$R = L_s / N$

Substituting value of R we get,

$L_s = (N \cdot v^3) / (c \cdot L_s)$

Or, $L_s^2 = (N \cdot v^3) / c$

Therefore, $L_s = \sqrt{[(N \cdot v^3) / c]}$

The total length of the valley curve is given by,

$L = 2 \cdot L_s = 2 \cdot \sqrt{[(N \cdot v^3) / c]}$

Where, L is the total length of the valley curve, N is the deviation angle, v is the design speed and c is the rate of change of centrifugal acceleration which may be taken as 0.6 m/s³.

Applying value of c = 0.6 m/s³ we get,

$L_s = 0.19 \sqrt{(N \cdot v^3)}$

$L = 0.38 \sqrt{(N \cdot v^3)}$

SAFETY CRITERIA

Sight distance is highly reduced during headlight driving conditions during night time. The sight distance within which the head lights can illuminate is known as head light distance and it should be equal to the stopping sight distance. There is no problem of overtaking operation at night because of low traffic and the fact that other vehicles with headlights can be seen from a considerable distance. This design provides adequate stopping sight distance for vehicles under headlight at night time at any part of the curve. It may be determined from two conditions:

- **Length of the valley curve is greater than the stopping sight distance**

At the lowest point of the valley curve, the sight distance will be minimum because at the bottom of the curve where there is minimum radius which is a property of the transition curve. From the geometry of the figure, we have,

Or, h1 + s tan α = as2
Where, a = N / (2L)

Then, h1 + s tan α = Ns2 / (2L)
Therefore,

L = (Ns2) / (2h1 + 2s tan α)
Where N is the deviation angle, s is the sight distance, h1 is the height of the headlight beam from the road surface and α is the head beam inclination in degrees.
Taking, h1 = 0.75m [NRS] and α≈1 ÌŠ we get,
L = (Ns2) / (1.5 + 0.035s)

- **Length of the valley curve is less than the stopping sight distance**
  - In this case, the minimum sight distance will be at the beginning of the curve because the headlight beam at the beginning of the curve will just hit outside of the curve. But at the bottom of the curve, the headlight beam will reach far beyond the end point of the curve. Hence the length of the curve is determined to assume that the vehicle is at the beginning of the curve. From the geometry of the figure, we have,

  - Or, h1 + s tan α = (s – L/2) * N
  - Therefore,

  - L = 2*s – (2h1 + 2s tan α) / N

Where N is the deviation angle, s is the sight distance, h1 is the height of the headlight beam from the road surface and α is the head beam inclination in degrees.
Taking, h1 = 0.75m [NRS] and α≈1 ÌŠ we get,
L = 2*s – (1.5 + 0.035s) / N

The expression above is approximate but it is satisfactory in practice because the gradients are very small. During design, both cases need to be calculated because we will not know prior which case has to be adopted. After calculation, we adopt the greater length among the two during design.

According to the specification of NRS 2045, the criteria to be adopted are that the stopping sight distance shall be equal to the headlight sight distance and the centripetal acceleration is limited to 0.3 m/s2.

Lowest and highest point of vertical curve

From the element of a vertical curve, we can write,

Length of a curve (L) = R * N

Where, R is the radius of the curve and N is the deviation angle.

Then, R = L / N

The highest point is calculated in a summit curve whereas the lowest is calculated in a valley curve. It is formed near the smaller gradient among the tangents. If n1 = +2% and n2 = -1.5% then summit curve is formed where the highest point is formed near the n2 grade i.e. near the end of the vertical curve (EVC).

Then, the distance of highest point from beginning of the curve (x1) = R * n1 = (L * n1) / N
Similarly from end of the curve (x2) = R * n2 = (L * n2) / N
And tangent correction for the curve or the height of the highest point of the curve is given by,

Or, y = x2 / (2*R) = (N * x2) / (2*L)
The lowest point of the curve is calculated in the same manner in the case of valley curve.

In the case of valley curve, if the curve is cubic parabola then the lowest point of the valley curve lies on the side of the flatter grade and the distance of the lowest point from the flatter gradient is given by:

Or, x1 = L * √ [n1 / (2*N)]
And, tangent correction (y) = (x3 * 2N) / (3*L2)

UNIT 3-ROAD MATERIALS

A wide variety of materials are used in the construction of roads these are soils (naturally occurring or processed), aggregates (fine aggregates or coarse aggregates obtained from rocks), binders like lime, bituminous materials, and cement, and miscellaneous materials used as admixtures for improved performance of roads under heavy loads and traffic.

Soil constitutes the primary material for the foundation, subgrade, or even thepavement (for low-cost roads with low traffic in rural areas). When the highwayis constructed on an embankment at the desired level, soil constitutes the primary embankment material; further, since all structures have to ultimately rest on and transmit loads to 'mother earth', soil and rock also serve as foundation materials.

Soil is invariably used after some process of stabilization such as compaction and strengthening by adding suitable admixtures for improving the performance of the road. Mineral aggregates obtained from rocks form the major component of the sub-bases and bases of highway pavements of almostall types.

1. Soil

1. Soils can be studied effectively if they are classified according to certain principles into a definite system. A system is an ordered grouping of certain elements in a discipline according to pre-defined principles. Just as classification or grouping is practised in scientific disciplines such as chemistry,zoology and botany, it is used in Geotechnical Engineering as well.

2. A soil classification system may be defined as a fundamental division of the various types of soil into groups according to certain parameters such as its physical properties, constituents or texture, field performance under load,

presence of water and so on. There are a few field identification tests have been developed for preliminary identification in the field.

Need for Soil Classification:

Soil deposits in nature are never homogenous in character; wide variations are observed in their properties and behaviour. Soils that exhibit similar averageproperties may be grouped as a class. Classification of soil is necessary to obtain an appropriate and fairly accurate idea of the properties and behaviour of a soil type.

A classification system is usually evolved with a view to assessing the suitability of a soil for specific use as a construction material or as a foundation material. In view of the wide variations in engineering properties of several soils, it is inevitable that in any system of classification, there will be borderline cases which may fall into groups that appear to be radically different under different systems of classification.

Hence, classification is taken only as a preliminary requirement to study the engineering behaviour of a soil; special tests may become necessary in any project of importance.

Requirements of a Soil Classification System:

The general requirements of an ideal soil classification system are:

    i.   It should have a scientific basis.

    ii.   It should be relatively simple and objective in approach.

    iii.   The number of groupings and properties used as the criteria should be limited.

    iv.   The properties considered should be relevant to the purpose of classification.

    v.   A generally accepted uniform soil terminology should be used.

    vi.   It should indicate the probable performance of the soil to a satisfactory degree of accuracy.

    vii.   Group boundaries should be drawn as closely as possible where significant changes in soil properties occur.

    viii.   It should be acceptable to all engineers.

Although several classification systems have been developed, some being relatively more elaborate and exhaustive than others, the following systems only will be considered**:**

(a) Textural classification

(b) PRA system of classification (Group index method)

(c) Unified soil classification System

(d) Indian Standard Soil classification system

2. Stone Aggregates:

Stone aggregate, or mineral aggregate, as it is called, is the most important component of the materials used in the construction of roads. These aggregates are derived from rocks, which are formed by the cementation of minerals by the forces of nature.

Stone aggregates are invariably derived by breaking the naturally occurring rocks

to the required sizes. They are used for granular bases, sub-bases, as part of bituminous mixes and cement concrete; they are also the primary component of a relatively cheaper road, called water-bound macadam.

A study of the types of aggregates, their properties, and the tests to determine their suitability for a specific purpose is of utmost importance to a highway engineer. Properties such as strength and durability of aggregates are generally influenced by their origin of occurrence, mineral constituents and the nature of the bond between the constituents.

Geological Classification of Rocks:

Geologically speaking, rocks are classified into the following categories:

(a) Igneous Rocks:

These are formed by the cooling, solidification and crystallisation of molten rock on the earth's crust at different depths. The minerals, their proportions and the rate of cooling of the magma have a bearing on the strength characteristicsof the rock.

Igneous rocks are, in general, stronger than the other two types. Granite, diorite and gabbro are intrusive rocks which form at deep layers in the earth's crust.

Basalt (or trap), andesite, rhyolite and dolerite are extrusive rocks which from atthe top layers of the earth's crust.

(b) Sedimentary Rocks:

Fine material or rock fragments and particles transported by water or wind and deposited in layers, get hardened in course of time to form sedimentary rocks (the time required is on geologic scale). They consist of a layered structure; the rock beds are stratified, they may be porous, and have relatively low strength.

Examples of siliceous variety are sandstone and argillite; those of calcareous variety are limestone and dolomite.

(c) Metamorphic Rocks:

These are formed by the modification and re-crystallisation of igneous rocks and sedimentary rocks by geological and natural agents such as temperature, pressure, moisture, humidity, and movement of rock beds.

Major changes occur in geologic time and form foliations. This kind of foliated structure makes these rocks comparably weaker than igneous rocks. Popular examples of metamorphic rocks are gneiss (from granite), slate (from shale) andschist.

Examples of un-foliated types are marble (from limestone) and quartzite (from sandstone). (Marble and gneiss are used for flooring and face work in buildings.)

Desirable Properties of Sand Aggregates:

The following properties are desirable in soil aggregates used the construction of roads:

(i) Strength:

It is the resistance to crushing which the aggregates used in road construction, especially in the top layers and wearing course, have to withstand the stresses due to wheel loads of the traffic in addition to wear and tear.

(ii) Hardness:

It is the resistance to abrasion of the aggregate at the surface. The constant rubbing or abrading action between the tyres of moving vehicles and the exposed aggregate at the road surface should be resisted adequately.

(iii) Toughness:

This is the resistance to impact due to moving traffic. Heavily loaded trucks and other vehicles cause heavy impact loads on the road surface while movingat high speeds, and while accelerating and decelerating. Even steel-typed vehicles, though moving slow, cause heavy impact on the aggregates exposed atthe surface. Hence, resistance to such impact forces is a desirable quality.

(iv) Durability:

It is the resistance to the process of disintegration due to the weathering action of the forces of nature. The property by virtue of which the aggregate withstands weathering is called soundness. This is also a desirable property.

(v) Cementation:

It is the ability of the aggregate to form its own binding material under traffic,providing resistance to lateral displacement. Limestone and laterite are examples of stones with good cementing quality. This becomes important in the case of water-bound macadam roads.

(vi) Appropriate Shape:

Aggregates maybe either rounded, cubical, angular, flaky, or elongated. Each shape is appropriate for a certain use. Too flaky and too elongated aggregateshave less strength and durability; so they are not preferred in road construction. Rounded aggregates are good for cement concrete because of the workabilitysuch aggregates provide. Cubical or angular aggregates have good interlockingproperties; since flexible pavements derive their stability due to interlocking,such aggregates are the preferred type for construction. Thus, the appropriateshape for a particular use is also a desirable property.

(vii) Adhesion with Bitumen:

The aggregates used in bituminous pavements should have less affinity to water than to bitumen; otherwise, the bituminous coating on the surface of the aggregate will get stripped off in the presence of water. So, hydrophobic characteristic is a desirable property for aggregates to be used in the construction of bituminous roads.

(viii) Attrition:

This is mutual rubbing of aggregates under traffic; adequate resistance to attrition is a desirable property.

(ix) Texture:

This is a measure of the degree of fineness or smoothness of the surface ofthe aggregate.

Gravels from river beds are fairly smooth; as a rule, fine grained rock is highly resistant to wear and is preferred for surface courses.

3. Bituminous Materials:

Bitumen was used as a bonding and water-proofing agent thousands of years ago. However, the use of bitumen for road-making picked up only in the nineteenth century. As the quest for fuels like petroleum to run automobilesgrew and the distillation of crude oil emerged as a major refining industry, the residues known as bitumen and tar found increasing use in constructing bituminous surfaces, which provided superior riding surface.

The definition for the term, bitumen, given by the American Society for Testing Materials (ASTM) runs thus:

"Bitumen is a hydrocarbon material of natural or pyrogenous origin, whichis in a gaseous, liquid, semi-solid, or solid state, and which is completelysoluble in carbon disulphide (CS_2)."

Of course, bitumen is found to be soluble to a large extent in carbon tetrachloride (CCl_4) also. Bitumen is a complex organic compound and occurs either as such in nature or can be obtained during the distillation of petroleum; itis generally non-volatile and resistant to most acids, alkalis and salts.

Bitumen occurring in nature as rock intrusions invariably contains inert inorganic materials or minerals; in such a case it is called asphalt. It is also found in lakes (as in Trinidad), in which case it is called lake asphalt. However, in American terminology, bitumen itself is termed asphalt, irrespective of whether it contains inorganic/mineral matter or not. In India, the British terminology is used for the terms bitumen and asphalt.

Important Properties of Bitumen:

1. Predominantly hydrocarbons, with small quantities of sulphur, nitrogenand metals.
2. Mostly (up to 99.9%) soluble in carbon disulphide (CS_2), and insoluble inwater.
3. Softens on heating and gets hardened on cooling.
4. Highly impermeable to water.
5. Chemically inert and unaffected by most acids, alkalis and salts.
6. No specific boiling point, melting point or freezing point; a form of 'softening point' is used in their characterisation.
7. Although generally hydrophobic (water repellent), they may be made hydrophilic (water liking) by the addition of a small quantity of surface- active agent.
8. Most bitumens are colloidal in nature.

Desirable Properties of Bitumen as a Road Material:

1. Workability – Bitumen should be fluid enough at the time of mixing so that the aggregates are fully coated by the binder. Fluidity is achieved either by heating or by cutting back with a thin flux or by emulsifying the bitumen.
2. Durability – There should be little change in viscosity within the usual range of temperatures in the locality.
3. Volatile constituents in bitumen should not be lost excessively at higher temperatures to ensure durability.
4. It should have enough ductility to avoid brittleness and cracking.
5. Strength and adhesion – The bitumen should have good affinity to the aggregates and should not be stripped off in the continued presence of water.
6. Cost-effectiveness.

A few more terms relating to bitumen/asphalt are:

Straight-Run Bitumen: Bitumen derived from the refining of petroleum for which the viscosity has not been adjusted by blending with flux oil or by softening with any cut-back oil or by any other treatment. It generally has high viscosity.

Asphalt Cement:

A binder consisting of bitumen, or a mixture of lake asphalt and bitumen or flux oils, specially prepared as per prescribed quality and consistency for direct use in paving, usually in the hot condition.

Oxidised or Blown Bitumen:

Bitumen obtained by further treatment of straight-run bitumen by running it, while hot, into a vertical column and blowing air through it. In this process, it attains a rubbery consistency with a higher softening point than before.

Cut-Back Bitumen:

Asphalt/bitumen dissolved in naphtha or kerosene to lower the viscosity and increase the workability.

Emulsified Bitumen:

A mixture in which asphalt cement, in a finely dispersed state, is suspended in chemically treated water.

Liquid Bitumen:

Include cut-backs in naphtha and kerosene, as also emulsified asphalts.

Flux-Oil:

A bituminous material, generally liquid, used for softening other bituminousmaterials.

Bitumen from Petroleum Refining:

The main source of bitumen is petroleum crude. Refining of petroleum crude involves fractional distillation. The crude oil is heated in a tube-still to about 200°C to 400°C and injected into a fractionating column. As the pressure is suddenly reduced, the volatile fractions with low boiling points get vaporised and go up the column, from where they are carried through condensers.

Gasoline, kerosene, diesel oil, and lubricating oils, constituting the light, medium and heavy distillates with gradually increasing boiling points, thus get collected. The heavy residue left at the bottom is collected as bitumen. Steam is injected into the fractionating column to help in the separation process of the fractions. The steam and vacuum distillation process is only a physical process and does not involve any chemical changes.

In modern refining processes, the distillation is carried out in stages. In the first stage, the temperature in the tube-still is kept relatively low (say 300°C to 350°C) and the light and medium fractions are separated in the fractionating column operating at atmospheric pressure.

The crude left is then passed through another still for subsequent transfer to another column operating under vacuum and injected with steam. The latest process dispenses with steam and relies on dry vacuum only, thus enabling a wide range of bitumen to be produced.

Paraffinic crudes yield, on distillation, an undesirable wax-like residue. Naphthenic crudes yield practically wax-free bitumen; crude from middle-east yields good bitumen. The heavy residue may be blown with air at high temperature in a converter to produce air-blown or oxidised bitumen.

They are stiff even at high atmospheric temperatures. Such bitumen are not used for pavements, but are good as roofing materials and water-proof paints. It is also used as filler material for cracks and joints in concrete pavements.

There are three types of cut-backs based on the diluent (dilutant or solvent)used:

1. Rapid-curing (RC) cutback – Bitumen blended with gasoline or naphtha,(highly

volatile, low viscosity)

2. Medium-curing (MC) cutback – Bitumen blended with kerosene or coal tarcreosote oil (medium viscosity)

3. Slow-curing (SC) cutback – Bitumen blended with gas oil (low viscosity,highly viscous)

Each of these has been categorized based on their initial kinematic viscosityvalues as follows:

1. RC 70, RC 250, RC 800, RC 3000

2. MC 30, MC 70, MC 250, MC 800, MC 3000

3. SC 70, SC 250, SC 800, SC 3000

Further details and specifications for these cutbacks are given in "IS: 217- 1988: Specification for cutback bitumen, Bureau of Indian Standards, NewDelhi, 1993".

Since cutbacks contain volatile solvent, some of which may enter water bodies and air, they may cause environmental pollution. Also, since the solvent is inflammable, it may increase the possibility of fire hazard and cause concerns related to safety during handling and application. Therefore, cutbacks are being gradually replaced by emulsions.

Bitumen Emulsions:

A bitumen emulsion is obtained by blending bitumen with water and an additive called an emulsifier. The emulsified suspension contains dispersedminute particles of bitumen (that is, oil in water). In a bituminous emulsion, bitumen is the 'dispersed' phase (minutely subdivided particles), while water is the 'continuous' phase in which it is not soluble. The amount of bitumen to be mixed with water may range from 40 to 70% depending upon the intended useof the suspension.

Based on the type of emulsifier used, the bitumen particles can be negatively charged or positively charged. If they are negatively charged, 'anionic bitumen emulsions' are obtained, and if they are positively charged, 'cationic emulsions' are got.

Fatty acids derived from mineral, vegetable or wood sources saponified with sodium or potassium hydroxide are used as emulsifiers for producing anionic emulsion. For cationic emulsions, the emulsifiers are generally amine salts produced by the reaction of organic amine or diamine with acetic acid or hydrochloric acid.

The type of emulsion should be selected based on the mineral composition of the aggregate used for the bituminous mix. For example, for an aggregate rich in silica (SiO_2) which has a strong electronegative charge on the surface, cationic emulsions are suitable with electropositive charge on the suspended bitumen particles. The mix then becomes electrostatically stable and produces a strong layer when compacted.

Bitumen emulsions, like cutback bitumens, are also classified into threetypes based on their setting times:

1. Rapid-setting emulsions (RS)

2. Medium-setting emulsions (MS)

3. Slow-setting emulsions (SS)

Setting, in this context, means separation of the emulsion. When the water in the emulsion evaporates, the minute bitumen particles in the emulsion coat the

surface of the aggregates; curing takes place, by which the compacted layer of the emulsion-aggregate mix hardens and attains strength. Therefore, rapid-setting emulsion sets and cures in a relatively quick manner.

"IS: 3117-2004: Anionic bitumen emulsions" covers anionic emulsions, while "IS: 8887- 2004: Cationic bitumen emulsions" covers cationic emulsions.

Setting and curing of emulsion mixes are affected by the following factors:

I. Gradation, dust, dampness, water absorption and mineral composition and surface charge of/on the aggregates.

II. Ingredients and quantity of the emulsion used.

III. Meteorological conditions like climate, weather, temperature, humidity, wind velocity, etc.

IV. Drainage conditions of the construction site.

Advantages of Emulsions:

1. Emulsions can be used under cold and damp weather conditions.
2. Strength properties of bitumen are preserved as they do not need hotmixing.
3. Better coating of aggregates due to low viscosity of the emulsion.
4. Ideal for patch repair work and sealing of cracks as no heating isrequired and better penetration into even minute cracks is possible.
5. Water-based nature of the emulsions makes them environment-friendly.
6. A lot of energy is conserved as there is no need for intensive heating(only warming is needed, if at all.)
7. **Limitations of Emulsions:**
1. The nature of the aggregate has to be verified before choosing an appropriate emulsion.
2. Setting time varies not only with the type of emulsion, but also with atmospheric conditions at the time of application.
3. Based on the particular need, care should be exercised in choosing the type of emulsion and the quantity needed for the desired grade of bituminous mix.
4. Storage time is relatively restricted.
5. Bitumen emulsions are more expensive than hot-mix bitumen.
6. In general, emulsion-based bituminous pavements using emulsions are not as good as hot- mix constructions for heavy traffic loads.

Tar:

Tar is a black or brown to black, viscous, non-crystalline material having binding property. This is, therefore, the other category of bituminous materials.

Tar is obtained from the destructive distillation of organic materials suchas coal, petroleum, oil, wood and peat, in the absence of air at about 1000°C. It is completely soluble in carbon tetrachloride (CCl_4). It contains more volatile

constituents than bitumen and is therefore more susceptible to change in temperature. Generally, tar is used for surface dressing on the wearing course since it has good adhesion in damp conditions.

Some more terms relating to tar are:

a. Coal tar – Tar produced by the destructive distillation of bituminous coal.

b. Coke-oven tar – A variety of coal tar obtained as a by-product from the destructive distillation of coal in the production of coke.

c. Oil-gas tar – A petroleum tar produced by cracking oils at high temperature in the production of oil-gas.

d. Water-gas tar – A petroleum tar produced by cracking oils at high temperature in the production of carburetted water-gas.

e. Refined tar – Produced from crude tar by distillation to remove water and to produce a residue of desired consistency.

f. Road tar – A tar refined in quality and consistency for use in paving of roads.

g. Pitch – Black or dark brown solid cementitious residue which gradually liquefies when heated and which is produced by distilling off the volatile constituents from tar.

Specifications for Road Tars:

Indian Standards classify road tars for paving purposes into five grades — RT1, RT2, RT3, RT4, and RT5, meant for specific purposes.

These are covered by "IS: 215-1995: Road tar: Specifications, Bureau of Indian Standards, New Delhi, 2000".

The grades and specific uses are given below in Table 6.12:

Table 6.12   (IS: 215-1995, 2000)

Grade		Specific uses
RT-1		Surface dressing for very cold weather conditions and at very high elevation on hill roads.
RT-2		Surface dressing under normal climatic conditions.
RT-3	(a)	Surface dressing and renewal coats.
	(b)	Pre-coating chippings; light chipping carpet.
RT-4		Premix tar macadam
RT-5		Grouting

Low Temperature Tar:

The coal-tar produced in the manufacture of coking coal requires carbonation at high temperatures above 1000°C. In view of the increasing demand for road tarsin recent years, a new technology known as low temperature carbonisation has come into vogue.

In this, the carbonisation of coal is carried out in the temperature range of 600°-750°C in a smokeless fuel process. The crude tar thus produced is successfully used for making road tars; these are known as low temperature tars.

Bitumen versus Tar:

A comparison of bitumen and tar is given below:

   i. Aggregates coated with tar exhibit lower stripping action than those coated with bitumen.

  ii. Tar is more susceptible to temperature than bitumen. It becomes liquid at relatively lower temperature.

 iii. Tar is not easily dissolved in petroleum solvents; so it can be preferredfor paving parking areas, where oils might drip from vehicles.

  iv. Since more setting time is required for tar, it may be processed at amixing plant and carried to the construction site.

   v. In view of the higher free carbon content, tar is more brittle thanbitumen.

  vi. As tars have more phenol content, they can get more easily oxidisedthan bitumen.

 vii. At higher temperatures, tar may be more easily affected than bitumen.

viii. As more time is required for tar to set, tar-paved roads need to beclosed to traffic for a longer time. Both bitumen and tar appear black in colour in a large mass, but appearbrown in thin films.

Tar-Bitumen Mixtures:

A mixture of tar and bitumen provides a binder of excellent quality as it has a decreased volume of insoluble benzene is decreased. Such mixtures have lower temperature susceptibility and reduced penetration value. Rheological properties of the binder also get altered. Generally, a mixture of tar and bitumen in equal proportions is considered to be an ideal binder.

Bituminous Mixes:

Bituminous mixes for paving purposes consist of coarse aggregate, fine aggregate, filler material, bitumen, and air voids, suitably proportional andblended to provide a strong, stable and durable pavement.

The main aim of mix design is to determine the optimum bitumen content that will hold the mineral aggregates of suitable gradation together as a compact layer that resists the traffic loads. The mix should have a certain minimum air voids to allow volume changes during service either because of temperature changes or repeated loading from the traffic.

Requirements of Bituminous Mixes:

The following are the important requirements of bituminous mixes for pavements:

(i) Stability:

This is the resistance to deformation under traffic loads; it is a function of inter-particle friction and cohesion offered by the bitumen binder. It is related to the density of the mix which is dependent on the voids content. The more the density, the more stable the mix; however, a minimum voids content is necessary to allow for volume changes which cannot be fully prevented.

(ii) Durability:

This is the resistance to weathering action and abrasion from traffic. Spalling, stripping and formation of pits, corrugations and potholes can result from weathering and traffic. Excessive strain may cause cracking or plastic failure.

(iii) Flexibility:

This is a measure of the resistance to long-term deformations and shapes of the road base, sub-base and subgrade; this depends on the flexural or bendingstrength of the pavement.

(iv) Skid Resistance:

The resistance of the surface of the pavement laid with the bituminous mix to skidding of the tyres of vehicles is called skid resistance. The surface texture should be such as to provide grip or friction even under wet conditions. This is important in the prevention of accidents.

(v) Workability:

This is the ease with which the mix can be placed in position and compacted. It depends on the aggregate characteristics like the size, shape texture and gradation, bitumen content and nature of the bituminous material.

(Vi)Economy:

The overall cost in achieving the desired qualities of the mix and the pavement should be a minimum, consistent with quality

.

The desired qualities of the bituminous mixes, therefore, have to be achieved by:

1. Using good quality aggregate, which is hydrophobic and has rough surface texture, with appropriate grading and voids content.

2. Using bituminous binder of the correct quality and consistency based on the specific purpose for which the pairing mix is intended.

3. Controlling the voids content and the bitumen content to achieve thedesirable qualities listed above.

4. Cement, Cement Mortar and Cement Concrete:

Cement concrete is a versatile material which has revolutionised civil engineering construction during the twentieth century. A fresh cement concrete mix consists of cement, mineral aggregates (coarse aggregate and fine aggregate), and water.

A well-designed cement concrete mix sets and hardens due to the binding property of the cements, forms a mix with minimum void space and on curing with water, provides a strong, stable and durable pavement for a highway, resisting repetitive impact from wheel loads and also withstanding adverse environmental conditions.

Thus, a cement concrete pavement is the most superior highway construction primarily from the point of view of strength and durability. The ingredients of the concrete mix, viz., the coarse aggregate (broken stone) and fine aggregate (sand) have to be selected carefully to satisfy the desirableproperties for concrete-making. Potable water is generally considered satisfactory making cement concrete. Cement is used also as an additive to soil to produce soil-cement used as the primarily material in the construction of low-cost roads.

Cement:

Cement is the most important ingredient of cement concrete or cementmortar (cement mortar is a suitable mixture of cement and fine aggregate orsand in appropriate proportions).

Cement mixed with water becomes a paste and spreads over the aggregates forming a thin film; chemical reactions take place leading to the formation of silicates and aluminates. Subsequently, setting takes place and in the presence of water, hydration takes place leading to hardening of the concrete.

The most common cement is what is now known as the Ordinary Portland Cement (OPC). Calcareous and silicate compounds are blended and heated to high temperatures (1500°C) to form clinkers of new chemical compounds,which when ground to fine particles result in 'cement'.

The primary ingredients of cement are:

(i) Tricalcium silicate ($3CaO.SiO_2$) ≈ 50%

(ii) Dicalcium silicate ($2CaO.SiO_2$) ≈ 22%

(iii) Tricalcium aluminate ($3CaO.Al_2O_3$) $\approx$ 9%

(iv) Tetracalcium aluminoferrite ($4CaO.Al_2O_3.Fe_2O_3$) $\approx$ 9%

(v) Miscellaneous compounds $\approx$ 10%

The silicates contribute to the immediate strength gain while the other ingredients are responsible for the long-term strength gain. The properties of cement can be modified by blending it in different admixtures in the manufacturing process.

The following are the different types cements widely used for specific purposes in India:

i. Ordinary Portland cement (OPC)

ii. Rapid hardening cement

iii. High alumina cement

iv. Low heat cement

v. Portland blast furnace slag cement

vi. White cement

Function of soil as highway Subgrade

Soil is used for the construction of the bottom most layer of the pavement, i.e. sub-grade. Here is a short details of the sub-grade and its function.:
Sub-grade is the layer of the pavement whose main function is to support theupper layers of the pavement and to provide the good drainage facility to the infiltrating rain water. It has to act as a single structure along with other layers of the pavement.

➤ Soil is compacted to its maximum dry density which can be achieved by using the optimum moisture content and the methods of compaction control. Strength has to be ensured which is required for the given design thickness of the pavement.

➤ Strength analysis and the thickness of pavement are inter linked because more thickness of the pavement is needed if the soil is weak but if the soil possess a good strength then less thickness is needed.

This is ensured by using the CBR (California Bearing Ratio) Test which is produced or was first used by the California State Highway Department. Using the CBR test and the empirical charts you can find out the thickness of the flexible pavement required above the sub-grade.

Tests On Soil:

Sub grade soil is an integral part of the road pavement structure as it providesthe support to the pavement from beneath. The sub grade soil and its properties are important in the design of pavement structure. The main function of the sub grade is to give adequate support to the pavement and for this the sub grade should possess sufficient stability under adverse climatic and loading conditions. Therefore, it is very essential to evaluate the sub grade by conducting tests.

The tests used to evaluate the strength properties of soils may be broadlydivided into three groups:

- Shear tests
- Bearing tests
- Penetration tests

Shear tests are usually carried out on relatively small soil samples in the laboratory. In order to find out the strength properties of soil, a number of representative samples from different locations are tested. Some of the commonly known shear tests are direct shear test, triaxial compression test, and unconfined compression test.

Bearing tests are loading tests carried out on sub grade soils in-situ with a load bearing area. The results of the bearing tests are influenced by variations inthe soil properties within the stressed soil mass underneath and hence the overall stability of the part of the soil mass stressed could be studied.

Penetration tests may be considered as small scale bearing tests in which thesize of the loaded area is relatively much smaller and ratio of the penetration to the size of the loaded area is much greater than the ratios in bearing tests. Thepenetration tests are carried out in the field or in the laboratory.

California Bearing Ratio: methods of finding CBR valued in the laboratoryand at site and their significance

California Bearing Ratio Test

In most cases, CBR decreases as the penetration increases. The ratio at 2.5 mm penetration is used as the CBR. In some case, the ratio at 5 mm may be greater than that at 2.5 mm. If this occurs, the ratio at 5 mm should be used. TheCBR is a measure of resistance of a material to penetration of standard plunger under controlled density and moisture conditions. The test procedure should be strictly adhered if high degree of reproducibility is desired. The CBR test may be conducted in re-moulded or undisturbed specimen in the laboratory. The test is simple and has been extensively investigated for field correlations of flexible pavement thickness requirement.

Test Procedure

- The laboratory CBR apparatus consists of a mould 150 mm diameter witha base plate and a collar, a loading frame and dial gauges for measuring the penetration values and the expansion on soaking.
- The specimen in the mould is soaked in water for four days and the swelling and water absorption values are noted. The surcharge weight is placed on the top of the specimen

in the mould and the assembly is placedunder the plunger of the loading frame.

- Load is applied on the sample by a standard plunger with dia of 50 mm atthe rate of 1.25 mm/min. A load penetration curve is drawn. The load values on standard crushed stones are 1370 kg and 2055 kg at 2.5 mmand 5.0 mm penetrations respectively.

CBR value is expressed as a percentage of the actual load causing the penetrations of 2.5 mm or 5.0 mm to the standard loads mentioned above.Therefore,

$$CBR = \frac{\text{load carries by specimen}}{\text{load carries by standard specimen}} \times 100$$

Two values of CBR will be obtained. If the value of 2.5 mm is greater than that of 5.0 mm penetration, the former is adopted. If the CBR value obtained from test at 5.0 mm penetration is higher than that at 2.5 mm, then the test is to be repeated for checking. If the check test again gives similar results, then higher value obtained at 5.0 mm penetration is reported as the CBR value. The average CBR value of three test specimens is reported as the CBR value of the sample.
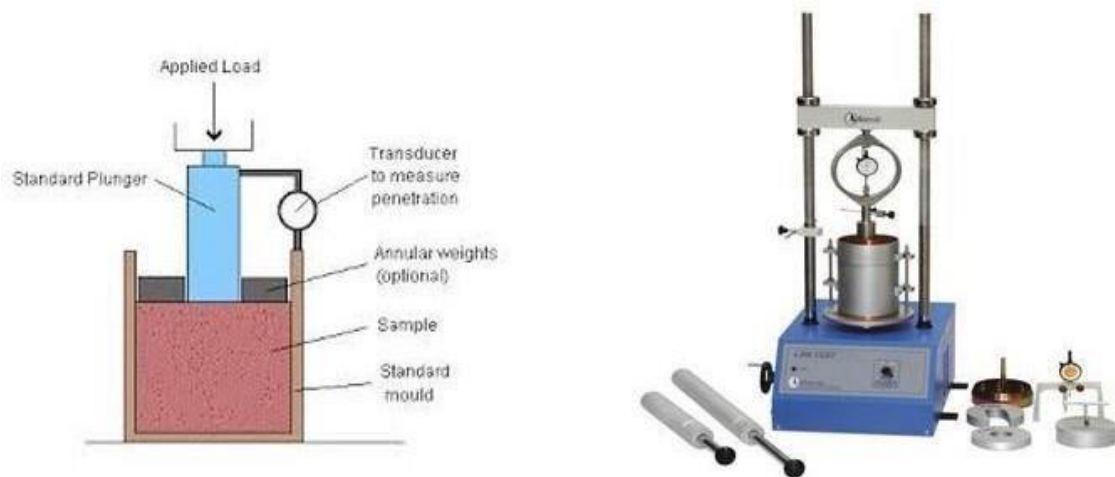
Fig.CBR Testing machine

Tests on aggregates:

Abrasion test:

Due to the movement of traffic, the road stones used in the surfacing course are subjected to wearing action at the top. Resistance to wear or hardnessis hence an essential property for road aggregates especially when used in wearing course. Thus road stones should be hard enough to resist the abrasion due to the traffic.
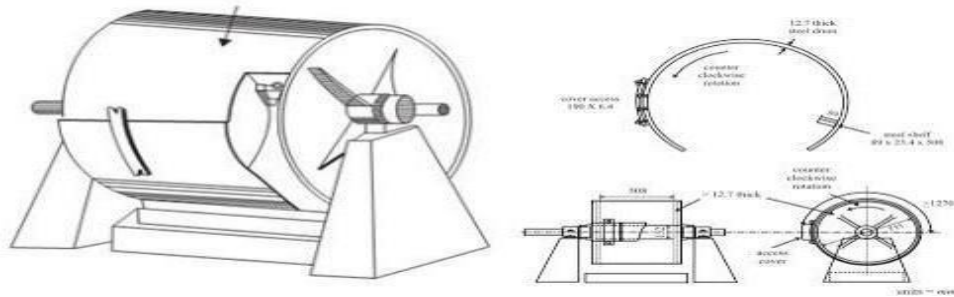
Fig. Los Angles Abrasion Machine

Procedure:

1) Take the clean and dried aggregates in an oven at 105-110° C.
2) Sieve the given aggregates in sieve size 20-12.5mm and weigh that aggregate in 2.5kg.
3) Again sieve the aggregate in sieve size is 12.5-10mm and take that aggregates in 2.5 k. i.e., W1 gm (2.5+2.5=5kg)
4) Pour the given taking aggregates into the los angles abrasion machine.
5) Put the steel balls into the abrasion machine after pouring the aggregates.
6) Start the machine and rotating the drum for 100 revolutions and stop the machine.
7) After stopping the machine, take out the aggregates and sieve the aggregates in 1.7mm sieve size and take the retained aggregates and note down its weight i.e, W2 gm.
8) Then, Los Angles Abrasion value= (W1-W2/W1)X100 %

Impact test:

Toughness is the property of a material to resist impact. Due to traffic loads the road stones are subjected to the pounding action or impact and there is possibility of stones breaking into smaller pieces. The road stones should therefore be tough enough to resist fracture under impact. A test designed to evaluate the toughness of stones i.e. the resistance of the stones to fracture under repeated impacts may be an impact test for road aggregate.

The aggregate impact value indicates a relative measure of the resistance of an aggregate to a sudden shock or an impact, which in some aggregate differs from its resistance to a slow compressive load. The method of tests specifies the procedure for determining the aggregate impact value of coarse aggregate.
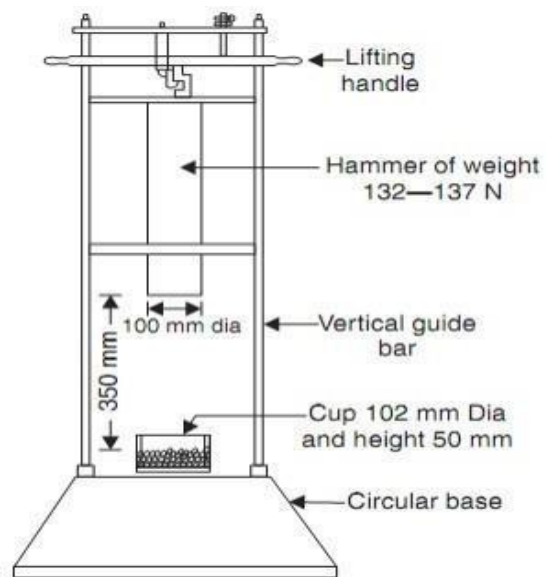
Fig. Impact test Machine

Procedure:

i. Take clean and dry aggregate and sieve on IS 12.5 mm and 10.00 mm sieve.

ii. Collect the aggregate passing IS 12.5mm sieve and retained on IS 10.0mm Sieve.

iii. Find the weight of empty cylindrical measure. Let the weight be 'a' g.

iv. Fill the aggregate in the cylindrical measure in three layers, tamping each layer 25 times with the rounded end of the tamping rod.

v. Roll the tamping rod over aggregate surface and remove excessaggregate, if any.

vi. Find the weight of the cylindrical measure with aggregate. Let the weightbe 'b' g. Thus the weight of aggregate = W1 = (b-a)

vii. Transfer all the aggregate from the cylindrical measure to the test cylinderin one layer and tamp the layer 25 times with the rounded end of the tamping rod.

viii. Fix the test cylinder firmly to the base of the impact tester.

ix. Adjust the height of fall of the plunger to 380+ 5mm and set the blow counter to zero.

x. Lift the plunger gently and allow it to drop. This is one blow. Give 15such blows.

xi. Take out the test cylinder and sieve the crushed material on IS 2.36 mm sieve. Find the weight of material passing the sieve. Let weight be W2 g.

xii. Find the weight of aggregate retained on this sieve. Let the weight be W3 g. Then,

Aggregate impact value = W2 / W1 * 100 % And

percentage of dust = W3 / W1* 100 %

Crushing strength test

The Principal mechanical properties required in road stones are (i) Satisfactory resistance to crushing under the roller during construction and (ii) adequate resistance to surface abrasion under traffic. Also stresses under rigid tyre rimsof heavily loaded animal drawn vehicles are high enough to consider the crushing strength of road aggregate as an essential requirement in India. Crushing strength of road aggregate may be determined either on aggregate or on cylindrical specimens cut out of rocks. These two tests are quite different is not only the approach but also is the expression of the results.

Aggregate used in road construction, should be strong enough to resist crushing under traffic wheel loads. If the aggregate are weak, the stability of the pavement stretches is likely to be adversely affected, the strength of coarse

aggregate is assessed by aggregate crushing test. The aggregate crushing value provides a relative measure of resistance to crushing under gradually applied compressive load. To achieve a high quality of pavement, aggregate possessing low aggregate value should be preferred.
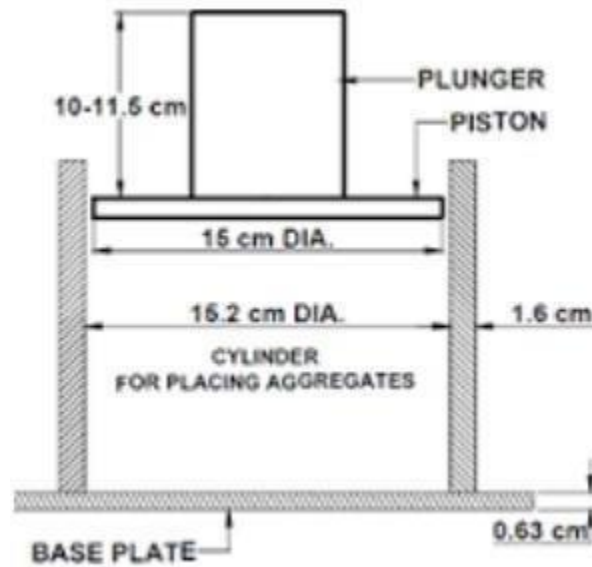
Fig. Arrangement of Crushing strength test

Testing Procedure:

i. Select clean and dry aggregate passing through IS 12.5 mm and retained on IS 10.0 mm sieve.

ii. Weight the empty cylindrical measure. Let the weight be 'a' g

iii. Fill the aggregate in the cylindrical measure in three layers, tamping each layer 25 times with the rounded end of the tamping rod. Weigh the cylindrical measure with aggregate. Let the weight be 'b' grams. Thus the weight of aggregate = W1 g

iv. Transfer the aggregate into the steel cylinder again in three layers tamping each layer 25 times

v. Place the plunger in the steel cylinder such that the piston rests horizontally over the aggregate surface.

vi. Keep the assembly of steel cylinder with plunger in the compressiontesting machine.

vii. Set the pointer to read zero and apply the compressive load of 40 tonnes.

viii. Stop the machine. Take out the assembly.

ix. Sieve the crushed material on IS 2.36 mm sieve and find he weight ofmaterial passing this sieve. Let the weight be W2 g.

x. Then Aggregate crushing value=W2 / W1*100 %

Specific Gravity and Water Absorption Test on Aggregates

The specific gravity of an aggregate is considered to be a measure of strength or quality of the material. Stones having low specific gravity are generally weaker than those with higher specific gravity values. The specific gravity test helps in the identification of stone.

Water absorption gives an idea of strength of rock stones having more water absorption are more porous in nature and are generally considered unsuitable unless they are found to be acceptable based on strength, impact and hardness.

Testing Procedure :

1. Take about 2kg of given aggregates passing IS 20mm sieve and retainedon 10mm sieve.
2. Keep the aggregate in density basket and then keep the basket in water.
3. Allow the aggregate and basket to be in water for 24 hours.
4. After 24 hours find the suspended weight of basket with aggregate.
5. Remove the basket out of water and remove the aggregate.
6. Keep the empty basket back in water and find the suspended weight.
7. Wipe the surface of aggregate using a cotton cloth to make them surface dry.
8. Find the weight of surface dry aggregate in air.
9. Keep the aggregate in oven at 110° C for 24 hours.
1
10. Now find the weight of dried aggregate in air.
11. Then specific gravity and Water absorption is calculated from the relation:

$$\text{Specific gravity} = \frac{W4}{W3-(W1-W2)}$$

$$\text{Water absorption} = \frac{W3-W4}{W4} * 100 \%$$

Soundness test

To study the resistance of aggregates to weathering action, by conducting accelerated weathering test cycle.

Testing Procedure

1. In order, to quicken the effects of weathering due to alternate wet-dry or freeze-thaw cycles in the laboratory, the resistance to disintegration of aggregate is determined by using saturated solution of sodium sulphate or magnesium sulphate.

2. Clean, dry aggregates of specified size is weighed and counted. Then immersed in the saturated solution of sodium sulphate or magnesium sulphate for 16 to 18 hours.

3. Then the aggregates are dried in an oven at 105-110°C to a constant weight, thus making one cycle of immersion and drying.

4. The number of such cycles is decided by prior agreement and then the specimens are tested. After completing the final cycle, the sample is driedand each fraction of aggregate is examined visually to see if there is any evidence of excessive splitting, crumbling or disintegration of the grains.

5. Sieve analysis is carried out to note the variation in gradation from original. The coarse aggregate fraction of each size range is sieved on specified sieve sizes.

Desirable value

IRC has specified 12percent as the maximum permissible loss in soundness test after 5 cycles with sodium sulphate, for the aggregate to be used in bituminous surface dressing, penetration macadam and bituminous macadam constructions.

UNIT 4-ROAD PAVEMENTS

Introduction:

A highway pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade. The pavement structure should be able to provide a surface of acceptable riding quality, adequate skid resistance, favorable light reflecting characteristics, and low noise pollution. The ultimate aim is to ensure that the transmitted stresses due to wheel load are sufficiently reduced, so that they will not exceed bearing capacity of the sub-grade. Two types of pavements are generally recognized as serving this purpose, namely flexible pavements and rigid pavements. This chapter gives an overview of pavement types, layers, and their functions, and pavement failures. Improper design of pavements leads to early failure of pavements affecting the riding quality.

Requirements of a pavement

An ideal pavement should meet the following requirements:

- Sufficient thickness to distribute the wheel load stresses to a safe value on the sub-grade soil,
- Structurally strong to withstand all types of stresses imposed upon it,
- Adequate coefficient of friction to prevent skidding of vehicles,
- Smooth surface to provide comfort to road users even at high speed,
- Produce least noise from moving vehicles,
- Dust proof surface so that traffic safety is not impaired by reducing visibility,
- Impervious surface, so that sub-grade soil is well protected, and
  Long design life with low maintenance cost.

Types of pavements

The pavements can be classified based on the structural performance into two, flexible pavements and rigid pavements. In flexible pavements, wheel loads are transferred by grain-to-grain contact of the aggregate through the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road). On the contrary, in rigid pavements, wheel loads are transferred to sub-grade soil by flexural strength of the pavement and the pavement acts like a rigid plate (e.g. cement concrete roads). In addition to these, composite pavements are also available. A thin layer of flexible pavementover rigid pavement is an ideal pavement with most desirable characteristics. However, such pavements are rarely used in new construction because of high cost and complex analysis required.

Flexible pavements

Flexible pavements will transmit wheel load stresses to the lower layers by grain-to-grain transfer through the points of contact in the granular structure(see Figure).
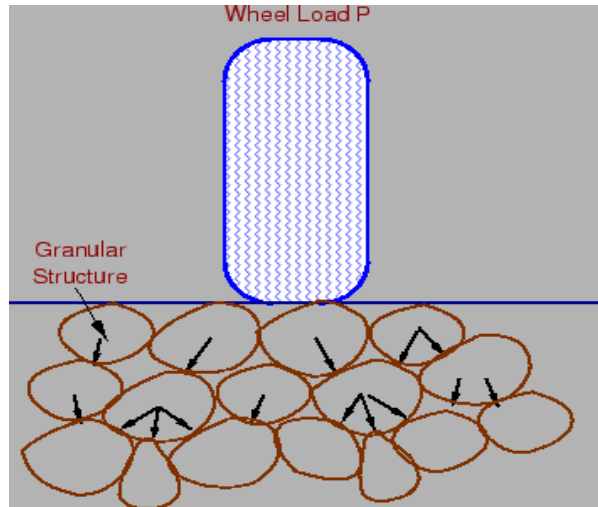
Figure : Load transfer in granular structure

The wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth. Taking advantage of this stress distribution characteristic, flexible pavements normally has many layers. Hence,the design of flexible pavement uses the concept of layered system. Based on this, flexible pavement may be constructed in a number of layers and the top layer has to be of best quality to sustain maximum compressive stress, in addition to wear and tear. The lower layers will experience lesser magnitude of stress and low quality material can be used. Flexible pavements are constructed using bituminous materials. These can be either in the form of surface treatments (such as bituminous surface treatments generally found on low volume roads) or, asphalt concrete surface courses (generally used on high volume roads such as national highways). Flexible pavement layers reflect the deformation of the lower layers on to the surface layer (e.g., if there is any undulation in sub-grade then it will be transferred to the surface layer). In the case of flexible pavement, the design is based on overall performance of flexible pavement, and the stresses produced should be kept well below the allowable stresses of each pavement layer.

Types of Flexible Pavements

The following types of construction have been used in flexible pavement:

- Conventional layered flexible pavement,
- Full - depth asphalt pavement, and
- Contained rock asphalt mat (CRAM).

Conventional flexible pavements are layered systems with high quality expensive materials are placed in the top where stresses are high, and lowquality cheap materials are placed in lower layers.

Full - depth asphalt pavements are constructed by placing bituminous layers directly on the soil sub-grade. This is more suitable when there is high traffic and local materials are not available.

Contained rock asphalt mats are constructed by placing dense/open graded aggregate layers in between two asphalt layers. Modified dense graded asphalt concrete is placed above the sub-grade will significantly reduce the vertical compressive strain on soil sub-grade and protect from surface water.

Typical layers of a flexible pavement

Typical layers of a conventional flexible pavement includes seal coat, surface course, tack coat, binder course, prime coat, base course, sub-base course, compacted sub-grade, and natural sub-grade (Figure 2).

Seal Coat:
Seal coat is a thin surface treatment used to water-proof the surface and to provide skid resistance.

Tack Coat:
Tack coat is a very light application of asphalt, usually asphalt emulsion diluted with water. It provides proper bonding between two layer of binder course and must be thin, uniformly cover the entire surface, and set very fast.

Prime Coat:
Prime coat is an application of low viscous cutback bitumen to an absorbent surface like granular bases on which binder layer is placed. It provides bonding between two layers. Unlike tack coat, prime coat penetrates into the layer below, plugs the voids, and forms a water tight surface.
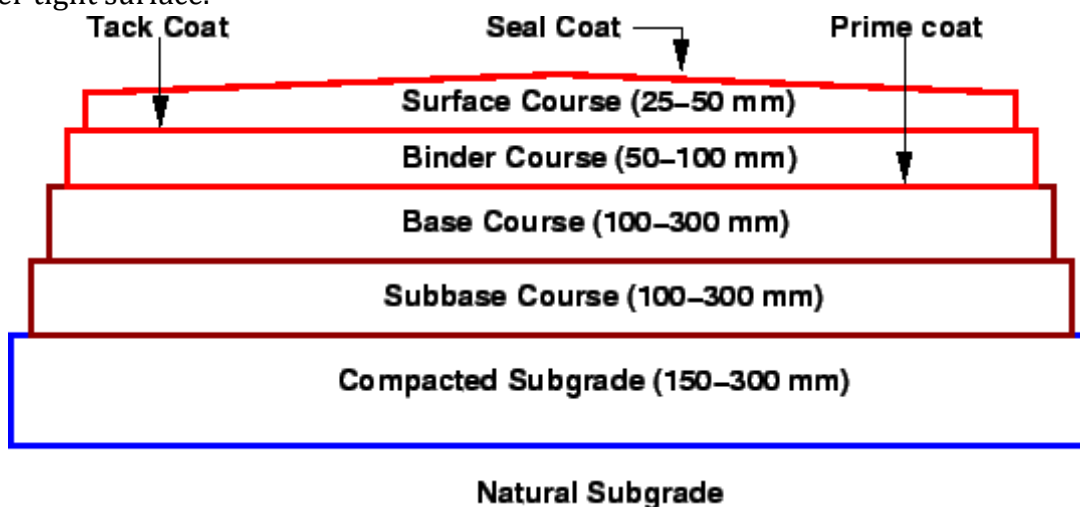
Figure: Typical cross section of a flexible pavement

Surface course

Surface course is the layer directly in contact with traffic loads and generally contains superior quality materials. They are usually constructed with dense graded asphalt concrete (AC). The functions and requirements of this layer are:

- It provides characteristics such as friction, smoothness, drainage, etc. Also it will prevent the entrance of excessive quantities of surface water into the underlying base, sub-base and sub-grade,
- It must be tough to resist the distortion under traffic and provide a smooth and skid- resistant riding surface,
- It must be water proof to protect the entire base and sub-grade from the weakening effect of water.

Binder course

This layer provides the bulk of the asphalt concrete structure. It's chief purpose is to distribute load to the base course The binder course generally consists of aggregates having less asphalt and doesn't require quality as high as the surface course, so replacing a part of the surface course by the binder course results in
more economical design.

Base course

The base course is the layer of material immediately beneath the surface of binder course and it provides additional load distribution and contributes to the sub-surface drainage It may be composed of crushed stone, crushed slag, and
other untreated or stabilized materials.

Sub-Base course

The sub-base course is the layer of material beneath the base course and the primary functions are to provide structural support, improve drainage, and reduce the intrusion of fines from the sub-grade in the pavement structure If the base course is open graded, then the sub-base course with more fines can serve as a filler between sub-grade and the base course A sub-base course is not always needed or used. For example, a pavement constructed over a high quality, stiff sub-grade may not need the additional features offered by a sub- base course. In such situations, sub-base course may not be provided.

Sub-grade

The top soil or sub-grade is a layer of natural soil prepared to receive the stresses from the layers above. It is essential that at no time soil sub-grade is overstressed. It should be compacted to the desirable density, near the optimum moisture content.

Failure of flexible pavements

The major flexible pavement failures are fatigue cracking, rutting, and thermal cracking. The fatigue cracking of flexible pavement is due to horizontal tensile strain at the bottom of the asphaltic concrete. The failure criterion relates allowable number of load repetitions to tensile strain and this relation can be determined in the laboratory *fatigue test* on asphaltic concrete specimens. Rutting occurs only on flexible pavements as indicated by permanent deformation or rut depth along wheel load path. Two design methods have been used to control rutting: one to limit the vertical compressive strain on the top of subgrade and other to limit rutting to a tolerable amount (12 mm normally). Thermal cracking includes both low-

Rigid pavements

Rigid pavements have sufficient flexural strength to transmit the wheel load stresses to a wider area below. A typical cross section of the rigid pavement is shown in Figure 3. Compared to flexible pavement, rigid pavements are placed either directly on the prepared sub-grade or on a single layer of granular or stabilized material. Since there is only one layer of material between the concrete and the sub-grade, this layer can be called as base or sub-base course.
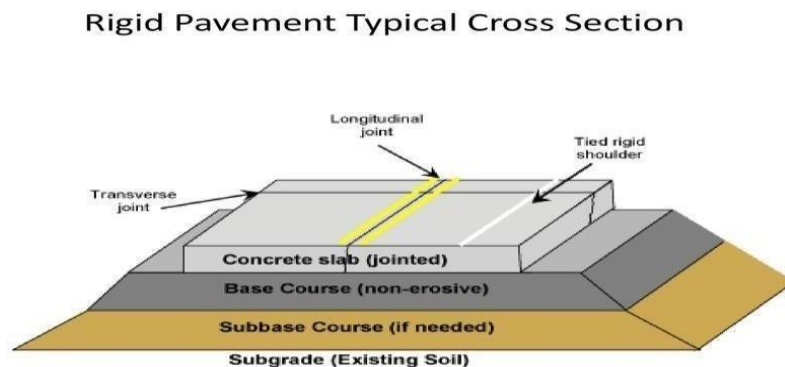
temperature cracking and thermal fatigue cracking.

Figure 3: Typical Cross section of Rigid pavement

In rigid pavement, load is distributed by the slab action, and the pavement behaves like an elastic plate resting on a viscous medium . Rigid pavements are constructed by Portland cement concrete (PCC) and should be analyzed by platetheory instead of layer theory, assuming an elastic plate resting on viscous foundation. Plate theory is a simplified version of layer theory that assumes the concrete slab as a medium thick plate which is plane before loading and to remain plane after loading. Bending of the slab due to wheel load and temperature variation and the resulting tensile

and flexural stress.
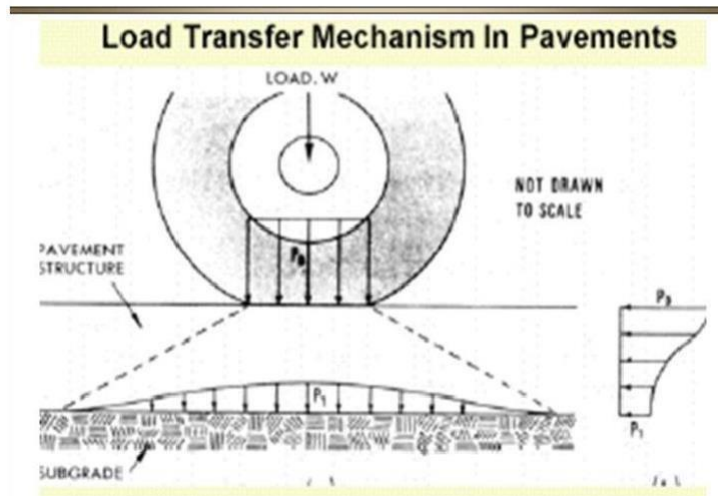
Figure 4 : Load transfer in rigid pavemet

Types of Rigid Pavements

Rigid pavements can be classified into four types:
- Jointed plain concrete pavement (JPCP),
- Jointed reinforced concrete pavement (JRCP),
- Continuous reinforced concrete pavement (CRCP), and
- Pre-stressed concrete pavement (PCP).

Jointed Plain Concrete Pavement:

are plain cement concrete pavements constructed with closely spaced contraction joints. Dowel bars or aggregate interlocks are normally used forload transfer across joints. They normally has a joint spacing of 5 to 10m.

Jointed Reinforced Concrete Pavement:

Although reinforcements do not improve the structural capacity significantly, they can drastically increase the joint spacing to 10 to 30m. Dowel bars are required for load transfer. Reinforcements help to keep the slab together even after cracks.

Continuous Reinforced Concrete Pavement:

Complete elimination of joints are achieved by reinforcement.

Failure criteria of rigid pavements:

Traditionally fatigue cracking has been considered as the major, or only criterion for rigid pavement design. The allowable number of load repetitions to cause fatigue cracking depends on the stress ratio between flexural tensile stress and concrete modulus of rupture. Of late, pumping is identified as an important failure criterion. Pumping is the ejection of soil slurry through the joints and cracks of cement concrete pavement, caused during the downward movement of slab under the heavy wheel loads. Other major types of distress in rigid pavements include faulting,

spalling, and deterioration.

Components of rigid pavement and there Functions:
1. Prepared soil subgrade.
2. Granular sub-base (GSB) or drainage layer.
3. Base course/ (DLC-Dry lean concrete).
4. CC pavement slab using PQC (paving quality concrete).

1. Prepared soil subgrade:

❖ The soil subgrade of rigid pavement consist of natural or selected soil from identified borrow pits fulfilling the specified requirements.

❖ The soil subgrade is well compacted to the desired density and to the required thickness.

❖ The soil subgrade is the lower most layer of the pavement structure which ultimately supports all other pavement layer and traffic loads.

❖ A good soil subgrade / well compacted and prepared soil subgrade gives long service life to the pavement.

2. Granular sub-base (GSB) or drainage layer:

❖ The GSB course has to serve as an effective drainage layer of the rigid pavement to prevent early failures due to excessive moisture content in the subgrade soil.

❖ Crushed stone aggregate are preferred In the granular subbase course as this material has high permeability and serves as a effective drainage layer.

❖ Coarse graded aggregates with low percent of fines (<5% finer than 75 micron sieve) will serve as good drainage layer.

❖ An effective drainage layer under the CC pavements have the following benefits:

   a. Increases in service life and improved performance of CC pavements.

   b. Prevention of early failures of the rigid pavements due to pumping and blowing.

   c. Protection of the subgrade against frost action in the frost suceptible areas.

3. Base course: (Dry lean concrete):

❖ The granular base course is generally provided under the CC pavement slab in low volume roads and also in roads with moderate traffic loads.

❖ On roads carrying heavy to very heavy traffic loads high quality base course materials such as dry lean concrete are preferred.

❖ In the base course of the CC pavement as they are designed for a life of 30 years or more with good maintenance. The CC pavement are expressed to provide a service life of 40 years or even more.

- ❖ The DLC layer provides a uniform support, high K value and excellent working platform for laying the PQC slab with a sensor paver.
- ❖ The suppression member is spread on the top of the DLC/ base course before laying the CC pavement slab.

4.CC pavement slab: (paving quality concrete (PQC):

- ❖ M-40 cement concrete mix with a minimum flexural strength of 45 kg/cm2 is recommended by the IRC for use in the CC-pavements of highways with heavy to very heavy traffic loads.
- ❖ The C pavement slab is extended to with stand the flexural stress caused by the heavy traffic loads and the warping effects in the CC pavements due to the temperature variations.
- ❖ The high quality CC mix with high flexural strength is used for the construction of PQC slab of the CC pavement.
- ❖ The CC pavement slab as considerable flexural strength and spreads the applied load/ wheel loads over a large area by slab action.
- ❖ The slab prevents the infiltration of excess surface water in to the sub-base.

Sub-grade   preparation:

This work shall consist of the preparation of subgrade in embankment, or in cut by scarifying, watering, compacting and shaping existing or previously placed material in accordance with these Specifications and to the lines, levels, grades, dimensions and cross sections shown on the Drawings or as required by the Engineer.

All subgrade material shall be from sources, which the contractor shall propose and which shall be approved by the Engineer. The material shall be free from roots, sods or other deleterious material and when compacted to 98% of maximum dry density determined in accordance with STP 4.3 shall have a 4 daysoaked CBR value of not less than 5%. Subgrade material shall satisfy the following criteria: · Liquid limit of soil fraction passing 0.425 mm sieve not to exceed 50% (STP 3.2) · Plasticity index of soil fraction passing 0.425 mm sievenot to exceed 15% (STP 3.2) Any subgrade material in cut or existing old embankment, which is found to be unsuitable, shall be removed and replaced as directed by the Engineer.

Construction Methods

The subgrade shall be prepared over the full width of the embankment including shoulders. Part width working may be allowed with the prior written approval of the Engineer. The subgrade shall be prepared in lengths of not less than 100 metres at any one time, unless otherwise approved by the Engineer. Subgrade material shall be scarified to a depth of 150 mm until the soil is fully loosened. Any lumps or clods shall be removed or broken to pass a 50 mm sieve. If the Drawings require the subgrade to be compacted for a depth greater than 150 mm, the work shall be carried out in more than one layer, the material in the upper layer being first removed in the case of road sections in cut. The moisture content of the sub-

grade material before compaction shall be within ±2% of the predetermined optimum moisture content established in accordance with STP 4.3 (Standard Compaction). The achieved dry density after compaction of the subgrade layer shall not be less than 98% of maximum dry density as determined in accordance with STP 4.3. When necessary, each layer, before being compacted, shall be allowed to dry or be watered to bring the moisture content to within ±2% of optimum to make possible its compaction to the required dry density. The material shall be so worked as to have a uniform moisture content through the entire layer. The subgrade material shall be compacted uniformly by use of adequate and appropriate compaction equipment. The compaction shall be done in a longitudinal direction along the embankment and shall generally begin at the outer edges and progress toward the centre in such a manner that each section receives equal compactive effort.

Samples to determine the compaction shall be taken regularly with a set of three samples for each 1,000 square metres of finished layer or as decided by the Engineer will be carried out according to STP 6.2. If the test results show that the density is less than the required dry density, the Contractor shall carry out further compaction to obtain at least the required dry density. The compacted subgrade layer shall be approved by the Engineer before the Contractor can commence a new layer.

The surface of the finished subgrade shall be neat and workmanlike and have the required form, super elevation, levels, grades and cross section. finished surface shall be constructed with a tolerance of 20 mm above or below the specified levels at any point.

Sub base Course:

A subgrade/sub base is made up of native soil that has been compacted to withstand the loads above it. It is a layer required in many structures such as pavements and slabs, although it needs to have certain characteristics. A subgrade might need special drainage structures to let water if it is composed of impermeable soil, and it should be graded to within plus or minus 1.5 inches of the specified elevation.

There is no consistency in regards to the terms of subbase and subgrade, but normally the subgrade is the native soil while the subbase is the layer of soil or aggregate on top of the subgrade.

Necessity of sub base:
❖ The need for a subbase - a layer of granular material placed on a prepared subgrade - depends on the frequency of heavy truck loadings. While mandatory for major highways, a subbase is seldom required for light- duty concrete pavements.
❖ Performance studies and surveys have shown the conditions for which a subbase is or is not required. With this information, an engineer can analyze

these conditions and rationally decide if a subbase layer is essential.

❖ The function of a subbase is to help prevent pumping of fine-grained, subgrade soils. Pumping, which leads to the loss of soil material beneath slab edges and joints, occurs when three factors exist in combination: pumpable soils, excess water under the pavement, and frequent heavy truck loads.

❖ In the absence of heavy truck traffic, which is the case for many streets, secondary roads, and parking lots, a subbase is not needed. For these pavements, good performance can be obtained by using appropriate subgrade preparation techniques aimed at providing uniform foundation support for the pavement.

Purpose of stabilization

The purpose of a stabilized base or sub-base layer is to provide a transitional load-bearing stratum between the pavement layer, which directly receives the wheel loadings of vehicular traffic, and the underlying sub-grade soil [1]. Stabilized base or sub-base materials may be used to provide support for either flexible or rigid pavements, but are more frequently used with flexible pavements. The key to strength development in stabilized base or sub-base mixtures is in the matrix that binds the aggregate particles together. The strength of the matrix is affected by the cementations material used in the mixture [2]. The amount of cementations material in a stabilized base or sub- base mix usually ranges from 5 to 10 percent by weight of the mix. The main concentration of the research is to determine various sand grain analyses and which of them is perfect for stabilization with cement to use instead of bricksor stone chips. This research also indicates the stability of the road with perfect sand cement mixing proportions.

MATERIALS

The components of a stabilized base or subbase mixture include aggregate, cementitious materials, and water.

Aggregates

Aggregates comprise the major portion of stabilized base. Normally, between80 to 95 percent by weight of a stabilized base or subbase mix may consist of aggregates. A wide range of different types and gradations of aggregates have been used in stabilized base and subbase mixtures. These include conventional aggregate sources, such as crushed stone or sand and gravel, and other aggregate materials, such as blast furnace slag, recycled paving materials, and bottom ash or boiler slag from coal-fired power plants. Reclaimed pavement materials have also been successfully recycled into stabilized base and subbase mixtures, as have some marginal aggregates. Aggregates used should have the proper particle size, shape, gradation, and particle strength to contribute to a mechanically stable mixture.

Cementitious Materials

The key to strength development in stabilized base or subbase mixtures is in the matrix that binds the aggregate particles together. The strength of the matrix is affected by the cementitious material used in the mixture. The amount of cementitious material in a stabilized base or subbase mix usually ranges from 5 to 10 percent by weight of the mix, but may in some cases comprise as much as up to 20 percent by weight if a lighter weight aggregate is used.

A number of different cementitious materials have been successfully used to bind or solidify the aggregate particles in stabilized base or subbase mixtures. The material that has been most frequently used is Portland cement.

In some parts of the United States, mainly west of the Mississippi River, fly ashfrom the burning of sub-bituminous coal is widely available and, because it exhibits self-cementing characteristics when mixed with water, it can be used byitself with no other cementitious material to bind aggregate particles together.

Coal fly ash, produced during the combustion of bituminous coal, is frequently used in stabilized base mixtures. Since this type of fly ash is a pozzolan, the mixtures in which it is used are often referred to as pozzolanic stabilized base (PSB) mixtures. Pozzolans are materials composed of amorphous siliceous or siliceous and aluminous material in a finely divided (powdery) form (similar in size to Portland cement particles) that will, in the presence of water, react with an activator to form compounds possessing cementitious properties. Pozzolan activators are alkaline materials that contain calcium and magnesium compounds present in sufficient amounts to chemically react in the presence of water with the silicate and aluminates in the pozzolan. Descriptions of various kinds of pozzolans and their specifications are provided in ASTM C618.

In PSB compositions, the fly ash is usually used in combination with eitherlime, Portland cement, or kiln dust, plus water, to form the matrix that cements the aggregate particles together. When used with a chemical reagent, this typeof fly ash normally comprises between 10 and 20 percent by weight of astabilized base or subbase mix. When used with lighter weight aggregates (such as coal bottom ash), the percentage of fly ash may be as high as 30 percent or more.

Types of stabilization:
Mechanical
Stabilization:
- ❖ In this technique mechanical energy is used (rollers, plate compactors, tampers etc. By choice or nature of soil) to improve the soil properties by compaction.
- ❖ Preferably for construction of embankment for roads, railways etc.

❖ Mechanical stability depends upon the degree of compaction. Normally, the compaction is done at optimum water content.

Uses—

- Simplest method of soil stabilization.
- To improve the sub-grades of low bearing capacity.
- Extensively used for construction of bases, sub-bases and surfacing of roads.

Factors Affecting the Mechanical Stabilization

The mechanical stability of the mixed soil depends upon the following factors.

1. **Mechanical Strength of the Aggregate—** The mixed soil is stable if the aggregates used have high strength. However, if the mixture is properly designed and compacted, even the aggregates of relatively low strength canprovide good mechanical stability.

2. **Mineral Composition—** the mechanical stability of the mixed soil depends upon the composition of the minerals. The minerals should be weather resistant.

3. **Gradation—** the gradation of the mixed soil should be such that the voids of the coarser particles are filled with finer particles to obtain a high density.

(1) Plasticity Characteristics—

❖ For mud roads surfacing, highly plastic soils are used as binders. They posses greater cohesion, moisture retention capacity and provide seal against downward movement of surface water.

❖ For base courses, the soils should have low plasticity to avoid excessive accumulation of water and the resulting loss of strength.

❖ The soil available at site may seldom meet both the requirements. It is necessary to mix soils from different sources to obtain desired mix.

Lime stabilization

There are basically five types of lime:

❖ High Calcium, quick lime (CaO)

❖ Hydrated, high calcium lime [Ca(OH)2]

❖ Dolomite lime (CaO+MgO)

❖ Normal, hydrated dolomitic lime [Ca(OH)2+MgO]

❖ Pressure, hydrated dolomitic lime [Ca(OH)2+Mg(OH)2]

- The quick lime is more effective than the hydrated lime, but the latter is more safe and convenient to handle. Generally, hydrated-lime is used. Itis also known as slaked lime.

- The higher the magnesium content of the lime, the less is the affinity for water and the less is the heat generated during mixing.

- The amount of lime required varies between 2 to 10% of the soil.

Lime stabilization is done by adding lime to soil. It is useful for the stabilizationof clayey soil.

> When lime reacts with soil there is exchange of cations in the absorbed water layer and a decrease in the plasticity of the soil occurs.
> The resulting material is more friable than the original clay, and is, therefore more suitable as sub-grade.

The following amount may be used as a rough guide:

❖ 2 to 5% for clay gravel material having less than 50% of silt-clay fraction
❖ 5 to 10% for soils with more than 50% of silt clay fraction
❖ About 10% for heavy clays used as bases and sub-bases
❖ For soils having particle size intermediate between (1) and (2) above, the quantity of lime required is between 3 to 7%.

Lime stabilization is not effective for sandy soils.

Construction Method— Construction methods used in lime stabilization aresimilar to those used in cement stabilization. However, the following points should be carefully noted.

- ❖ The reaction in the case of lime is slow, there is no maximum time limit between the addition of lime to the soil and the completion of compaction.
- ❖ Lime may be added in the form of slurry instead of dry powder.
- ❖ A rest period of 1 to 4 days is generally required after spreading lime over a heavy clay before final mixing is done.
- ❖ The soil-lime is compacted to the required maximum dry density.
- ❖ After compaction, the surface is kept moist for 7 days and then covered with a suitable wearing coat.

Cement stabilization

- ❖ Most commonly used for road construction.
- ❖ Heavy clays are difficult to pulvirize and not suitable.
- ❖ Well graded sand and gravel mixtures with upto 10% fine binder Material (passing#200sieve).
- ❖ Quantity of cement to be determined on trial basis in lab. (minimum strength required 3.5n/mm2—7 days cube strength.
- ❖ Compaction to be completed within two hours after laying mixing with water.

A. Central plant method: faster construction, expansive, dry mix and thenWet thoroughly, spreading and compaction.

B. Mix in place method: similar to agriculture rotary cultivator, firstly soil is Pulverized then dry cement is spread over, then water sprinkled in layers, again Remixed and shaped to camber., compacted using rollers.

(1) Normal Soil-Cement—

- ❖ It consists of 5 to 14% of cement by volume.
- ❖ Cement is sufficient to produce a hard and durable material.
- ❖ Sufficient water be used for hydration requirement & workability
- ❖ It is weather resistant and strong and used for stabilizing sandy and other low plasticity soils.

(2) Plastic Soil-Cement—

- ❖ It consists of 5 to 14% of cement by volume,
- ❖ It has more water to have wet consistency similar to that of plastering mortar at the time of placement.
- ❖ Used for water proof lining of canals and reservoirs
- ❖ Used for protection of steep slopes against water erosion.
- ⬚

Fly ash stabilization

Fly ash is a byproduct from burning coal which makes steam to generate

electricity. When burning coal, combustion particles rise out of the combustion

chamber with flue gasses. They are captured in filters to prevent them from reaching the atmosphere and collected for disposal or <u>beneficial reuse</u>. These particles are called fly ash.

There are two types of fly ash, Class C and Class F. Class C has self- cementing properties and is used in the production of concrete as a substitutefor <u>Portland Cement</u>, and as a chemical stabilizing & modifying agent to dry and/or strengthen poor soils. Class F has very little self-cementing properties, but can be combined with additives such as quicklime, hydrated lime, or cement(portland or hydraulic) to create cementitious compounds for the same purposes.

Fly Ash in Modification, Stabilization and FDR

Soil Modification: Given its cementitious properties, fly ash can dry down wet soils and increase the strength of each fill layer. It works best in sandy/silty soils, but can be very effective in lean clays as well.

Soil & Base Stabilization: Using fly ash to strengthen the top 8"-14" of subgrade can decrease the thickness of aggregate base and/or pavement (asphalt or concrete) needed to achieve the structural design strength of the overall pavement section. Again, it works best in sandy, silty soils, but is also a great option to stabilize existing aggregate base when performing parking lot repairs or upgrades.

Full Depth Reclamation: Depending on the existing make-up of the current pavement & aggregate base, fly ash can be used by itself, or in combinationwith other additives, to rehabilitate entire pavement sections in place.

Base Course:
Preparation of base course
It is the layer immediately under the wearing surface (Applies whether the wearing surface is bituminous or cement concrete and or more inch thick or is but a thin bituminous layer). As base course lies close under the pavement surface it is subjected to severe loading. The material in a base course must beof extremely high quality and its construction must be done carefully.

Types of Base Course

1. Granular Base Course

Fly ash yields less strength gain than <u>Portland Cement</u>. Typically, you need to use twice as much product to achieve similar results when compared to cement. However, if a fly ash source is within proximity of a project, utilizingit could be a better value and is worth comparing.

2. Macadam Base
3. In-water bound Macadam
4. Treated Bases

Brick soling
Soling in the construction field is the bottom-most layer of any component ofthe structure. It may be under floor or road. Soling may consist of bricks, stone cutting or such other building material having good crushing strength. t is one ofthe most common techniques used for soil stabilization.

Water Bound Macadam

The concept of water bound macadam road was suggested by John Macadam, who was a Scottish engineer. The road whose wearing course consists of clean crushed aggregates, mechanically interlocked by rolling and bound together with filler material and water laid on a well compacted base course, is called water bound macadam (W.B.M) road.

This is constructed as village road serves as a base for bituminous roads. In most of the roads projects, in the first phase, W.B.M roads are constructed and when the funds are available, the surfacing is done with the premix carpet bituminous

macadam or cement concrete. So a water-bound macadam road is considered as the mother of all types of road construction.

Wet-mix Macadam

Aggregates used are of the smaller sizes, varies between the 4.75 mm to 20 mm sizes and the binders(stone dust or quarry dust having PI(Plasticity Index) not less than 6%) are premixed in a batching plant or in a mixing machine. Thenthey are brought to the site for overlaying and compaction.

The PI(plasticity Index) of the binding material is kept low because it should be a sound and non plastic material. If the plasticity index is more then there arethe chances of the swelling and more water retention properties. So this value should be kept in mind.

Comparison of the WBM and WMM road construction: Although the costof construction of the WMM is said to be more than that of the WBM sub-base and bases but the advantages given below will compensate for that. Here are the points of difference:

1. The WMM roads are said to be more durable.
2. The WMM roads gets dry sooner and can be opened for traffic withing less time as compare to the WBM roads which take about one month for getting dry.
3. WMM roads are soon ready to be black topped with the Bituminous layers.
4. WMM roads are constructed at the faster rate.
5. The consumption of the water is less in case of the WMM roads.
6. Stone aggregates used in WBM is larger in size which varies from 90 mm to 20 mm depending upon the grade but in case of the WMM size varies from 4.75 mm to 20 mm.
7. In case of WBM, stone aggregates, screenings and binders are laid one after another in layers while in WMM, aggregates and binders are premixed in the batching plants and then brought to the site for overlaying and compacting.
8. Materials used in the WBM are the stone aggregates, screeningsand binder material (Stone dust with water) while in WMM material used are only stone aggregates and binders.
9. Quantity of the WBM is generally measured in cubic meters while that of the WMM in square meters.

Surfacing:
☐ Surface dressing

❖ A Surface Dressing is a process of spraying a road surface with bituminous binder and then covering the binder with clean, crushed aggregate or natural gravel.

❖ These layers are then rolled in order to press the aggregate into the binder film.

❖ Traffic movement commences the process of chipping movement which will

produce eventually an interlocking matrix.

The main objective of adopting surface dressing as a wearing coat over bituminous macadam is to achieve water proofed, anti skid but comparatively less expensive wearing coat which can last for more duration as compare toother wearing surfaces.

(i) Premix carpet

Premix carpet (PC) is the oldest hot mix in India. It is a good, economical, bituminous wearing course mix to be placed directly on water bound macadam (WBM) of low-volume rural roads. The premix carpet is also provided with a bituminous sand seal coat to minimize direct penetration of rainwater into it.

(ii) Semi dense carpet

The semi-dense bituminous concrete mixes have neither dense or open graded characteristics. It consists of the so called pessimum voids when they are fully constructed. This will create the separation of aggregate and the bitumen in the BM layer.

Bituminous concrete(BC)

- BC is a dense graded bituminous mix used as wearing course for heavily trafficked roads.
- BC mix consists of coarse aggregates, fine aggregates, filler and binder blended as per marshall mix design.

Quality control operations involved are:

- Design of mix in laboratory, and control of mixing, laying and rollingtemperatures
- Density, Marshall Stability, Flow, Air Voids, Retained Stability, BitumenContent, Gradation of aggregates are controlled
- Riding quality is a control

Grouting

Grouting is generally a mixture of cement, sand and water.Different type of grouting are used for different purposes but generally They areused in the purpose of repairing of concrete cracks, filling seams and gaps in tiles, seal and fill gaps for waterproofing courses, and for soil stabilization in boring well and foundation. It is also used to give extra strength to the foundations of load-bearing structures.

Grouting in civil engineering refers to the injection of pumpable materials into a soil or rock formation to change its physical characteristics. It is one of the ways ground water can be controlled during civil engineering works.

Grouting is suitable where soil permeability would create a heavy demand on pumping or where ground conditions mean it may be economicallyinefficient to bore wells. Grout may also be used in the formation of pile foundations, ground anchors, under-reaming, underpinning, in road construction, dam construction, and other applications.

Different materials may be used for grouting depending upon factors such as the soil or rock type and the area to be grouted. However, the basic process is the same:

the soil or rock is injected with fluid grout which sets and reduces or acts as a sealant on the material's permeability.

Grouting is relatively costly and so wastage must be controlled. This is achieved by the use of additives which improve the gelling properties of the grout and limit its spread through the ground.

UNIT 5-HILL ROADS

Introduction:

- Roads constructed in mountains region is called hill roads.
- There are different considerations while designing hill roads as compare to plain area roads.
- Types of curve used in hill roads is of different than plain road.
- All geometric parameters will gets changes while designing hill roads such as- Curves, Super elevation, SSD, OSD, Extra Widening, etc.

Components parts of Hill Roads

1. Road Bed
2. Side Drain
3. Parapet Drain
4. Catch Water Drains
5. Brest Wall
6. Retaining Wall
7. Cross Drains

Road Bed

- The pavement potion of hill road is called road bed.
- **Function:** To resist stresses developed due to moving traffic.

Side Drain

- Drain provided on the sides of road is called side drain.
- Side drains runs parallel to the length of road.
- **Function:** To collect and drain off rain water collected from camber of road.

Parapet Wall

- Wall which is provided above the formation level in the down side slopeis called parapet well.
- **Function:** Protection to the traffic against falling down the hill slope.

Catch Water drain

- It is drain provided on higher slope running parallel to the length of road.
- **Function:** To make intercept for runoff coming from top of hill and divert water in to nearby cross drains.

Brest Wall

- The wall constructed to upside slope is called retaining wall.
- **Function:** Protect road from sliding of upside slope.

GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING
GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

Structural Design -1

For 4th Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)
Prepared By
Sasmita Kumari Das
(Lecturer in civil Engineering)

CONTENT

CHAPTER-1

INTRODUCTION TO DESIGN AND DETAILING

2.1 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.

2.2 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.

2.3   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

<u>WORKING STRESS METHOD OF DESIGN</u>

2.1   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 concrete3.0	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/mm² 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'.

(a) Rectangular Section with Reinforcement
(b) Strain Distribution
(c) Stress Distribution
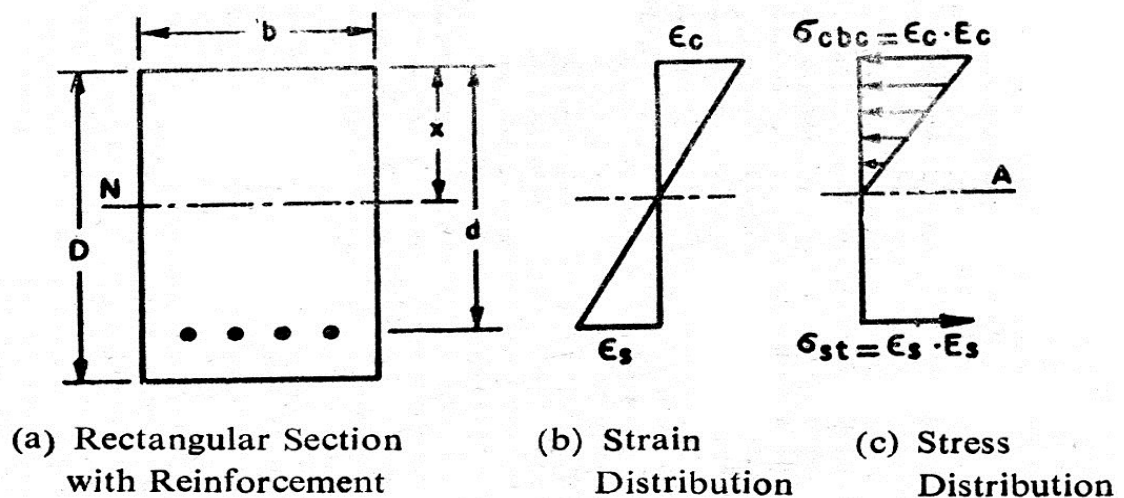
Fig.2.1 (a-c)

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= $\dfrac{1}{1 + \sigma_{st}\Big/ m.\sigma_{cbc}}$

Total compressive force=$\frac{b.x.\sigma_{cbc}}{2}$

Total tensile forces = $\sigma_{st}.A_s$

Z= Lever arm= $d - \dfrac{x}{3} = d - \dfrac{k.d}{3} = d\left(1 - \dfrac{k}{3}\right) = j.d$

Where j is called the lever arm constant.

Moment of resistance = MR= $\dfrac{b.x}{2}.\sigma_{cbc}.jd = \dfrac{kd.j}{2}.\sigma_{cbc}.b.d = \dfrac{1}{2}.k.j.\sigma_{cbc}.b.d^2 = Q.b.d^2$

Where Q is called moment of resistance constant and is equal to $\dfrac{1}{2}.k.j.\sigma_{cbc}$

(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.

$$\text{Moment of resistance} = \sigma_{st}.A_s.\left(d - \frac{x}{3}\right) = \sigma_{st}A_s.j'd \qquad where \quad j' = 1 - \frac{k'}{3}$$

$$\text{Since } p = \frac{A_s.\times 100}{b.d}$$

Moment of resistance

$$= \sigma_{st}.p.\frac{b.d}{100}.j' \times d = \frac{\sigma_{st}.p.j'}{100}.b.d^2 = Q'.Bb.d^2 \qquad where \quad Q' = \frac{\sigma_{st}.p.j'}{100}$$

(a) Rectangular Section with Reinforcement   (b) Strain Distribution   (c) Stress Distribution
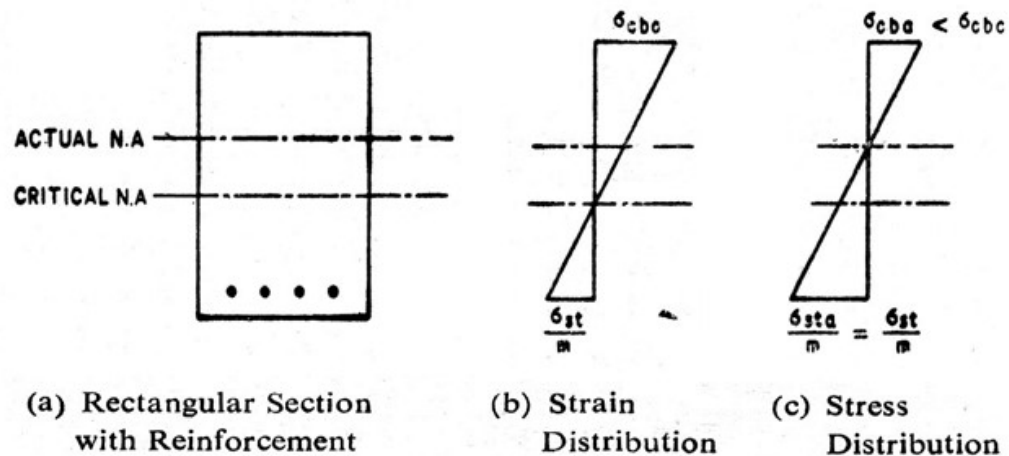
Fig.2.2 (a-c)

(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
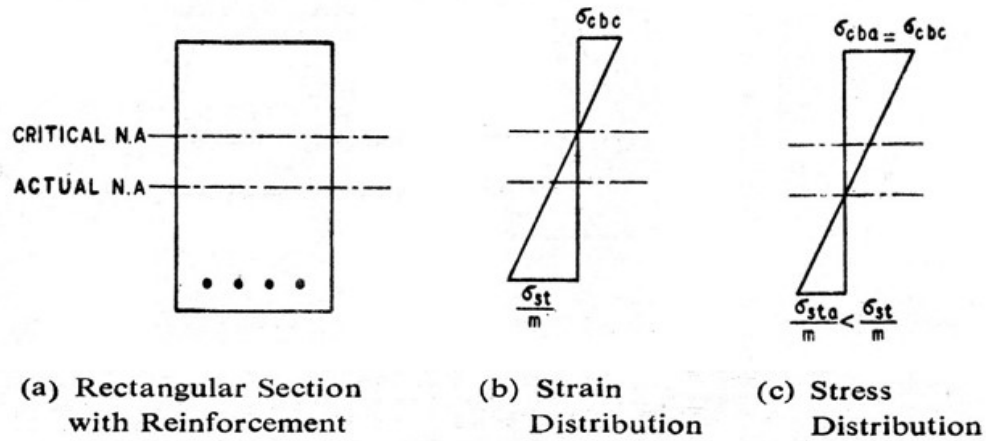
Fig.2.3 (a-c)

$$\text{Moment of resistance} = b.x.\sigma_{cbc}.\frac{1}{2}.\left(d - \frac{x}{3}\right) = \frac{\sigma_{cbc}}{2}.b.x.d.\left(1 - \frac{k'}{3}\right)$$

$$= \frac{\sigma_{cbc}}{2}.b.x.d.j' = \frac{1}{2}.\sigma_{cbc}.k'.j'.b.d^2 = Q'.b.d^2 \quad where\, Q' = \frac{\sigma_{cbc}}{2}.k'.j' = Constant$$

2.2 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.*

$$b.x.\frac{x}{2} = m.A_s.(d - x)$$

$$A_s = \frac{b.x^2}{2(m)(d - x)} \quad \therefore \quad M = \frac{\sigma_{st}.b.x^2}{2.m.(d - x)}\left(d - \frac{x}{3}\right) = B.M.\text{ to be resisted}$$

Solve for '*x*', and then A_s can be calculated.

(b) If *B.M.* is more than *M*, design the section as over-reinforced.

$$M = \frac{\sigma_{cbc}}{2}.b.x.\left(d - \frac{x}{3}\right) = B.M. \text{ to be resisted.} \quad \text{Determine '}x\text{'. Then } A_s \text{ can be obtained by taking}$$

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.

$$x = \frac{1}{1 + \dfrac{\sigma_{st}}{m.\sigma_{cbc}}}.d$$

(iii) Check if (i) > (ii)- the section is over-reinforced

   (i)<(ii)- the section is under-reinforced

(iv)   Calculate *M* from relation

$$M = b.x.\frac{1}{2}.\sigma_{cbc}.\left(d - \frac{x}{3}\right) \text{ for over-reinforced section}$$

and $M = \sigma_{st}.A_s.\left(d - \dfrac{x}{3}\right)$ 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.

$$b.\dfrac{x^2}{2} = m.A_s.(d - x))$$

(ii)   Determine lever arm, $z = d - \dfrac{x}{3}$

(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.

$$BM = \dfrac{\sigma_{cba}}{2}.b.x.z$$

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
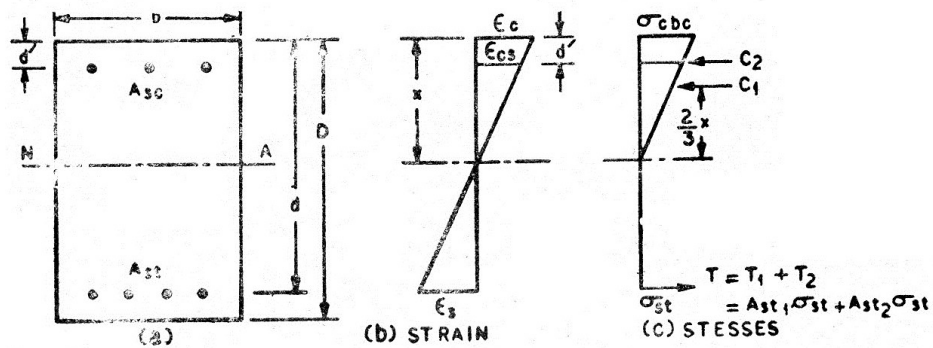
Fig.2.4 (a-c)

Since strains are proportional to the distance from $N.A.$,

$$\frac{Strain\ in\ top\ fibre\ of\ concrete}{Strain\ in\ Compression\ Steel} = \frac{x}{x-d'}$$

$$\frac{\sigma_{cbc}/E_C}{\sigma_{sc}/E_s} = \frac{x}{x-d'}$$

$$\frac{\sigma_{cbc}}{\sigma_{sc}} \cdot \frac{E_s}{E_c} = \frac{x}{x-d'}$$

$$\sigma_{sc} = \sigma_{cbc} \cdot \frac{x}{x-d'} \cdot m$$

Since $\sigma_{cbc}.\dfrac{x-d'}{x}$ 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 (σsc) whichever is less.

$$\sigma_{sc} = 1.5m\ \sigma'_{cbc}$$

Total equivalent concrete area resisting compression

$(x . b\text{-}A_{sc}) +1.5m\ A_{sc} = x .b +(1.5m\text{-}1)A_{sc}$

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,

$$M = b.x.\dfrac{\sigma_{cbc}}{2}.(d - \dfrac{x}{3})+(1.5m-1)A_{sc}.\sigma_{cbc}.\dfrac{x-d'}{x}.(d-d') = Q.b.d^2 +(1.5m-1)A_{sc}.\sigma_{cbc}.\dfrac{x-d'}{x}(d-d')$$

$$= M_1 + M_2$$

$Where\ M_1 = Moment\ of\ resis\tan ce\ \ of\ the\ \ balanced\ \sec tion = M_{bal}$

$\qquad M_2 = Moment\ \ of\ \ resis\tan ce\ \ of\ \ the\ \ compression\ \ \ steel$

$$Area\ of\ tension\ \ steel = A_{st1} = \dfrac{M_1}{\sigma_{st}.j.d}.$$

$$Area\ of\ \ tension\ steel\ \ equivalent\ \ to\ compression\ steel = A_{st2} = \dfrac{M_2}{\sigma_{st}\ (d-d')}$$

Thus the total tensile steel A_{st} shall be:

$$\therefore \qquad A_{st} = A_{st1} + A_{st2}$$

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
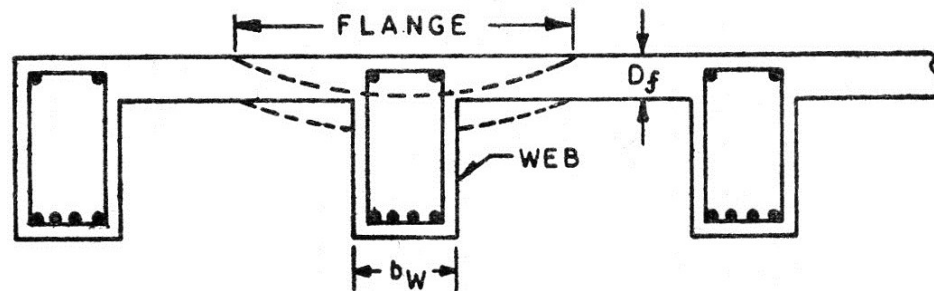
Fig.2.5

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.

SECTION AT X-X
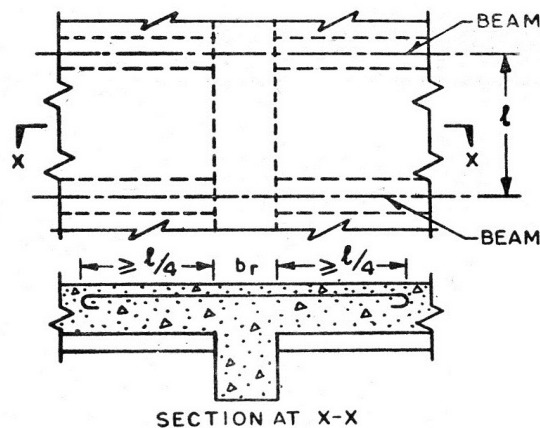
Fig.2.6

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.
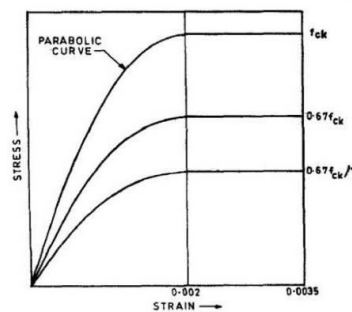
STRESS-STRAIN CURVE FOR CONCRETE   STRESS BLOCK PARAMETERS
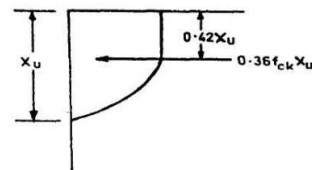
For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor $\gamma_m = 1.5$ shall be applied in addition to this.

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.

(a) Strain diagram   (b) Stress diagram
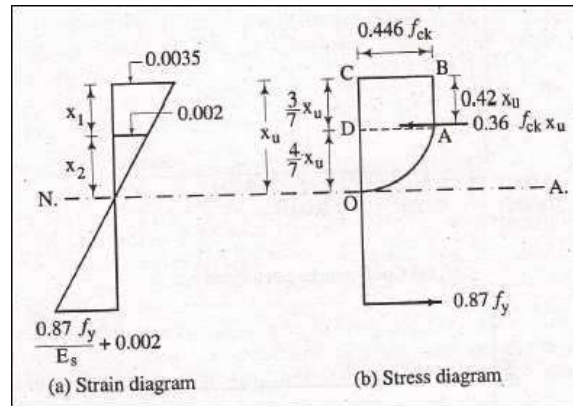
4)   the tensile strength of the concrete is ignored.

5)   the stresses in the reinforcement are derived from representative stress – strain curve for the type of steel used.
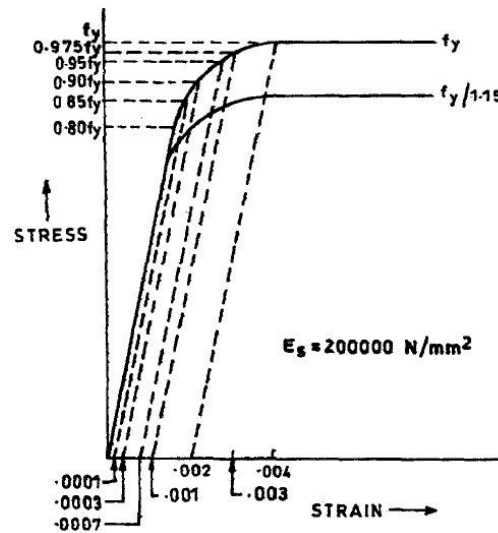
Cold Worked Deformed Bar

STRESS — STARIN CURVE FOR STEEL
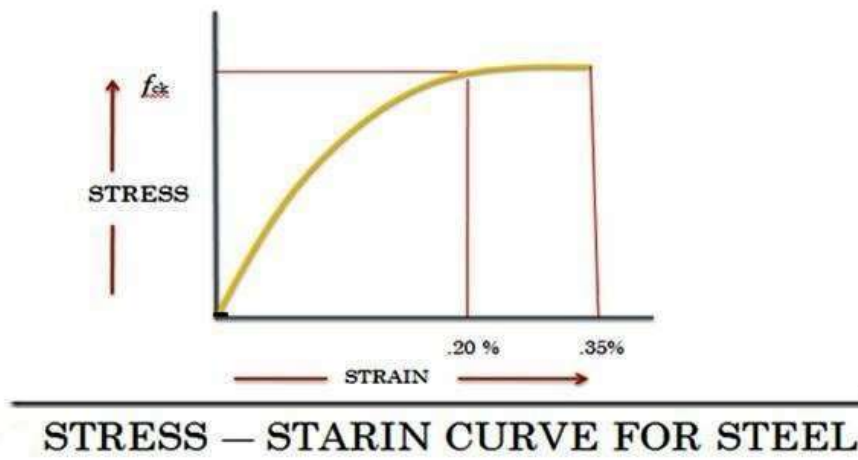
6) the maximum strain in tension reinforcement in the section at failure shall not be less than

$$\frac{f_y}{1.15E_s} + 0.002 = \frac{0.87f_y}{E_s} + 0.002$$

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

<div align="center">

- Flexure
- Compression
- Shear
- Torsion

</div>

This state corresponds to the maximum load carrying capacity.

<div align="center">

Types of reinforced concrete beams

</div>

a)   Singly reinforced beam

b)   Doubly reinforced beam

c)   Singly or Doubly reinforced flanged beams

<div align="center">

Singly reinforced beam

</div>

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.
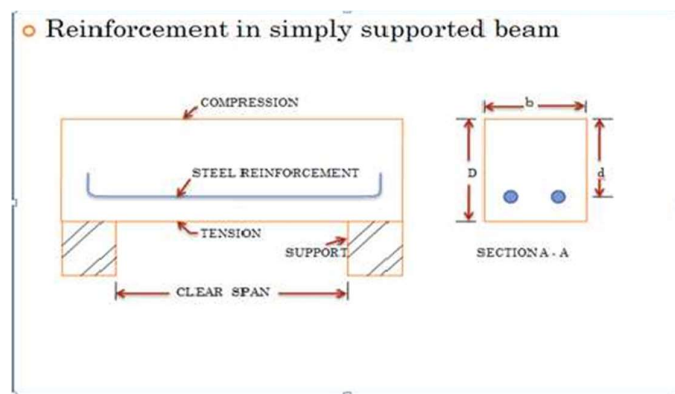
○ Reinforcement in a cantilever beam
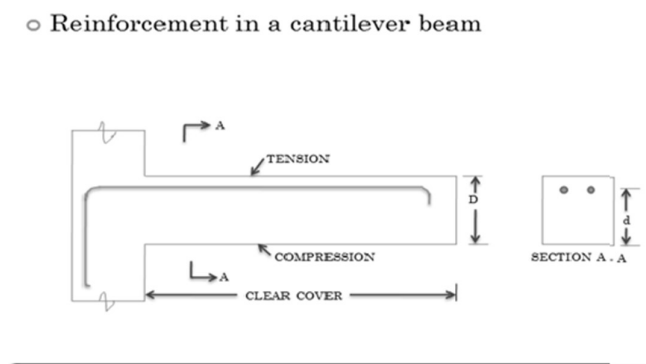
TYPES OF BEAM SECTIONS

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 $\dfrac{X_u}{d} = \dfrac{X_{u\,max}}{d}$ $\Rightarrow$ Balanced If $\dfrac{X_u}{d} < \dfrac{X_{u\,max}}{d}$ $\Rightarrow$ Under Reinforced If $\dfrac{X_u}{d} > \dfrac{X_{u\,max}}{d}$ $\Rightarrow$ Over Reinforced $\dfrac{X_u}{d} = \dfrac{0.87 f_y . A_{st}}{0.36 b . d f_{ck}}$ $\begin{array}{cc} f_y & \dfrac{X_{u\,max}}{d} \\ 250 & 0.53 \\ 415 & 0.48 \\ 500 & 0.46 \end{array}$

2	Calculate Moment of Resistance	Grade of Concrete & Steel, Size of beam & Reinforcement Provided	1) If $\dfrac{x_u}{d} = \dfrac{x_{u,max}}{d}$, balanced $M.R = M_u = 0.36.\dfrac{x_{u,max}}{d}(1-0.42\dfrac{x_{u,max}}{d})b.d^2.f_{ck}$
			2) If $\dfrac{X_u}{d} < \dfrac{X_{u\,max}}{d}$ Under Reinforced M.R=M_u= $0.87f_y.A_{st}.d(1-\dfrac{A_{st}.f_y}{b.d.f_{ck}})$ or $M.R = 0.87f_y.A_{st}.d(1-0.42\dfrac{x_u}{d})$
			3) If $\dfrac{X_u}{d} > \dfrac{X_{u\,max}}{d} \Rightarrow$ 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; $M.R = M_u = 0.36.\dfrac{x_{u,max}}{d}(1-0.42\dfrac{x_{u,max}}{d})b.d^2.f_{ck}$ For finding A$_{st}$ use the equation $0.87f_y.A_{st}.d(1-\dfrac{A_{st}.f_y}{b.d.f_{ck}})$ or $M.R = 0.87f_y.A_{st}.d(1-0.42\dfrac{x_u}{d})$

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}.

$M_{u2} = M_u - M_{u,lim}$
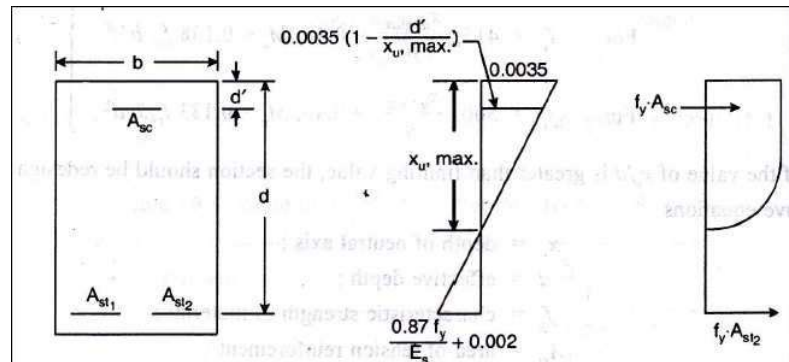
The lever arm for the additional moment of resistance is equal to the distance between the centroids of tension and compression reinforcement, (d – d').

$M_{u2} = 0.87\, f_y.A_{st2}(d - d') = A_{sc}.(f_{sc} - f_{cc})(d - d')$

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 $0.0035 \left(1 - \dfrac{d'}{x_u, max.}\right)$

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/mm², $\dfrac{X_{u\,max}}{d} = 0.48$

$$M_{u,lim} = 0.36.\frac{x_{u,max}}{d}(1-0.42\frac{x_{u,max}}{d})b.d^2.f_{ck}$$

$$= 0.36 \times .48(1\text{-}0.42 \times 0.48) \times 250 \times 460^2 \times 20$$

$$= 146 \times 10^6 \text{N.mm}$$

a) For M_u = 100 KN.m < 146 KN.m

Area of steel required is obtained from , $M_u = 0.87f_y.A_{st}.d(1-\dfrac{A_{st}.f_y}{b.d.f_{ck}})$

$100 \times 10^6 = 0.87 \times 415 \times A_{st} \times 460 \left(1\text{-} \dfrac{A_{st} \times 415}{250 X 460 X 20}\right)$

A_{st} =686 or 4850 mm², taking minimum steel 686mm²

b) M_u = 146 KN.m =$M_{u,lim}$= 146 KN.m

      x_u = $x_{u,max}$

Area of tension reinforcement required

$$\frac{X_{u,max}}{d} = \frac{0.87f_y.A_{st}}{0.36b.df_{ck}}$$

$$A_{st} = \frac{0.48 X 0.36 X 20 X 250 X 460}{0.87 X 415} = 1100mm^2$$

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 $\dfrac{0.0035(x_{u,max} - d')}{x_{u,lim}} = \dfrac{0.0035(0.48 X 460 - 40)}{0.48 X 460} = 0.002866$

$f_{sc} = 360.8 N/_{mm}{}^2$

$(200-146) \, X \, 10^6 = 360.8. \, A_{sc}(460-40)$

$A_{sc} = 356 mm^2$

A_{st1} = Area of tension reinforcement corresponding to $M_{u,lim}$

$146 \, X \, 10^6 = 0.87 \, X \, 460 \, X \, 415 A_{st1}(1 - \dfrac{A_{st} X 415}{250 X 460 X 20}$)

$A_{st1} = 1094 mm^2$

$A_{st2} = A_{sc}. \, f_{sc} / 0.87 \, X415 = 356 mm^2$

$A_{st} = A_{st1} + A_{st2} = 1094 + 356 = 1450 mm^2$

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

$M_u = M_{u,lim}$

$M_{u,lim} = 0.36. \dfrac{x_{u,max}}{d}(1 - 0.42 \dfrac{x_{u,max}}{d})b.d^2 . f_{ck}$

$M_{u,lim} = 0.36 \, X \, 0.48 \, X \, 20(1-0.42 \, X \, 0.48) \, bd^2$

$= 2.759 b \, d^2$

Assuming width of beam as 250 mm

$d = \sqrt{\dfrac{65 X 10^6}{2.759 X 250}} = 307 mm$

Area of reinforcement

$\dfrac{X_{u,max}}{d} = \dfrac{0.87 f_y . A_{st}}{0.36 b . d f_{ck}}$

$0.48 = \dfrac{0.87 \, X \, 415 X A_{st}}{0.36 X 20 X 250 X 307}$

$A_{st} = 734.66 \ mm^2$

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

$\dfrac{d}{d'} = \dfrac{50}{450} = 0.11$, next higher value 0.15 may be adopted.

A_{sc} =area compression reinforcement = 2 π 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 $(b_f - b_w) \times D_f$. It is shown in the figure beam (a) is equivalent to beam (b) + beam (c).
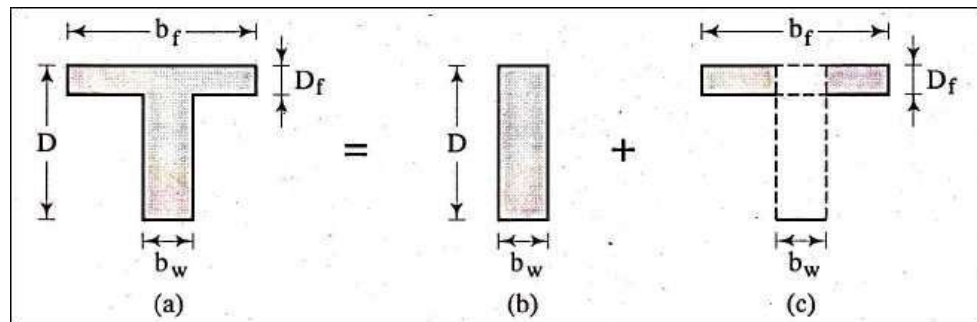
The flanged beam analysis and design are analogous to doubly reinforced rectangular beam. In doubly reinforced beams additional compressive is provided by adding reinforcement in compression zone, whereas in flanged beams, this is provided by the slab concrete, where the spanning of the slab is perpendicular 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.

5.2 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.
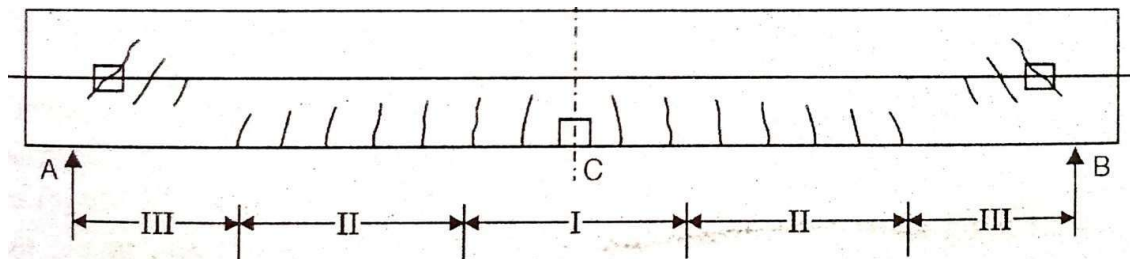
Fig-5.2.1 DIFFERENT REGION OF CRACKS IN BEAMS

(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.

5.3 MECHANISM OF SHEAR TRANSFER IN REINFORCE CONCRETE BEAM WITHOUT SHEAR.

(a) Diagonal tension crack   (b) Flexural shear crack
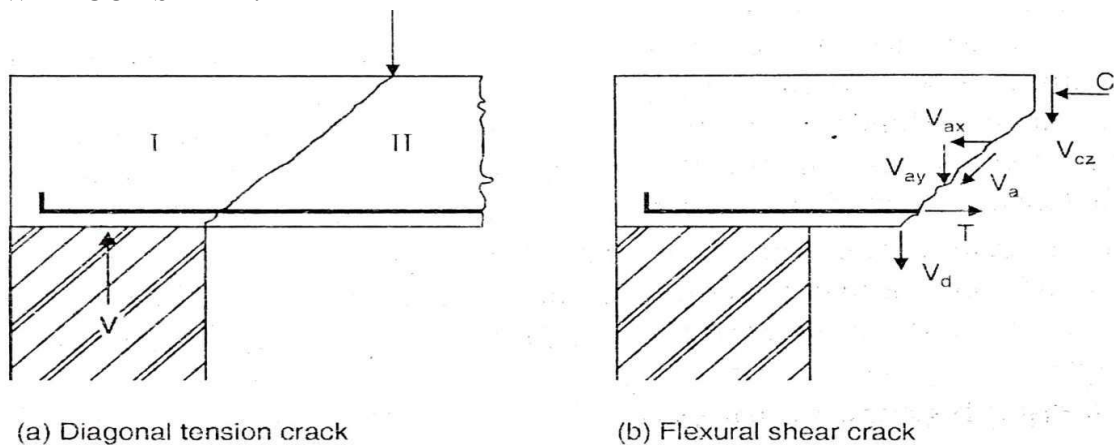
Fig- 5.3.1

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.

5.4. 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
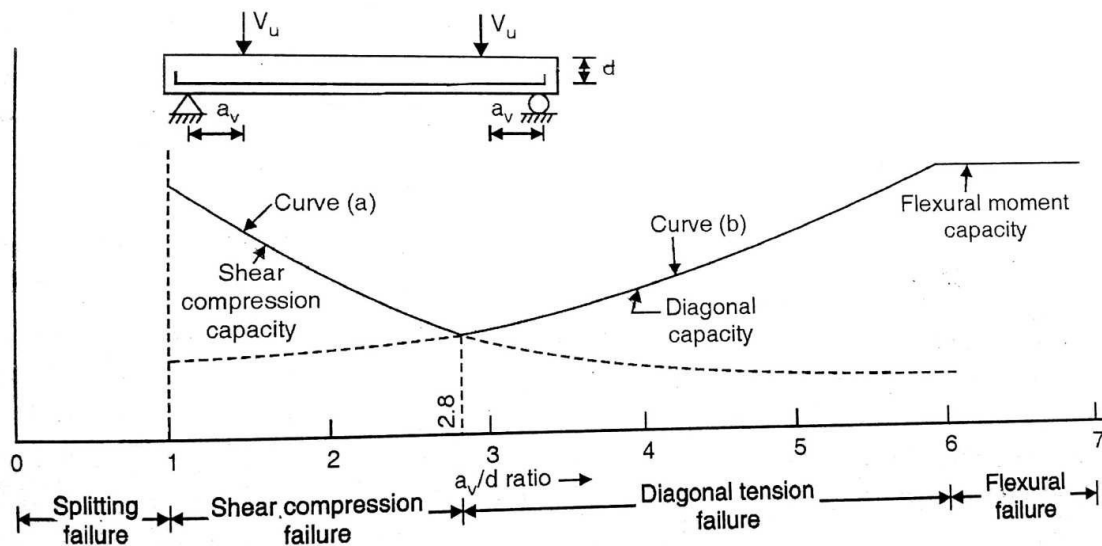
Fig – 5.4.1. EFFECT OF a_v / d ON SHEAR STRENGTH OF R C BEAM

CASE I : $a_v / d < 1$ (Deep Beams) : Splitting or compression failure:

(a) Tied Arch Action   (b) Failure modes
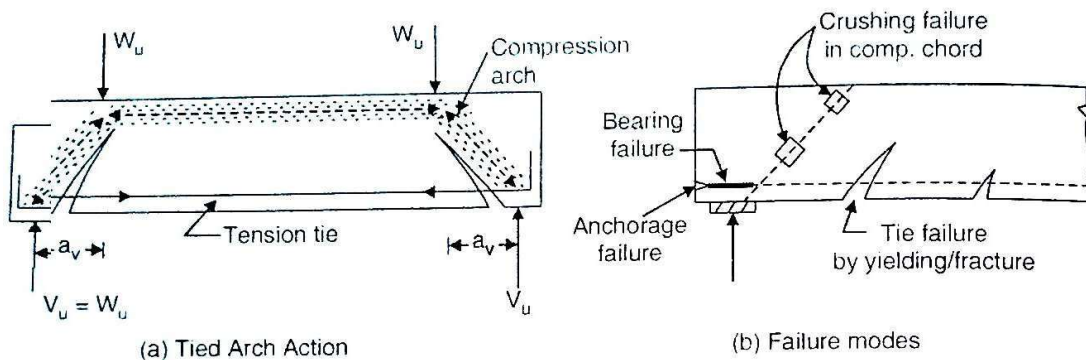
Fig – 5.4.2. CASE I : $a_v / d < 1$ (DEEP BEAMS)

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.

(a) Shear compression   (b) Shear tension
         failure   failure
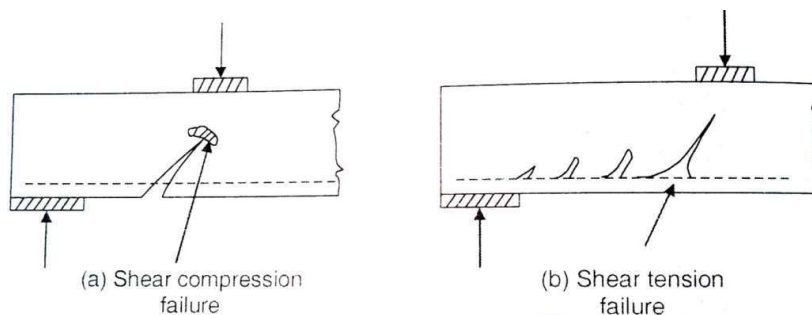
Fig- 5.4.3   CASE II : $1 < a_v / d < 2.8$

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.
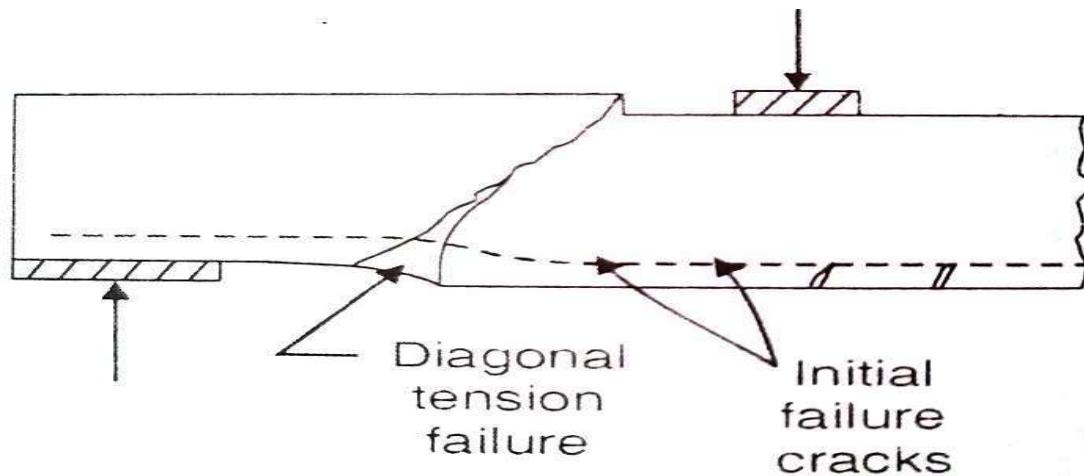
Fig- 5.4.4   CASE III : 2.8< a_v / d < 6

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.

5.5. 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 δ.

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 $\delta = 1 - \dfrac{P_w}{3.45 \, A\!E_g}$.

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)

$100 \dfrac{A_{st}}{bd}$	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).

$$\tau_c = \frac{0.85 \sqrt{0.8\, f_{ck}}\; (\sqrt{1 + 5\,\beta} - 1)}{6\,\beta} \qquad \qquad \text{...... 5.6.1}$$

where

$$\beta = \frac{0.8\, f_{ck}}{6.89\, p_t}, \quad \text{but not less than } 1$$

$$p_t = \frac{100\, A_{st}}{bd} \quad \text{(percentage steel in rib width only)}$$

$$0.8\, f_{ck} = \text{cylinder strength in terms of cube strength}$$

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 $\left(\dfrac{400}{d}\right)$ = the correction factor for depth and should not be less than 1

$\left(\dfrac{f_{ck}}{25}\right)$ = 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 :

$$\delta = 1 - \frac{P_{ut}}{3.45\, A_g} \qquad \ldots 5.6.4$$

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.

5.7. 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*.
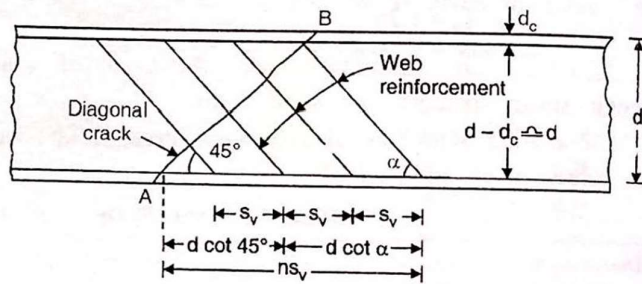
FIG. 5.7.1 SHEAR RESISTED BY WEB STEEL

Let

V_{us} = Ultimate shear carried by shear (or web) reinforcement

f_{yd} = 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

$\therefore$

$$V_{us} = n\ A_{sv}\ f_{yd}\ \sin \alpha \qquad \ldots\ldots 5.7.1\ (a)$$

In order to get the value of n, we have from geometry,

$$n\ s_v = d \cot 45° + d \cot \alpha \quad \text{or} \quad n = \frac{d \cot 45° + d \cot \alpha}{s_v} = \frac{d\ (1 + \cot \alpha)}{s_1} \quad \ldots 5.7.1\ (b)$$

Substituting the value of n and $f_{yd}\ (= 0.87\ f_y)$ in Eq. 7.13 (a) we get

$$V_{us} = \frac{d\ (1 + \cot \alpha)}{s_v} \cdot A_{sv}\ (0.87\ f_y) \sin \alpha \ = \frac{0.87\ f_y\ A_{sv}\ d}{s_v} (\sin \alpha + \cos \alpha) \qquad \ldots 5.7.2$$

Rearranging the above, we get

$$s_v = \frac{0.87\ f_y\ A_{sv}\ d}{V_{us}} (\sin \alpha + \cos \alpha) \qquad \ldots\ldots 5.7.2\ (a)$$

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 = $A_\emptyset$ X No of legs.

Special Cases :

(i) **Bars inclined at 45^0.** (i.e. $\alpha = 45^0$)

If $\alpha = 45°$, $\qquad V_{us} = \dfrac{0.87\ f_y\ A_{sv}\ d}{s_v}\ (\sqrt{2})$ $\qquad$ 5.7.3

$\qquad$ ..... 5.7.3 (a)

or $\qquad\qquad\qquad s_v = \dfrac{0.87\ f_y\ A_{sv}\ d}{V_{us}}\ \sqrt{2}$

(ii) Bars inclined at $90°$ *(i.e. vertical stirrups)*

$\qquad\qquad V_{us} = \dfrac{0.87\ f_y\ A_{sv}\ d}{s_v}$ $\qquad$ 5.7.4

$\qquad$ ...... 5.7.4 (a)

or $\qquad\qquad\qquad s_v = \dfrac{0.87\ f_y\ A_{sv}\ d}{V_{us}}$

(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 $n = 1$

$\qquad\qquad V_{us} = 0.87\ f_y\ .\ A_{sv}\ \sin\alpha$ $\qquad$ 5.7.5

5.8. 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.
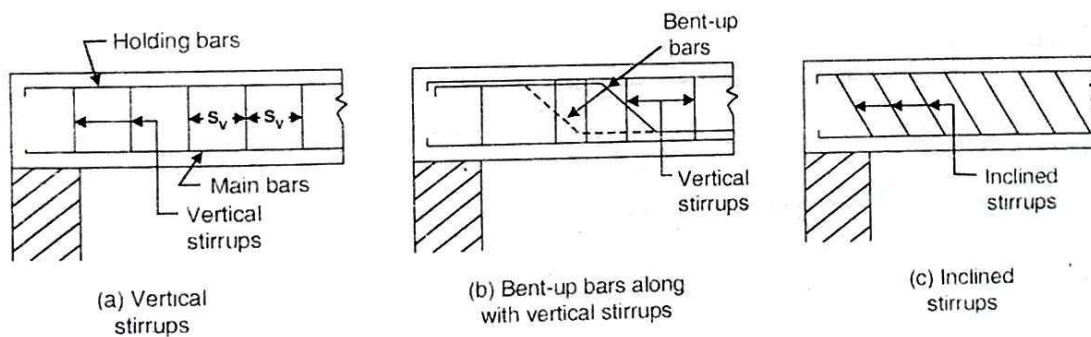
FIG. 5.8 TYPES OF SHEAR REINFORCEMENT

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.

5.9. 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.

(a) One legged   (b) Two legged   (c) Four legged   (d) Six legged
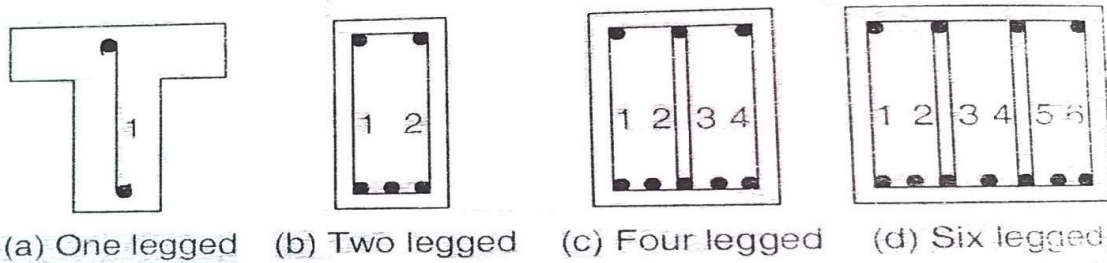
FIG-5.9. FORMS OF VERTICAL STIRRUPS

The strength of shear reinforcement in the form of vertical stirrups is given by

$$V_{us} = \frac{0.87 \, f_y A_{sv} d}{S_v} \qquad \ldots\ldots\ldots 5.9.1$$
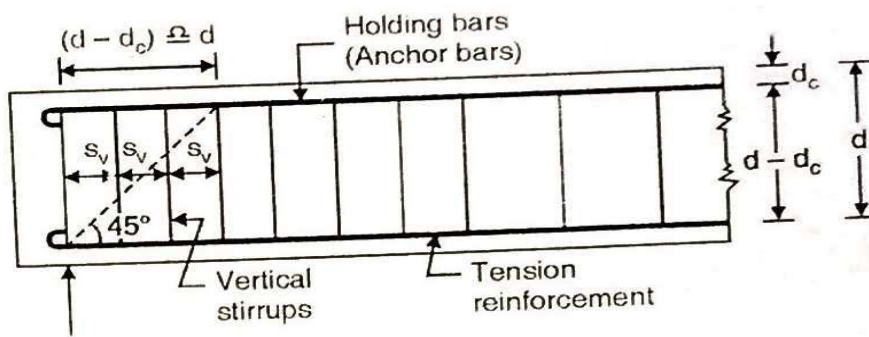
FIG. 5.9.1 SPACING OF VERTICAL STIRRUPS

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,

$$V_{us} = 0.87\, f_y \cdot A_{sv} \times \frac{d}{S_v}$$

$$S_v = \frac{0.87\, f_y \cdot A_{sv} \cdot d}{V_{us}} = \frac{0.87\, f_y \cdot A_{sv} \cdot d}{V_u - V_{uc}} \qquad \text{.... } 5.9.4$$

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 spacing 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 first 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.
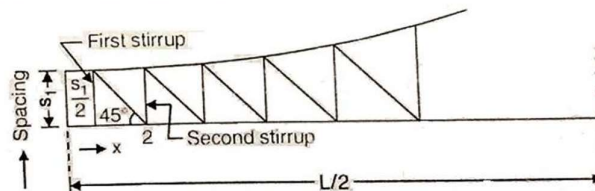
FIG. 5.9.2 SPACING DIAGRAM.

5.10. 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/mm², which *shall not be taken greater than 415 N/mm².*

Hence spacing based on minimum shear reinforcement is given by

$$s_v \leq \frac{0.87\,f_y \cdot A_{sv}}{0.4\,b} \leq \frac{2.175\,f_y\,A_{sv}}{b} \qquad \text{.... 5.10.2}$$

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 $\dfrac{0.87\,f_y \cdot A_{sv}}{s_v} = 0.4\,b$

$$\qquad \text{..... 5.10.3}$$

Thus, $V_{us,\,mm} = \left(0.87\dfrac{f_y\,A_{sv}}{s_v}\right) d = (0.4\,b)\,d = 0.4\,bd$

Thus, shear carried by concrete and that carried by minimum stirrups is given by

$$V_{u,\,mm} = \tau_c \cdot bd + 0.4\,bd \qquad \text{..... 5.10.4}$$

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 $\qquad \tau_c = 0.48$ N/mm^2 for M 20 concrete and $100 A_{st}/bd = 0.5$

Also, from Table 5.3 $\qquad \tau_{c, max} = 2.8$ 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 $\qquad V_{us} = V_u - V_{uc} = 150000 - 48000 = 102000$ 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 $\qquad s_v = \frac{0.87 \, f_y \, A_{sv}}{V_{us}} \cdot d$

Using two legged stirrups of 10 mm dia. bars, $A_{sv} = 2 \dfrac{\pi}{4} (10)^2 = 157.08$ mm^2

$$\therefore \qquad s_v = \frac{0.87 \times 250 \times 157.08}{102000} \times 400 \approx 134 \text{ mm}$$

Again we know $\qquad s_v = \frac{0.87 \, f_y \, A_{sv}}{(\tau_v - \tau_c) \, b} = \frac{0.87 \times 250 \times 157.08}{(1.5 - 0.48) \, 250} \approx 134$ 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.

$V_{uD} = 225 - 75 \times 0.7 = 172.5$ kN.

And $r_v = (172.5 \times 10^3) / (300 \times 500) = 1.15$ N/ mm^2.

From Table-5.1 for 100 A_s/bd = 0.75%, we get $r_c = 0.56$ 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

∴ Spacing $s_v = \dfrac{0.87\,f_y \cdot A_{sv} \cdot d}{V_{us}} = \dfrac{0.87 \times 250 \times 157.1 \times 500}{88500} = 193$ mm ≃ 190 mm (say)

Spacing corresponding to minimum shear reinforcements is

$$s_v = \frac{0.87\,f_y\,A_{sv}}{0.4\,b} = \frac{0.87 \times 250 \times 157.1}{0.4 \times 300} = 284.7 \text{ m} \simeq 280 \text{ mm} \quad (\text{say})$$

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}.

$$∴ \qquad V_{ux} = \frac{V_{u,max}}{3}\,x = \frac{225000}{3}\,x = 75000\,x$$

$$∴ \qquad V_{us} = V_{ux} - V_{uc} = 75000\,x - 84000$$

$$s_v = 280 = \frac{0.87 \times 250 \times 157.1 \times 500}{75000\,x - 84000}$$

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.

5.1 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)].
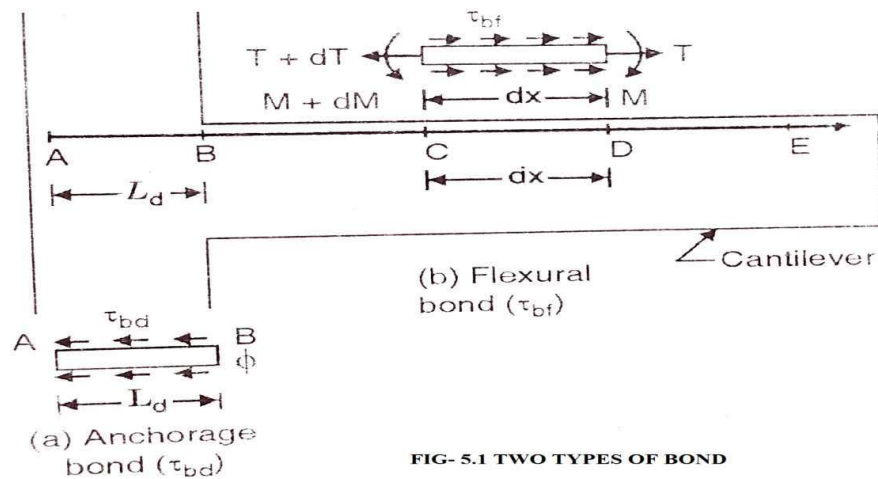
FIG- 5.1 TWO TYPES OF BOND

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 .
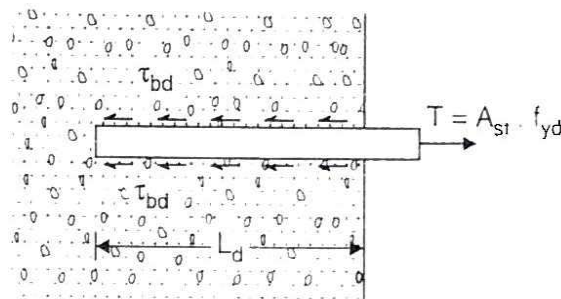
FIG- 5.2

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.

5.3 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%.

5.4 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

$$L_d = \phi \sigma_s / 4 \, \tau_{bd} = k_d \phi \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 5.4.1$$

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

$$L_{ds} = \phi \, \sigma_{sc} / 5 \, \tau_{bd} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 5.4.3$$

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 $L_{dr} = L_d (A_{st} \text{ required} \div A_{st} \text{ provided})$

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.

5.5 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 developed 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
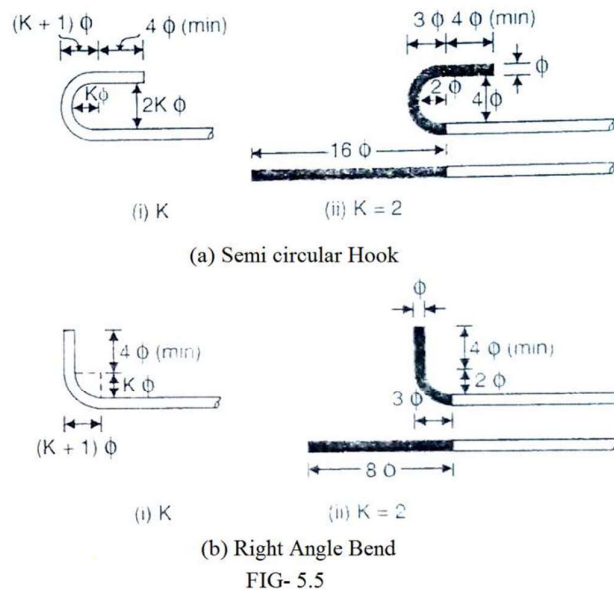
FIG- 5.5

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.

5.6 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.

5.7 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.

5.8. 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

$$L_d \leq M_1/V + L_o \quad ....5.8.1$$

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 eqn - 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.
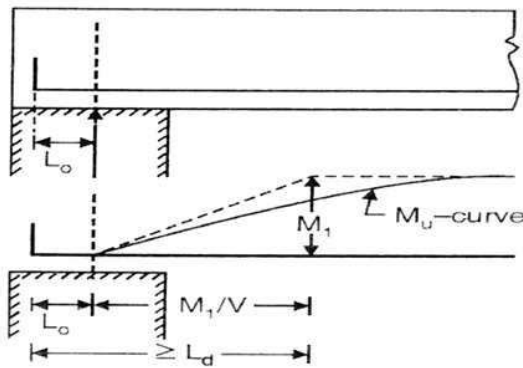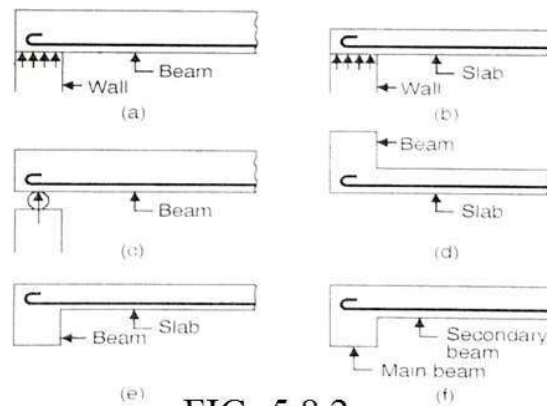
FIG-5.8.1   FIG- 5.8.2

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.
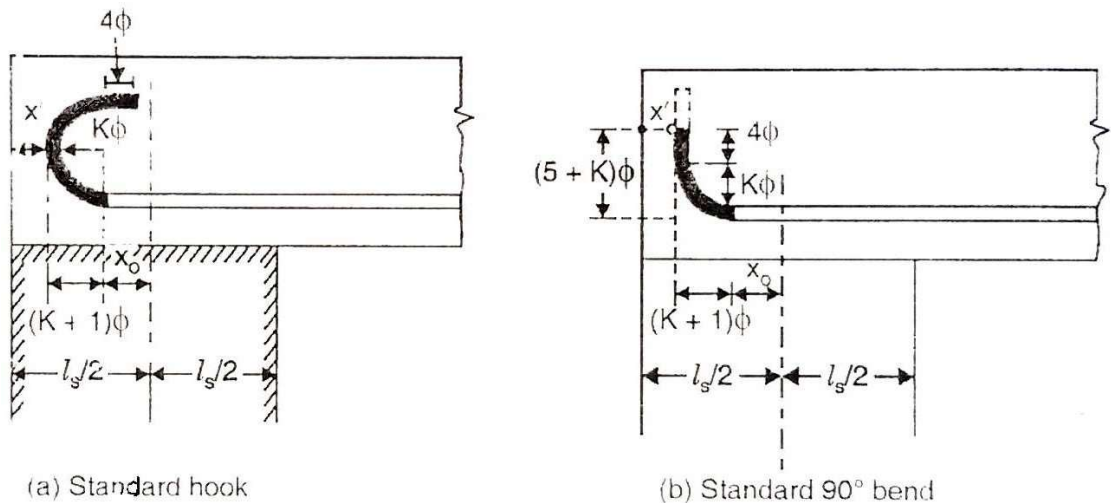
FIG. 5.8.3   COMPUTATION OF L_0

(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 $(K+1)\varphi$. 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 $x_0 = \dfrac{l_s}{2} - x' - (K + 1)\,\varphi$

$\therefore \qquad L_0 = \left(\dfrac{l_s}{2} - x' - (K + 1)\,\varphi \right) + 16\,\varphi = \dfrac{l_s}{2} - x' + (15 - K)\,\varphi$ 5.84

Taking $K = 2$ for mild steel bars, $L_0 = \dfrac{l_s}{2} - x' + 13\,\varphi$ .... 5.84(a)

Taking $K = 4$ for HYSD bars, $L_0 = \dfrac{l_s}{2} - x' + 11\,\varphi$ .... 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 $8\,\varphi$ (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 $x_0 = \dfrac{l_s}{2} - x' - (K + 1)\,\varphi$

$\therefore \qquad L_0 = \left(\dfrac{l_s}{2} - x' - (K + 1)\,\varphi \right) + 8\,\varphi = \dfrac{l_s}{2} - x' + (7 - K)\,\varphi$ 5.8.5

Taking $K = 2$ for mild steel bars, $L_0 = \dfrac{l_s}{2} - x' + 5\,\varphi$ .... 5.8.5(a)

Taking $K = 4$ for HYSD bars $L_0 = \dfrac{l_s}{2} - x' + 3\,\varphi$ ..... 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_{st1}) 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 φ (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.
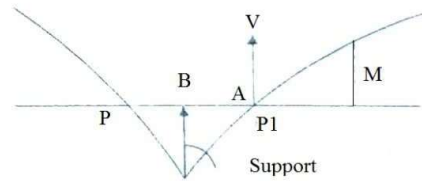
FIG. 5.9 DEVELOPMENT LENGTH AT A POINT OF INFLEXION

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)].
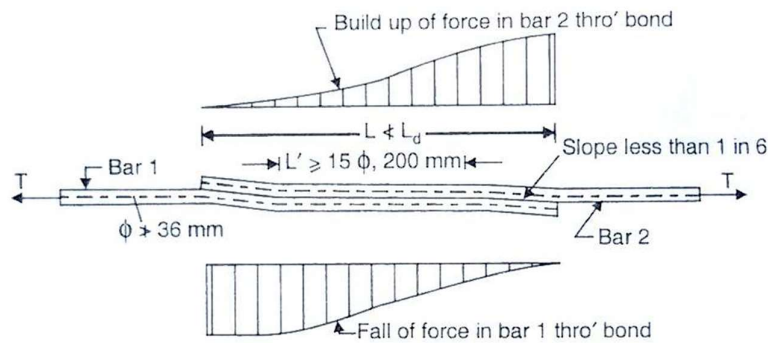
(a) Configuration of lapped portion & transfer of force thro' development bond
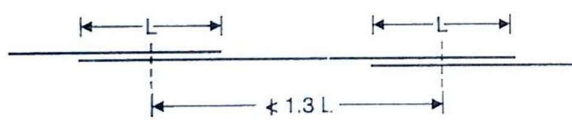
(b) Staggered splicing of bar
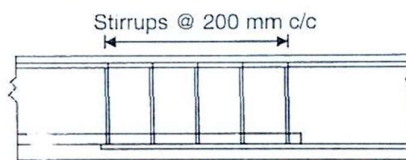
(c) Stirrups at splice point

(d) Use of spirals in lap splices for large diameter bars
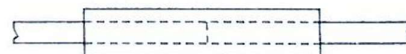
(e) Mechanical joint for $\phi > 36$ mm

(f) Butt welding
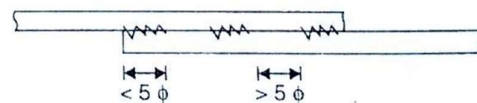
(g) Lap welding of bars

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 σ_y $= 415\ N/mm^2$

Moment of resistance $M_1 = 0.87\ \sigma_y\ A_t\ (d - 0.42\ x)$

$$x = \frac{0.87\,\sigma_y\,A_t}{0.36\,\sigma_{ck}\,b} = \frac{0.87 \times 415 \times 628}{0.36 \times 20 \times 250} = 126\ mm\ <\ x_m \qquad \text{OK}$$

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.

Development length $L_d = \dfrac{\phi\,\sigma_s}{4\,\tau_{bd}} = \dfrac{0.87 \times 415\phi}{4 \times (1.6 \times 1.2)} = 47\ \phi$

If the bar is given a 90° bend at the centre of support, its anchorage value

$$L_o = 8\ \phi = 8 \times 20 = 160\ mm$$

$$L_d \leq 1.3\ M_1/V + L_o$$

$$47\ \phi \leq \left[\frac{1.3 \times 93.45 \times 10^6}{165 \times 1000}\right] + 160$$

or, $\phi \leq 19\ mm$

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_0 to 240 mm. It gives

$$\phi \leq 20.8 \text{ mm} \qquad\qquad \text{OK}$$

The arrangement of 90° bend is shown in Fig. 8.19a.

Alternatively

Provide a U bend at the centre of support, its anchorage value,

$$L_o = 16\phi = 320 \text{ mm}$$
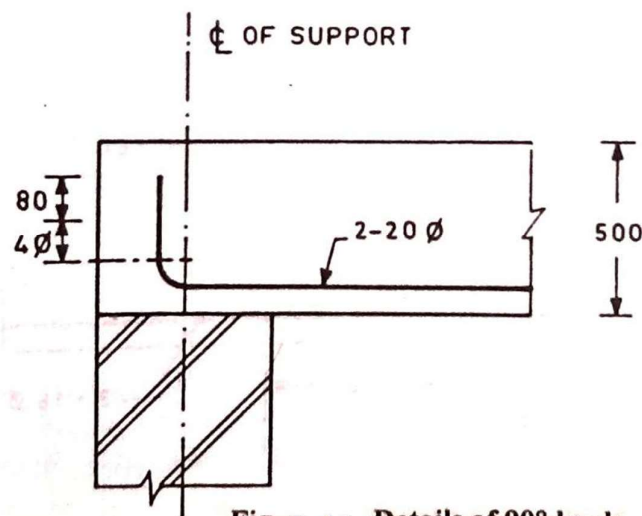
Fig. Ex 1.1 **Details of 90° hook**

$L_d \leq 1.3 \, M_1/V + L_0.$

$$47\phi \leq \left[\frac{1.3 \times 93.45 \times 10^6}{165 \times 1000}\right] + 320$$

Or.　$\phi \leq 22.47$ 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.
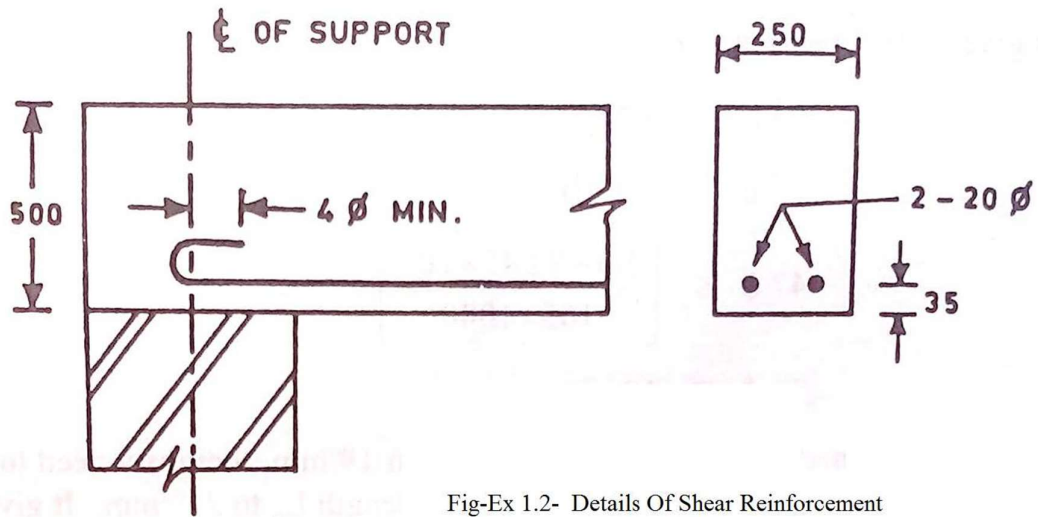
Fig-Ex 1.2- Details Of Shear Reinforcement

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 ?
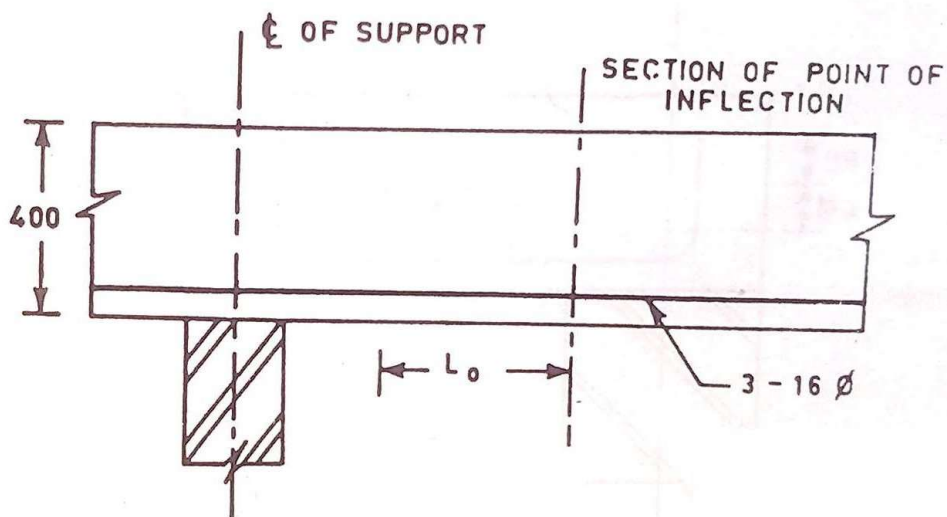
Fig- Ex 1.3 - Section of Continious Beam

Depth of neutral axis $\quad x \quad = \quad \dfrac{0.87\,\sigma_y\,A_t}{0.36\,\sigma_{ck}\,b} = \dfrac{0.87 \times 415 \times 3 \times \pi/4 \times 16^2}{0.36 \times 20 \times 250}$

$\qquad\qquad = \quad 120 \text{ mm} \ < \ x_m \ (= 0.48\, d) \qquad\qquad\qquad \text{OK}$

Moment of resistance $M_1 \quad = \quad 0.87\,\sigma_y\,A_t\,(d - 0.42\,x)$

$\qquad\qquad = \quad 0.87 \times 415 \ \times 603\,(367 - 0.42 \times 120) = 68.90 \times 10^6 \text{ Nmm}$

Development length $\quad L_d \quad = \quad \dfrac{\sigma_s\,\phi}{4\,\tau_{bd}}$

$\qquad$ Bond stress $\ \tau_{bd} \quad = \quad 1.6 \times 1.2 \text{ N/mm}^2$ for M20 mix and HSD steel

or $\qquad\qquad L_d \quad = \quad \dfrac{0.87 \times 415\phi}{4 \times 1.6 \times 1.2} = 47\,\phi$

$\qquad$ Anchorage length $\ L_o = \ $ greater of d or $12\,\phi$

$\qquad\qquad = \ $ greater of 367 mm, or $12 \times 16 = 192$ mm

$\qquad\qquad = \ 367$ mm

$\qquad\qquad L_d \ \le \ \dfrac{M_1}{V} + L_o$

or $\qquad\qquad 47\,\phi \ \le \ \dfrac{68.9 \times 10^6}{150 \times 1000} + 367 \quad$ or, $\quad \phi \le 17.6$ 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)

7.1 Control of deflection and ensuring lateral stability in beams as per Clause 23.2 &   23.3 of   IS-456.

7.2 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:
  7.1 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 $100b^2/d$.

<h2 style="text-align:center;color:red;">Span/depth ratios and modification factors</h2>

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 $b_f d$.		

7.2 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

$$d_{bal} = \sqrt{\frac{M_u}{Q_{lim} \, Xb}}$$

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 $\frac{M_u}{bd^2}$, 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 $d_{bal} = \sqrt{\frac{M_u}{Q_{lim} \, Xb}}$ (Assume b)

For a given factored moment, width & depth of Section,final

mu$_{lim}$ = Q$_{lim}$xbd^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 $r_v = \dfrac{V_u}{bd}$

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) $V_{us} = \dfrac{0.87 f_y \, \cancel{E}_{sv} \times d}{S_v}$

Vus =Exces S.F to be   resisted

Asv=Area of shear reinforcement.

SV = Spacing of strirrups

$Sv = \dfrac{0.87 f_y \, A_{sv}}{d} \; V_{us}$

(iii) For inclined stirrups

$V_{us} = \dfrac{0.87 \, f_y \, \cancel{E}_{sv} \times d \, (sin\alpha + cos\,\alpha)}{S_v}$

Or

$Sv \dfrac{0.87 \, fy \, \cancel{E}sv \, d(Sin\alpha + Cos\alpha)}{Vus}$

(iv) For Bent up bars

$V_{us} = 0.87 \, f_y \, A_{sv} \cdot sin\alpha$

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°

Minimum shear reinforcement .

Spacing of shear stirrups should not exceed the following.

$$\frac{Asv}{bSv} \geq \frac{0.4}{0.87\,fy}$$

(i) $S_v = \dfrac{0.87\,fy\,Æsv}{0.4\,b}$

(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 = $30 \times \dfrac{4^2}{8}$ = 60 kNm and $V_{u,max}$ = $30 \times \dfrac{4}{2}$ = 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.

$$\frac{Mu}{bd^2} = \frac{60 \times 10^6}{230\ x\ 412\ x412} = 1.54$$

From SP 16 table no 2 P_t = 0.474

$$A_{st} = \frac{0.474 \times 230 \times 412}{100} = 449 mm^2$$

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 mm² < $A_{st, provided}$ < 910 mm².

Provide 4 no. 12mm # =4x113 =452 mm²

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

$$\tau_v = \frac{60 \ x \ 10^3}{230 \ x \ 412} = 0.633 \ N/mm^2.$$

$$\frac{100 \ A_s}{bd} = \frac{2 \ x \ 113 \ x \ 100}{230 \ x \ 412} = 0.238$$

$.r_c$ =0.332 N/mm² <0.633 N/mm².

∴ 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 45º 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 - τ_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 $\frac{0.87 fy \ .Æsv.d}{Vus}$

Where A_{sv} = 2 x 28 = 56 mm²

∴ $S_v = \dfrac{0.87 \times 250 \times 56 \times 412}{14.27 \ 10^3} = 351$ 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) $S_v = \dfrac{0.87\,fy\,\text{Æsv}}{0.4\,b}$

(ii) 0.75 d (d = Effective depth)

(iii) 300 mm c/c distance

Using 2legged 6 mm ø mild steel stirrup $S_v = \dfrac{0.87 \times 250 \times 56}{0.4 \times 232}$ = 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$_{lim}$ x bd^2

$$= 0.36 f_{ck} \, b. \, x_{umax} \, (d - 0.42 x_{umax})$$

& A$_{stlim}$ = $\dfrac{M_{ulim}}{0.81 \, f_y \, (d - 0.42 xumax)}$

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, $A_{sc} = \dfrac{Mu2}{f_{sc}(d-d')}$

(iv) Corresponding Ast Tensile stepA$_{st2}$ can be found out from

$$A_{st2} = \dfrac{Æ_{sc} f_{sc}}{0.87 \, f_y}$$

(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 = $\dfrac{46.00 \, kN/m}{Total \, 49.45 \frac{kN}{m},}$ *say* 50 kN/m

Factor load = 1.5 x 50 = 75 kN/m

M$_u$ = $\dfrac{5^2}{8}$x 75 = 243.4 kNm

V$_u$ = $\dfrac{5}{2}$ x 75 = 187.5 kN

Moment steel:

Assuming 2 layers of 20 mm diameter bars

D = 600 – 30 – 20 -10 = 540 mm

$\dfrac{Mu}{bd2} = \dfrac{243.4.x \, 10^6}{230 \, x \, 540^2}$ = 3.63 > 2.76

∴ Doubly reinforced section

M$_{u,lim}$ = 2.76 x 230 x 540^2 x 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

$\dfrac{d'}{d} = \dfrac{40}{545} = 0.07$, consider $\dfrac{d'}{d} = 0.1$

Stress in compression steel, fsc = 353 N/mm² (refer to table f of S.P-16)

$$A_{sc} = \dfrac{Mu2}{f_{sc}(d-d')} = \dfrac{58.3 \times 10^6}{353(540-40)} = 330 \text{ mm}^2$$

Corresponding tension steel

$$A_{st2} = \dfrac{A_{sc}f_{sc}}{0.87\, f_y} = \dfrac{330\times353}{0.87\times415} = 323 \text{ mm}^2$$

$$A_{st1} = A_{st,lim} = P_{t,lim} \times \dfrac{bd}{100}$$

$$= 0.96 \times \dfrac{230 \times 540}{100} = 1192 \text{ mm}^2$$

Ast = Ast1 + Ast2 = 1192 + 323 mm² = 1515 mm²

Asc = 2-16 # = 402 mm²

Ast = 5-20# = 1570 mm² (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:

$$r_v = \dfrac{187.5 \times 10^3}{230 \times 540} = 1.51 \text{ N/mm}^2 < 2.8 \text{ N/mm}^2$$

$$\dfrac{100\, A_s}{bd} = \dfrac{100 \times 5 \times 314}{230 \times 540} = 1.26$$

$$r_c = 0.672 \text{ N/mm}^2$$

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 $\dfrac{0.87fy\, .A_{sv}.d}{Vus}$

Where A_{sv} = 2 x50 = 100 mm²

$$\therefore S_v = \frac{0.87 \times 415 \times 100 \times 540}{63.5 \times 10^3} = 307 \text{ mm} \dots\dots\dots\dots\dots\dots\dots (1)$$

At distance of 550 mm from support, where two bars are bent

$V_u = 187.5 - (0.540 \times 75) = 147 \text{ kN}$

$V_{us} = 147 - 0.672 \times 230 \times 540 \times 10^{-3} = 63.5 \text{ kN}$

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) $S_v = \dfrac{0.87\, fy\, \text{Æ}sv}{0.4\, b}$

(ii) 0.75 d (d = Effective depth)= 0.75 x 540 =405 mm

(iii) 300 mm c/c distance

Using 2legged 8 mm # stirrup $S_v = \dfrac{0.87 \times 415 \times 100}{0.4 \times 230} = 392.4 \text{ mm}$

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.

Simply Supported Beams

(a) Elevation

(b) Section A-A
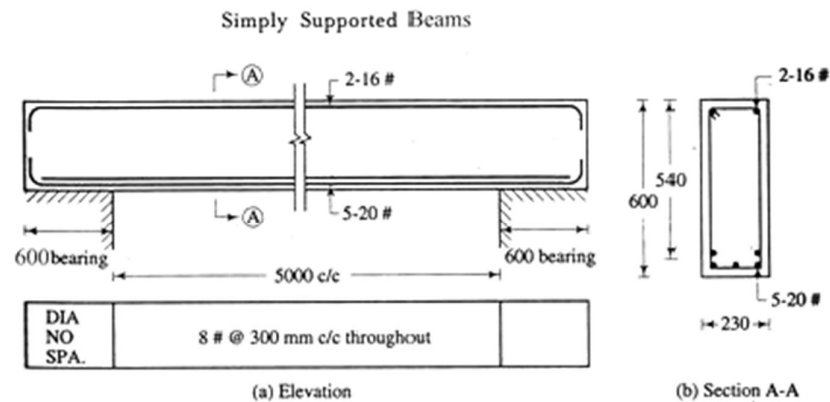
7.3:Design of T beams as per IS 456 for bending and shear

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 $A_t = \frac{Mu}{0.87 \times f_y \times Z}$

(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 $D_f \leq \frac{3}{7} x_u$

$Mu = 0.36f_{ck}\,b.x_u\,(d - 0.42x_u) + 0.446\,f_{ck}\,(b_f - b_w) \times D_f\,(d - \frac{D_f}{2})$

When $D_f \geq \frac{3}{7} x_u$

$Mu = 0.36f_{ck}\,b.x_u\,(d - 0.42x_u) + 0.446\,f_{ck}\,(b_f - b_w) \times D_f\,(d - \frac{Y_f}{2})$

Where Y$_f$ =0.15 x_u+0.65D_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) $V_{us} = \dfrac{0.87 f_y\,\text{Æ}_{sv} \times d}{S_v}$

   Vus =Exces S.F to be   resisted
   Asv=Area of shear reinforcement.
   SV = Spacing of strirrups

   $Sv = \dfrac{0.87 f_y\,A_{sv}}{d}\ V_{us}$

(vii)   For inclined stirrups

   $V_{us} = \dfrac{0.87\,f_y\,\text{Æ}_{sv} \times d\,(sin\alpha + cos\,\alpha)}{S_v}$

   Or

   $Sv\,\dfrac{0.87\,fy\,\text{Æsv}\,d(Sin\alpha + Cos\alpha)}{Vus}$

(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°

e Minimum shear reinforcement .

Spacing of shear stirrups should not exceed the following.

$$\frac{Asv}{bSv} \geq \frac{0.4}{0.87\,fy}$$

(i) $S_v = \dfrac{0.87 \, fy \, \text{Æsv}}{0.4 \, b}$

(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 × 314 = 1570 mm².

The section is now checked for moment of resistance.

$$b_f = \frac{l_0}{6} + b_w + 6 \, D_f \ngtr 3000$$

$$= \frac{7000}{6} + 230 + 6 \times 100 = 1996 \text{ mm.}$$

Use

$$b_f = 1990 \text{ mm}$$

$$F_{tc} = 0.36 \, f_{ck} \, b_f \, D_f$$

$$= 0.36 \times 20 \times 1990 \times 100 \times 10^{-3} = 1432.8 \text{ kN}$$

$$F_{ts} = 0.87 \, f_y \, A_{st}$$

$$= 0.87 \times 415 \times 1570 \times 10^{-3} = 566.8 \text{ kN}$$

$$F_{tc} > F_{ts}$$

∴ Neutral axis lies in flange.

Equating the forces

Total compression = total tension

$$0.36 \, f_{ck} \, b_f \, x_u = 0.87 \, f_y \, A_{st}$$

$$036 \times 20 \times 1990 \, x_u = 0.87 \times 415 \times 1570$$

∴

$$x_u = 39.56 \text{ mm}$$

$$x_{u,max} = 0.48 \, d = 0.48 \times 540 = 259.2 \text{ mm}$$

$$x_u < x_{u,max}.$$

∴ Under-reinforced section.

$$M_u = 0.87 \, f_y \, A_{st} \, (d - 0.42 \, x_u)$$
$$= 0.87 \times 415 \times 1570 \times (540 - 0.42 \times 39.56) \times 10^{-6}$$
$$= 296.7 \text{ kNm} > 275.6 \text{ kNm} \dots\dots\dots\dots\dots \text{(O.K.}$$

Let 2 bars bent up at $1.25 \times 600 = 750$ mm from the face of the support.

(b) *Check for deve*

At support, $A_{st} = 3 \times 314 = 942$ mm².

$$0.36 \, f_{ck} \, b_f \, x_u = 0.87 \, f_y \, A_{st}$$
$$\therefore \quad 0.36 \times 20 \times 1990 \, x_u = 0.87 \times 415 \times 942$$
$$x_u = 23.74 \text{ mm.}$$
$$M_{u1} = 0.87 \, f_y \, A_{st} \, (d - 0.42 \, x_u)$$
$$= 0.87 \times 415 \times 942 \times (540 - 0.42 \times 23.74) \times 10^{-6}$$
$$= 180.2 \text{ kNm}$$
$$V_u = 157.5 \text{ kN.}$$

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 $L_0 = 12 \, \#$

$$\therefore \quad 1.3 \times \frac{180.2 \times 10^6}{157.5 \times 10^3} + 12 \, \# \geq 47 \, \#$$
$$\therefore \quad 42.5 \geq \#$$
$$\#_{\text{provided}} = 20 \text{ mm} \dots\dots\dots\dots\dots\dots\dots\dots\dots \text{(Safe)}$$

(c) *Check for shear:*

As the ends are confined by compressive reaction, shear at distance d will be used at the support.

$$\therefore \quad V_u = 157.5 - 0.540 \times 45 = 133.2 \text{ kN}$$
$$\frac{100 \, A_s}{b \, d} = \frac{100 \times 942}{230 \times 540} = 0.758$$
$$\tau_c = 0.562 \text{ N/mm}^2$$
$$\tau_v = \frac{133.2 \times 10^3}{230 \times 540} = 1.06 \text{ N/mm}^2 > \tau_c$$

∴ Shear design is necessary.

2-20 # bars can resist a shear of

$$0.87 \times 415 \times 2 \times 314 \times \sin 45° \times 10^{-3} = 160.32 \text{ kN.}$$
$$V_{us} = V_u - \tau_c \, b \, d$$
$$= 133.2 - 0.562 \times 230 \times 540 \times 10^{-3} = 63.40 \text{ kN.}$$

Bent bars share 50 % = 31.7 kN

Stirrups share 50 % = 31.7 kN.

Using 6 mm ϕ M.S. two-legged stirrups, $A_{sv} = 56$ mm²

Use 2-10 # as anchor bars. The designed section is shown in fig.

(a) Elevation

(b) Section A-A
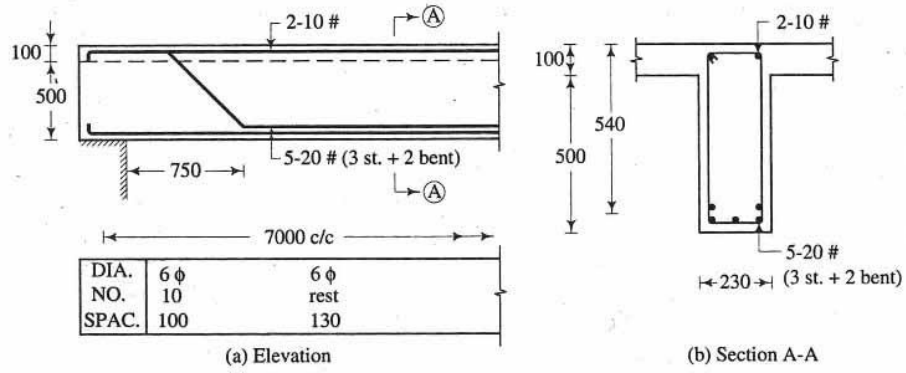
CHAPTER O8
INTRODUCTION TO SLABS BY LIMIT STATE METHOD

SLABS

8.1 :   One way slab & two way slab

8.2 :   One way & cantilever slabs as per IS-456 for bending & shear.

8.3:   Explain the action of two way slabs with defected shape.

8.4 :   Provision for two way slab as per IS-456-2000

8.4.1:   Middle strip & edge strip

8.4.2 :   B.M Co-Efficient

8.4.3 :   Torsion reinforcement

8.4.4 :   Design of two way slab as per IS-456-2000 & SF-16 hand hook

8.4.5 :   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 $\leqslant$ 2lx slabs bend in both directions. Such slabs are called two way spanning slab.

$$\frac{ly}{lx} < 2$$

8.2 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 $= \dfrac{area\ of\ one\ bar\ x\ 1000}{required\ area\ in\ mm^2\ per\ meter}$

Distribution Steel   on Step   Main Steel

$\left(\text{Simply supported one way slab}\right)$

Main Steel   Distribution Steel
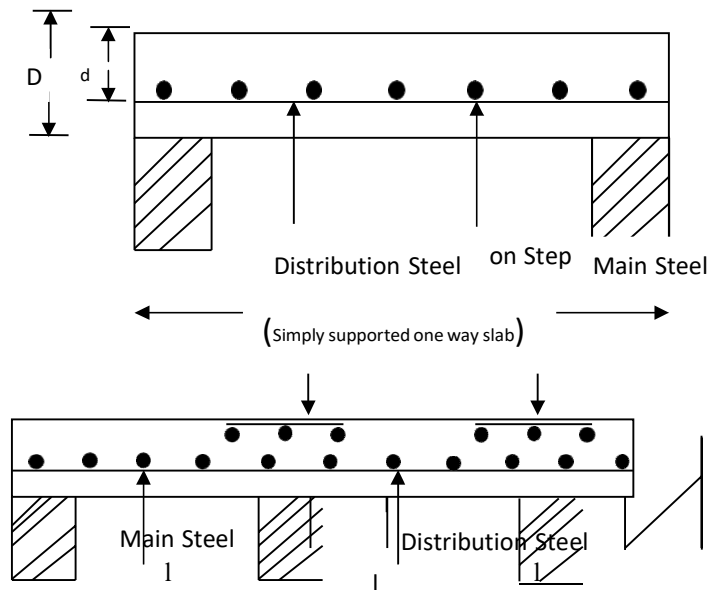
2. Reinforcement requirement

   (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 $1/8^{th}$ 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 τ_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 $\cancel{12}$ mm clear cover = 15mm

Ø> 12mm, clear cover = 20mm

Development Length:

0.15L 50%   50%, 0.15L

Beam

(With out Bent bars)
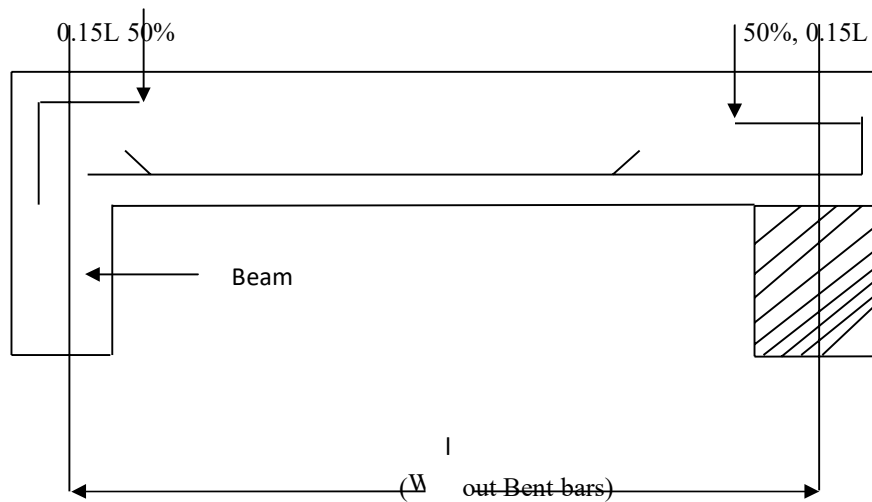
(Typical details of simply supports slab)

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/m².

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/mm². Basic $\dfrac{\text{span}}{d}$ ratio = 20. Also modification factor from fig. 8-1 is 1.45.

Then permissible $\dfrac{\text{span}}{d}$ ratio = $20 \times 1.45 = 29$. The depth $d = \dfrac{3100}{29} = 106.9$ 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.

$$DL = 0.13 \times 25 = 3.25 \text{ kN/m}^2$$

$$\text{Floor finish} = 1.00 \text{ kN/m}^2$$

$$\text{Live load} = \underline{2.00 \text{ kN/m}^2}$$

$$\text{Total} \quad 6.25 \text{ kN/m}^2$$

$$\text{Factored load} = 1.5 \times 6.25 = 9.4 \text{ kN/mm}^2.$$

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.

$$\text{Maximum moment} = 9.4 \times \frac{3.11^2}{8} = 11.36 \text{ kNm.}$$

$$\text{Maximum shear} = 9.4 \times \frac{3}{2} = 14.1 \text{ kN (based on clear span).}$$

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)

$d_{\text{provided}} = 130 - 15 \text{ (cover)} - 5 \text{ (assume 10 } \phi \text{ bar)}$

$= 110 \text{ mm} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \text{ (O.K)}$

Design for flexure:

$$\frac{M_u}{b\,d^2} = \frac{11.36 \times 10^6}{1000 \times 110^2} = 0.939$$

$$p_t = \frac{100\,A_{\text{st}}}{b\,d} = 0.281$$

$\therefore$ $A_{\text{st}} = \dfrac{0.281 \times 1000 \times 110}{100} = 309 \text{ mm}^2.$

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 $= \dfrac{0.15}{100} \times 1000 \times 130 = 195 \text{ mm}^2,$ using mild steel.

Maximum spacing $5 \times 100 = 500$ 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 $= \dfrac{13.5 \times 10^3}{1000 \times 111} = 0.122$ N/mm^2 < 0.364 N/mm^2 $\dots\dots$ (safe)

Check for development length:

Refer to fig. 10-5.

$L_0 = 145$ mm. L_0 is limited to $12\ \phi = 96$ mm or

$$d = 100 \text{ mm, whichever is greater.}$$

$$L_0 = 100 \text{ mm.}$$

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 $\qquad\qquad 15.4 \text{ mm} \geq \#$ .... (O.K.)

Check for deflection :

$$\text{Basic } \frac{\text{span}}{d} \text{ ratio} = 20$$

$$p_t = \frac{100 \times 335}{1000 \times 111} = 0.3$$

$$\text{service stress} = 0.58 \times 415 \times \frac{303}{335} = 218 \text{ N/mm}^2.$$

$$\text{modification factor} = 1.62$$

$$\text{permissible } \frac{\text{span}}{d} \text{ ratio} = 20 \times 1.62 = 32.4$$

$$\text{actual } \frac{\text{span}}{d} \text{ ratio} = \frac{3100}{111}$$

$$= 28 < 32.4 \text{ .... (O.K.)}$$

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

$$= 3 \times 100 = 300 \text{ mm.}$$

Actual spacing $= 150$ mm < 300 mm .... (O.K.)

For distribution steel, maximum spacing permitted

$$= 5 \times 100$$

$$= 500 \text{ or } 450 \text{ mm, i.e., } 450 \text{ mm.}$$

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**

6 mm φ @ 140 c/c
8 mm # @ 150 mm c/c alternate bent

335   335

3350 c/c
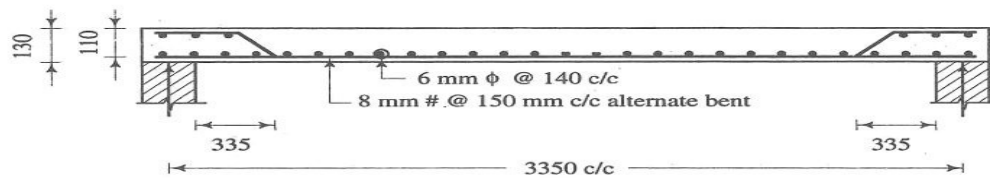
Design the slab S2 – S1 of above figure , if it is to be used for residential purpose at the free end of slab S1 there is a concrete parapet of 75mm thick and 1 m high. The materials are M20 grade concrete and HYSD reinforcement of grade Fe 415 . Use IS:875 for live loads.

Solution

Estimation of loads:

For slab S2 live load shall be 2 kN/m². For slab s1 which is a balcony slab live load shall be 3 kN/m². Assume 120 mm thick slab.

Slab S_2 :   Self load $= 0.12 \times 25$ $= 3 + 0 \text{kN/m}^2$

Floor finish $= 1 + 0 \text{ kN/m}^2$

Live load $= 0 + 2 \text{ kN/m}^2$

Total $\underline{= 4 + 2 \text{ kN/m}^2}$

P_u $= 1.5(4+2)$

$= (6+3) \text{ kN/m}^2$

Slab$_1$:   Self load $= 3 + 0 \text{kN/m}^2$

Floor finish $= 1 + 0 \text{ kN/m}^2$

Live load $\underline{= 0 + 3 \text{ kN/m}^2}$

Total $= 4 + 3 \text{ kN/m}^2$

P_u $= 1.5(4+3)$

$= 6+4.5 \text{ kN/m}^2$

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 $= \frac{9 \times 3}{2} - \frac{4.32}{3} = 12.06$ kN

Point of zero shear from A $= \frac{12.06}{9} = 1.34$ m.

Maximum positive moment $= 12.06 \times 1.34 - \frac{1.34^2}{2} \times 9$

$$= 16.16 - 8.08 \text{ kNm.}$$

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 $= \frac{1.2^2}{2} \times 10.5 + 1.2 \times 2.8$

$$= 7.56 + 3.36 = 10.92 \text{kNm.}$$

Maximum shear at B, $V_u,BA = \frac{9 \times 3}{2} + \frac{10.92}{3} = 13.5 + 3.64 = 17.14 \ kN$

$$V_u,BC = 10.15 \times 1.2 + 2.8 = 15.4 \ kN.$$

C) Moment steel:

Maximum moment = 10.92 kN.

$$d_{required} = \sqrt{\frac{10.92 \times 10^6}{1000 \times 2.76}} = 62.9 \ mm$$

$$d_{provided} = 120 - 15 - 5 \ \text{(assume 10\# bar)}$$

$$= 100 \text{ mm}.. \text{(O.K)}$$

$$\frac{M_u}{bd^2} \ (+) = \frac{8.08 \times 10^6}{1000 \times 100 \times 100} = 0.81$$

$$Pt = 0.236$$

$$\text{Ast } (+) = \frac{0.236 \times 1000 \times 100}{100} = 2.36 \ mm^2$$

$$\frac{M_u}{bd^2} \ (-) = \frac{10.92 \times 10^6}{1000 \times 100 \times 100} = 1.09$$

$$\text{Pt} = 0.324$$

$$\text{Ast } (-) = \frac{0.324 \times 1000 \times 100}{100} = 324 \ mm^2$$

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.

$$\text{Minimum steel} = \frac{0.12}{100} \times 1000 \times 120 = 144 \ mm^2 \ .$$

Remaining positive moment bars give As $= \frac{294}{2} = 147$ mm^2 Thus bar can be bent up.

$$\text{Distribution steel} = \frac{0.15}{100} \times 1000 \times 120 = 180 \ mm^2 \ .$$

Provide 6 mm Ⓒ about 150 mm c/c =187 mm^2 .

For negative moment reinforcement

$$\text{Ld} = 47 \ \#$$

$$\text{Ld} = 47 \times \frac{(8+10)}{2} = 423 \text{ mm}.$$

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 :

$$\text{At A, Mul} = 0.87 \ f \ A_{st} \ (d - \frac{f_y \cancel{A}_{st}}{bf_{ck}})$$

$$= 0.87 \times 415 \times 147 \ (100 - \frac{415 \times 417}{1000 \times 20}) \times 10^{-6}$$

$$= 5.15 \text{ kNm}.$$

$$\text{Vu} = 12.06 \text{ kN}.$$

Consider L0 = 8 #

Then $1.3 \dfrac{M_{ul}}{V_u} + L_o \geq L_d$

$1.3 \times \dfrac{5.15 \times 10^6}{12.06 \times 10^3} + 8 \ \# \ \geq 47 \ \#$

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.

$1.3 \times (5.15 \times 10^6 / 13.09 \times 10^3 \ ) + 8 \# \geq 47 \#$

$\# \leq 13.11$ 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=13.09x10³/1000x100=0.131N/mm²

$\dfrac{100 \, \cancel{E}_s}{bd} = 100x147/1000x100 = 0.147$

τc=0.28

kx τc=0.28x1.3=0.364N/mm²>τv

Span BC

:,V_u, =17.14 KN

Shear stress τ_v=17.14 x10³/1000x100=0.171N/mm²

$\dfrac{100 \, \cancel{E}_s}{bd} = 100x378/1000x100 = 0.378$

τc=0.28

kx τc=0.421x1.3=0.547N/mm²>τv

Longitudinal section through $S_2 - S_1$
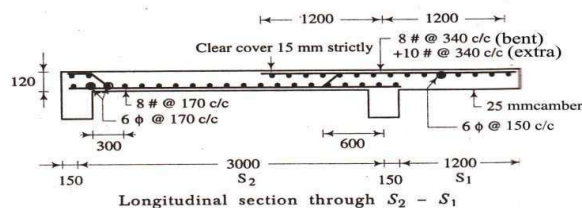
Two way Slabs:

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
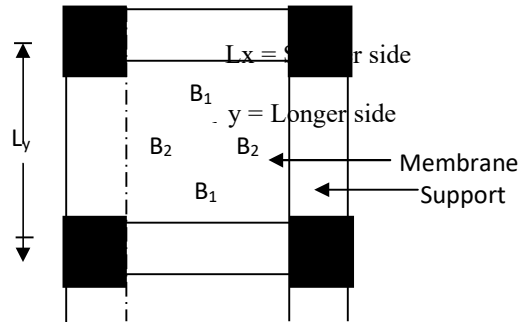
Mx = ∂x.w.Lx^2

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.
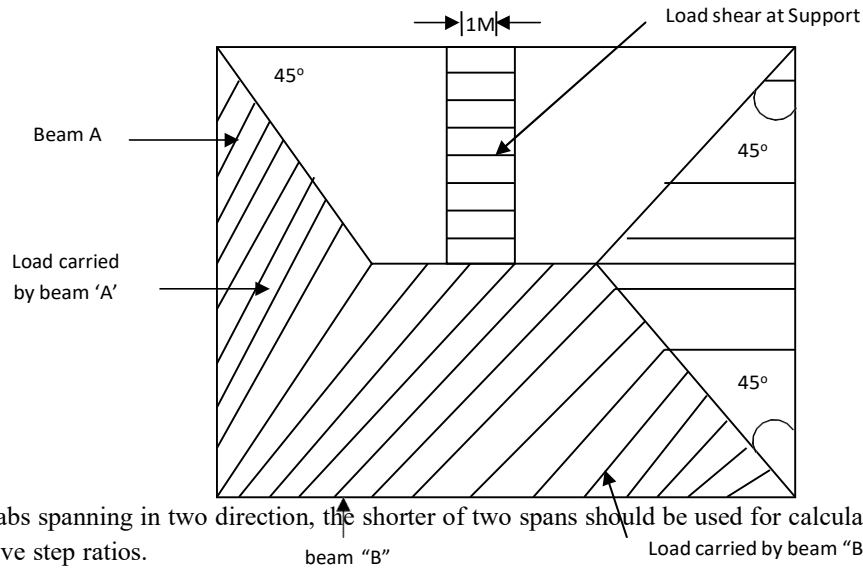
For slabs spanning in two direction, the shorter of two spans should be used for calculating the span to effective step ratios.

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
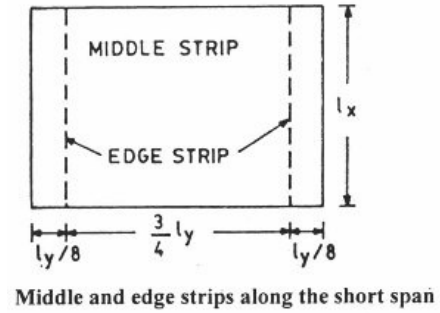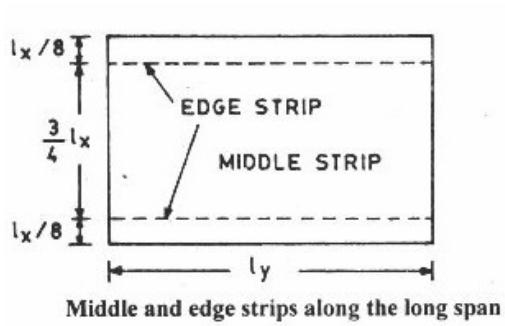
Bending moment Coefficients with torsion reinforcement

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

IS 456 : 2000

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:
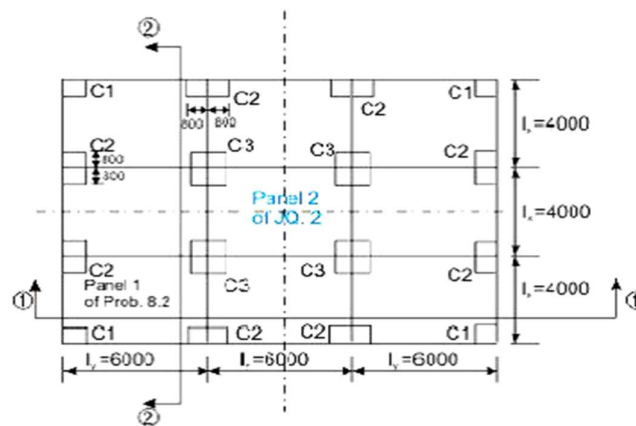
Fig. 8.19.7: Problem 8.2 (panel 1) and TQ 2 (panel 2)

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 $\tau cmax$ = 2.8 N/mm^2. The computed shear stress τv = 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
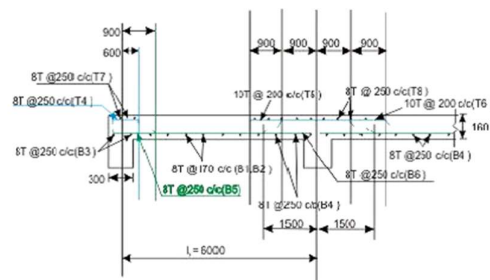
Fig. 8.19.8: Problem 8.2, Sec 1-1 of Panel 1 of Fig. 8.19.7

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
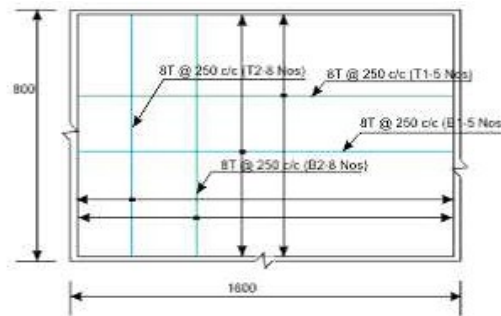
Fig. 8.19.10(c): Corners C2

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.

$$l_x = 4.3 + 0.16 = 4.46 \text{ say } 4.5 \text{ m.}$$

$$l_y = 6.55 + 0.16 = 6.71 \text{ say } 6.75 \text{ m.}$$

$$\text{Dead load: self } 0.18 \times 25 = 4.5 \text{ kN/m}^2$$

$$\text{floor finish} = 1.0 \text{ kN/m}^2$$

$$\text{Live load (residence)} = \underline{2.0 \text{ kN/m}^2}$$

$$\text{Total} \quad 7.5 \text{ kN/m}^2$$

For 1 m wide strip

$$P_u = 1.5 \times 7.5 = 11.25 \text{ kN/m.}$$

$$\frac{l_y}{l_x} = \frac{6.75}{4.5} = 1.5$$

$$M_{ux} = 0.104 \times 11.25 \times 4.5^2 = 23.7 \text{ kNm}$$

$$M_{uy} = 0.046 \times 11.25 \times 4.5^2 = 10.48 \text{ kNm}$$

$$d_{required} = \sqrt{\frac{23.7 \times 10^6}{1000 \times 2.76}} = 92.7 \text{ mm}$$

$$d_{short} = 180 - 15 \text{ (cover)} - 5$$

$$= 160 \text{ mm} > 92.7 \text{ mm} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \text{(O.K.)}$$

$$d_{long} = 160 - 10 = 150 \text{ mm}$$

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 mm^2 in short span and 8 mm ϕ about 230 mm c/c = 217 mm^2 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:

$$\text{Long span } V_u = 11.25 \times 2.25 = 25.31 \text{ kN.}$$

$$M_{u1} = 0.87 \times 415 \times 217 \left(150 - \frac{415 \times 217}{1000 \times 20}\right) \times 10^{-6}$$

$$= 11.40 \text{ kNm.}$$

Assuming $L_0 = 8 \phi$

$$1.3 \times \frac{11.40 \times 10^6}{25.31 \times 10^3} + 8 \phi \geq 47 \phi$$

which gives $\qquad \phi \leq 15.01$ mm .... (O.K.)

$$\text{short span } V_u = 11.25 \times 2.25 = 25.31 \text{ kN.}$$

$$M_{u1} = 0.87 \times 415 \times 436 \left(160 - \frac{415 \times 436}{1000 \times 20}\right) \times 10^{-6}$$

$$= 23.76 \text{ kNm.}$$

Assuming $L_0 = 8 \phi$

$$1.3 \times \frac{23.76 \times 10^6}{25.31 \times 10^3} + 8 \phi \geq 47 \phi$$

which gives $\qquad \phi \leq 31.3$ mm .... (O.K.)

Note that the bond is usually critical along long direction.

Check for shear:

This is critical along long span

$$\text{Shear stress} \qquad \tau_v = \frac{25.31 \times 10^3}{1000 \times 150} = 0.169 \text{ N/mm}^2$$

$$\frac{100 A_s}{bd} = \frac{100 \times 217}{1000 \times 150} = 0.145$$

$$\tau_c = 0.28 \text{ N/mm}^2$$

$$k\tau_c = 0.28 \times 1.2 = 0.336 \text{ N/mm}^2. \dots\dots\dots\dots\dots \text{(O.K.)}$$

Check for deflection:

The deflection shall be checked along short span.

$$\text{Basic } \frac{\text{span}}{d} \text{ ratio} = 20$$

$$\frac{100\,A_s}{b\,d} = \frac{448 \times 100}{1000 \times 160} = 0.28.$$

$$\text{service stress} = 0.58 \times 415 \times \frac{435}{448} = 234 \text{ N/mm}^2$$

Note that $A_{st,req}$ is used here.

Modification factor $= 1.5$

Permissible $\dfrac{\text{span}}{d}$ ratio $= 20 \times 1.5 = 30$

actual $\dfrac{\text{span}}{d}$ ratio $= \dfrac{4480}{160} = 28$ .... (O.K)

Check for cracking:

Maximum spacing permitted for short span steel

$$= 3 \times 160 = 480 \text{ or } 300 \text{ mm, i.e., } 300 \text{ mm}$$

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
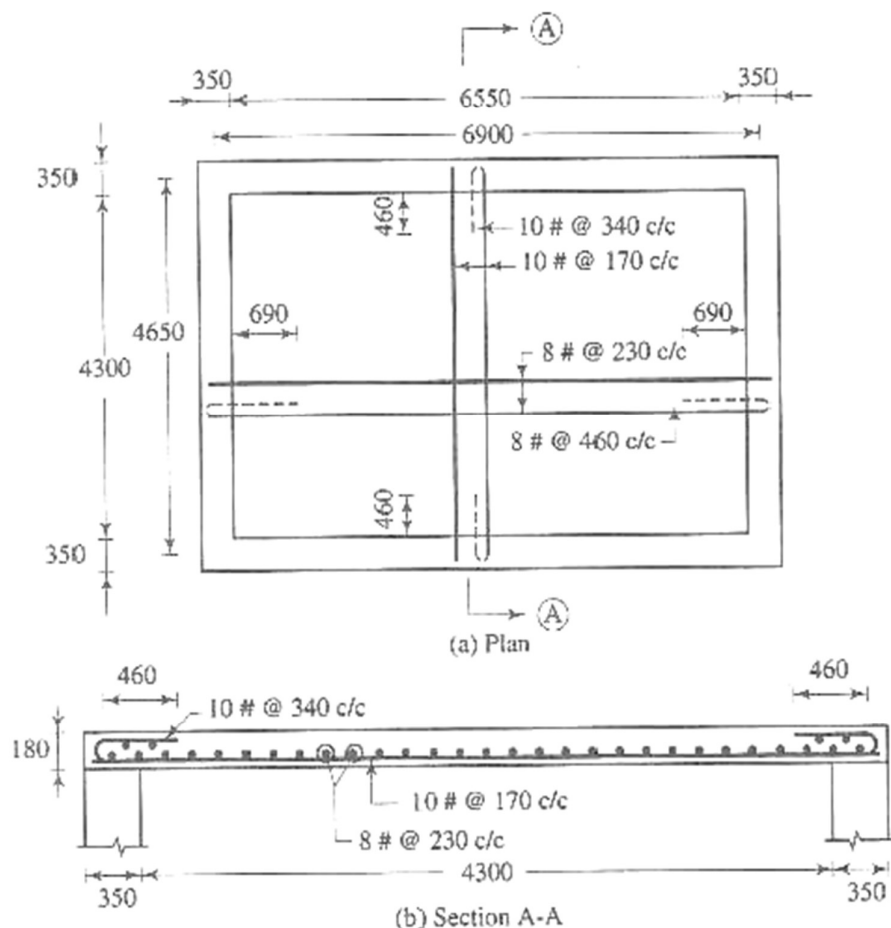
Note: It is possible to bend short span reinforcement alternately. Rework the problem

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 $0.67 \, \partial_{ck} / 1.5$ or $0.446 \, \partial_{ck}$.
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.
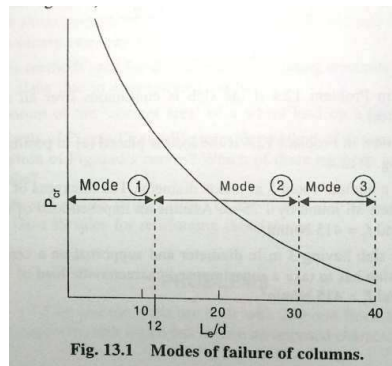
Mode :1 Pure compression failure

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.

Braced and unbraced frames
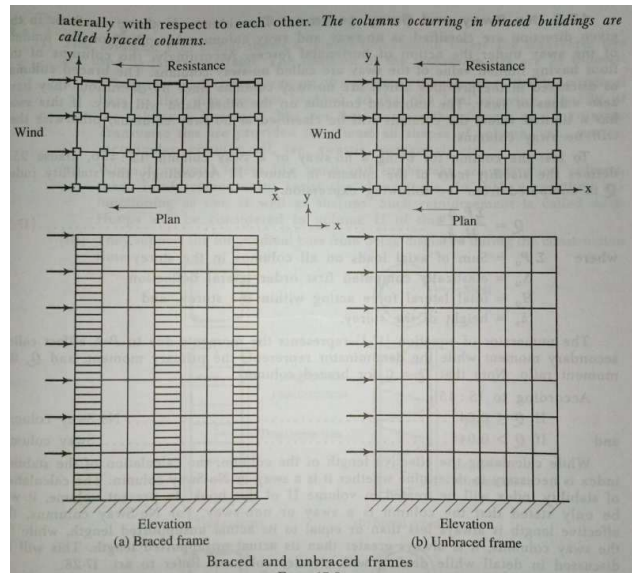
(2) **Unbraced columns :** A unbraced frames no special bracing systems are provided to resist horizontal forces. In other words the sidesway or joint translation do occur in such columns. The columns shall have to be designed to resist the lateral loads. The column those occur in the buildings where 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).
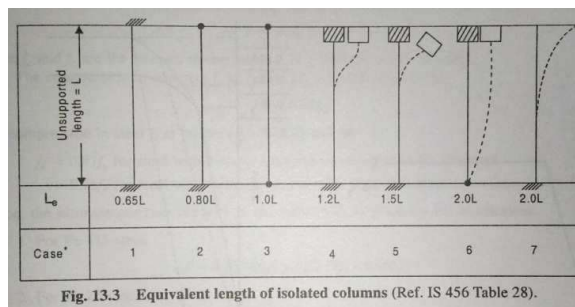
Design formula for Short column –

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

(a) Section   (b) Strain diagram   (c) Stress diagram

Axially loaded column
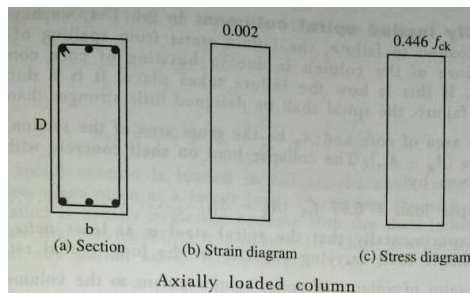
The code adopts the critical value of 0.75 f_y for all grades of steel for finding out the pure axial load carrying capacity of the column. Accordingly

$$P_{uz} = 0.446 f_{ck} A_c + 0.75 f_y A_{sc}$$

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

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \ldots \ldots$$

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 –

$$P_u = 0.4 f_{ck} \left(A_g - \frac{P \! \! \!E g}{100} + 0.67 f \, \frac{P \! \! \!E g}{100} \right.$$

Where A_g = Gross area of cross section

P   = Percentage of reinforcement.

Dividing both sides by A_g

$$\frac{P_u}{A_g} = 0.4 f_{ck} \left(1 - \frac{P A_g}{100}\right) + 0.67 f_y \frac{P}{100}$$

$$= 0.4 f_{ck} \frac{P}{100} \left(0.67 f_y - 0.4 f_{ck}\right)....$$

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, $P = A_c \left(0.45 f_{ck}\right) + A_s \left(0.75 f_y - 0.45 f_{ck}\right)$
2. For Fe 250 steel , $P = A_c \left(0.45 f_{ck}\right) + A_s \left(0.87 f_y - 0.45 f_{ck}\right)$

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

$P_u = 0.9 \left(0.45 f_{ck} A_c + 0.75 f_y A_s\right)$

$P_w = 0.4 f_{ck} A_c + 0.67 f_y A_s$ 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,

$$e_{min} = \frac{L0}{500} + \frac{D}{30} = 3000/500 + 600/30 = 26mm.$$

As 26mm is greater than the minimum specified 20mm, use e_{min} = 26mm. then $\frac{e_{min}}{D}$ = 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.

<u>Minimum Longitudinal and Transverse reinforcement –</u>

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
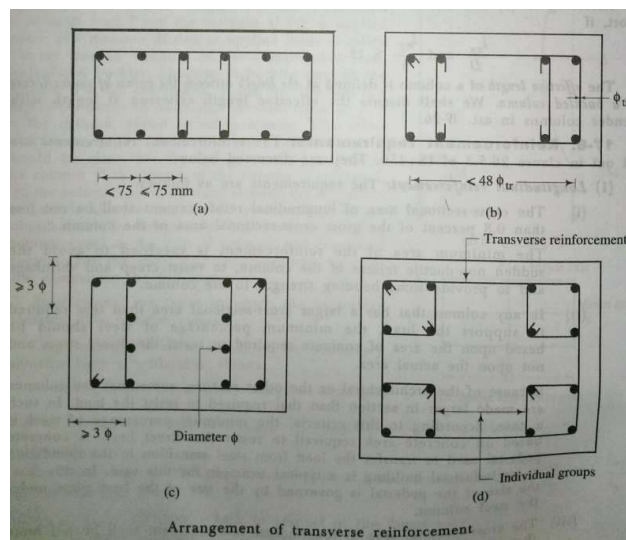
Design of short column by IS 456 and SP 16 –

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

$$P = percentage \ of \ steel = 100 \frac{A_s}{A_g}$$

The area of steel and concrete are given by

$$A_s = \frac{P \, A_g}{100}$$

$$A_c = A_g - A_s = A_g \left(1 - \frac{P}{100}\right)$$

Rewritting the equation with above quantities , we obtain

$$\frac{P_u}{A_g} = 0.4 f_{ck} \left(1 - \frac{P}{100}\right) + 0.67 f_y \frac{P}{100}$$

$$P_u = \left[0.4 f_{ck} \frac{P}{100} \left(0.67 f_y - 0.4 f_{ck}\right)\right] A_g$$

Chart 13.1   Design of Axially Loaded Columns f_y = 415 (Chart 25 of SP 16).
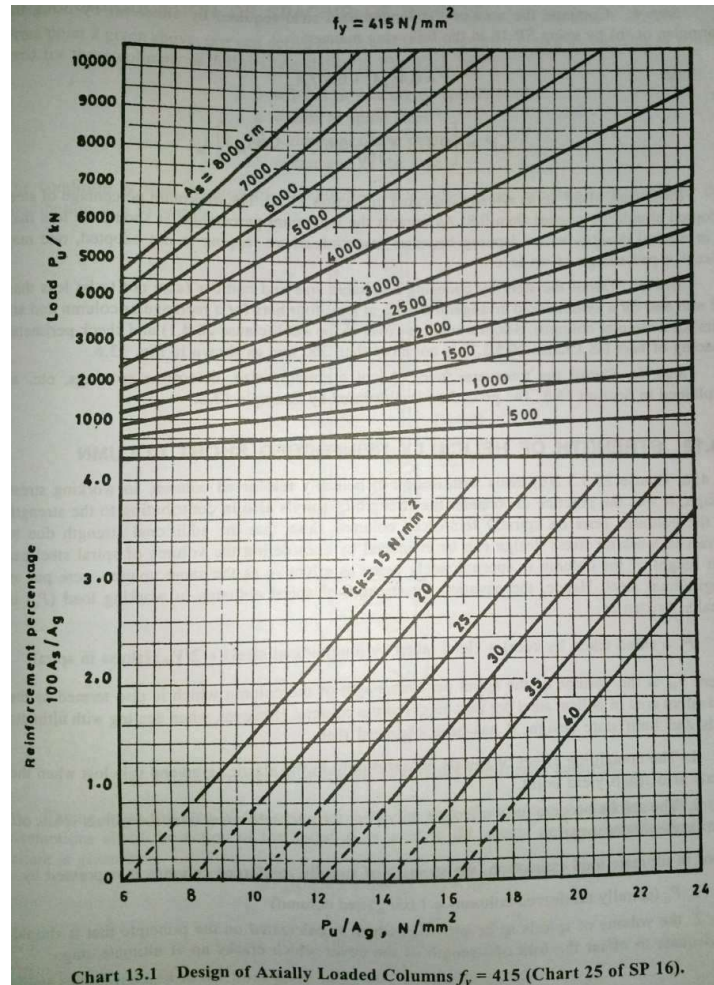
Chart 24 to 26 of SP 16 have been prepared from these formula for Fe 250 , Fe 415 and Fe 500 and f_{ck} = 15, 20, 30, 35, and 40.

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 (p_c) 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.
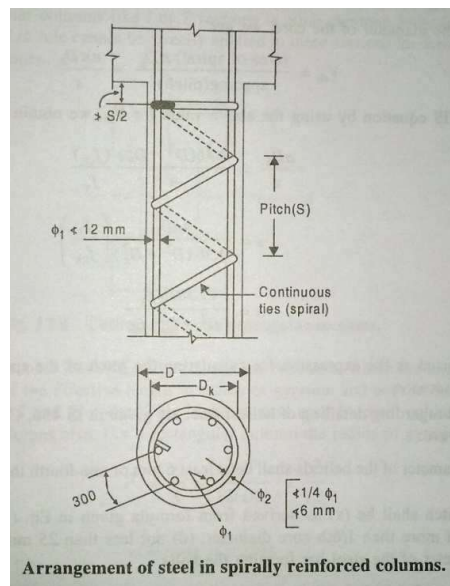
Taking A_k as the area of the core and A_g as the area of cross section and using the same assumption about the action of the spiral as is used in the elastic design, the relationship at failure is given by

$$2V_{sh}(0.87f_y) = 0.63 f_{ck} (A_g - A_k)$$

$$V_{sh} = 0.36 (A_g - A_k)\left(\frac{f_{ck}}{f_y}\right)$$

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

$$\frac{aD_k}{s} = \frac{0.36\ (D^2 - D_k^2)}{4}\ \frac{f_{ck}}{f_y}$$

$$S = \frac{4aD_k)}{0.36\ (D^2 - D_k^2)}\ \frac{f_y}{f_{ck}}$$

$$S = \frac{11.1aD_k f_y)}{f_{ck}(\ D^2 - D_k^2)}\ \ \ \ ....(1)$$

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

$$\frac{L_e}{D} = 4500 / 400 = 11.25 \ \ \ \ \ \ \text{(IS456 Cl.25.1.2)}$$

As columns with slenderness less than 12 can be considered as short , the column is short.

Step 2: Eccentricity considerations

$$e_{min} = \frac{L0}{500} + \frac{D}{30} \ \text{not less than 20mm.} \ \ \ \text{(IS 456 Cl . 25.4)}$$

$$= 7000 / 500 + 400/30$$

$$= 14 + 13.3$$

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

$P_u = 2300$ KN

$Step\ 2$: 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)

$L_e = 1.0L = 3000$mm

$\frac{L_e}{b}$ = 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 10³ = 0.4 x 20 x (400² - A_s) + 0.67 x 415 x A_s

A_s = 3777 mm², P = 3777/ 400² x 100 = 2.36%

$This\ is\ more\ than\ 0.8\%\ and\ less\ than\ 6\%.\ hence\ o.k$

(b) By SP 16 , A_g = 1600 cm², P = 2300 KN. P = 2.4% (SP 16 chart 25)

A_s = (2.4x400x400) / 100 = 3840mm² (Use 8T -25 / 3927mm²)

$Step\ 5$: 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 = $\frac{11.1aD_k f_y)}{f_{ck}(\ D^2 - D_k^2)}$ =(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

$D_{core} = 320$mm

D/20 = 400/20 = 20mm.

$e_{min} = \frac{L_0}{500} + \frac{400}{30} = \frac{4500}{500} + \frac{400}{30} = 22.3$mm > 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 $400^2 / 4 = 125.6$ x 10^3 mm², $\frac{P}{1.05} = \frac{900}{1.05} = 857$ KN.

857 x $10^3 = [.4$ x 30 x (125600 - A_s) + 0.67 x 415 $A_s]$

$= 1507$ x $10^3 + A_s$ (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 = $\frac{900 x \ 10^3}{1.05 \ x \ 0.4 \ x \ 30}$ = 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

PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTINGS
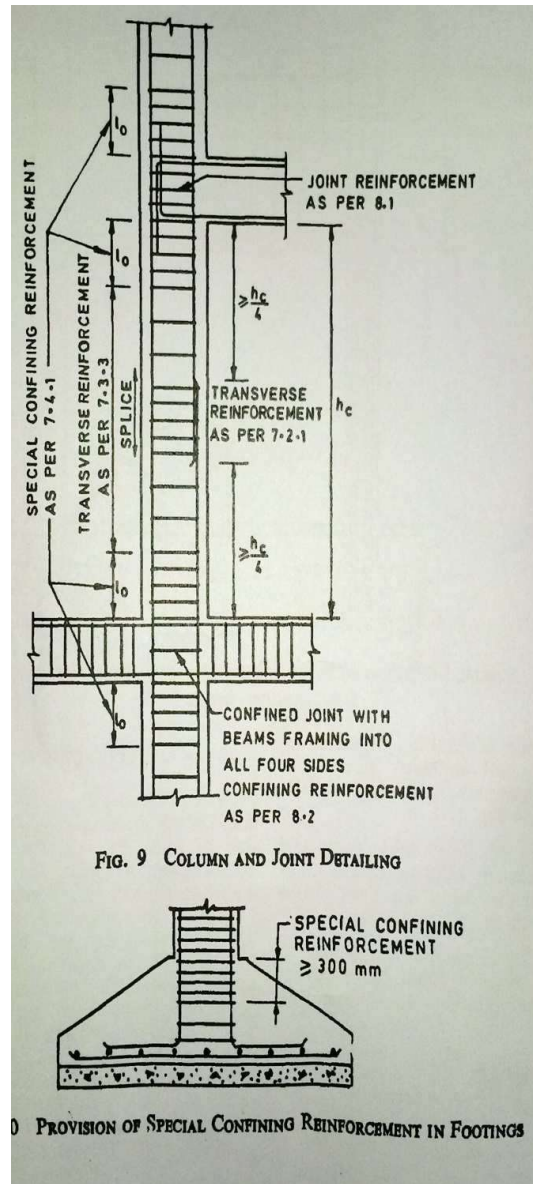
CHAPTER – 10

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 Pe 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 $0.1 f_{ck}$.

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 $p_{min} = 0.24 \sqrt{f_{ck}}/f_y$; 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 $p_{max} = 0.025$.

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.
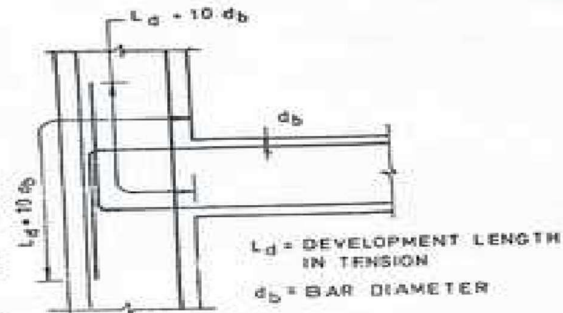
FIG. 1   ANCHORAGE OF BEAM BARS IN AN EXTERNAL JOINT

L_d = DEVELOPMENT LENGTH IN TENSION

d_b = BAR DIAMETER

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 2d 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.

L_d = DEVELOPMENT LENGTH IN TENSION

d_b = BAR DIAMETER
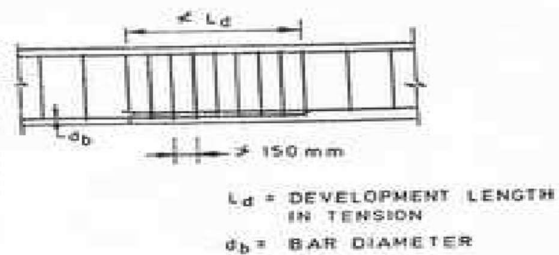
FIG. 2   LAP, SPLICE IN BEAM

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.

IS 13920 : 1993

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:

$$V_{u,a} = V_a^{D+L} - 1\cdot4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$

and $V_{u,b} = V_b^{D+L} + 1\cdot4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$, and

ii) for sway to left:

$$V_{u,a} = V_a^{D+L} + 1\cdot4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

and $V_{u,b} = V_b^{D+L} - 1\cdot4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$,

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.
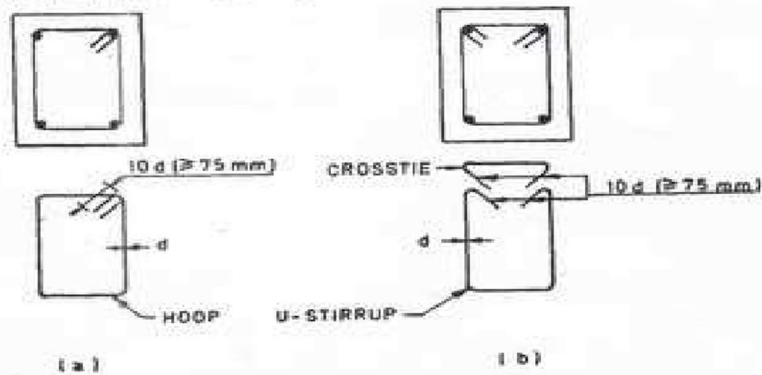
FIG. 3 BEAM WEB REINFORCEMENT

$$V_a^{D+L} = V_b^{D+L} = \frac{1.2(D+L)}{2}$$

$$V_{u,a} = V_a^{D+L} - 1.4 \left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}} \right]$$

$$V_{u,b} = V_b^{D+L} + 1.4 \left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}} \right]$$

(SWAY TO RIGHT)

$$V_{u,a} = V_a^{D+L} + 1.4 \left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

$$V_{u,b} = V_b^{D+L} - 1.4 \left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

(SWAY TO LEFT)
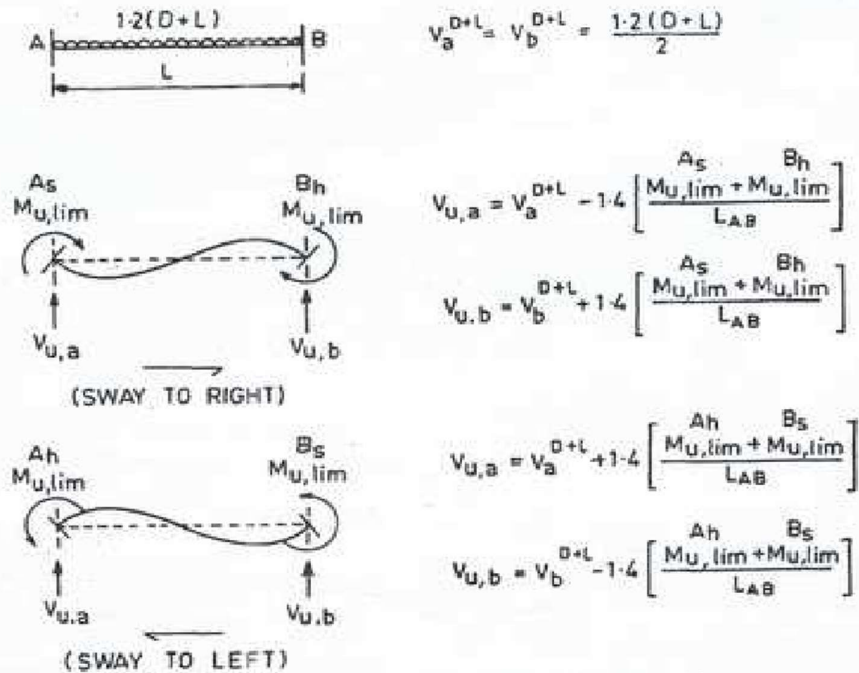
FIG. 4   CALCULATION OF DESIGN SHEAR FORCE FOR BEAM

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 th central half of the member length. It should l proportioned as a tension splice. Hoops shall l provided over the entire splice length at spaci not exceeding 150 mm centre to centre. N more than 50 percent of the bars shall l

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.

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 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 $0.1 f_{ck}$ 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.

MIN 2 BARS FOR FULL LENGTH
ALONG TOP AND BOTTOM FACE
AS ≥ P MIN. Bd
AS ≤ P MAX. Bd

50mm MAX   50 mm MAX

2d   HOOP SPACING   2d
$\not> \dfrac{d}{2}$

HOOP SPACING
≤ d/L AND 8 db

B = BREADTH OF BEAM
db = DIAMETER OF LONGITUDINAL
BAR
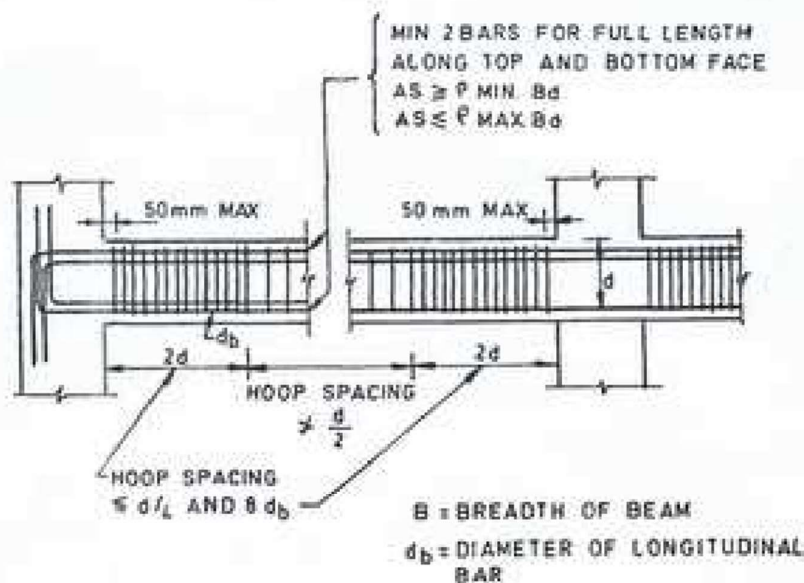
FIG. 5   BEAM REINFORCEMENT

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).

MINIMUM LONGITUDINAL
AND TRANSVERSE STEEL
AS PER IS 456 : 1978
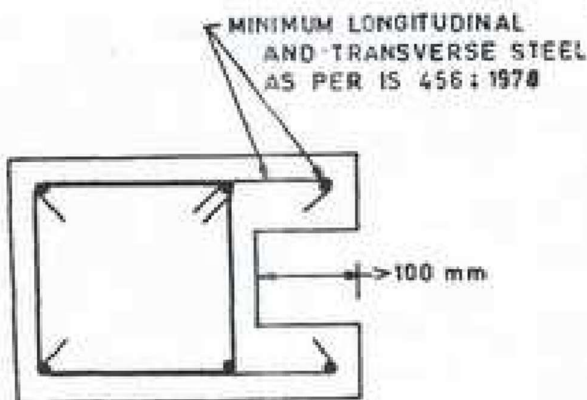
>100 mm

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

$$V_a = 1.4 \left[\frac{M_{u,\,lim}^{bL} + M_{u,\,lim}^{bR}}{h_{st}} \right]$$

where $M_{u,\,lim}^{bL}$ and $M_{u,\,lim}^{bR}$ 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
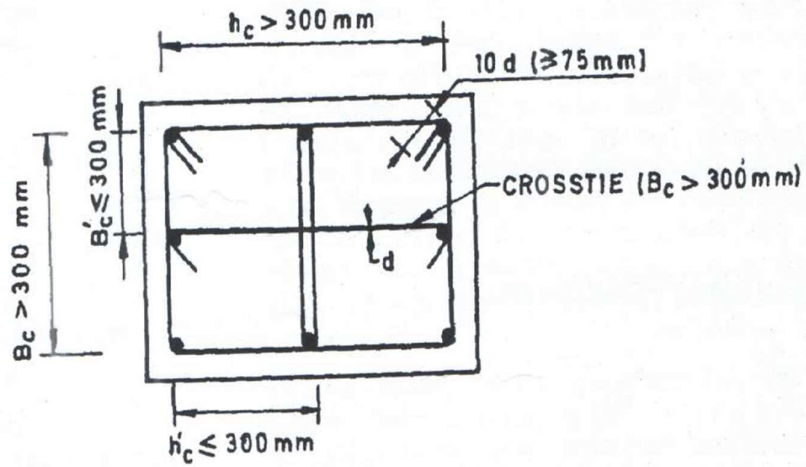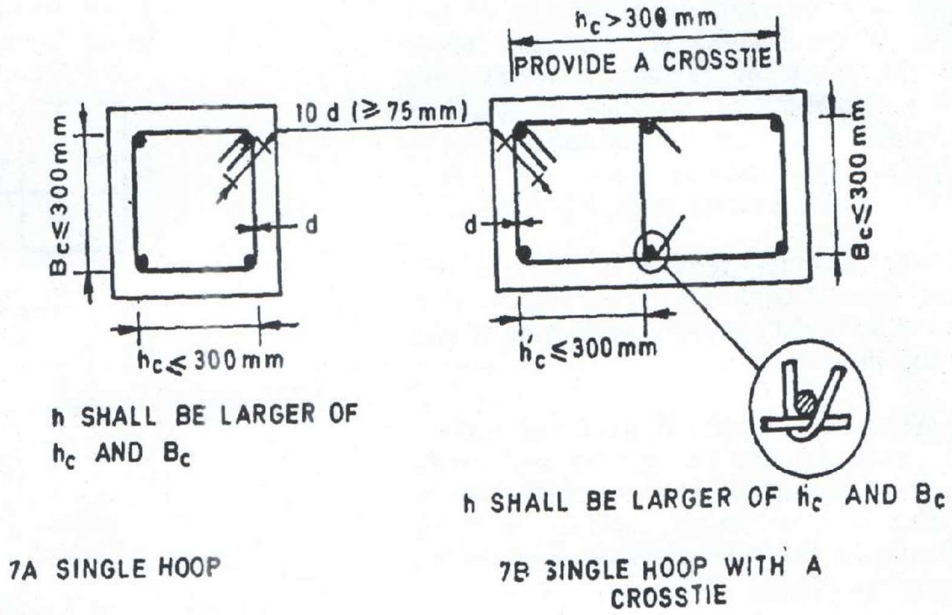
FIG. 7 TRANSVERSE REINFORCEMENT IN COLUMN

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

$$V_u = 1.4 \left(\frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{st}} \right)$$
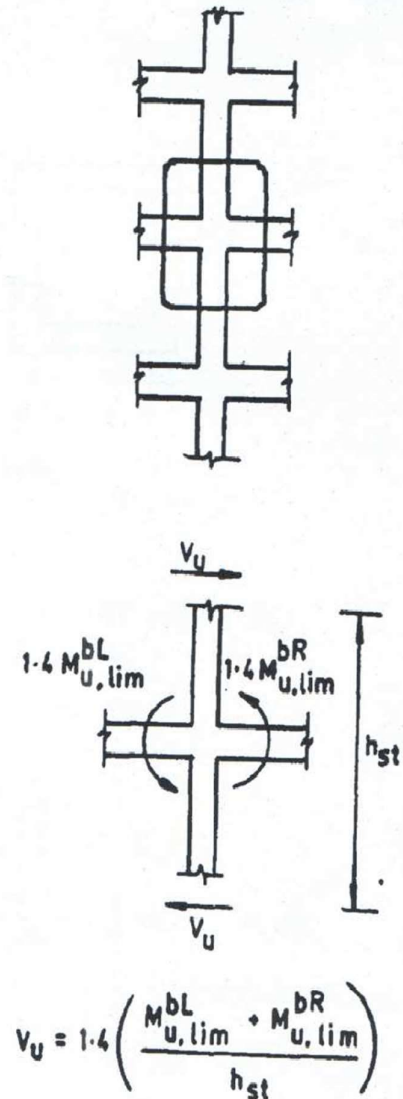
FIG. 8 CALCULATION OF DESIGN SHEAR FORCE FOR COLUMN

IS 13920 : 1993
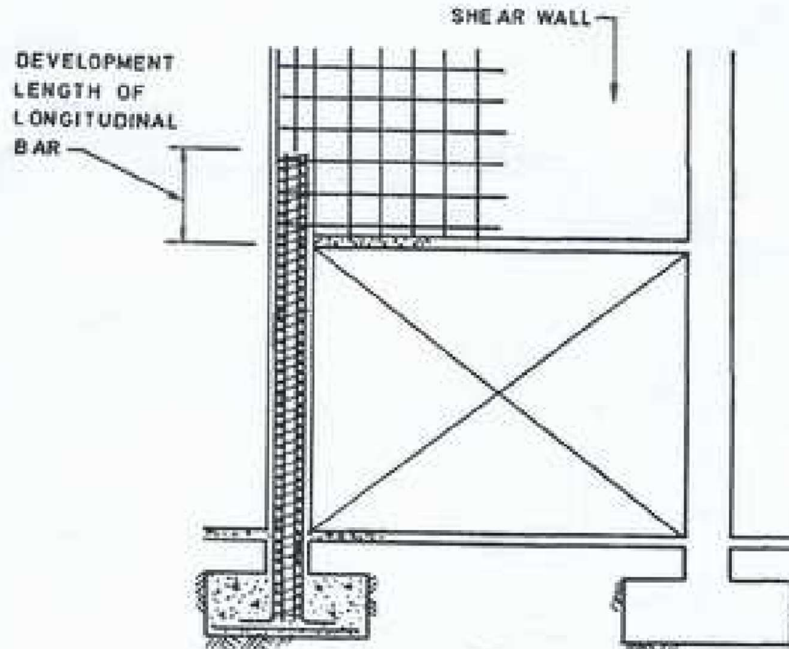
FIG. 11 SPECIAL CONFINING REINFORCEMENT REQUIREMENT FOR COLUMNS UNDER DISCONTINUED WALLS

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

$$A_{sh} = 0.09 \, SD_k \frac{f_{ck}}{f_y} \left[\frac{A_g}{A_k} - 1.0 \right]$$

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 × 40 + 2 × 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

$$A_{sh} = 0.18 \, Sh \frac{f_{ck}}{f_y} \left[\frac{A_g}{A_k} - 1.0 \right]$$

where

h = longer dimension of the rectangular confining hoop measured to its outer
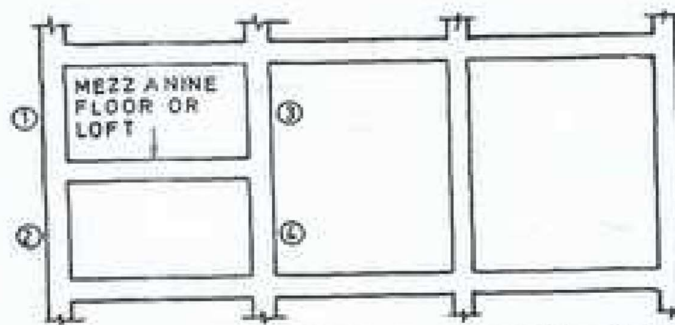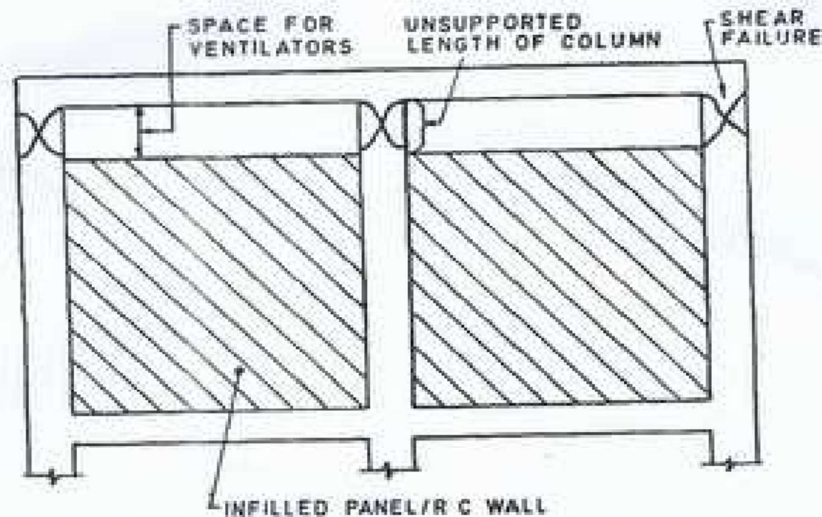
(1), 2), (3) and (4) relatively stiff columns — They attract parge seismic shear force.

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
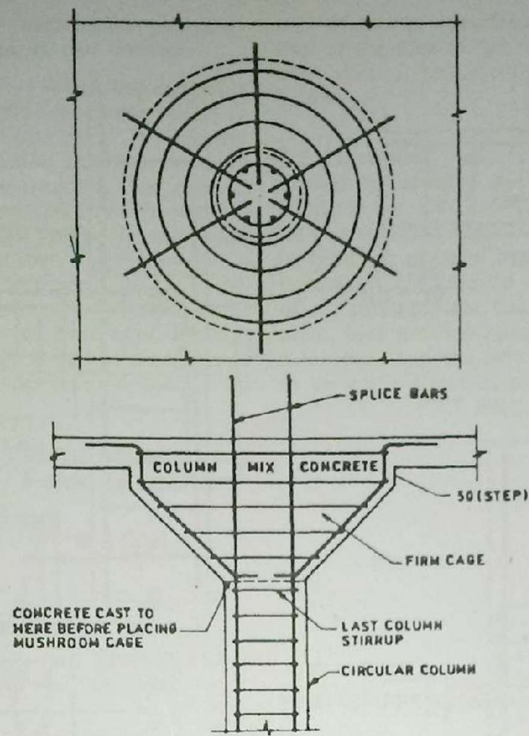
FIG. 7.12 REINFORCEMENT OF MUSHROOM HEADS

Labels on Fig. 7.12:
- SPLICE BARS
- COLUMN MIX CONCRETE
- 50 (STEP)
- FIRM CAGE
- CONCRETE CAST TO HERE BEFORE PLACING MUSHROOM CAGE
- LAST COLUMN STIRRUP
- CIRCULAR COLUMN

Labels on Fig. 7.13:
- COLUMN BARS STRAIGHT THROUGH JUNCTION
- STIRRUP HANGER BARS STOP SHORT OF COLUMN FACE
- TOP BARS, PRIMARY BEAM BARS PLACED ABOVE SECONDARY BEAM
- PRIMARY BEAM
- SECONDARY BEAM
- BOTTOM BARS STOP SHORT OF COLUMN FACE
- BOTTOM SUPPORT BARS
- BOTTOM BARS STOP SHORT OF COLUMN FACE

FIG. 7.13 BEAM-COLUMN INTERSECTION

94

HANDBOOK ON CONCRETE REINFORCEMENT AND DETAILING

SP : 34(S&T)-1987

U-TYPE BARS

SHEAR STIRRUPS

COLUMN TIES

7.14A FIXED END JOINT IN A COLUMN

L_d (IN TENSION)

M

L_d

U-TYPE BARS

BEAM

END COLUMN

FIG. 7.15 Typical Details of a Beam-Column Junction at Exterior Column

ROOF LEVEL

LAP

7.14B TERMINATION OF COLUMN BARS INSIDE A SLAB

Fig. 7.14 Typical Details of End Joints in a Column

$hy \geqslant h/2$

FIG. 7.16 Truss Anology for Corbel Design

MAIN STEEL

$\leqslant$ 1 BAR DIA.

L_d

MAIN STEEL BAR WELD

TRANSVERSE BAR OF EQUAL STRENGTH WELDED TO MAIN STEEL

NOMINAL STEEL FOR ANCHORING STIRRUPS

SHEAR REINFORCEMENT

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

GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING
GOLANTHARA, BERHAMPUR

LECTURE NOTES
ON

E.M & S.T

For 5th Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)

Prepared By

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1. ENTRPRENUERSHIP

Concept of Entrepreneurship

Entrepreneurship is the ability and readiness to develop, organize and run a business enterprise, along with any of its uncertainties in order to make a profit. The most prominent example of entrepreneurship is the starting of new businesses.

WHAT IS ENTREPRENEURSHIP?

In economics, entrepreneurship connected with land, labour, natural resources and capital can generate a profit. The entrepreneurial vision is defined by discovery and risk-taking and is an indispensable part of a nation's capacity to succeed in an ever-changing and more competitive global marketplace.

Meaning of Entrepreneur

The entrepreneur is defined as someone who has the ability and desire to establish, administer and succeed in a startup venture along with risk entitled to it, to make profits. The best example of entrepreneurship is the starting of a new business venture. The entrepreneurs are often known as a source of new ideas or innovators, and bring new ideas in the market by replacing old with a new invention.

It can be classified into small or home business to multinational companies. In economics, the profits that an entrepreneur makes is with a combination of land, natural resources, labour and capital.

In a nutshell, anyone who has the will and determination to start a new company and deals with all the risks that go with it can become an Entrepreneur.

What are the 4 Types of Entrepreneurship?

It is classified into the following types:

Small Business Entrepreneurship-

These businesses are a hairdresser, grocery store, travel agent, consultant, carpenter, plumber, electrician, etc. These people run or own their own business and hire family members or local employee. For them, the profit would be able to feed their family and not making 100 million business or taking over an industry. They fund their business by taking small business loans or loans from friends and family.

Scalable Startup Entrepreneurship-

This start-up entrepreneur starts a business knowing that their vision can change the world. They attract investors who think and encourage people who think out of the box. The research focuses on a scalable business and experimental models so, they hire the best and the brightest employees. They require more venture capital to fuel and back their project or business.

Large Company Entrepreneurship-

These huge companies have defined lifecycle. Most of these companies grow and sustain by offering new and innovative products that revolve around their main products. The change in technology, customer preferences, new competition, etc., build pressure for large companies to create an innovative product and sell it to the new set of customers in the new market. To cope with the rapid technological changes, the existing organisations either buy innovation enterprises or attempt to construct the product internally.

Social Entrepreneurship-

This type of entrepreneurship focuses on producing product and services that resolve social needs and problems. Their only motto and goal is to work for society and not make any profits.

Characteristics of Entrepreneurship:

Not all entrepreneurs are successful, there are definite characteristics that make entrepreneurship successful. A few of them are mentioned below:

- **Ability to take a risk-** Starting any new venture involves a considerable amount of failure risk. Therefore, an entrepreneur needs to be courageous and able to evaluate and take risks, which is an essential part of being an entrepreneur.
- **Innovation-** It should be highly innovative to generate new ideas, start a company and earn profits out of it. Change can be the launching of a new product that is new to the market or a process that does the same thing but in a more efficient and economical way.
- **Visionary and Leadership quality-** To be successful, the entrepreneur should have a clear vision of his new venture. However, to turn the idea into reality a lot of resources and employees are required. Here, leadership quality is paramount because leaders impart and guide their employees towards the right path of success.
- **Open-Minded-** In a business, every circumstance can be an opportunity and used for the benefit of a company. For example, Paytm recognised the gravity of demonetization and acknowledged the need for online transactions would be more, so it utilised the situation and expanded massively during this time.

- **Flexible-** An entrepreneur should be flexible and open to change according to the situation. To be on the top, a businessperson should be equipped to embrace change in a product and service, as and when needed.
- **Know your Product-**A company owner should know the product offerings and also be aware of the latest trend in the market. It is essential to know if the available product or service meets the demands of the current market, or whether it is time to tweak it a little. Being able to be accountable and then alter as needed is a vital part ofentrepreneurship.

Importance of Entrepreneurship:

- **Creation of Employment-** Entrepreneurship generates employment. It provides an entry-level job, required for gaining experience and training for unskilled workers.
- **Innovation-** It is the hub of innovation that provides new product ventures, market, technology and quality of goods, etc., and increase the standard of living of people.
- **Impact on Society and Community Development-** A society becomes greater if the employment base is large and diversified. It brings about changes in society and promotes facilities like higher expenditure on education, better sanitation, fewer slums, a higher level of homeownership. Therefore, entrepreneurship assists the organisation towards a more stable and high quality of community life.
- **Increase Standard of Living-** Entrepreneurship helps to improve the standard of living of a person by increasing the income. The standard of living means, increase in the consumption of various goods and services by a household for a particular period.
- **Supports research and development-** New products and services need to be researched and tested before launching in the market. Therefore, an entrepreneur also dispenses finance for research and development with research institutions and universities. This promotes research, general construction, and development in the economy.

Functions of an Entrepreneur

Functions of an Entrepreneur – Risk Bearing, Administrative, Decision-Making and Distributive Functions

An entrepreneur frequently has to wear many hats. He has to perceive opportunity, plan, organize resources, and oversee production, marketing, and liaison with officials. Most importantly he has to innovate and bear risk.

The main functions of an entrepreneur are grouped in three categories:

1. Risk bearing functions,

2. Administrative and decision-making functions, and

3. Distributive functions (responsibility of the organizer).

1. <u>Risk Bearing Functions:</u>

It is the most important and specific function of an entrepreneur. Every business involves some amount of risk. The production of goods and services is always related to future demands. The future demand is uncertain and unpredictable, because it is influenced by the changes in fashion or taste and liking of the consumers.

The price structure, value of money, climatic conditions and government policies are some other important factors that affect the demand of a commodity. All these factors are variable and as such an exact estimation of future demand is a difficult exercise to work o:

Since this unpredictable task is undertaken by the entrepreneur, he has to bear the risk. If his estimations prove to be wrong, then in the entire business sphere, no other factor of production shares the loss incurred by the entrepreneur.

It is the main reason why the entrepreneur becomes entitled for the surplus that is remaining with him from the sale proceeds of the product, after distributing the shares to other factors. This surplus is termed as profit of the business.

2. <u>Administrative and Decision-Making Functions:</u>

i. Conceiving the Idea of Business:

The entrepreneur conceives the idea of a particular business which suits his nature, skill and resources. He makes a thorough (intensive and extensive) study of the condition of market and business prospects. After making a thorough study of economic viability, he decides the business that he has to star

ii. <u>Estimation of Details of Business and Implementation of the Same:</u>

After arriving at a conclusion about the nature of business, the entrepreneur works out the details of business, i.e., what, how and when to produce and from where the resources are to be arranged. With all these estimations, he makes an all-out effort to give a practical shape to his plans, organizes various factors of production and sets them to function in proper harmony.

iii. <u>Supervision and Control of Business Activities:</u>

The entrepreneur has to supervise and control the day-to-day business activities to accomplish the business objectives. For this he properly coordinates between various factors of production. As the risk (success or failure) of business operations directly affect his economy, he keeps a vision and control on the business affairs and avoids unnecessary expenditures.

He is required to take a numerous decisions and has to get these decisions properly implemented.

iv. <u>Innovation:</u>

Innovation is one of the most important functions of an entrepreneur. An entrepreneur uses information, knowledge and intuition to come up with new products, new methods of reducing costs of a product, improvement in design or function of a product, discovering new markets or new ways of organization of industry. Through innovation, an entrepreneur converts a material into a resource or combines existing resources into new and more productive configurations.

It is the creativity of an entrepreneur that results in invention (creation of new knowledge) and innovation (application of knowledge to create the new products, services or processes).

3. Distributive Functions:

The entrepreneur organizes different factors of production and sets them to work. It, therefore, becomes his responsibility to make proper allocation of funds for each factor of production, i.e., each factor of production must be properly remunerated.

The remuneration here refers to an important decision as to what should be the share of each factor of production in the sale proceeds of the entire product. The remuneration should be just and equitable and the payment to each factor should be commensurate, so that each factor is fully satisfied.

If the factors of production remain dissatisfied, they will not be able to deliver their best to the entrepreneur. So, it is the entrepreneur, who has to ultimately suffer. Hence, it is very essential for the entrepreneur to perform distributive functions with extreme care and caution.

barriers to entrepreneurship

hareEconomic innovation and job formation is stalled throughout the world. Entrepreneurs can change all that, say four professors. To do so, entrepreneurs must overcome eleven barriers facing them.

First: Corrupt and unsupportive business environment

Lack of supportive and market-augmenting governmental regulations serve as a barrier to entrepreneurship. Russia leads all other large nations in having an unsupportive business environment because they lack rule of law, have poorly defined contract and property laws, enforce regulations inconsistently, allow rampant corruption and bribing, allow regulatory authorities and inspectors to act in a predatory nature which therefore requires friendly ties with government officials and bureaucrats to smooth the way for businesses to operate. China has similar issues. See the accompanying list where pictured is Russian dissident- businessman, Boris Berezhovsky, now exiled in London, England. Berezhovsky was one of the Russian oligarchs who acquired massive wealth by taking control of state assets after the fall of communism. He also has links to the poisoned Russian spy Alexander Litvinenko.

Tied for second: Employee related difficulties

Building an employee asset base for the enterprise is one of the more daunting and sometimes overlooked tasks. Entrepreneurs must find and select the best-qualified employees who are motivated and willing to grow with the venture. Then they must ensure the employees do not leave. The professors say this task becomes a barrier when employees' expectations increases, governmental regulations related to labor employment is hardened, and employee costs grow. Employee cost is more than pay. It includes healthcare, workers' compensation, social security tax, and health and safety regulations.

Just ask Charles "Chip" Starnes, who was recently held hostage by his own employees in a plant in China. It has been widely reported that Starnes was held hostage by current employees who did not receive severance pay that 30 workers received because they were

being laid off. Starnes was moving a plastic-injection-molding division to Mumbai, India, where production costs are lower. After being barricaded in the plant for almost a week, Starnes reached an agreement after nearly a week to pay two months' wages, totaling almost $300,000, to the remaining 97 employees.

Tied for second: Severe market entry regulations

Governmental rules, taxation, environmental regulations, lending requirements and licensing are all barriers to entrepreneurship. Most countries, the United States included, proscribe or license market entry and the creation of new firms to protect incumbents in certain industries and professions. Entry procedures, or "red tape," vary such that entrepreneurs need one day to register an enterprise in one country and up to 20 weeks in another. Other barriers to entrepreneurship are predatory tax behavior of authorities, lack of property rights and tax disadvantages.

Tied for second: Shortage of funds and resources

Finding the money to start up an enterprise is a leading barrier to entrepreneurship. Without funds, any person cannot begin to organize, train, develop and sell product.

Fifth: Lack of Entrepreneurship Opportunities

Venture creation requires existing marketplace opportunities with possibilities known to the entrepreneur and favorable odds for success for entrepreneurial "spirit" to succeed.

Sixth: Lack of Entrepreneurial Capacity

Opportunities go untried until someone comes along with an eye for possibility and a can-do attitude. Some cultures may discourage entrepreneurial capacity resulting in a low rate of new firm entrance. It is like having an oil well field without knowledgeable people to mine the wells. Entrepreneurial capacity is the existence of people with entrepreneurship qualities, willingness and motivation to initiate new ventures.

Seventh: Lack of Adequate Entrepreneurship Training

Training and education can be a robust incubator for new ventures. This includes training in technical skills, managerial skills, entrepreneurial skills and entrepreneurship.

Tied for eighth: Lack of Appropriate Technical and Practical Skills

People tend to use the skills they have acquired to pursue entrepreneurial initiative. Lacking the appropriate skills and knowledge inhibits economic development.

Tied for eighth: Lack of Market Experience

The essence of leadership is first learning and doing before leading. Therefore, the capability to start a business is propelled by previous education and work experience. Rushing into a new market because it looks attractive and rewarding without having some experience and background in it can be fatal. Experience in a related business before start-up is positively correlated to the probability of success.

Tenth: Fear of Failure

Entrepreneurs have to decide whether to take action so they don't miss the boat, while knowing that hasty action may cause them to sink the boat.

Eleventh: Aversion to Risk

A psychological barrier closely related to the fear of failure is aversion to risk. Entrepreneurs must take initiative, create structure with a social-economic mechanism and accept risk of failure. Entrepreneurs have to be risk takers while those who are risk averse will seek the security if an existing establishment.

Entrepreneurs vs

ManagersWho is an

Entrepreneur?

Very basically speaking, an entrepreneur is a one-man show that runs entrepreneurship. However, such a person usually has some unique attributes that allow him to be successful in his endeavors. He is essentially an initiator and a leader. He brings business ideas to fruition thus starting off his venture.
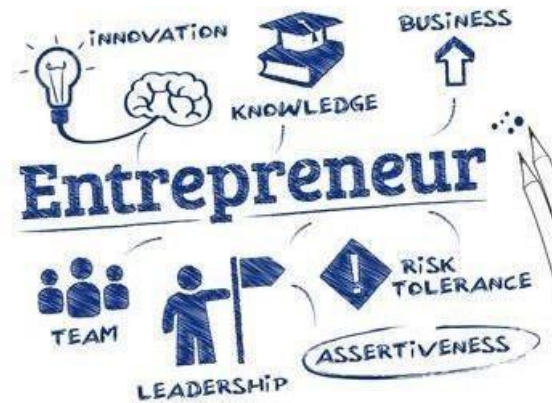
A successful entrepreneur is usually a responsible person. He is accountable for the success or the failure of his venture, and he takes this responsibility very seriously. And since he is the only person in-charge he is automatically the leader. In fact, leadership qualities are one of the main aspects of an entrepreneur.

Who is a Manager?

A manager, on the other hand, is not an owner of an enterprise. Instead, he is the one that is responsible for the management and administration of a group of people or a department of the organization. His day to day job is to manage his employees and ensure the organization runs smoothly.

A manager must possess some of the same qualities as an entrepreneur, like leadership, accountability, decisiveness etc. He must also be a good manager of people. So qualities such as warmth and empathy are also very important in a manager.

Now that we have a brief idea about their qualities and roles, let us take a look at the difference between them

Entrepreneur vs Manager

Entrepreneur	Manager
Entrepreneur is visonary and bears all financial risks.	Manager works for salary, and does not have to bear any risks.
Focuses on starrting and expanding the business ideas	Focus on daily smooth functioning of business
Key motivation for Entrepreneur is achievments	Mangers motivation comes from the power that comes with their position
Reward for all the efforts is profit he earns from the enterprise	Remuneration is the salary he draws from the company
Entrepreneur can be informal and casual	Manager's approach to every problem is very formal

Difference between Entrepreneur and Manager

- The key **difference between an entrepreneur and a manager** is their standing in the company. An entrepreneur is a visionary that converts an idea into a business. He is the owner of the business, so he bears all the financial and other risks. A manager, on the other hand, is an employee, he works for a salary. So he does not have to bear any risks.

- The focus of an entrepreneur lies in starting the business and later expanding the business. A manager will focus on the daily smooth functioning of the business.

- For an entrepreneur the key motivation is achievements. But for the managers, the motivation comes from the power that comes with their position.

- The reward for all the efforts of an entrepreneur is the profit he earns from the enterprise. The manager is an employee, so his remuneration is the salary he draws from the company.

- The entrepreneur can be informal and casual in his role. However, a manager's approach to every problem is very formal.

- The entrepreneur by nature is a risk taker. His has to take calculated risks to drive the company further. A manager, on the other hand, is risk-averse. His job is to maintain the status quo of the company. So he cannot afford risks.

Forms OF Business Ownership

Business organization is the single-most important choice you'll make regarding your company. What form your business adopts will affect a multitude of factors, many of which will decide your company's future. Aligning your goals to your business organization type is an important step, so understanding the pros and cons of each type is crucial.

Your company's form will affect:

- How you are taxed

- Your legal liability
- Costs of formation
- Operational costs

There are 4 main types of business organization: sole proprietorship, partnership, corporation, and Limited Liability Company, or LLC. Below, we give an explanation of each of these and how they are used in the scope of business law.

Sole Proprietorship

The simplest and most common form of business ownership, sole proprietorship is a business owned and run by someone for their own benefit. The business' existence is entirely dependent on the owner's decisions, so when the owner dies, so does the business.

Advantages of sole proprietorship:

- All profits are subject to the owner
- There is very little regulation for proprietorships
- Owners have total flexibility when running the business
- Very few requirements for starting—often only a business license

Disadvantages:

- Owner is 100% liable for business debts
- Equity is limited to the owner's personal resources
- Ownership of proprietorship is difficult to transfer
- No distinction between personal and business income

Partnership

These come in two types: general and limited. In general partnerships, both owners invest their money, property, labor, etc. to the business and are both 100% liable for business debts. In other words, even if you invest a little into a general partnership, you are still potentially responsible for all its debt. General partnerships do not require a formal agreement—partnerships can be verbal or even implied between the two business owners.

Limited partnerships require a formal agreement between the partners. They must also file a certificate of partnership with the state. Limited partnerships allow partners to limit their own liability for business debts according to their portion of ownership or investment.

Advantages of partnerships:

- Shared resources provides more capital for the business
- Each partner shares the total profits of the company
- Similar flexibility and simple design of a proprietorship
- Inexpensive to establish a business partnership, formal or informal

Disadvantages:

- Each partner is 100% responsible for debts and losses

- Selling the business is difficult—requires finding new partner
- Partnership ends when any partner decides to end it

Corporation

Corporations are, for tax purposes, separate entities and are considered a legal person. This means, among other things, that the profits generated by a corporation are taxed as the "personal income" of the company. Then, any income distributed to the shareholders as dividends or profits are taxed again as the personal income of the owners.

Advantages of a corporation:

- Limits liability of the owner to debts or losses
- Profits and losses belong to the corporation
- Can be transferred to new owners fairly easily
- Personal assets cannot be seized to pay for business debts

Disadvantages:

- Corporate operations are costly
- Establishing a corporation is costly
- Start a corporate business requires complex paperwork
- With some exceptions, corporate income is taxed twice

Limited Liability Company (LLC)

Similar to a limited partnership, an LLC provides owners with limited liability while providing some of the income advantages of a partnership. Essentially, the advantages of partnerships and corporations are combined in an LLC, mitigating some of the disadvantages of each.

Advantages of an LLC:

- Limits liability to the company owners for debts or losses
- The profits of the LLC are shared by the owners without double-taxation

Disadvantages:

- Ownership is limited by certain state laws
- Agreements must be comprehensive and complex
- Beginning an LLC has high costs due to legal and filing fees

Types of industry concept

There are four types of industry. These are primary, secondary, tertiary and quaternary.

Primary industry involves getting raw materials e.g. mining, farming and fishing.

Secondary industry involves manufacturing e.g. making cars and steel.

Tertiary industries provide a service e.g. teaching and nursing.

Quaternary industry involves research and development industries e.g. IT

What Is a Startup?

The term startup refers to a company in the first stage of its operations. Startups are founded by one or more entrepreneurs who want to develop a product or service for which they believe there is a demand. These companies generally start with high costs and limited revenue which is why they look for capital from a variety of sources such as venture capitalists.

KEY TAKEAWAYS

- A startup is a company that's in the initial business stage.
- Until the business gets off the ground, a startup is often financed by its founders. and the startup attracts outside investment.
- There are many different ways to fund startups including family and friends, venture capitalists, crowdfunding, and credit.
- Startups also have to consider where they'll do business and their legal structure.

Understanding Startups

Startups are companies or ventures that are focused around a single product or service that the founders want to bring to market. These companies typically don't have a fully developed business model and, more importantly, lack adequate capital to move on to the next phase of business. Most of these companies are initially funded by their founders.

Many startups end up turning to others for more funding—family, friends, and venture capitalists. Silicon Valley in California is known for its strong venture capitalist community and is a popular destination for startups, but is also widely considered the most demanding arena. Startups can use this seed capital to invest in research and to develop their business plans. Market research helps determine the demand for a product or service, while a comprehensive business plan outlines the company's mission statement, future visions, and goals as well as management and marketing strategies.

Dotcoms were the most common type of startup in the 1990s. Venture capital was extremely easy to obtain during this time due to a frenzy among investors to speculate on the emergence of these new types of businesses. Unfortunately, most of these internet startups eventually went bust due to major oversights in their underlying business plans such as a lack of sustainable revenue. However, there were a handful of companies that survived when the dotcom bubble burst. Both Amazon and eBay are examples of such companies.

So many startups fail within the first few years. That's why this initial period is so important. Entrepreneurs need to find money, create a business model and business plan, hire key personnel, work out intricate details such as equity stakes for partners and investors, and plan for the long run. Many of today's most successful companies began as startups— Facebook, Airbnb, Uber, SpaceX, and Ant Financial—and ended up becoming publicly- traded companies.

The first few years are very important for startups—a period during which entrepreneurs should concentrate on raising capital and developing a business model.

Special Considerations
Location

Startups must decide whether their business is conducted online, in an office/home office or in a store. The location depends on the product or service being offered. For example, a technology startup selling virtual reality hardware may need a physical storefront to give customers a face-to-face demonstration of the product's complex features.

Legal Structure

Startups need to consider what legal structure best fits their entity. A sole proprietorship is suited for a founder who is also the key employee of a business. Partnerships are a viable legal structure for businesses that consist of several people who have joint ownership, and they're also fairly straightforward to establish. Personal liability can be reduced by registering a startup as a limited liability company (LLC).

Funding

Startups often raise funds by turning to family and friends or by using venture capitalists. This is a group of professional investors that specialize in funding startups. Crowdfunding has become a viable way for many people to get access to the cash they need to move forward in the business process. The entrepreneur sets up a crowdfunding page online, allowing people who believe in the company to donate money.

Startups may use credit to commence their operations. A perfect credit history may allow the startup to use a line of credit as funding. This option carries the most risk, particularly if the startup is unsuccessful. Other companies choose small business loans to help fuel growth. Banks typically have several specialized options available for small businesses— a microloan is a short-term low-interest product tailored for startups. A detailed business plan is often required in order to qualify.

Entrepreneurship support agencies at national, state, district levelsDIC

District Industries Centers (DICs) provide full assistance to the **entrepreneurs** who are going to start the business on their own and in their regional places. ... These **DIC** programs can take over the responsibilities in order to promote cottage and small scale industries at district level effectively.

NSIC

National Small Industries Corporation (**NSIC**), is an ISO 9001-2015 certified Government of India Enterprise under Ministry of Micro, Small and Medium Enterprises (MSME). **NSIC** has been working to promote, aid and foster the growth of micro, small and medium enterprises in the country.

OSIC

THE ODISHA SMALL INDUSTRIES CORPORATION LTD. (OSIC) was established

on 3rd April, 1972 as a wholly owned Corporation of Government of Odisha. The basic objective of the Corporation is to aid ,assist and promot the MSMEs in the State for their sustained growth and development to gear up the industrialization process in the State.

SIDBI

Its purpose is to provide refinance facilities and short term lending to industries, and serves as the principal financial institution in the Micro, Small and Medium Enterprises (MSME) sector. **SIDBI** also coordinates the **functions** of institutions engaged in similar activities.

NABARD

NABARD is a Development Bank with a mandate for providing and regulating credit and other facilities for the promotion and development of agriculture, small-scale industries, cottage and village industries, handicrafts and other rural crafts and other allied economic activities in rural areas with a view to promoting …

KVIC

Rural Area as declared under **KVIC** Act 2006 - Scheme to be implemented by **KVIC**, **KVIB and DIC** ("Rural Area" means the area comprised in any Village and includes the area comprised in any town, the population of which does not exceed twenty thousand or such other figure as the Central Government may specify from time to …

Technology Business Incubators(TBI)

The need for instruments such as TBI has been recognised the world over for initiating technology led and knowledge driven enterprises. Studies also show that such mechanisms help not only in the growth of technology based new enterprises but also in improving their survival rate substantially (from 30 per cent to over 70 per cent). TBIs also facilitate speedy commercialisation of research outputs. The TBIs usually provide following types of services:-

- Market survey/ marketing assistance
- Business planning and training
- Organising management/ technical assistance
- Assistance in obtaining statutory approvals
- Information dissemination on product ideas/technologies
- Syndicating finances
- Arranging legal and IPR services
- Using facilities of the Host Institute (HI) at nominal charges
- Work space for a limited period
- Common facilities of TBI such as communication, conference, computers

Thus, the TBIs besides providing a host of services to new enterprises (and also to existing SMEs in the region) also facilitate an atmosphere congenial for their survival and growth. The essential feature of a TBI is that the tenant companies leave the incubator space within 2-3 years.

Objectives

TBIs are promoted to achieve the following objectives:

- Creation of technology based new enterprises
- Creating value added jobs & services
- Facilitating transfer of technology
- Fostering the entrepreneurial spirit
- Speedy commercialisation of R&D output
- Specialised services to existing SMEs

Criteria for selection of location

Ideally a TBI should be located near a source of technology and knowledge i.e. around R&D Institutions/Academic Institutions or it should have strong links with such institutions to ensure optimal use of the already existing expertise and facilities thus keeping the cost of the TBI on lower side. Locating TBIs in such location could also reduce time lag between technology development and its commercialisation. Further, as the success of a TBI largely depends on its location and management besides quality of tenant enterprises, following aspects relating to the Host Institution (HI) need to be kept in view while selecting location of the TBI:

- R&D track record and subsequent commercialisation of R&D output
- Dedicated team of R&D persons
- Industrial milieu in the region
- Proximity to other R&D/academic institutions
- Infrastructure, facilities and expertise available
- Strong commitment and willingness of the HI

Thrust Areas

Each TBI would focus on not more than 2-3 thrust areas. The thrust areas for a TBI would be identified based on the following:

- Expertise and facilities available in the HI
- Track record of the HI in the chosen areas
- Industrial climate in the region
- Market potential/demand in the region

To begin with, TBIs are proposed to be promoted in following selected thrust areas which have potential for faster growth:

- Information & Communication Technology (ICT)/Internet of Things (IOT)
- Healthcare
- Manufacturing
- Agriculture and allied fields
- Clean-Tech
- Energy
- Water
- Services

Facilities required

The TBIs should mainly draw upon the existing facilities available in the HI including land and building. However, certain essential facilities, which need to be created in a TBI are given below:

- Modern work space
- Communication facilities
- Computing facilities
- Vital equipment needed in identified area
- Library & information centre
- Training and conference facilities

Sponsorship

The TBI may be promoted by the selected HI and DST jointly. The HI has to provide the requisite land and building for the TBI. Other related and interested agencies could also be involved as sponsors.

Activities

Each TBI would be required to plan and undertake specified activities based on the identified thrust areas. However, the following set of activities is suggested as general guidelines:

- Provide specialized services to existing SMEs in the region
- Facilitate technology commercialization
- Consultancy
- Training including short courses
- Technology related IPR issues, legal and quality assurance services
- Marketing
- Assistance in obtaining and other clearances
- Common facilities
- Assistance in preparation of business plans
- Technology shows/ technology clinics/ trade fairs

Organisation Status

The TBI should itself represent a dynamic model of sustainable business operation and generate revenue as well as profits. As per the guidelines of the Department regarding legal status of new Technology Business Incubators, it has become mandatory to register the new TBIs as an autonomous body functioning as a society registered under societies act of 1860/or as a non profit making section 25/ section 8 company. The affairs of the TBI should be managed by an Advisory Board. The Board of the TBI should help not only in development of a strategic plan containing quantifiable objectives to achieve the desired results but also in managing the TBI efficiently and effectively. The Board should have representation from the promoters and reputed professionals. This may include representatives of DST, SIDBI, HI, Industry, VC companies, Entrepreneurs, student bodies and tenants of the TBI. A committee should also be set up for selection of tenant firms.

Staff Structure

The day to day operations of the TBI would be looked after by the Chief Executive Officer/Managing Director and a team of selected personnel which may include one or two professionals having technical/managerial qualification and relevant industry experience to look after areas such as business planning, technology transfer, training and consultancy. In addition, an accounts cum administrative officer and one secretarial assistant may be inducted in the core team. To cater to the specialised and need based services, the TBI should have a panel of experts/ consultants. Their services may be hired as and when required on payment basis. Security and house keeping services may be arranged on contract basis.

Role of the HI

The Host Institution has to play an important role not only in the establishment of the TBI project but also in its smooth and efficient functioning. Only those institutions/ organisations that can provide land and built-up space for TBI and are also willing to share available facilities and expertise would be considered for setting up of the TBI.

Host Institute should demonstrate its commitment and responsibility towards the TBI project. The HI will provide a suitable built up area where-in the TBI could be set up besides provision of utilities such as electricity and water. The HI will also ensure availability of following facilities to the tenants of the TBI on mutually agreed charges:

- Lab/testing facilities
- Library
- Mainframe computer
- Faculty support

Estimated Project Cost

Each TBI should prepare a detailed project proposal and work out the cost, based on actual requirements. A project implementation schedule may be prepared covering the key activity of the project. Since TBI is software intensive, greater focus should be laid on providing value-added services rather than facilities (hardware) to its tenants. Wherever possible, duplication of the facilities already existing in HI may be avoided and only need-based facilities may be proposed in the initial phase of the project.

Self Sufficiency

Each TBI is expected to become self-sufficient within a period of five years from the date of sanction of the project. The TBI should, however, start earning from the very first year of its operation. The TBI should appoint a Project Manager with relevant experience and exposure to the business environment.

Science & Technology Entrepreneurship Park (STEP)

The Science Parks and similar initiatives help in creating an atmosphere for innovation and entrepreneurship; for active interaction between academic institutions and industries for sharing ideas, knowledge, experience and facilities for the development of new technologies and their rapid transfer to the end user.

The Science & Technology Entrepreneurs Park (STEP) programme was initiated to provide a re-orientation in the approach to innovation and entrepreneurship involving education, training, research, finance, management and the government. A STEP creates the necessary climate for innovation, information exchange, sharing of experience and facilities and opening new avenues for students, teachers, researchers and industrial managers to grow in a trans-disciplinary culture, each understanding and depending on the other's inputs for starting a successful economic venture. STEPs are hardware intensive with emphasis on common facilities, services and relevant equipments.

The major objectives of STEP are to forge linkages among academic and R&D institutions on one hand and the industry on the other and also promote innovative enterprise through S&T persons.

Objectives

- To forge a close linkage between universities, academic and R&D institutions on one hand and industry on the other.
- To promote entrepreneurship among Science and Technology persons, many of whom were otherwise seeking jobs soon after their graduation.
- To provide R&D support to the small-scale industry mostly through interaction with research institutions.
- To promote innovation based enterprises.

Facilities and Services Provided by STEPs

- It offers facilities such as nursery sheds, testing and calibration facilities, precision tool room/central workshop, prototype development, business facilitation, computing, data bank, library and documentation, communication , seminar hall/conference room , common facilities such as phone, telex, fax, photocopying. It offers services like testing and calibration, consultancy.
- Training, technical support services, business facilitation services, database and documentation services, quality assurance services and common utility services .

The department has so far catalysed 15 STEPs in different parts of the country, which have promoted nearly 788 units generating annual turnover of around Rs. 130 crores and employment for 5000 persons. More than 100 new products and technologies have been developed by the STEPs / STEP promoted entrepreneurs. In addition, over 11000 persons have been trained through various skill development programmes conducted by STEPs.

STEPs are autonomous bodies registered as societies under the Societies Registration Act.

Role of Host Institution

In order to achieve synergetic benefits and also to harness the knowledge and expertise available in academic and R&D institutions of excellence, every STEP needs to be promoted around a host institution which could launch, sustain and help the STEP grow. Therefore, the host institution has to play an important and crucial role in promotion and growth of a STEP.

The host institution should aim at optimum usage of its facility by STEP. For this purpose, a periodic assessment of the priorities must be undertaken based on which re-allocation of resources might become necessary. During the stages of planning and implementation of the STEP project, the host institution must not lose sight of its slated goals and objectives with respect to academic excellence.

STEP Model

Though a workable 'STEP-Model' has been evolved by an Expert Committee chaired by the late Prof. Y. Nayudamma, however, each STEP would have to carve out a niche for itself with regard to the types of products to be developed based on the availability of facilities and expertise in the host institution and also the industrial climate of the region.

Each project envisages active involvement and participation of agencies such as the host institution, ultimate user of the facilities, financial institutions, government agencies and STEP management.

QUESTIONS

1. Who can be an entrepreneur?
2. State the characteristics of entrepreneurship.
3. What are the barriers to entrepreneurship?
4. Differentiate between entrepreneur and manager?
5. Write Short notes on
     I. Sole proprietorship
     II. NABARD
     III. TBI
6. What is a startup?

2. Market survey and opportunity Identification

(Business Planning)Business planing

A business plan is a written description of your business's future. That's all there is to it--a document that desribes what you plan to do and how you plan to do it. If you jot down a paragraph on the back of an envelope describing your business strategy, you've written a plan, or at least the germ of a plan.

Business plans can help perform a number of tasks for those who write and read them. They're used by investment-seeking entrepreneurs to convey their vision to potential investors. They may also be used by firms that are trying to attract key employees, prospect

for new business, deal with suppliers or simply to understand how to manage their companies better.

So what's included in a business plan, and how do you put one together? Simply stated, a business plan conveys your business goals, the strategies you'll use to meet them, potential problems that may confront your business and ways to solve them, the organizational structure of your business (including titles and responsibilities), and finally, the amount of capital required to finance your venture and keep it going until it breaks even.

Sound impressive? It can be, if put together properly. A good business plan follows generally accepted guidelines for both form and content. There are three primary parts to a business plan:

- **The first is the *business concept***, where you discuss the industry, your business structure, your particular product or service, and how you plan to make your business a success.
- **The second is the *marketplace section***, in which you describe and analyze potential customers: who and where they are, what makes them buy and so on. Here, you also describe the competition and how you'll position yourself to beat it.
- **Finally, the *financial section*** contains your income and cash flow statement, balance sheet and other financial ratios, such as break-even analyses. This part may require help from your accountant and a good spreadsheet software program.

Breaking these three major sections down even further, a business plan consists of seven key components:

1. Executive summary
2. Business description
3. Market strategies
4. Competitive analysis
5. Design and development plan
6. Operations and management plan
7. Financial factors

In addition to these sections, a business plan should also have a cover, title page and table of contents.

How Long Should Your Business Plan Be?
Depending on what you're using it for, a useful business plan can be any length, from a scrawl on the back of an envelope to, in the case of an especially detailed plan describing a complex enterprise, more than 100 pages. A typical business plan runs 15 to 20 pages, but there's room for wide variation from that norm.

Much will depend on the nature of your business. If you have a simple concept, you may be able to express it in very few words. On the other hand, if you're proposing a new kind of business or even a new industry, it may require quite a bit of explanation to get the message across.

The purpose of your plan also determines its length. If you want to use your plan to seek millions of dollars in seed capital to start a risky venture, you may have to do a lot of explaining and convincing. If you're just going to use your plan for internal purposes to manage an ongoing business, a much more abbreviated version should be fine.

Who Needs a Business Plan?

About the only person who doesn't need a business plan is one who's not going into business. You don't need a plan to start a hobby or to moonlight from your regular job. But anybody beginning or extending a venture that will consume significant resources of money, energy or time, and that is expected to return a profit, should take the time to draft some kind of plan.

Startups. The classic business plan writer is an entrepreneur seeking funds to help start a new venture. Many, many great companies had their starts on paper, in the form of a plan that was used to convince investors to put up the capital necessary to get them under way.

Most books on business planning seem to be aimed at these startup business owners. There's one good reason for that: As the least experienced of the potential plan writers, they're probably most appreciative of the guidance. However, it's a mistake to think that only cash- starved startups need business plans. Business owners find plans useful at all stages of their companies' existence, whether they're seeking financing or trying to figure out how to invest a surplus.

Established firms seeking help. Not all business plans are written by starry-eyed entrepreneurs. Many are written by and for companies that are long past the startup stage. WalkerGroup/Designs, for instance, was already well-established as a designer of stores for major retailers when founder Ken Walker got the idea of trademarking and licensing to apparel makers and others the symbols 01-01-00 as a sort of numeric shorthand for the approaching millennium. Before beginning the arduous and costly task of trademarking it worldwide, Walker used a business plan complete with sales forecasts to convince big retailers it would be a good idea to promise to carry the 01-01-00 goods. It helped make the new venture a winner long before the big day arrived. "As a result of the retail support up front," Walker says, "we had over 45 licensees running the gamut of product lines almost from the beginning."

These middle-stage enterprises may draft plans to help them find funding for growth just as the startups do, although the amounts they seek may be larger and the investors more willing. They may feel the need for a written plan to help manage an already rapidly growing business. Or a plan may be seen as a valuable tool to be used to convey the mission and prospects of the business to customers, suppliers or others.

Plan   an   Updating   Checklist

Here are seven reasons to think about updating your business plan. If even just one applies to you, it's time for an update.

1. A new financial period is about to begin. You may update your plan annually, quarterly or even monthly if your industry is a fast-changing one.
2. You need financing, or additional financing. Lenders and other financiers need an updated plan to help them make financing decisions.
3. There's been a significant market change. Shifting client tastes, consolidation trends among customers and altered regulatory climates can trigger a need for plan updates.
4. Your firm develops or is about to develop a new product, technology, service or skill. If your business has changed a lot since you wrote your plan the first time around, it's time for an update.
5. You have had a change in management. New managers should get fresh information about your business and your goals.
6. Your company has crossed a threshold, such as moving out of your home office, crossing the $1 million sales mark or employing your 100th employee.
7. Your old plan doesn't seem to reflect reality any more. Maybe you did a poor job last time; maybe things have just changed faster than you expected. But if your plan seems irrelevant, redo it.

Finding the Right Plan for You

Business plans tend to have a lot of elements in common, like cash flow projections and marketing plans. And many of them share certain objectives as well, such as raising money or persuading a partner to join the firm. But business plans are not all the same any more than all businesses are.

Depending on your business and what you intend to use your plan for, you may need a very different type of business plan from another entrepreneur. Plans differ widely in their length, their appearance, the detail of their contents, and the varying emphases they place on different aspects of the business.

The reason that plan selection is so important is that it has a powerful effect on the overall impact of your plan. You want your plan to present you and your business in the best, most accurate light. That's true no matter what you intend to use your plan for, whether it's destined for presentation at a venture capital conference, or will never leave your own office or be seen outside internal strategy sessions.

When you select clothing for an important occasion, odds are you try to pick items that will play up your best features. Think about your plan the same way. You want to reveal any positives that your business may have and make sure they receive due consideration.

Types   of   Plans
Business plans can be divided roughly into four separate types. There are very short plans, or miniplans. There are working plans, presentation plans and even electronic plans. They require very different amounts of labor and not always with proportionately different results. That is to say, a more elaborate plan is not guaranteed to be superior to an abbreviated one, depending on what you want to use it for.

- *The Miniplan.* A miniplan may consist of one to 10 pages and should include at least cursory attention to such key matters as business concept, financing needs, marketing plan and financial statements, especially cash flow, income projection and balance sheet. It's a great way to quickly test a business concept or measure the interest of a potential partner or minor investor. It can also serve as a valuable prelude to a full-length plan later on.

Be careful about misusing a miniplan. It's not intended to substitute for a full-length plan. If you send a miniplan to an investor who's looking for a comprehensive one, you're only going to look foolish.

- *The Working Plan.* A working plan is a tool to be used to operate your business. It has to be long on detail but may be short on presentation. As with a miniplan, you can probably afford a somewhat higher degree of candor and informality when preparinga working plan.

A plan intended strictly for internal use may also omit some elements that would be important in one aimed at someone outside the firm. You probably don't need to include an appendix with resumes of key executives, for example. Nor would a working plan especially benefit from, say, product photos.

Fit and finish are liable to be quite different in a working plan. It's not essential that a working plan be printed on high-quality paper and enclosed in a fancy binder. An old three- ring binder with "Plan" scrawled across it with a felt-tip marker will serve quite well.

Internal consistency of facts and figures is just as crucial with a working plan as with one aimed at outsiders. You don't have to be as careful, however, about such things as typos in the text, perfectly conforming to business style, being consistent with date formats and so on. This document is like an old pair of khakis you wear into the office on Saturdays or that one ancient delivery truck that never seems to break down. It's there to be used, not admired.

- ***The Presentation Plan.*** If you take a working plan, with its low stress on cosmetics and impression, and twist the knob to boost the amount of attention paid to its looks, you'll wind up with a presentation plan. This plan is suitable for showing to bankers, investors and others outside the company.

Almost all the information in a presentation plan is going to be the same as your working plan, although it may be styled somewhat differently. For instance, you should use standard business vocabulary, omitting the informal jargon, slang and shorthand that's so useful in the workplace and is appropriate in a working plan. Remember, these readers won't be familiar with your operation. Unlike the working plan, this plan isn't being used as a reminder but as an introduction.

You'll also have to include some added elements. Among investors' requirements for due diligence is information on all competitive threats and risks. Even if you consider some of only peripheral significance, you need to address these concerns by providing the information.

The big difference between the presentation and working plans is in the details of appearance and polish. A working plan may be run off on the office printer and stapled together at one corner. A presentation plan should be printed by a high-quality printer, probably using color. It must be bound expertly into a booklet that is durable and easy to read. It should include graphics such as charts, graphs, tables and illustrations.

It's essential that a presentation plan be accurate and internally consistent. A mistake here could be construed as a misrepresentation by an unsympathetic outsider. At best, it will make you look less than careful. If the plan's summary describes a need for $40,000 in financing, but the cash flow projection shows $50,000 in financing coming in during the first year, you might think, "Oops! Forgot to update that summary to show the new numbers." The investor you're asking to pony up the cash, however, is unlikely to be so charitable.

- ***The Electronic Plan.*** The majority of business plans are composed on a computer of some kind, then printed out and presented in hard copy. But more and more business information that once was transferred between parties only on paper is now sent electronically. So you may find it appropriate to have an electronic version of your plan available. An electronic plan can be handy for presentations to a group using a computer-driven overhead projector, for example, or for satisfying the demands of a discriminating investor who wants to be able to delve deeply into the underpinnings of complex spreadsheets.

Small Scale Industries (SSI)

Small scale industries are those industries in which the manufacturing, production and rendering of services are done on a small or micro scale. These industries make a one-time investment in machinery, plants, and industries, but it does not exceed Rs 1 Crore.

1. Introduction of SSI

1. <u>Introduction of SSI</u>

Essentially the small scale industries are generally comprised of those industries which manufacture, produce and render services with the help of small machines and less manpower. These enterprises must fall under the guidelines, set by the Government of India.

The SSI's are the lifeline of the economy, especially in developing countries like India. These industries are generally labour-intensive, and hence they play an important role in the creation of employment. SSI's are a crucial sector of the economy both from a financial and social point of view, as they help with the per capita income and resource utilisation in the economy.

<u>Examples and Ideas of Small Scale Industries</u>

- Bakeries
- School stationeries
- Water bottles
- Leather belt
- Small toys
- Paper Bags
- Photography
- Beauty parlours

2. <u>Characteristics of SSI</u>

o **Ownership**: SSI 's generally are under single ownership. So it can either be a sole proprietorship or sometimes a partnership.

o **Management**: Generally both the management and the control is with the owner/owners. Hence the owner is actively involved in the day-to-day activities of the business.

o **Labor Intensive**: SSI's dependence on technology is pretty limited. Hence they tend to use labour and manpower for their production activities.

- **Flexibility**: SSI's are more adaptable to their changing business environment. So in case of amendments or unexpected developments, they are flexible enough to adapt and carry on, unlike large industries.

- **Limited Reach**: Small scale industries have a restricted zone of operations. Hence, they can meet their local and regional demand.

- **Resources utilisation**: They use local and readily available resources which helps the economy fully utilise natural resources with minimum wastage.

3. Role in the Indian economy

- **Employment**: SSI's are a major source of employment for developing countries like India. Because of the limited technology and resource availability, they tend to use labour and manpower for their production activities.

- **Total Production**: These enterprises account for almost 40% of the total production of goods and services in India. They are one of the main reasons for the growth and strengthening of the economy.

- **Make in India**: SSI's are the best examples for the Make in India initiative. They focus on the mission to manufacture in India and sell the products worldwide. This also helps create more demands from all over the world.

- **Export contribution**: India's export industry majorly relies on these small industries for their growth and development. Nearly half of the goods that are exported from India are manufactured or produced by these industries.

- **Public Welfare**: These industries have an opportunity to earn wealth and create employment. SSI's are also important for the social growth and development of our country.

4. Objectives of SSI

The objectives of the small scale industries are:

- To create more employment opportunities.

- To help develop the rural and less developed regions of the economy.

- To reduce regional imbalances.

- To ensure optimum utilisation of unexploited resources of the country.

- To improve the standard of living of people.

- To ensure equal distribution of income and wealth.

- To solve the unemployment problem.

- To attain self-reliance.

o To adopt the latest technology aimed at producing better quality products at lower costs.

5. <u>Registration of SSI</u>

SSI registration is a registration provided by the Ministry of MSME. A business should obtain SSI registration in order to be eligible for a number of schemes, subsidies and other incentives provided by the Government to such SSI's. SSI registration can be obtained online too. Let's look at the process of SSI registration online:

Overview   of   SSI   registration
SSI registration is provided by the Ministry of Micro, Small and Medium Enterprises through the Directorate of Industries of the State Government. The main logic behind the SSI registration is to set up new SSI businesses in India. SSI registration helps the business to be eligible for a number of subsidies given by the Government. We can also get SSI registration online.

<u>Eligibility Criteria for SSI registration</u>

SSI registration can be obtained for:

o Manufacturing enterprise; and

o Service enterprise

<u>For manufacturing enterprise, if the investment in plant and machinery (excluding land& buildings) is within any of the following levels:</u>

Micro Enterprises: Investment of up to Rs.25 lakhs in plant and machinery Small Enterprises: Investment of up to Rs.5 crores in plant and machinery Medium Enterprises: Investment of up to Rs.10 crores in plant and machinery

<u>For service enterprise, if the investment in plant and machinery (excluding land & buildings) is within any of the following levels:</u>

Micro Enterprises: Investment of up to Rs.10 lakhs in equipment Small Enterprises: Investment of up to Rs.2 crores in equipment Medium Enterprises: Investment of up to Rs.5 crores in equipment If the investment is done within the above-mentioned limits then the SSI registration has to be obtained.

<u>Benefits of obtaining SSI registration</u>

o There are various tax rebates offered to SSI's

o A credit for Minimum Alternate Tax (MAT) is allowed to be carried forward for up to 15 years instead of 10 years

o There are many government tenders which are only open to the SSI.

o They get easy access to credit.

- Once registered the cost of acquiring a patent, or the cost of setting up the industry reduces as many rebates and concessions are available.

- Business registered as SSI are given higher preference for government license and certification.

Business registered as SSI are given higher preference for government license andcertification.

- To do the registration the SSI owner has to fill a single SSI online registration form. It can be done in the offline mode as well.

- If a person wants to get registration for more than one industry then also he/she can opt for an individual SSI registration done.

- To get registered he/she has to fill a single form which is available at the website.

- The documents required for the SSI registration are Aadhar number, industry name, address, bank account details and some common information.

- Here, the person can provide self-certified certificates.

- No registration fees is required for the registration.

- Once the SSI registration form is filled and uploaded, very soon you will obtain the SSI registration number.

Ancillary unit

Ancillary industries are those which manufacture parts and components to be used by larger industries. Eg- Companies like GE (ancillary) produce engines for the aircraft industry. The programme of ancillarisation includes motivation of public and private sector units to offload production of components, parts, sub-assemblies, tools, intermediates, services etc., to ancillary units. The programme of ancillary development has specific advantages both for large as well as small industries and also for the total economy of the country. The large scaleunits have the advantages in the form of savings in investments, inventories, employment of labour, etc. and getting the items of the desired specifications, while the small scale units have the advantage of getting assured market for their products, availability of technical assistance and improved technology from the parent units. This programme also helps in overall economy of the country. Small Industry Development Organisation (SIDO) is a nodal agency of the Central Government and Ancillary Division at Headquarters continued its function for the promotion of ancillarisation programme in the country. Constant liaison has been maintained with Administrative Ministries both at Central & State Levels, Department of Public enterprises, public/private sector undertakings and other industrial developmental agencies through various programmed such as Vendor Development Programmes, Buyer- Sellers Meet, Ancillary Exhibition, Seminars, Workshops, State Level Ancillary Advisory Meetings, Plant Level Committee Meetings and PSUs and visit to public/private sector undertakings for the promotion of small ancillary & sub-contracting units. Sub-contracting exchanges are functioning as a part of major SISIs in the country at important cities for the promotion of fruitful and lasting contracts between large & medium undertakings and small scale ancillary units. The spare capacity for different facilities as available with the competentsmall scale units are registered with these SCXs. These SCXs also obtain such items from large units which are required by them and can be manufactured in the small scale sector.

These SCXs organise contacts between Buyers & Sellers by way of organising Vendor Development Programmes, Buyers & Sellers Meet and Exhibition, etc.

Tiny unit

The **Tiny** (Micro) **Small scale industries** have played a vital role in the overall economic development of our country where millions of people are unemployed, where most of the **entrepreneurs** are capable of making only small investment and where there is chronic shortage of resources to produce sophisticated machinery and ...**Tiny Industry**: **Tiny** Scale **industry** is one in which the investment in plant and. machinery is less than Rs.25 lakhs irrespective of the location of the unit. • Small Scale **Industry**: An **industrial** undertaking in which the investment in plant.

What is the Service Sector?

The service sector produces intangible goods, more precisely services instead of goods, and according to the U.S. Census Bureau, it comprises various service industries including warehousing and transportation services; information services; securities and other investment services; professional services; waste management; health care and social assistance; and arts, entertainment, and recreation. Countries with economies centered around the service sector are considered more advanced than industrial or agricultural economies.

KEY TAKEAWAYS

- The service sector is the third sector of the economy, after raw materials production and manufacturing.
- The service sector includes a wide variety of tangible and intangible services from office cleaning to rock concerts to brain surgery.
- The service sector is the largest sector of the global economy in terms of value-added and is especially important in more advanced economies.

Understanding Service Sector

The service sector, also known as the tertiary sector, is the third tier in the three sector economy. Instead of the product production, this sector produces services maintenance and repairs, training, or consulting. Examples of service sector jobs include housekeeping, tours, nursing, and teaching. By contrast, individuals employed in the industrial or manufacturing sectors produce tangible goods, such as cars, clothes, or equipment.

Among the countries that place heavy emphasis on the service sector, the United States, the United Kingdom, Australia, and China rank among the top. In the United States, the Institute for Supply Management (ISM) produces a monthly index that details the general state of business activity in the service sector. This index is regarded as a metric for the overall economic health of the country because approximately two-thirds of U.S. economic activity occurs in the service sector.

According to the CIA World Factbook, the following countries are the largest by service or tertiary output as of 2018:

1. United States: $15.5 trillion
2. China: $6.2 trillion
3. Japan: $3.4 trillion
4. Germany: $2.5 trillion
5. United Kingdom: $2.1 trillion

6. France: $2.0 trillion
7. Brazil: $1.5 trillion
8. India: $1.5 trillion
9. Italy: $1.4 trillion
10. Canada: $1.2 trillion

The Service Sector in the Three-Part Economy

The service or tertiary sector is the third piece of a three-part economy. The first economic sector, the primary sector, covers the farming, mining, and agricultural business activities in the economy. The secondary sector covers manufacturing and business activities that facilitate the production of tangible goods from the raw materials produced by the primary sector. The service sector, though classified as the third economic sector, is responsible for the largest portion of the global economy's business activity.

Technology in the Service Industry

Technology, specifically information technology systems, is shaping the way businesses in the service sector operate. Businesses in this sector are rapidly placing more focus on what is becoming known as the [knowledge economy](#), or the ability to surpass competitors by understanding what target customers want and need, and operate in a way that meets those wants and needs quickly with minimal cost. In nearly all industries within the sector, businesses adopt new technology to bolster production, increase speed and efficiency, and cut down on the number of employees required for operation. This cuts down on costs and improves incoming revenue streams.

Time schedule planning

1. Create a routine

No matter what you are working on, create a routine. Block times for specific activities, and stick with the plan. Turn your calendar into a bunch of blocks, and put activities into those blocks. Whatever is not planned, you don't do. If you want free time, plan it.
Your routine may change through the year, but at any given time it's better to have a plan. For example, if you are working on launching a company, and need to do customer discovery, coding and hiring, then prioritize and block specific times for each activity. A calendar app is a very useful tool

2. Group meetings and calls into blocks

For example, if you need to have outside meetings, block two and a half days a week for those meetings, and go to the outside meetings only during those times. Do the same thing for in-office meetings. This way you are not only creating a chunk of time for meetings, you are also creating other blocks of time that you will be able to important work. Do the same thing with calls, and book them all back to back.

3. Optimize time for different meeting types

Personally, I am now a big fan of 30-minute meetings and 10-minute calls. I think 10-minute calls are a great way to initially connect with someone or give someone quick advice. You can do a Google Hangout or Skype if you prefer to see the person instead of just hearing

them. The reason 10-minute calls work is because people skip BS and get to the point. Try it. Ten minutes is actually a lot of time, if you focus. I prefer to do these calls on Fridays, when I am usually working from home.

I am not a big fan of introductory coffee meetings, lunches and dinners. I am a huge fan of coffee and meals with people I already know. Those meetings are typically productive and fun, but the first time you are meeting someone, it's more productive to do a call or an actual 30-minute meeting in the office.

Here are the types of meetings you might want to book:

- 30-minute meeting in the office to get to know someone or catch up
- 45-minute meeting outside of the office. Allow 15 minutes for travel.
- 10-minute call to help someone who needs advice
- 15-minute daily standup -- great for startups and engineering teams
- 30-minute weekly staff meeting

Whatever meetings you hold, group them into blocks depending on your particular schedule. If you feel like a particular type of meeting needs more or less time, then adjust the block accordingly.

4. Use appointment slots

There is a great feature in Google Calendar called Appointment Slots. It allows you to book a chunk of time, and then split it into pieces. For example, I can book three hours of outside meetings and then split it into three meetings -- one hour each. Or I can book one hour of calls and split it into six calls at 10 minutes each. There is also a bunch of specific tools, such as doodle, that do that too.

The next step is to create bit.ly links for different blocks of time. You can have a link for your outside meetings, another link for 30-minute inside meetings and yet another one for 10- minute calls. You then share these links, and they can book the time with you. I've done this with Techstars candidate companies and it was amazingly effective. It minimized the back and forth on email and saved a ton of time for me and the companies.

Related: How to Use Technology to Increase Productivity, Not Distract You

This won't work with everyone, because some people may find this rude. In any case, if you are not comfortable sending the link to a someone, then you can use your own appointment slots, suggest a few meeting times, and then book the specific slot yourself.

If you are asking someone to meet, always propose several specific alternative times such as Tuesday at 4:30 p.m. or 5 p.m. on Wednesday. David Tisch gave a great talk that covers scheduling meetings and many more basics of communication.

5. Block time for email

This is the most important tip in the whole post. Email will own you unless you own it. To own your email you must avoid doing it all the time. To do that you need to schedule the time to do your email. It is absolutely a must. In fact it is so important that I wrote a whole entire post about managing email.

6. Plan your exercise and family time

Unless you put it on the calendar, it won't get done. Well, that applies to your exercise and time with your family. Whether you go in the morning, afternoon or evening, do it

three times a week or every day, put exercise time on the calendar. My friend and mentor Nicole Glaros makes it very clear that her mornings, until 10 a.m., belong to her. She hits the pavement or the gym, depending on the weather, and rarely deviates from her routine.

The same applies to planning time with your family and significant others. If you are a workaholic like me, you will end up stealing time from your family unless you book it in advance and train yourself to promptly unplug. Many people in the industry have talked about planning family time. My favorite is Brad Feld, who talks about it a lot.

7. __Actually manage your time__

I think about my time a lot. I think about where it goes. I think about where can I get more of it, and how to optimize it. When I was running GetGlue, I had an assistant who was managing my time. She was awesome. But when I joined Techstars, I decided that I will manage my calendar myself. I have to confess that I am happy about this decision.

I find myself thinking about what I am doing, who am I meeting with and why a lot more. I meet with a lot of people every week. My schedule is particularly insane during the selection process. Yet, because I manage my calendar, follow a routine, plan meetings in blocks and use appointment slots, I find myself less overwhelmed and less stressed.

Agencies to be contacted for Project Implementation

Project Implementation Unit is a technical **project** management **unit** that serves as the technical secretariat for the GPC. PIU reports directly to GPC and is tasked to **implement** daily tasks, and manage and oversee the **project** development.

Setting up an Implementation Agency

- What type of an Implementation Agency to set up – Society, Trust or Section 8 Company
- How to set up the Implementation Agency? What are the rules and regulations?
- Registration requirements:
  - o With Registering bodies
  - o Income Tax
  - o Service Tax
  - o Home Ministry
  - o CAPART
- Setting up the Registered Office
- Opening a Bank Account
- Things to keep track of with regard to all regulations and compliances?

Governance of the Implementation Agency

- How to ensure effective governance mechanisms are in place, with adequate checks and balances
- How to build on-time reporting system and effective documentation?

Funding

- How to build a working revenue model and improve fundraising abilities?
- How to develop correct financial and reporting system with funders?
- How to obtain various tax exemptions to support fund raising?
- What are the ways to reach financial sustainability?

Project Development & Management

- How to develop impactful projects?
- How to create projects that are scalable and can get funded?
- What are the strongest areas to build your activities around?

Scaling

- How to ensure the current actions will lead to future growth?
- How to expand the projects to maximize impact?

Skill-building

- Where to find professionals to hire and how to develop their skills further?
- Which skills are currently required in the market that can get the target client into future jobs?

Evaluation and assessment

- How to evaluate the efficiency of a project and maximize impact?

Assessment of Demand and supply and Potential areas of Growth

What Is Demand?
Demand is the **amount of goods or services that consumers are willing to pay at each price point**. It is based on wants and needs and the ability to pay. If consumers are unable to pay for goods and services, demand does not exist. When the price of a good or service rises, demand decreases. Conversely, if the price of a good or service falls, demand goes up. This law of demand represents an inverse relationship between price and quantity demanded.

Take the price of gasoline, for example. When the price of a gallon of gasoline increases, consumers start finding ways to reduce consumption by combining errand trips, taking vacations closer to home, forming carpools or commuting by mass transit.

However, the demand for a product is not infinite; people usually want only a certain amount of a good. Purchasing additional quantities of a product yields less and less satisfaction.

How to Graph a Demand Curve
A demand curve can be illustrated by constructing a graph with price plotted on the vertical axis and quantity demanded on the horizontal. The demand curve will slope downward from left to right as price declines and demand increases. Demand curves can be flat or steep, or they can be straight or curved. The relationship between price and demand will be unique for each good or service being analyzed.

What Is Supply?
Supply is the **amount of goods or services available or produced**, based on a number of factors such as input resources, labor, technology and regulations. Let's consider again the price of gasoline.

If a new gas reserve is discovered unexpectedly, the supply suddenly increases. Because gas becomes less scarce, prices become more competitive to beat out other suppliers for sales volumes. Prices will decrease to the level where the demand matches, because demand will naturally increase: Cheaper gas is more attractive than its more expensive counterpart.

How to Graph a Supply Curve
Graphically, a supply line is represented by an upward sloping curve from left to right with price on the vertical axis and quantity plotted on the horizontal axis. In the case of both

supply and demand relationships, all other variables are assumed to remain constant when constructing these graphs.

What Is Supply and Demand Equilibrium?

Since the data for the supply and demand curves can both be plotted on the same graph, they will intersect at an equilibrium point. This is the price/quantity point where consumers and producers are satisfied with their decisions, and the market is in balance. The quantity supplied equals the quantity demanded.

The laws of supply and demand ensure that the market always recalibrates to equilibrium. When the market price is higher than the equilibrium price, the supply quantity will be greater than the quantity demanded, resulting in excess supply. Producers will then have the incentive to cut prices down to the equilibrium level to sell this excess supply.

If the market price is lower than equilibrium, competition between buyers creates excess demand. Producers then have the opportunity to raise prices up to the equilibrium level.

What Factors Affect Demand?

The factors that affect demand cause a shift of the entire demand curve to either the left or the right. This is different from a movement along the demand curve, which would be a result of a price change.

The factors that affect the demand curve are changes in:

Tastes and preferences. Consumer tastes are constantly changing, and demand for products rises and falls as a result. Kale is an example of changing tastes. For years, kale was used as a decoration for commercial buffets, then its health benefits become more publicized. As a result, demand for kale rose and prices increased.

Income level. When consumer incomes increase, they are able to demand and buy more normal goods, which are products whose demand goes up as income rises. For example, suppose a car costs $25,000 and 19 million buyers are willing to pay this price. Now, suppose consumer incomes increased; with more money available to spend, 21 million people can afford to pay $25,000 for the car.

In this case, demand for cars increased with a rise in income, and the demand curve shifted to the right.

Prices of substitutes. An increase in the price of one product can increase the demand for its substitute. Coca-Cola and Pepsi are excellent examples of this effect. If Pepsi increases its price, consumers will quickly switch to buying more Coke.

Complements. Complementary goods are products typically bought together, like yoga classes and mats or bread and butter. If the price for yoga classes falls, more people will sign up, increasing the demand for mats. A drop in the price of bread will increase the demand for butter to put on the bread.

Expectation of future prices. If consumers expect prices to drop in the near future or go on sale, they will delay their purchases, shifting the demand curve to the left. English muffins, for example, are frequently offered at buy one, get one free. Consumers know this, so rather than pay full price for a single package, they wait for the special offer.

The opposite is true for anticipated price increases, shifting the demand curve to the right in advance of price rises.

Changes in buyer demographics. Changing demographics affect the demand for different products. For instance, the increasing percentage of elderly people in the population increases the demand for nursing homes, hearing aids and in-home health care.

What Factors Affect Supply?

The factors that affect the shifts in a supply curve are changes in:

Cost of production. Changes in the costs to manufacture a product will cause the producer to modify production volume. Suppose a car manufacturer receives an increase in the price of steel and raises the price of cars to cover the increase. Consumers will demand a lower

quantity of cars at the higher price, causing the manufacturer to reduce output and shift the supply curve to the left.

Technology. Improvement in technology that reduces the cost of production will enable producers to lower selling prices and sell more cars. This shifts the supply curve to the right.

Number of suppliers. The addition of new suppliers increases the quantities available at the same prices and shifts the supply curve to the right.

Government regulations. Some government regulations can increase the cost of production. As a result, a manufacturer might reduce the quantity supplied because the profit is reduced, shifting the curve to the left.

What Is the Importance of Supply and Demand Analysis?

All business managers and consumers use supply and demand analysis to make decisions. Business owners analyze the factors that affect supply and demand curves to determine what volume to produce and how to price their products. Consumers make buying decisions, either consciously or instinctively, based on their wants and needs and perceived value received at particular price points.

The basics laws of supply and demand form the foundation of a competitive, capitalistic environment.

growth potential

Growth potential is an organization's future ability to generate larger profits, expand its workforce and increase production. In the business sense, an organization's growth potential depends heavily upon its leadership's expectations for success, and the quantitative and qualitative measures used to determine expansion readiness.

Growth potential can be gauged from an organization's planned movement into new markets, the development of new product lines, the employment of more effective marketing techniques, or other methods that grow a business from a niche market to a more volume operation. Growth potential is often a barometer for investment interest from public and private investors, venture capitalists and other stakeholders.

Identification of Business Opportunity

I. 1. IDENTIFICATION OF BUSINESS OPPORTUNITY Made and Drafted By- Krishnkant Chaturvedi Gwalior (MP), INDIA Email: krishna.chaturvedi37@gmail.com WHAT IS OPPORTUNITY? Opportunity is defined as a situation that enables an entrepreneur to offer marketable products or services to interested buyers or end users 1KRISHNKANT CHATURVEDI

II. 2. TYPES OF SITUATIONAL FACTORS •Product or service is still not in existence •Product or service is already in the market but failed to satisfy the customers – so need to be improved EMERGENCE OF OPPORTUNITY • when people decide they have certain needs and want to be satisfied, or when people discovered a problem of some kind that can be helped by a product or service. • The presence of unfulfilled needs and want and/or problems alerts the entrepreneur to the potential opportunity. • The entrepreneur later creates a business that is able to fulfill the needs or want and/or solve the problem. 2KRISHNKANT CHATURVEDI

III. 3. OPPORTUNITY IDENTIFICATION Opportunity identification is a process that involved the search for and discovery of business opportunities APPROACH TO OPPORTUNITY IDENTIFICATION 1. Observe changes in the environment 2. Recognize a need that customers have that is not being satisfied 3. Recognize problems and find ways to solve it OBSERVING CHANGES IN THE ENVIRONMENT Changes in the environment give rise to needs and wants and/or

problems, and an opportunity emerges Important environment forces to observe include : 1. Economic forces 2. Social forces 3. Technological advances 4. Political and regulatory statues 3KRISHNKANT CHATURVEDI

IV. 4. Examples of How Changes in the Environment Provides Openings for New Product and Service Opportunities Structure of Population and Income Number of teenagers higher than number of elderly and children Cyber cafes, Cineplex's, recording studios People have higher purchasing power Passenger cars, household furniture, DVD Social Increase incident of housebreaking Grills, alarm, sensor, security systems Increase interest in fitness Fitness center, dancing class, in-house exercise equipment, health food store Increase mobility of population Hand phone, laptop computers Increasing predominance of dual-income families leaves less time to cook at home Restaurants, food delivery services Technological Advances Advances in biotechnology Biotech-related pharmaceutical products, foodproducts, veterinary products Development of the internet E-commerce, improved communication Increase pressure to improve economicperformance Online marketing, cost control services Government Policies and Regulations Increased driving standards Smoke emission control, helmet, seatbelt 4KRISHNKANT CHATURVEDI

V. 5. RECOGNIZE NEEDS AND WANTS Opportunity occurs whenever there is a need and want to fulfill. The term "needs" refer to basic needs that the consumer must have in order to live while the term "wants" refers to a personal desire for something that is more than a basic need. RECOGNIZE PROBLEM AND FIND SOLUTION Problems can be recognized by observing the challenges that people encounter in their daily lives. Solution to the problem represented a business opportunity OPPORTUNITY IDENTIFICATION PROCESS Search for Changes in the environment Recognize needs and wants, and solutions Discovery of Opportunity 5KRISHNKANT CHATURVEDI

VI. 6. ABILITY TO SEARCH AND DISCOVER BUSINESS OPPORTUNITIES •Experience and exposure •Knowledge and skills •"Special alertness" •Social network •Creativity •Vigilant MECHANISM TO IDENTIFY OPPORTUNITIES • Customers • Retailers and distributors • Business associates • Bankers • Consultants •Employees • Others EVALUATION AND SELECTION PROCESS The evaluation and selection process involves judging the viability of the opportunity and assessing its potential. 6KRISHNKANT CHATURVEDI

VII. 7. CRITERIA FOR EVALUATION AND SELECTION 1. Can make money and has potential for growth 2. Less competition 3. Good fit between entrepreneur and opportunity 4. Has competitive advantage 5. Workable and efficient 6. Not against the norms and values of the community 7. Conform with laws and regulations 7KRISHNKANT CHATURVEDI

Final product selection

There are three basic stages/steps involved in **product**/venture **selection**. These are idea generation, evaluation, and choice. The starting point for idea generation could be a simple analysis of the **business's** strengths and weakness.

CONCEPT OF **PRODUCT SELECTION** • **Product selection** is a decision process, in which the design team selects one or few **product** concept for further development. • An **entrepreneur** establishes a business unit with a modest investments in a small scale level.

QUESTIONS

1. What is Business Planning?
2. Write short note on
   I. SSI
   II. Time schedule planning
3. What is demand and supply explain?
4. Differentiate between an ancillary unit and Tiny unit?

3.Project Report Preparation

PROJECT REPORT PREPARATION • Project report: The document which contains the required details like the personal biodata of the entrepreneur, the product to be manufactured, its design manufacturing process, row materials, machines and equipments required, staff nedded, the amount of money required, the source of finance, cost of goods to be produced market demand of product and likely profits anticipated etc, is called a project report. • Need of Project Report:

1. The project report is like a Roadmap. It describes the direction the enterprise is going in, what its goals are, where it wants to be and how it is going to get there. It also enables the entrepreneur to know together he is preceding the right direction.

2. It helps the entrepreneur is getting provisional permanent registration the project from the district industries centre.

3. It helps in allotment of industrial plot or shed for the project from state industrial development corporation.

4. It helps the entrepreneur in obtaining working capital loan or term loan from Banks/State Financial Corporation/other financial institution.

5. It helps in securing supply of scare raw materials required for the product to be manufactured.

6. It help the entrepreneur in establishing techno economic viability of the oroject.

Contents of a project report:
1. Objective and scope of the report.
2. Promotor's profile
3. Location
4. Land and Building
5. Plant and Machinery
6. Production Process
7. Other utilities
8. Raw materials
9. Market Potentials
10. Personal
11. Financial Implications
12. Sources of Finances
13. Financial viability of the project
14. Schedule of project implementation.

• Praliminary Project Report (PPR): A preliminary project report is a brief summary of a project describing the expected inputs and outputs like finance, manpower, machinery, material , technology, expenses, production, sales and profit etc of a project before the project is actually implemented.

• Detailed Project Report (DPR): Detailed project report is a detailed elaboration of each and every information and estimates mentioned in the preliminary project report while preparations a detailed project report (DPR) the entrepreneur may take the help of experts to do the job preparation of DPR requires a lot of time and hence it is a each and every item is necessary in a D.P.R.

• Project Appraisal: The exercise of project appraisal simply means the assessment of a project in term of its economic technical social and financial viability.

• Need of Project Appraisal: Project appraisal is a costs and benefits analysis of different aspects of proposed projects with an objective to adjudge its viability. A project involves employment of scare resources and entrepreneur needs to appraise various alternative projects before allocating the scare resources for the best project. Thus, project appraisal helps selecting the best project among available alternative projects. For appraising a project its economic, financial technical market managerial and social aspect are analyzed.

• Economic Analysis: The economic aspects of appraisal are functional as they logically precede all other aspects. This is so because the bank will not finance a project unless it stands assured that the project represents high priority use of a region's resources. An economic and social analysis looks at the project from the view pint of the whole economy, asking whether the latter will show benefits sufficiently greather than project costs to justify investment in it. i. Increased employment ii. Increased output iii. Increased services iv. Increased Govt. Revenues v. Higher Earning vi. Improves income distribution vii. Higher standard of living viii. Increased national income

2. Financial Analysis: Finance is one of the most important pre-required to establish an enterprise. It is finance only that facilities an entrepreneur to bring together the labour of one machine of another and raw materials of yet another to combine them to produce goods. An entrepreneur conceives the idea of starting a new enterprise with the ultimate objectives of proposed project are required to be analyzed with the help of various financial statements.

Fixed capital normally called fixed assets are those tangible and material facilities which purchased once are used again and again.

i. Financial soundness
ii. Efficient operation
iii. Cost production
iv. Return on Investment
v. Prospects of marketing
vi. Profitability
vii. . Effective controls
viii. Budgeting
ix. Pricing

3. Technical feasibility: While making project appraisal the technical feasibility of the project also needs to be taken into consideration implies to mean the adequacy of the proposed plant and equipment to produce the product within the prescribed norms. As regards know how it denotes the availability of technical knowledge for the proposed assets and machinery. It should be ensured whether that know how is available with the entrepreneur or is to be procured from elsewhere. If the project requires any collaboration then the terms and conditions of the collaboration should also be spelt and comprehensively and carefully. While assessing the technical feasibility of the project, the following aspects need to be covered in the report.

a. Process technology

b. Economic size of the project.
 c. Technical know-how and consultancy

While assessing the technical feasibility of the project, the following aspects need to be covered in the report. a. Process technology
b. Economic size of the project.
 c. Technical know-how and consultancy
. 4. Managing competence: Managing ability or competences play an important role in making an enterprise a success or otherwise strictly speaking in the absence of managing competences, the project which is otherwise feasible may fast. On the contrary even a poor project may become a successful one with good managerial ability. Hence, while doing project appraisal the managing competence or talent of the promoter should be taken into consideration. Research studies report that most of the enterprises fall sick because of lack of managerial competence or mismanagement
. 5. Market/Commercial Analysis: Before the production actually starts the entrepreneur needs to anticipate the possible market for the product. He has to anticipate who will be the possible customers for this product and where and when his product will be sold. This is because production has no value for the producer unless it is sold.

QUESTIONS
1. What is a project report?
2. What details are required for preparation of project report?
3. Write short note on
    I.   Preliminary project report
    II.   Detailed project report
    III. Technical feasibility

4.   **Management Principles**

Management:

Management in all business and organizational activities is the act of getting people together to accomplish desired goals and objectives using available resources efficiently and effectively.

Principles of Management
There are 14 Principles of Management described by Henri Fayol.
1.Division **of Labour**
- Work of all kinds must be divided and allotted to various persons according to their expertise in a particular area.
- Subdivision of work makes it simpler and results in efficiency.
- It also helps in increasing speed, accuracy in performance.
- Specialization leads to efficiency.

2. Party of Authority & Responsibility
- Authority & responsibility are co-existing.

- If authority is given to a person, he should also be made responsible.
- In a same way, if anyone is made responsible for any job, he should also have concerned authority..
- Authority without responsibility leads to irresponsible.

3. Principle of One Boss

- A sub-ordinate should receive orders and be accountable to one and only one boss at a time
- Unity of command provides the enterprise a disciplined, stable & orderly existence.
- It creates harmonious relationship between superiors and sub-ordinates.

4. Unity of Direction

- There should be one plan for a group of activities having similar objectives.
- Related activities should be grouped together.
- Efforts of all the members of the organization should be directed towards common goal.
- Without unity of direction, unity of action cannot be achieved.

5. Equity

- Equity means combination of fairness, kindness & justice.
- The employees should be treated with kindness & equity.
- It implies that managers should be fair and impartial.
- They should give similar treatment to people of similar position.
- They should not discriminate with respect to age, caste, sex, religion, relation etc.

6. Order

- Proper & systematic arrangement of things and people.
- Arrangement of things is called material order and placement of people is called social order.
- Material order- There should be safe and specific place for every article.
- Social order- Selection and appointment of most suitable person on the suitable job.

7. Discipline

- Discipline means sincerity, obedience, respect of authority & observance of rules and regulations of the enterprise .
- Subordinate should respect their superiors and obey their order.
- It is an important requisite for smooth running of the enterprise.
- Discipline is not only required on path of subordinates but also on the part of management.

VIII. Initiative

- Workers should be encouraged to take initiative in the work assigned to them.
- Initiate actions without being asked to do so.
- Management should provide opportunity to its employees to suggest ideas, experiences& new method of work.
- It helps in developing an atmosphere of trust and understanding.
- They can be encouraged with the help of monetary & non-monetary incentives.

IX. Fair Remuneration

- Remuneration to be paid to the workers should be fair & rewarding of the efforts.
- It should accord satisfaction to both employer and the employees.
- Logical & appropriate wage rates.
- Fayol also recommended provision of other benefits such as free education, medical & residential facilities to workers.

X. Stability of Tenure

- Employees should not be moved frequently from one job position to another.
- Employees should be appointed after keeping in view principles of recruitment & selection procedure.
- Time is required for an employee to get used to a new work and to succeed. And if he is removed the time, effort and money spent on training the worker will go waste.
- Stability of job creates team spirit and a sense of belongingness.

XI. SCALAR CHAIN

- Fayol defines scalar chain as The chain of superiors ranging from the ultimate authority to the lowest .
- Every orders, instructions, messages, requests, explanation etc. has to pass through Scalar chain.
- For the sake of convenience & urgency, this path can be cut short and this short cut is known as Gang Plank.
- A Gang Plank is a temporary arrangement between two different points to facilitate quick & easy communication.

XII. Sub-Ordination of Individual Interest to General Interest

- An organization is much bigger than the individual.
- As far as possible, reconciliation should be achieved between individual and group interests.
- But in case of conflict, individual must sacrifice for bigger interests.

XIII. Espirit De'Corps

- It refers to team spirit i.e. harmony in the work groups and mutual understanding among the members.
- Spirit De' Corps inspires workers to work harder.
- Fayol cautioned the managers against dividing the employees into competing groups because it might damage the moral of the workers and interest.
- There should be proper co-ordination of work at all levels
- Efficient employees should be rewarded and those who are not up to the mark should be given a chance to improve their performance.

XIV. Centralization & De-Centralization

- Centralization means concentration of authority at the top level. In other words, centralization is a situation in which top management retains most of the decision making authority.
- Decentralization means disposal of decision making authority to all the levels of the organization. In other words, sharing authority downwards is decentralization. According to Fayol, Degree of centralization or decentralization depends on no. of factors like size of business, experience of superiors, dependability & ability of subordinates etc

Functions of management

Planning

It is the basic function of management.
- What to do, when to do & how to do.
- It bridges the gap from where we are & where we want to be".
- It is an intellectual activity and it also helps in avoiding confusion, uncertainties, risks, wastages etc.

Organizing

It is the process of bringing together physical, financial and human resources and developing productive relationship amongst them achieve goal.

- Identification of activities.
- Classification of grouping of activities.
- Assignment of duties.
- Delegation of authority and creation of responsibility.
- Coordinating authority and responsibility relationships.

Staffing

The main purpose of staffing is to put right man on right job.

- Manpower Planning
- Recruitment, Selection & Placement.
- Training & Development.
- Remuneration.
- Performance Appraisal.
- Promotions & Transfer.

4.Directing

Direction of management which deals directly with influencing, guiding, supervising, motivating subordinate for the achievement of organizational goals. Direction has following elements: Supervision- Implies overseeing the work of subordinates by their superiors. Motivation- Means inspiring, encouraging the sub-ordinates Leadership- Guides the work of subordinates in desired direction. Communications- Is the process of passing information, experience, opinion etc from one person to another.

5. Controlling

The purpose of controlling is to ensure proper progress is being made towards the objectives and goals and to correct any deviation. Therefore controlling has following steps: a. Establishment of standard performance. b. Measurement of actual performance. c. Comparison of actual performance with the standards and finding out deviation if any. d. Corrective action.

Level of Management in an organsation

The three levels of management provide a separation between the managerial positions of the organization. The administrative rank of an organization worker determines the extent of authority, the status enjoyed, and the chain of command that can be controlled by the worker. There are three levels of management found within an organization, where managers at these levels have different roles to perform for the organization to have a smooth performance, and the levels are:

1. Top-Level Management/ Administrative level
2. Middle-Level Management/ Executory
3. Low-level Management/ Supervisory

The levels of Management and Their Functions are Discussed Below:

1. Top Level Management

Top-Level Management is also referred to as the administrative level. They coordinate services and are keen on planning. The top-level management is made up of the Board of Directors, the Chief Executive Officer (CEO), the Chief Financial Officer (CFO) and the Chief Operating Officer (COO) or the President and the Vice President. The Top-level management controls the management of goals and policies and the ultimate source of authority of the organization. They apply control and coordination of all the activities of the firm as they organize the several departments of the enterprise which would include their budget, techniques, and agendas. Top-level management is accountable to the shareholders for the performance of the organization. There are several functions

performed by the top-level management, but three of them are the most important, and they are: To lay down the policies and objective of the organization Strategizing the plans of the enterprise and aligning competent managers to the departments or middle level to carry them out. Keeping the communication between the enterprise and the outside world

2. Middle Level of Management

READ Features of Organizations 9/25/2020 Three Levels of Management and Their Functions - Management Study HQ There are three levels of management found,Managem… Middle-level Management is also referred to as the executory level, they are subordinates of the top-level management and are responsible for the organization and direction of the low-level management. They account for the toplevel management for the activities of their departments. The middle-level managers are semi- executives and are made up of the departmental managers and branch manager. They could be divided into senior and junior middle-level management if the organization is big. They coordinate the responsibilities of the sub-unit of the firm and access the efficiency of lower-level managers. The middle-level managers are in charge of the employment and training of the lower levels. They are also the communicators between the top level and the lower level as they transfer information, reports, and other data of the enterprise to the top-level. Apart from these, there are three primary functions of the middle-level management in the organization briefed below: To carry out the plans of the organization according to policies and directives laid down by the top-level management. To organize the division or departmental activities. To be an inspiration or create motivation for junior managers to improve their efficiency.

3. Lower Level of Management

The lower level Management is also referred to as the supervisory or the operative level of managers. They oversee and direct the operative employees. They spend most of their time addressing the functions of the firm, as instructed by the managers above them. The lower-level managers are the first line of managers as they feature at the base of operations, so they are essential personnel that communicates the fundamental found,Managem… problems of the firm to the higher levels. This management level is made up of the foreman, the line boss, the shift boss, the section chief, the head nurse, superintendents, and sergeants. They are the intermediary, they solve issues amidst the workers and are responsible for the maintenance of appropriate relationships within the organization. They are also responsible for training, supervising, and directing the operative employees. The lower level managers represent the management to the operative workers as they ensure discipline and efficiency in the organization. The duty of inspiration and encouragement falls to them, as they strengthened the workforce. They also organize the essential machines, tools, and other materials required by the employees to get their job done.

QUESTIONS

1. Define the term Management.
2. Explain in details about principles of management.
3. What are the functions of management?
4. Briefly explain about levels of management in an organization?

Functional areas of management:
a)Fuctions of production are

- Selection of product and designing.
- Selection of production process.
- Estimation of right production capacity.
- Production planning.
- Production control.
- Quality and cost control.
- Inventory Control.
- Maintenance and replacement of Machines.

Productivity:

Productivity is **a measure of economic performance that compares the amount of goods and services produced (output) with the amount of inputs used to produce those goods and services**.

Quality Control:

Quality control (QC) is a procedure or set of procedures intended to ensure that a manufactured product or performed service adheres to a defined set of quality criteria or meets the requirements of the client or customer.

Production planning and control:

Production planning and control manages and schedules the allocation of human resources, raw materials, work centers, machinery, and production processes. It finds the most efficient way to produce finished goods with the lead times needed to meet production demand.

b)Inventory management:

Inventory management helps companies identify which and how much stock to order at what time. It tracks inventory from purchase to the sale of goods. The practice identifies and responds to trends to ensure there's always enough stock to fulfill customer orders and proper warning of a shorta

Models/Techniques of inventory management:

Three of the most popular inventory control models are Economic Order Quantity (EOQ), Inventory Production Quantity, and ABC Analysis. Each inventory model has a different approach to help you know how much inventory you should have in stock. Which one you decide to use depends on your business.

c)Financial management:

Functions of fianancial management:

- Financial Planning and Forecasting. As a part of financial management function, financial managers have to do financial planning.
- Cash Management.
- Estimating Capital Expenses.
- Determining Capital Structure.
- Procurement of Funds.
- Investment of Funds.

- Surplus Disposal.

Management of working capital:

Working capital management ensures the best utilisation of a business's current assets and liabilities for the company's effective operation. The main aim of managing working capital is to monitor a company's assets and liabilities to maintain adequate cash flow and meet short-term business goals.

Costing:

Cost management is the process of planning and controlling the costs associated with running a business. It includes collecting, analyzing and reporting cost information to more effectively budget, forecast and monitor costs.

Break even analysis:

A break-even analysis is a financial calculation that weighs the costs of a new business, service or product against the unit sell price to determine the point at which you will break even. In other words, it reveals the point at which you will have sold enough units to cover all of your costs.

Book keeping:

Bookkeeping is the process of recording your company's financial transactions into organized accounts on a daily basis. It can also refer to the different recording techniques businesses can use. Bookkeeping is an essential part of your accounting process for a few reasons.

Journl Entry:

Journal entries record all transactions for a business. Transactions are broadly defined as any financial activity that impacts the business. They are not limited to the buying and selling of goods and services, but include any exchange of monetary value, such as interest payments, depreciation, expenses, or payroll.

Petty Cash:

A petty cash fund is a small amount of company cash, often kept on hand (e.g., in a locked drawer or box), to pay for minor or incidental expenses, such as office supplies or employee reimbursements.

P & L account:

The purpose of the profit and loss statement, also referred to as the P&L, is to show you, and any investors, whether your small business is profitable. A profit and loss statement displays the company's revenue and expenses, which, when combined, result in the net income.

Balance Sheet:

A balance sheet gives you a snapshot of your company's financial position at a given point in time. Along with an income statement and a cash flow statement, a balance sheet can help business owners evaluate their company's financial standing.

d) Marketing Management

Marketing management is especially important for smaller businesses because **it gives thema level footing to compete with larger players in the field**. Thorough customer research, creative campaigns and marketing strategies and positive branding can go a long way in providing a brand with an edge over its competition.

Marketing techniques:

1) Specialization and niche targeting.

2) Visible Experts.

3) Blogs and articles.

4) A lead-generating website.

5) Search engine optimization (SEO)

6) Speaking engagements.

7) Trade show participation.

4Ps:

Four Ps of marketing is a marketing concept that summarizes the four key factors of anymarketing strategy. The four Ps are: **product, price, place, and promotion**

e) Human Resource Management: Functions of personnel Management:

- Recruiting.
- Hiring.
- Determining wages and salaries.
- Administering benefits.
- Providing employee incentives.
- New employee orientation.
- Training and development.
- Performance appraisals.

QUESTIONS

1. What are the functions of production management?

2. Define Productivity?

3. What are the techniques used for Inventory management?

4. What are the 4Ps of marketing management?

5. State the functions of human resource management?

6. Define Break Even Analysis?

Leadership:

Leadership means **creating and planning, securing resources, and looking out for and improving errors**. Leadership is about motivating people to work together and cooperatewith themselves and in some cases, other teams, to achieve a certain goal.

Importance of leadership:

Leadership is important because **it inspires, motivates, and sets an example for people to accomplish positive changes in the world**. Leaders establish a vision, provide a plan of action, and build strong relationships with their followers. As a result, they guide people to accomplish incredible feats together.

Qualities of leader:

- Integrity.
- Delegation.
- Communication.
- Self-Awareness.
- Gratitude.
- Learning Agility.
- Influence.
- Empathy.

Functiopns of leader

- Setting goals. A leader's most important function is to set goals for team members to encourage them to work confidently and enthusiastically.
- Organising.
- Take initiatives.
- Cooperation among employees.
- Motivation and direction.
- Liaison between workers and management.
- Policy making.

Leader vs manager:

Basis	Manager	Leader
Origin	A person becomes a manager by virtue of his position.	A person becomes a leader on basis of his personal qualities.
Formal Rights	Manager has got formal rights in an organization because of his status.	Rights are not available to a leader.
Followers	The subordinates are the followers of managers.	The group of employees whom the leaders leads are his followers.
Functions	A manager performs all five functions of management.	Leader influences people to work willingly for group objectives.
Necessity	A manager is very essential to a concern.	A leader is required to create cordial relation between person working in and for organization.
Stability	It is more stable.	Leadership is temporary.
Mutual Relationship	All managers are leaders.	All leaders are not managers.
Accountability	Manager is accountable for self and subordinates behaviour and performance.	Leaders have no well defined accountability.

Concern	A manager's concern is organizational goals.	A leader's concern is group goals and member's satisfaction.
Followers	People follow manager by virtue of job description.	People follow them on voluntary basis.
Role continuation	A manager can continue in office till he performs his duties satisfactorily in congruence with organizational goals.	A leader can maintain his position only through day to day wishes of followers.

Motivation:

Motivation is **the process that initiates, guides, and maintains goal-oriented behaviors**. It is what helps you lose extra weight, for instance, or pushes you to get that promotion at work. In short, motivation causes you to act in a way that gets you closer to your goals.

Characterstics:

Alignment of aims, purpose and values between staff, teams and organization is the most fundamental aspect of motivation. The better the alignment and personal association with organizational aims, the better the platform for motivation.

Factors affecting motivation:

- Leadership style.
- Recognition and appreciation.
- Meaning and purpose.
- Positive company culture.
- Professional development opportunities.
- Job advancement opportunities.
- Financial benefits.
- Flexible work schedules.

Maslow's hierarchy Needs(pyramid):

There are five levels in Maslow's pyramid. From the bottom of the hierarchy upwards, theneeds are: **physiological (food and clothing), safety (job security), love and belonging needs (friendship), esteem, and self-actualization**.
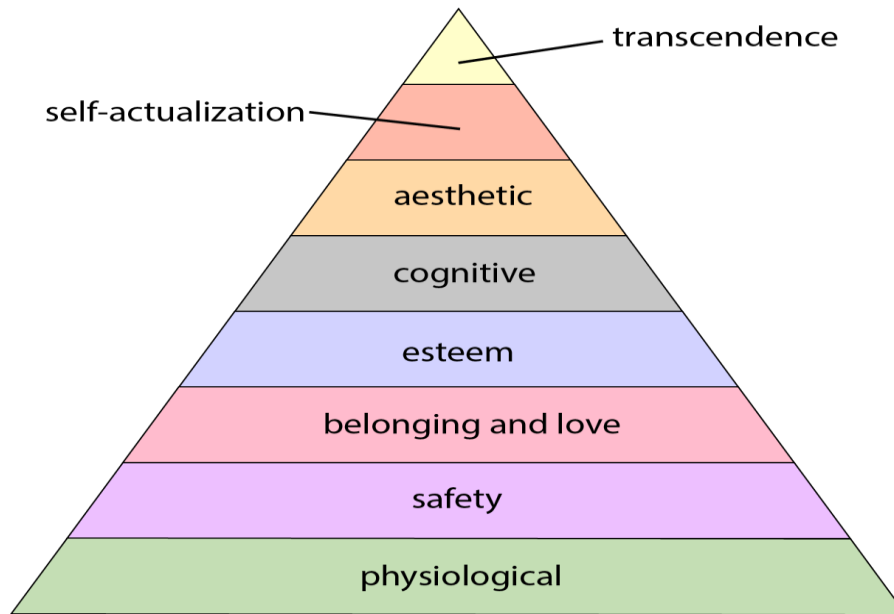
b) <u>Methods of motivation:</u>

- Make your business a pleasant .
- Be a respectful, honest and supportive manager.
- Offer employee rewards.
- Give them room to grow.
- Share positive feedback.
- Be transparent.
- Offer flexible scheduling.
- Offer food in the workplace.

Importance of Communication in organisation.

- The Basis of Co-ordination.
- Fluent Working.
- The Basis of Decision Making.
- Increases Managerial Efficiency.
- Increases Cooperation and Organizational Peace.
- Boosts Morale of the Employees.

Types of communication:

- Formal communication Being formal, clear and specific is a great way to ensure a proper flow of information in the workplace.
- Informal communication.
- Horizontal communication.
- Vertical communication.

Barriers to communication:

- Dissatisfaction or Disinterest With One's Job.

- Inability to Listen to Others.
- Lack of Transparency & Trust.
- Communication Styles (when they differ)
- Conflicts in the Workplace. ...
- Cultural Differences & Language.

QUESTIONS

1. What is leadership?

2. Explain the qualities and functions of a leader?
3. Differentiate between Leader and manager?
4. What is Motivation?
5. Briefly explain
    I. The types of communication
    II. Barriers to communication

Human relationship and performance in organization:

Human relations refer to the ability to interact in a healthy manner with others and build strong relationships. From the perspective of managers in a company, it involves the process of creating systems and communication channels to enable group employee relationships as well as strong one-on-one relationships. Additionally, it includes the process of training people for specific roles, addressing their needs, resolving conflicts between employees or between management and employees and creating a positive workplace culture.

By developing key human relations skills, managers and employees both can build and maintain stronger relationships within a workforce and maintain a positive work environment. Here are the most critical human relations skills:

The results showed that superior-subordinate relationship have **positive relation with job satisfaction and organization commitment**. The study concluded by showing that strong leader-member exchange led to employee job satisfaction, which may result in improved organizational commitment.

Maintain discipline among employees. Maintain good work performance by accepting only quality work from subordinates. Build cooperation and morale among other employees. Be loyal to management and to the company.

Relationship among employee may be of following type:

- Direct Single Relationships:
- Direct Group Relationships:
- Cross-Relationships:

Total Quality Management

Total Quality management is defined as a continuous effort by the management as wellas employees of a particular organization to ensure long term customer loyalty and customer satisfaction. Remember, one happy and satisfied customer brings ten new customers along with him whereas one disappointed individual will spread bad word of mouth and spoil several of your existing as well as potential customers.

You need to give something extra to your customers to expect loyalty in return. **Quality canbe measured in terms of durability, reliability, usage and so on**.

There are five major steps to TQM, and each are essential to successful implementation.

- Commitment and Understanding from Employees.
- Quality Improvement Culture.
- Continuous Improvement in Process.
- Focus on Customer Requirements.
- Effective Control.

Accidents:

1. Taking shortcuts
2. Being overconfident

3. Starting a task with Incomplete instructions

4. Poor housekeeping
5. Ignoring procedures
6. Mental distractions from work
7. Failure to pre-plan work

The following steps may be taken to prevent industrial accidents:

* Proper Safety Measures

 * Proper Selection

 * Safety Conscious

* Enforcement of Discipline

* Incentive

 * Safety Committees

* Proper maintenance of Machines and Equipment

* <u>Safety Training etc.</u>

<u>GENERAL SAFETY RULES</u>

A. Introduction:

   1. The following general safety rules are to be followed by all applicable departments.

B. Responsibilities:

   1. Environmental Health and Safety Department:
      a. Develop and implement all general safety rules to be followed by employees.

      b. Evaluate the general safety rules annually or as needed to ensure they are up to date.

      c. Be a resource for any employee with questions on general safety rules.

      d. Create and implement training programs for general safety rules.

   2. Manager's, Supervisor's, and Department Heads:

      a. Abide by all general safety rules set forth in the Safety Manual.

      b. Ensure all employees are attending training sessions as required.

      c. Guarantee that employees are following all general safety rules and provide disciplinary action when necessary if they are not.

   3. Employees:

      a. Abide by all general safety rules set forth in the Safety Manual.

          b. Attend any training sessions online or in the classroom as required by your supervisor or the EHS department.

          c. Provide any additional items that may need to be addressed under general safety rules.

C.   Policies, Practices, and Procedures:

  1. General Safety Rules:

     a. Observe and abide all warning signs and safety notices.

     b. Smoking is prohibited everywhere on UNE property. UNE is now a smoke-free facility.

     c. No running or horseplay is allowed in the workplace.

     d. Guard all floor openings and edge of working platforms greater than 3 feet to prevent a fall.

     e. Do not take unsafe short cuts or by-pass any safety devices or controls.

     f. Never throw debris, tools or other objects from any height.

     g. Report all unsafe acts, unsafe conditions or incidents/accidents to your supervisors and safety personnel.

     h. Refer to Safety Data Sheet (SDS) before handling any hazardous materials.

     i. For any hazardous work, safe operating procedure/safe work procedures must be referred and adhered to.

     j. Label all chemical containers and store them in designated chemical storage area.

     k. Do not deface or change any manufacturer's labels on incoming chemicals or industrial products.

     l. Know the location of the nearest emergency equipment including fire extinguishers, fire alarms, fire blankets, eye wash stations, safety showers, AED's, etc.

     m. Familiarize yourself with the emergency evacuation procedure and the location of the nearest exits and muster points. A list of muster point locations for each building can be found in UNE's Annual Clery Report on the Safety and Security portion of the UNE website.

     n. Do not obstruct or block any exits, escape passage way or any emergency response equipment.

     o. Follow the manufacturer's operating instructions when working with machinery.

p. Report all work related illness and injuries right away and submit an accident report.

2. General Tool and Device Safety:

The employer is responsible for the safe condition of tools and equipment used by employeesbut the employees have the responsibility for properly using and maintaining tools.

a. All electrical tools will be in good repair.

b. Saw blades, knives, or other tools should be directed away from aisle areas and other employees working in close proximity. Knives and scissors must be sharp. Dull tools can be more hazardous than sharp ones.

c. Appropriate personal protective equipment, e.g., safety goggles, gloves, etc., should be worndue to hazards that may be encountered while using portable power tools and hand tools.

d. Floors should be kept as clean and dry as possible to prevent accidental slips with or arounddangerous hand tools.

e. Keep all tools in good condition with regular maintenance.

PPE means personal protective equipment or equipment you use to guarantee your (own) safety.

Use PPE always and anywhere where necessary. Observe the instructions for use, maintain them well and checkregularly if they still offer sufficient protection. But when do you use what type of protection?

These 7 tips will help you on your way.

1. SAFETY FOR THE HEAD

Wearing a **helmet** offers protection and can prevent head injuries. Select a sturdy helmet that is adapted to the
working conditions. These days you can find many elegant designs and you can choose extra options such as anadjustable interior harness and comfortable sweatbands.

2. PROTECT YOUR EYES
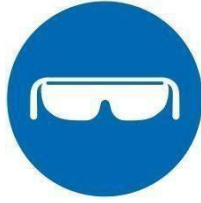
The eyes are the most complex and fragile parts of our body. Each day, more than 600 people worldwide sustain eye injuries during their work. Thanks to a good pair of **safety glasses**, these injuries could be prevented. Do youcome into contact with bright light or infrared radiation? Then **welding goggles or a shield** offer the ideal protection!

3. <u>HEARING PROTECTION</u>

Do you work in an environment with high sound levels? In that case it is very important to consider hearing protection. **Earplugs** are very comfortable, but earmuffs are convenient on the work floor as you can quickly putthese on or take them off.

4. <u>MAINTAIN A GOOD RESPIRATION</u>

Wearing a **mask** at work is no luxury, definitely not when coming into contact with hazardous materials. 15% ofthe employees within the EU inhale vapours, smoke, powder or dusk while performing their job. **Dust masks** offer protection against fine dust and other dangerous particles. If the materials are truly toxic, use a **full-face mask**. This adheres tightly to the face, to protect the nose and mouth against harmful pollution.

5. <u>PROTECT YOUR HANDS WITH THE RIGHT GLOVES</u>

Hands and fingers are often injured, so it is vital to protect them properly. Depending on

the sector you work in,you can choose from gloves for **different applications**:

- protection against vibrations
- protection against cuts
- by sharp materials
  protection against
  cold or heat
- protection against bacteriological risks
- protection against splashes from diluted chemicals.

6. <u>PROTECTION FOR THE FEET</u>

Even your feet need solid protection. **Safety shoes** (type Sb, S1, S2 or S3) **and boots** (type S4 or S5) are the ideal solution to protect the feet against heavy weights. An **antiskid sole** is useful when working in a damp environment, definitely if you know that 16,2% of all industrial accidents are caused by tripping or sliding. On slippery surfaces, such as snow and ice, **shoe claws** are recommended. Special socks can provide extra comfort.

7. <u>WEAR THE CORRECT WORK CLOTHING</u>

Preventing accidents is crucial in a crowded workshop. That is why a good visibility at work is a must: a **high- visibility jacket and pants made of a strong fabric** can help prevent accidents. Just like the hand protection,there are versions for different applications.

<u>WORST-CASE SCENARIO</u>

Prevention is better than cure. A smart thing is to be prepared for the worst. A **classic first-aid kit** is no luxurybut a **first-aid kit for the eyes** can also be an essential first aid. If the employee comes into contact with chemicals, a **safety shower** is mandatory, so that he can rinse the substances off his body at any moment.

PREVENTING ACCIDENTS: PICTOGRAMS

Not only is preparing your workshop for accidents a smart thing to do, it is even smarter to organise your
workshop in such a way that no serious accidents can take place. A simple way to make your workshop safer is to use pictograms: indicating flammable materials, the necessary

use of hearing protection, indicating emergency exits …

You can find all pictograms in the 'Labels & decals' catalogue under 'Safety'.

<u>QUESTIONS</u>

1. What is Total quality management?

2. Explain the general safety rules for accidents ?

8. LEGISLATION

Intellectual property right (IPR)

Intellectual Property Rights in general refers to the set of intangible assets including invention, creation, and contribution to the contemporaneous field of knowledge which is owned and legally protected by an individual or company from the outside use or implementation without approved consent.

Types of Intellectual Property

- Patents
- Trademarks
- Copyrights

Patents:

Patent is an intellectual property right, to make, use or sell an invention for a limited period of 20 years. A Patent is granted to an entity on disclosing a technical invention, which may be a product (system/apparatus/device) or process (method/manufacturing) or both, which is novel or new and demonstrate inventive step or technological advancement over the existing technologies.

Trademarks:

A trademark is a sign capable of distinguishing the goods or services of one enterprise from those of other enterprises. Trademarks are protected by intellectual property rights

In principle, a trademark registration will confer an exclusive right to the use of the registered trademark. This implies that the trademark can be exclusively used by its owner, or licensed to another party for use in return for payment. Registration provides legal certainty and reinforces the position of the right holder, for example, in case of litigation.

Copyrights:

The word copyright is a mixture of two words – 'copy' and 'right'. To be more precise copyright means 'right to copy', wherein only the creator or his authorised person has a right to reproduce a work. In simple words, a legal right which is possessed by the owner of Intellectual property is a copyright.

Salient Features of the Factories Act, 1948

1. **OBJECTIVE**

The main objective of the Act is to ensure adequate safety measures and promote health and welfare of the workers employed in factories as well as to prevent haphazard growth of factories. The Factories Act is meant to provide protection to the workers from being exploited and also provides for improvement of the working conditions within the factory premises.

2.   COVERAGE OF THE ACT

The coverage of the Act is confined to the: -
- ❑ factories using power and employing 10 or more workers on any working day of the preceding twelve months;
- ❑ factories not using power and employing 20 or more workers on any working day of the preceding twelve months; and the
- ❑ factories specially notified under Section 85 of the Factories Act by the State Govts./Union Territories.

3.   IMPORTANT CONCEPTS AND DEFINITIONS

(a) Factory means any premises including the precincts thereof:-

i) wherein ten or more workers are working, or were working on any day of the preceding twelve months and in any part of which a manufacturing process is being carried on with the aid of power, or is ordinarily so carried on, or

ii)   wherein twenty or more workers are working or were working on any day of the preceding twelve months, and in any part of which a manufacturing process is being carried on without the aid of power, or is ordinarily so carried on.

(b) Worker means a person (employed directly or through any agency including a contractor) with or without the knowledge of the principal employer, whether for remuneration or not in any manufacturing process, or in cleaning any part of the machinery or premises used for a manufacturing process, or in any kind of work incidental to or connected with the manufacturing process, or the subject of manufacturing process (but does not include any member of the armed forces of the Union).

(c)   Manufacturing process means any process for: -

i) Making, altering, repairing, ornamenting, finishing, packing, oiling, washing, cleaning, breaking up, demolishing or otherwise treating or adapting any article or substance with a view to its use, sale, transport, delivery or disposal or

ii) Pumping oil, water, sewage or any other substance or

iii) Generating, transforming or transmitting power or

iv)   Composing types for printing, printing by letter press, lithography, photogravure of other similar process or book binding,

v) Constructing, reconstructing, repairing, refitting, finishing or breaking up ships or vessels,

vi) Preserving or storing any article in cold storage.

(d)   Hazardous Process means any process or activity in relation to an industry

specified in the First Schedule where, unless special care is taken, raw materials used therein or the intermediate or finished products, by- products, wastes or effluents thereof would cause material impairment of the health of the persons engaged in or connected therewith, or results in the pollution of the general environment, provided that the State Government may, by notification in the Official Gazette, amend the First Schedule by way of addition, omission or variation of any industry specified in the said Schedule.

(e) **Fatal Injury** means injury resulting from industrial accident which caused death to the worker.

(f) **Non-Fatal** Injury means injury resulting from industrial accident, which prevented injured worker from attending to work for a period of 48 hours or more immediately following the injuries.

(g) **Adult** means a person who has completed his eighteenth year of age.

(h) **Adolescent** means a person who has completed his fifteenth year of age but has not completed his eighteenth year.

(i) **Child** means a person who has not completed his fifteenth year of age.

(j) **Average Daily Number of Workers Employed** The average daily number of workers employed is calculated by dividing the aggregate number of attendances on all the working days (that is, mandays worked) by the number of working days in the year. Attendance on separate shifts (e.g. night and day shifts) should be counted separately. Days on which the factory was closed for whatever cause and days on which the manufacturing process was

not carried on should not be treated as working days. Partial attendance for less than half a shift on a working day should be ignored, while attendance for half a shift or more on such day should be treated as full attendance.

(k) **Mandays Worked** Mandays Worked means total number of attendances during a calendar year.

(l) **Average Number of Hours Worked per week** The Average number of hours worked per week means the total actual hours worked by all workers during the year excluding the rest intervals but including overtime worked divided by the product of average number of workers employed daily in the factory and 52 weeks or Average number of hours worked per week = total number of man hours worked/number of weeks in which production process has taken place.* average number of workers employed daily. In case the factory has not worked for the whole year, the number of weeks during which the factory worked should be used in place of 52

THE PAYMENT OF WAGES ACT, 1936

(m)   [ACT 4 OF 1936]
(n) [23rd April, 1936]

(o) An Act to regulate the payment of wages of certain classes of 1[employed persons].

(p) Whereas it is expedient to regulate the payment of wages to certain classes of [employed persons];

(q) It is hereby enacted as follows:-

(r) Short title, extent, commencement and application.- (1) This Act may be called the Payment of Wages Act, 1936.

(s) 2[(2) It extends to the whole of India 3[***].]

(t) It shall come into force on such date 4as the Central Government may, by notification in the Official Gazette, appoint.

(u) It applies in the first instance to the payment of wages to persons employed in any 5[factory, to persons] employed (otherwise than in a factory) upon any railway by a railway administration or, either directly or through a sub-contractor, by a person fulfilling a contract with a railway administration, 6[and to persons employed in an industrial or other establishment specified in sub-clauses (a) to (g) of clause (ii) of Section 2].

(v) The 7[Appropriate Government] may, after giving three months' notice of its intention of so doing, by notification in the Official Gazette, extend the provisions of 8[this Act or any of them to the payment of wages to any class of persons employed in 9[any establishment or class of establishments specified by the Central Government or a State Government under sub-clause (h) of clause (ii) of section 2.]

Responsibility for payment of wages.- Every employer shall be responsible for the payment of all wages required to be paid under this Act to persons employed by him and in case of persons employed,--

(a) in factories, if a person has been named as the manager of the factory under [clause (f) of sub-section (1) of Section 7 of the Factories Act, 1948 (63 of 1948)];

[(b) in industrial or other establishments, if there is a person responsible to the employer for the supervision and control of the industrial or other establishments;]

(c) upon railways (otherwise than in factories) if the employer is the railway administration and the railway administration has nominated a person in this behalf for the local area concerned.

(d) in the case of contractor, a person designated by such contractor who is directly under his charge; and

(e) in any other case, a person designated by the employer, the person so nominated or the person so designated, as the case may be, shall be responsible for such payment.

(1) Notwithstanding anything contained in sub-section (1), it shall be the responsibility of the employer to make payment of all wages required to be made under this Act in case the contractor or the person designated by the employer fails to make such payment.]

4. Fixation of wage-periods.- Every person responsible for the payment of wages under Section 3 shall fix periods (in this Act referred to as wage-periods) in respect of which such wages shall be payable.

(2) No wage-period shall exceed one month.

5. Time of payment of wages.- (1) The wages of every person employed

upon or in -

(a) any railway, factory or [4][industrial or other establishment] upon or in which less than one thousand persons are employed, shall be paid before the expiry of the seventh day.

6. Deductions for absence from duty.- (1) Deductions may be made under clause (b) of sub-section (2) of Section 7 only on account of the absence of an employed person from the place or places where, by the terms of his employment, he is required to work, such absence being for the whole or any part of the periodduring which he is so required to work.

(2) The amount of such deduction shall in no case bear to the wages payable to the employed person in respect of the wage-period for which the deduction is made a larger proportion than the period for which he was absent bears to the total period, within such wage-period, during which by the terms of his employment, he was required to work:

Provided that, subject to any rules made in this behalf by [5][the appropriate Government], if ten or more employed persons acting in concert, absent themselves without due notice (that is to say without giving the notice which is required under the terms of their contracts of employment) and without reasonable cause, such deduction from any such person may include such

QUESTIONS

1. Define IPR?
2. Write short note on
   I. Patent
   II. Trademark
   III. Copyright
3. What are the features of payment of Wages act 1936?

Introduction to Components of IoT

With the increasing bandwidth availability and cheap hardware cost, IoT is changing the industry landscapes in scale. From medical, construction, governance to Insurance, BFS the use cases of IoT are spreading day by day. The IT spending of multinational corporations, banks, and other institutesare also increasing along with all these developments. Though the hardware, software and infrastructure requirements of Industry grade IoT differ from domain to domain, the basic components remain the same. In this article, we will discuss the standard components of an IoT

system.

Key Components of IoT

We can list down the below components as key parts of an IoT ecosystem.

- Sensors or End Devices
- Network or Connectivity Layer
- Security Layers
- Compute Engines
- Technology and Governance Standards
- Intelligent Insights and Actions

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1. Sensors or End Devices

For any IoT use case, the components of the endpoint are sensors. Sensors capture electric pulse oranalog signals which are passed through the IoT ecosystems. Based on the use case and domains RFID, temperature sensors, light sensors, electromagnetic sensors, etc. are

used. For example,

smartphones and smart wearables are equipped with sensors like accelerometer, Gyroscope

sensors, etc. Data obtained from these IoT endpoints can be used in various domains

like Humanactivity recognition, medical stability, etc. Based on the use case and

precision requirements

sensors can be chosen keeping the following parameters in mind

- Accuracy of the input
- readings Reliability

    percentage of the inputs

- The purpose of the use case, for example, sensors required for a temperature-

    dependentuse case, will differ from use cases based on motions.

- Industry grade IoT systems can be integrated with multi-technology, cross-
    functional and cross-vendor products. Based on the complexity and compatibility
    sensors are chosen fora particular use case.

2. Network or Connectivity Layer

In a typical IoT ecosystem, sensors are connected with computation layers and intelligent

layers vianetwork or connectivity layers. IoT endpoints need to be always connected with

various other components seamlessly over the connectivity layer. Based on the scale of

the implementations IoT components can be connected overs LANs, MANs or WANs. It

can also be connected through

telephony networks like LTE (Long Term Evolution or popularly known as 4G

Network) or light-based technologies like Li-Fi (where light is used as a mode of

communication to maintain interconnections). For local use cases, Bluetooth and Wi-

Fi can also be used.

An IoT network consists of various network components like routers, gateways, switches,
various

network protocols, etc. Based on the use case and domain proper network infrastructure is

neededto be chosen.

3. Security Layers

The heart of any industry-grade IoT user story is 'data'. In a standard use case, analog or

digital signal is acquired by sensors and the signal is then converted to a format on top of which AI/ML

components can work. In the total flow of data, proper security systems and methodologies needto be enforced. The data can be compromised in any layers starting from the data acquisition to

business insights derivations. We can enforce proper security by using strong encryption in variouslayers of communication, using proper firmware and anti-malware systems, etc.

- **Intelligence:** This will be essential for smart product

    - **Connectivity:** This feature will be responsible for network accessibility and compatibilityfeatures of the devices and hence one of the prime characteristics.

    - **Sensing:** Like collecting the information basing the retrieval capacity and providing it forintelligent decision

    - **Expressing:** This will enable interactivity with humans and the world.

    - **Energy:** Without this, there will not be the creation of our devices. Energy harvesting andproper infrastructure to charge and all will be important features for our IoT devices

    - **Safety:** The prime feature on which the customer rely and use the product. Hence no compromise on this is allowed and all the details need to be checked and validated. Forsome feature, if you see there are even government regulations as well.

Benefits of IoT Technology

There are many benefits of IoT technology which is a niche technology and some of them are listedbelow:

1. Business Opportunity: There will be enhanced opportunities for the business with IoT and new revenue tracks can be easily developed. IoT driven products and innovations will be an asset to theorganization.

2. Enhanced and Efficient Asset Utilisation: with IoT, there will be improved tracking of the assetsand efficient products can be achieved on a real-time basis.

3. Enhance Device Communication: Establishing the interaction between machines will be boomand IoT will one step further in this process.

4. Automation and Control: IoT with its device interaction and connectivity strives towards achieving automation with minimal or no human intervention since machines can lead to a fasterand timely output.

5. Time-Saving and Efficiency: IoT which uses the machine to machine interaction will help us toutilize human time efficiently and also the work gets faster results which will define its efficiency.

The IoT is a complex system with a number of characteristics. Its characteristics vary from one
domain to another. Some of the general and key characteristics identified during the research studyare as follows:

1. <u>Intelligence</u>

IoT comes with the combination of algorithms and computation, software & hardware that makes itsmart. Ambient intelligence in IoT enhances its capabilities which facilitate the things to respond in an intelligent way to a particular situation and supports them in carrying out specific tasks. In spite of all the popularity of smart technologies, intelligence in IoT is only concerned as means of interaction between devices, while user and device interaction is achieved by standard input
methods and graphical user interface.

2. <u>Connectivity</u>

Connectivity empowers Internet of Things by bringing together everyday objects. Connectivity ofthese objects is pivotal because simple object level interactions contribute towards collective
intelligence in IoT network. It enables network accessibility and compatibility in the things. With thisconnectivity, new market opportunities for Internet of things can be created by the networking of
smart things and applications.

3. <u>Dynamic Nature</u>

The primary activity of Internet of Things is to collect data from its environment, this is achieved with the dynamic changes that take place around the devices. The state of these devices changedynamically, example sleeping and waking up, connected and/or disconnected as well as the
context of devices including temperature, location and speed. In addition to the state of the device,the number of devices also changes dynamically with a person, place and time.

4. <u>Enormous scale</u>

The number of devices that need to be managed and that communicate with each other will bemuch larger than the devices connected to the current Internet. The management of data

generated from these devices and their interpretation for application purposes becomes more critical. Gartner (2015) confirms the enormous scale of IoT in the estimated reportwhere it stated that 5.5 million new things will get connected every day and 6.4 billion connected things will be inuse worldwide in 2016, which is up by 30 percent from 2015. The report also forecasts that the

number of connected devices will reach 20.8 billion by 2020.

5. <u>Sensing</u>

IoT wouldn't be possible without sensors which will detect or measure any changes in the environment to generate data that can report on their status or even interact with theenvironment.

Sensing technologies provide the means to create capabilities that reflect a true awareness of the physical world and the people in it. The sensing information is simply the analogue input from thephysical world, but it can provide the rich understanding of our complex world.

QUESTIONS

1. Define IOT?
2. Briefly explain the component of Internet of things?
3. What are the applications of IOT in day to day life?

GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING
GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

Estimating & Cost Evaluation-II

For 5TH Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)
Prepared By
Snehasikta Tripathy
(Lecturer in Civil Engineering)

COURSE OUTCOMES
On completion of the course, students will be able to
1. Create detailed estimate of culverts and bridges
2. Prepare estimates of irrigation structures
3. Prepare estimates of a macadam road and a national highway in cutting and filling
4. Prepare detailed estimates for septic tank and soak pits
5. Prepare detailed estimates of miscellaneous works
6. Comprehend the management practices in Public Works Department

C. **TOPIC WISE DISTRIBUTION OF PERIODS**

Chapter	Name of topics	Hours
1.	Detailed estimate of culverts and bridges	12
2.	Estimate of irrigation structures	14
3.	Detailed estimate of roads	12
4.	Detailed estimates of miscellaneous works	12
5.	PWD accounts works	10
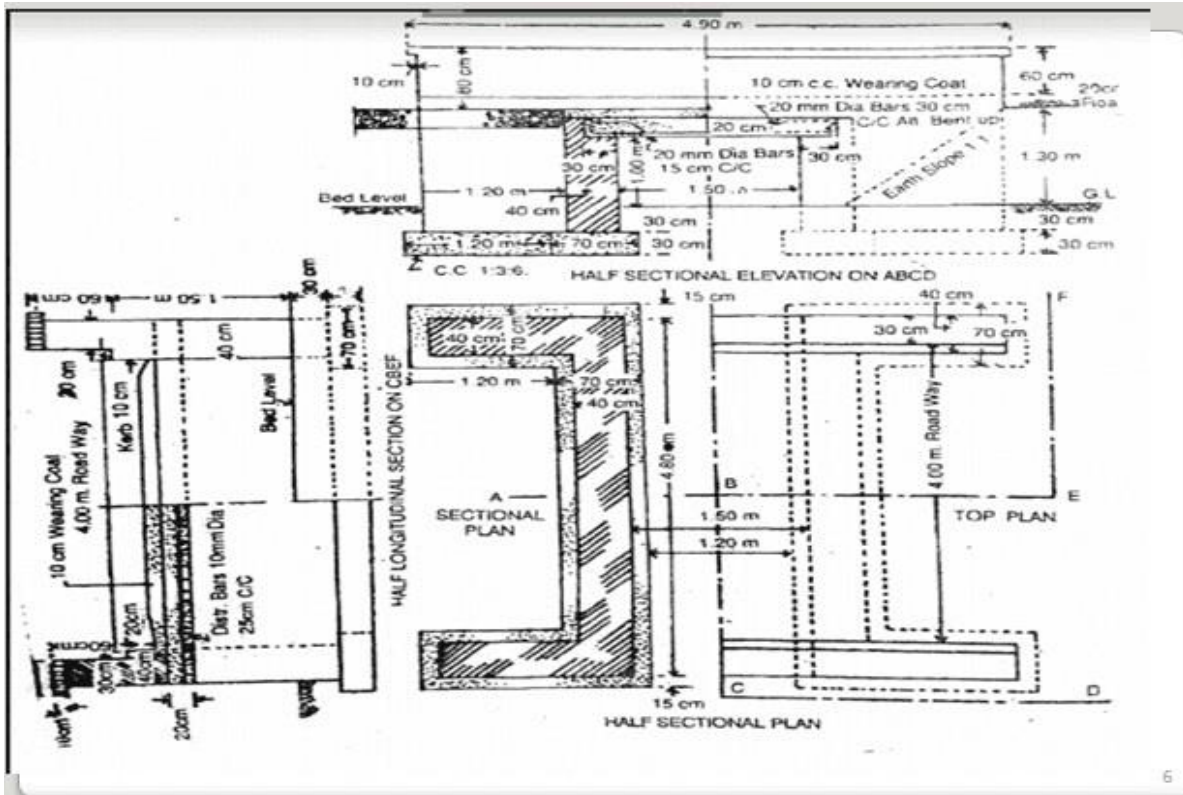
RCC SLAB CULVERT

Details of Measurement and Calculation of Quantities (Ex. 1, Page No-376-377 B.N. DUTTA)

Item No.	Particulars of items of works	No.	Length M	Breadth M	Height Or Depth m	Quantity	Explanatory notes
1.	**Earthwork in excavation in** foundation –						
	Abutments …	2	5.10	0.70	0.60	4.28	
	Wings Walls …	4	1.20	0.70	0.60	2.02	
					Total	6.30	cu m
2.	**Cement Concrete 1:3:6** In foundation with stone ballast –						(1/2 of earthwork in excavation in item 1)
	Abutments …	2	5.10	0.70	0.30	2.14	
	Wings walls …	4	1.20	0.70	0.30	1.01	
					Total	3.15	cu m

Item No.	Particulars of items of works	No.	Length m	Breadth M	Height Or Depth m	Quantity	Explanatory notes
3.	**I-class brick work in I : 4 cement mortar-** Abutments …	2	4.80	0.40	1.50	5.76	(Up to top of R.C.C slabs.)
	Wing walls …	4	1.20	0.40	1.50	2.88	
	Parapets up to kerb …	2	4.70	0.40	0.30	1.13	(Above R.C.C. slab up to kerb.)
	Parapets above Kerb …	2	4.70	0.30	0.50	1.41	(Above kerb excluding coping.)
	Parapet coping …	2	4.90	0.40	0.10	0.39	
					Total	11.57	
	Deduct- Bearing of R.C.C slab in abutment						
4.		2	4.80	0.30	0.20	0.57	
	R.C.C. work 1:2:4 in slab excluding steel and its bending but including centering, shuttering and binding steel			Net	Total	11.00	Cu m
		1	4.80	2.10	0.20	2.016 Cu m	No deduction for volume of steel.
5.	Steel bars including bending in R.C.C. Work- 20 mm dia. Bars – Main straight bars 30 cm c/c … (No.= 4.80 +1 =17) . 30	17	2.38	-	-	40.46 Cu m	L = 2.10 – 2 side covers + 2 hooks = 2.10-(2X4 cm) +(18X20 mm) = 2.3 m

	Description	No.	Length	Breadth	Height	Contents	Remarks
	Main bent up bars 30 cm c/c … (No.=$\frac{4.80}{.30}$= 16)	16	2.54	-	-	40.64 m	Adding one depth, 1 cm for two bent ups L=2.38+.16 = 2.54 m
			Total	81.10	@2.47	Kg m =200.32 kg	
	10 mm Dia. Bars – Distributing bottom Bars 25 cm c/c …	9	4.90	-	-	44.10 m	L = 4.8 – 2 end covers + 2 hooks =4.80-(2X4 cm)+(18X10mm)=4. 9 m
	Distributing top bars	4	4.90	-	-	19.60	
	Total		63.70 m	@.62 kg	=	39.49 kg	
			Total	Of	Steel	239.81 kg	2.398 quintal In between parapets
6.	Cement concrete 1:2:4 Wearing coat …	1	4.00	2.30	0.10	0.92cum	
7.	Cement Pointing 1:2 in walls- Face wall from 10 cm below G.L up to bottom of coping	2	4.70	-	2.10	19.74	
	Inner side of parapet excluding coping	2	4.70	-	0.80	7.52	Ht. = (20+10+50) =0.80 mm
	Coping (inner edge, top, outer edge and outer and side) …	2	4.90	0.70	-	6.86	B=(10+40+10+10) cm = 0.70 m
	Ends of parapet ….	4	-	0.40	0.20	0.32	Up to kerb Above kerb
	Ends of parapet ….	4	-	0.30	0.50	0.60	Edge and under side
	Ends of coping ….	4	-	0.40	0.20	0.32	
				Total		35.36	Including 10 cm below G.L and edge of R.C.C slab.
	Deduct- Rectangular opening	2	1.50		1.30	3.90	
	Triangular portion below earth slope	2	(1/2 X	1.3 X 1.3		1.69	Sq. mm
			Total of	Deduct ion		5.59	
			Net	Total		29.77	

6

HUME PIPE CULVERT

Fig. 8-14

ESTIMATING AND COSTING

Details of Measurement and calculation of Quantities

Item No.	Particular of items	No	Length m	Breadth m	Height Or Depth m	Quantity	Explanatory notes

1.	Earthwork in excavation
	    In foundation ——
	    Face walls ....
		2	3.10	0.80	0.80	3.97	

	Wing walls inclined Portion	4	$\frac{2.3 + 1.8}{2}$	$\frac{0.8 + 0.7}{2}$	0.80	4.92	Average length and average breadth.
	Wing walls triangular corner	4	(1/2 X0.6	X 0.8)	0.80	0.77	Area of triangle.
	Turn walls	4	$\frac{0.95+0.80}{2}$	0.70	0.80	1.96	Average length.
	Under pipe	1	9.80	3.10	0.15	4.56	
					Total	16.18 Cu.m	

2.	Cement concrete 1:4:8
	    In foundation ——
	    Face walls ....
		2	3.10	0.80	0.30	1.49	

	Wing walls inclined Portion	4	$\frac{2.3+1.8}{2}$	$\frac{0.80+0.70}{2}$	0.30	1.85	
	Wing walls inclined Portion	4	(1/2.X0.6	X.0.8)	0.30	0.29	
	Turn walls	4	$\frac{0.95+0.80}{2}$	0.70	0.30	0.74	
	Upper pipe and in Between pipe up to half Height	1	9.80	3.10	0.50	15.19	Thicknees $=15+\frac{70}{2}$
	Deduct half of pipes ...	3	9.80X1/2	$\frac{\pi X\ 0.7^2}{4}$	Total	19.56 5.66	=50 cm=0.50 m
					Total	13.90	cu m
3.	First class brick work in 1:6 cement sand mortor Face wall — Footing — 50cm breadth	2	4.00	0.50	0.50	2.00	Breadth means thickness of wall.
Item No.	Particular of items	No	Length m	Breadth m	Height Or	Quantity	Explanatory notes

				Depth m		
	Above footing 40 cm Breadth .…	2	3.80	0.40	1.60	4.86cum	
	Parapet 30 cm breadth	2	3.80	0.30	0.30	0.68	
	Coping — 35 cm breadth	2	4.00	0.35	0.10	0.28	
	Wing wall — 1st step —50 cm breadth	4	1.10	0.5	$\frac{0.5+0}{2}$	0.55	
	2nd step — 40 cm breadth — (i) straight portion .…	4	1.80	0.40	0.30	0.86	
	(ii) Sloping portion .…	4	1.80	0.40	$\frac{0.40+0.0}{2}$	0.58	} Average height
	3rd step — 30 cm breadth	4	1.90	0.30	$\frac{0.70+0}{2}$	0.80	
	Turn wall— 40 cm breadth	4	$\frac{0.8+0.75}{2}$	0.40	0.50	0.62	
	Turn wall— 30 cm Breadth	4	$\frac{0.8+0.7}{2}$	0.30	0.30	0.27	
					Total	11.49	cu m
4.	Cement pointing 1:2 in Exposec surfaces above G.L— Face walls outer sides	2	3.10	——	1.4	8.68sqm	Up to road level Above road level } Including coping.
	Face wall parapet outer side	2	3.80	——	0.65	4.94	Ht =20+30+10+5 = 65 cm =0.65m
	Parapet inner faces .…	2	3.80	——	0.70	5.32	Including kerb Offset of 10 cm Average height.
	Wing walls vertical face	4	2.30	0.30	$\frac{1.40+0.50}{2}$	8.74	
	Wing wall top .…	4	2.30	——	0.30	2.76	L = Perimeter = 80+30+70
	Turn walls vertical face three sides	4	1.80	0.3		2.16	= 180 cm = 1.80 m

Item No.	Particular of items	No	Length m	Breadth m	Height Or Depth m	Quantity	Explanatory notes
	Turn walls top ….	4	$\frac{0.8+0.7}{2}$		——	0.90	
					Total	33.50	Sq m
5.	Hume pipe heavy type 60 cm dia. Including collar joint ….	3	10.80			32.40	L= 10+0.4+0.4 = 10.8 m

ESTIMATE OF A CANAL FALL
PAGE.NO-440 BN DUTTA
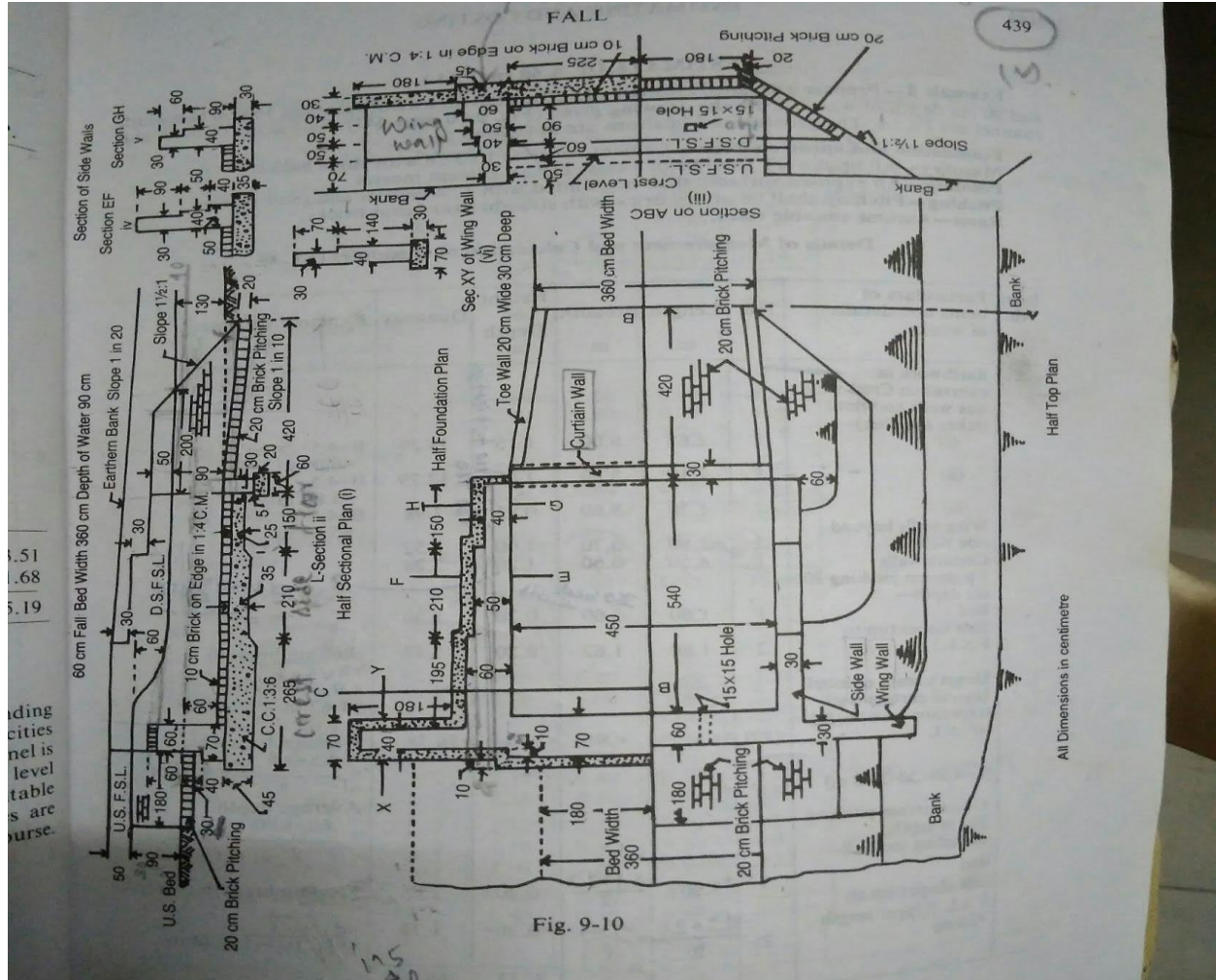
Fig. 9-10

11

Item No.	Particulars of item and details of work	No.	Length m	Breadth m	Height or Depth m	Quantity m	Explanatory notes
1.	E/W in excavation						
	.Crest wall —	1	2.65	6.00	1.15	18.29	B=4.5+2X0.6+2X0.15
	.Side wall —	1	2.10	5.80	1.05	12.79	= 6 m
	.Floor — Wing wall beyond side wall---	1	1.50	5.60	0.95	7.98	B=4.5+2X0.5+2X0.15 = 5.80 m
		2	1.80	0.70	1.00	2.52	B=4.5+2X0.4+2X0.15 = 5.60 m
	.Curtain walls upstream side pitching 20 cm depth Bed ----	1	4.50	0.60	1.20	3.24	
		1	1.80	3.60	0.20	1.30	
	.Side slope (up to F.S.L) ---	2	1.80	1.62	0.20	1.17	
	.Down stream channel beyond curtain wall. Trapezium section(bd+sd^2)L.	(4.0	5X0.8+1^1/$_2$	X0.8^2)	X3.90	16.38	Sloping breadth = h$\sqrt{S^2+1}$ = 0.9 $\sqrt{(1^1/2^2+1}$ = 1.62 m
	.Down stream pitching 20 cm depth, excluding toe wall --- Bed –	1	3.90X	$\frac{4.1+3.2}{2}$	X0.20	2.85	Average breadth = $\frac{4.5+3.6}{2}$ = 4.05 m Average depth = $\frac{0.60+1.0}{2}$ = 0.80 m
	Side slope up to FSL(Upper length=2.0m)	2	$\frac{4.2+2.0}{2}$	X1.44	X0.20	1.79	Sloping breadth at middle = d$\sqrt{S^2+1}$ =0.8$\sqrt{1^1/_2{}^2+1}$ = 1.44 m
					C.O	68.31	
	Curve portion ---	2	πX0.6^2	(area) 0.20	X0.20	0.45	
	toe wall ---	2	3.90		0.30	0.47	Taken as Quadrant of sphere
					Total	69.23	

Item No.	Particulars of item and details of work	No.	Length m	Breath m	Ht or Depth m	Quantity m	Explanatory notes
	Deduct for set back of wing wall ….	2	0.60	0.10	1.15	0.14	
				Net	Total	69.09 cu m	
2.	Cement concrete 1:3:6 in foundation and —						
	(i) Crest wall ….	1	2.65	6.00	0.45	7.16	
	(ii) side walls ….	1	2.10	5.80	0.35	4.26	
	(iii) floor ….	1	1.50	5.60	0.25	2.10	
	Wing wall beyond side wall	2	1.80	0.70	0.30	0.76	
	Curtain wall ….	1	4.50	0.60	0.20	0.54	
					Total	14.82	
3.	Deduct for set back of wing wall ….	2	0.60	0.10	1.15	0.14	
				Net	Total	14.68 cu m	
	1-class brick work in 1:4 cement mortar crest wall —						
	1st step ….	1	4.50	0.70	0.40	1.26	As per cross sec.BC
	2nd step ….	1	4.50	0.60	0.90	2.43	
	Side wall —						
	(i) 1st step	2	2.35	0.60	0.40	1.13	As per cross sec.EF
	2nd step	2	2.35	0.50	0.50	1.18	
	3rd step	2	2.35	0.40	0.50	0.94	
	4th step	2	2.35	0.30	0.70	0.99	As per cross sec.GH
	(ii) 1st step	2	2.10	0.50	0.40	0.84	
	2nd step	2	2.10	0.40	0.50	0.84	
	3rd step	2	2.10	0.30	0.90	1.13	
	(iii) 1st step	2	1.50	0.40	0.90	1.08	
	2nd step	2	1.50	0.30	0.60	0.54	
						12.36	As per cross sec. XY
	Wing wall beyond						

14

Item No	Particulars of item and details of work	No.	Length m	Breath m	Height or Depth m	Quantity m	Explanatory notes
.	side wall	2	1.80	0.40	0.40	0.58	
	….	2	1.90	0.40	0.50	0.76	
		2	2.00	0.40	0.50	0.80	
		2	2.10	0.30	0.70	0.88	
	Curtain wall	1	4.50	0.30	0.40	0.54	
	Toe wall	2	3.90	0.20	0.30	0.47	
					Total	16.39 cu m	Down stream in between walls
4.	Brick edge Floor in 1:8 cement mortar Including pointing …	1	5.40	4.50	—	24.30 sq m	
5.	Cement pointing in 1:3 cement mortar – Crest wall (upstream face top and down stream (face) ….	1	4.50	—	2.40	10.80	Ht = 0.6+0.6+1.2 = 2.40 m
	Side wall inner face (i)	2	1.80	—	2.00	7.20	
	(ii)	2	2.10	—	1.70	7.14	
	(iii)	2	1.50	—	1.40	4.20	
	Side wall portin Above crest wall	2	0.60	—	0.80	0.96	Full length of 30 cm wall
	Vertical faces of stepping vertical face of end	2X2	—	0.30	0.30	0.36	
	Top of side walls	2	—	0.40	0.90	0.72	
	Top of curtain wall	2	—	0.30	0.60	0.36	Triangular portions of slope
	Top of toe walls	2	6.00	0.30	—	3.60	
	Wing wall top face	1	4.50	0.30	—	1.35	
	Wing wall up-stream	2	3.90	0.20	—	1.56	
		2	2.10	0.30	—	1.26	
	Side Triangular portion above slope	2	½(2.10	X 1.40)	—	2.94	Dimension same as in item 1)
					Total	42.45 sq m	
6.	Brick –pitching — Up-stream bed	1	1.80	3.60	0.20		
	Up-stream side slop	2	1.80	1.62	0.20	1.30	
	Down- stream bed	1	3.90X	$\frac{4.1+3.2}{2}$	X0.20	1.17	
	Down - stream Side					2.85	
	Slopes	2	4.2+2.0	X1.44	X0.20	1.79	

	Side curved portions	2	3.14×0.6^2	(area)	X0.20	0.45	
					Total	7.56 cu m	

DRAINAGE SYPHON

Due to relative levels sometime it is required to lower the bed of the irrigation channel or the drainage channel at their crossing. When the bed of the irrigation channel is depreed and taken under nala or stream it is known as irrigation siphon. When the bed of the nala or stream is depressed and taken under the irrigation channel it is known as drainage syphon. The syphon crossing may be of rectangular closed masonry channel or of circular brickwork of R.C.C OR Hume pipe of the required diameter and number. Approach and exit may be through masonry drop pit or of masonry sloped channel. The down stream end is kept lower than the up stream end by at least 15cm for better flow. An estimate of a small drainage syphon has been given in Example-7 of BN Dutta.

Example-7 Prepare a detailed estimate of a drainage syphon across a minor from the given drawing,.

Foundation concrete shall be of 1:4:8 cement concrete with brick ballast. All brickwork shall be of 1:4 cement mortar. Exposed surfaces of brickwork shall be struck pointed with 1:2 cement mortar. Brick pitching shall be of dry brick with straight over burnt bricks.

Assume suitable rates for the different items of work.

17

DRAINAGE SYPHON ACROSS A MINOR

Longitudinal Section

B — Sectional Plan
Showing all Footings

Top Plan

Fig. 9.9

All Dimensions in centimetre

Drop Pit 180×180

Bank

Slope

Berm

Slope

Bed of Minor

Slope

Berm

Slope

Bank

Bed of Nala

Brick Pitching

Bank of Nala

18

DRAINAGE SYPHON

Cross Sections

G
L
H.F.L. 40
80 1:1
180
10 cm Dry Brick Pitching

Cross Section EF
Showing Nala

G
L
H.F.L.
80 1:1
120 30
30 180
30
10
10 cm Brick in
1:3 Cement Mortar
240
270

Cross Section CD Showing Drop Pit and Nala

10
30
80
G
L
100
15cm R.C.C.Slab
30
80
100
30
150
10 cm Brick in
60 1:3 Cement Mortar
30
240
460

20

Item No.	Particulars of item and details of work	No.	Length m	Breath m	Height or Depth m	Quantity Cu m	Explanatory notes

21

Item No.	Particulars of item and details of work	No.	Length m	Breath m	Height or Depth m	Quantity Cu m	Explanatory notes
1	Earthwork in excavation in foundation —						
	Syphon duct	1	9.50	2.40	1.60	36.48	For bed level of nala
	Drop pit .	2	2.10	2.70	1.60	18.14	
	Wing walls	4	1.25	1.10	1.60	8.80	
					Total	63.42 cu m	
2	Cement Concrete 1:4:8With brick ballast —						
	Syphon duct	1	9.50	2.40	0.30	6.84	
	Drop pit	2	2.10	2.70	0.30	3.40	
	Wing walls	4	1.25	1.10	0.30	1.65	
					Total	11.89 cu m	
3	First class brick work in 1:4 cement mortar — Syphon duct side walls	2	9.20	0.30	1.30	7.18	
	Drop pit walls .	2X2	2.10	0.30	1.30	3.28	
		2	1.80	0.30	1.30	1.40	
	Wing wall —						
	1st step 70cm walls	4	1.25	0.70	0.60	2.45	
	2nd step 60cm walls	4	1.25	0.60	0.60	1.80	Upto top of slab
	2nd step 60 cm Walls above slab	2	4.60	0.60	0.20	1.10	
	3rd step 50 cm	2	4.60	0.50	1.00	4.60	

	wall						
	4th step 40 cm wall	2	4.60	0.40	0.80	2.94	
	5th step 30 cm wall						
	(Parapet)	2	4.60	0.30	0.30	0.83	
	Coping	2	4.70	0.35	0.10	0.33	
					Total	25.91 cu m	
4	R.C.C slab of siphon						
	Duct including steel reinforcement complete work	1	9.20	2.10	0.15	2.90 cu m	
5	10 cm thick brick floor in 1:3 cement mortar including 1:2 cement pointing —						
	Floor of syphon duct	1	9.20	1.50	— — —	13.80	
	Floor of drop pit ...	2	1.80	1.80		6.48	
					Total	20.28 sq m	
6	Cement struck Pointing 1:2 —						
	Syphon duct inner faces	2	9.20		1.00	18.40	
	Drop pit 3 vertical faces	2X3	1.80	—	1.20	12.96	
	Drop pit 3 top faces	2	6.0	—	0.30	3.42	L = 2x180+240 = 600 cm
	Parapet wall inner Face top and outer	2	4.60		2.30	21.16	Ht= 20+10+30+10+35 +10+5+110
	Face up to G.L			—			= 230 cm
		2	1.80		1.20	4.32	

	Outer face of wing Wall above slab ... Triangular portion of Outer face of wing wall ...	2X2	($\frac{1}{2}$X0.8	— X 0.8)	=	1.28	
					Total	61.54 sq m	
7	10 cm dry brick pitching with straight over burnt brick — Bed of nala Side slopes of nala	2 2X2	3.00 3.00	1.80 1.13	— —	10.80 13.56	Thin pitching unit in area basis
					Total	24.36 sq m	Up and down stream Sloping breadth = $\sqrt{0.8^2+0.8^2}$=1.13 m

NUMERICAL ON ESTIMATE OF WBM ROADS

Estimate the items involved for construction of WBM road from the following data:

Length of the road 100m, metalled width = 5.5 mt

Thickness of the grade – l metal solving = 80mm

Wearing coat of grade – ll metal loose consolidated to 80mm thick.Surface of the road is to be finished with a coat of bitumen as given below:

1st finishing coat:12mm chips @0.018 m^3 and bitumen @1.22 kg per square meter of road surface.

2nd finishing coat: 6mm chips @ 0.01 m^3 and bitumen @ 1.22 kg per square meter of road surface.

Consumption of fuel @ 0.42 kg per kg of bitumen.

Ans – Length of the road = 100m

Metalled width = 5.5 mt

Area of the road surface = 5.5 x 100 = 550 sqm

Thickness of grade 1 metal solving

    = 80mm = 0.08mt.

Quantity required = 5.5 x 0.08 x 100 = 44 cum.

Thickness of grade ll metal consolidated thickness

    = 80mm = 0.08mt

Quantity required = 5.5 x 0.08 x 100 = 44cum

1st finishing coat: 12mm chips @ 0.018 m^3 per square meter

For 550 sqm   chips required

$$= 550 \times 0.018 = 9.9 \text{ cum}$$

Bitumen required

$$= @ 1.22 \text{ kg per } m^2 \text{ of road surface}$$

$$= 550 \times 1.22 \text{ kg} = 671 \text{ kg}$$

For 2nd finishing coat : 6 mm chips @ 0.01 m3 per square meter of road surface

quantity required

$$= 550 \times 0.01 = 5.5 \text{ cum}$$

Bitumen required

$$= @ 1.22 \text{ kg per } m^2 \text{ of road surface}$$

$$= 550 \times 1.22 \text{ kg} = 671 \text{ kg}$$

For 1st coat and 2nd coat bitumen required

$$= 671 + 671 = 1342 \text{ kg}$$

Consumption of fuel @ 0.42 kg per kg of bitumen.

Consumption of fuel $= 1342 \times .42 = 563.64$ kg.

ROAD ESTIMATE

EXAMPLE 1 :-Reduced level (R.L) of ground along the center line of a proposed road from chainage 10 to chainage 20 are given below. The formation level at the 10th chainage is 107 and the road is in downward gradient of 1 in 150 up to the chainage 14 and then the gradient changes to 1 in 100 downward. Formation width of road is 10 metre and side slopes of banking are 2:1 (Horizontal : Vertical). Length of the chain is 30 metre.

Draw longitudinal section of the road and a typical cross-section and prepare an estimate of earthwork at the rate of Rs-275.00 % cu m.

R.L of Formation 107.00

Chainage	10	11	12	13	14	15	16	17	18	19	20
R.L of ground	105.00	105.60	105.44	105.90	105.42	105.30	105.00	104.10	104.62	104.00	103.3

Gradient   Down gradient 1 in 150   Down gradient 1 in 100

Calculation of Quanties of Earthwork (EX-3 OF BN DUTTA)   B = 10 m, S = 2

SOLUTION:-

Height of Bank	2.00	1.20	1.16	0.50	0.78	1.60	0.60	1.20	0.38	0.70	1.10
R.L. of Formation	107.00	106.80	106.60	106.40	106.20	105.90	105.60	105.30	105.00	104.70	104.40
R.L. of Ground	105.00	105.60	105.44	105.90	105.42	104.30	105.00	104.10	104.62	104.00	103.30
Distance in Metre	300	330	360	390	420	450	480	510	540	570	600
Chainage	10	11	12	13	14	15	16	17	18	19	20
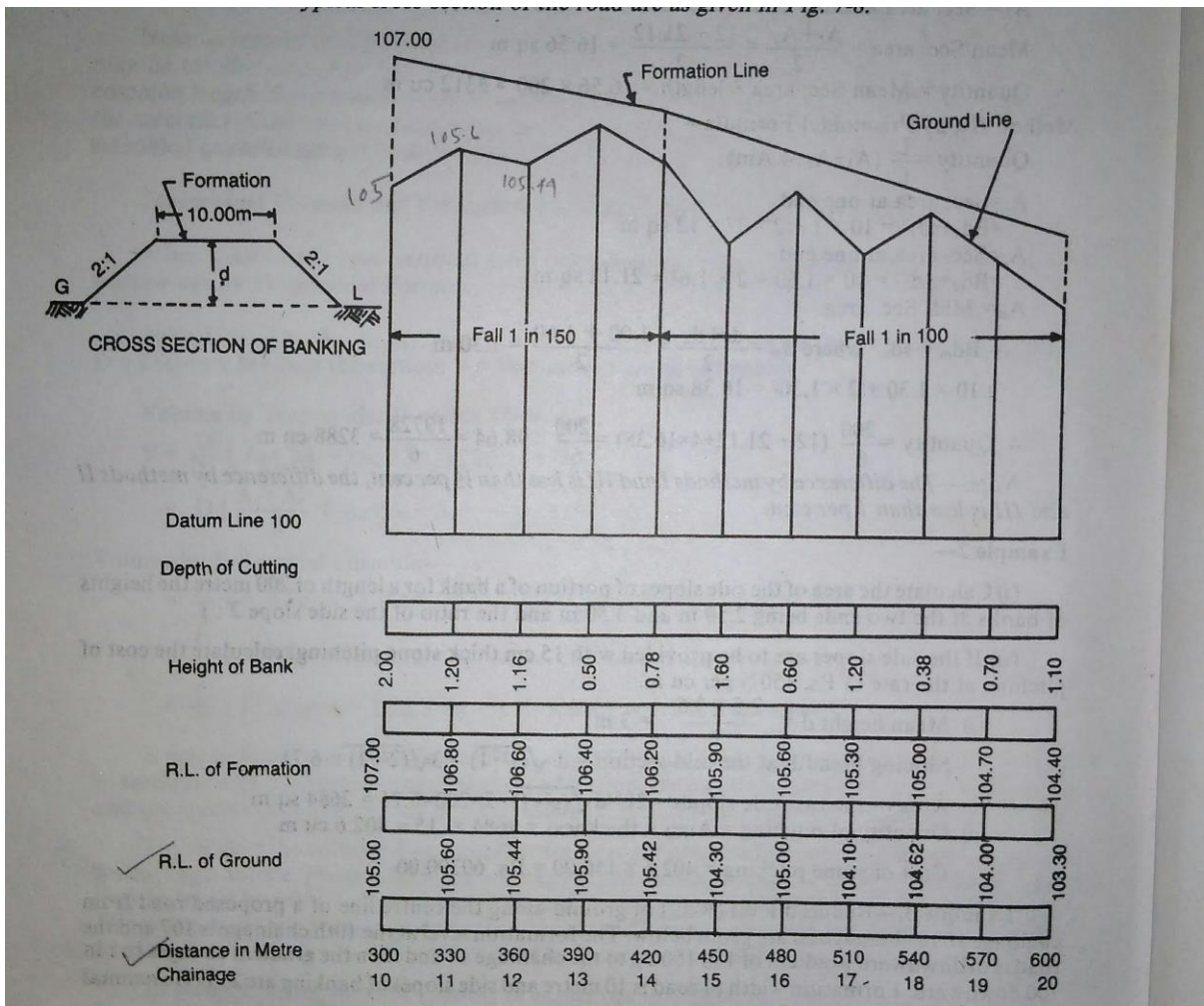
Stations Or Chainage	Length	Height Or Depth Diff.Of G.L.and F.L	Mean height Or Depth d	Central Area Bd	Side Area Sd^2	Total sec.area Bd+sd2	Length In betw. Stations L	Quantity (Bd+sd2)XL	
								Banking	Cutting
m	m	m	m	M^2	M^2	M^2	M^2	M^3	M^3
10	300	2.00	---	---	---	---	---	---	---
11	330	1.20	1.60	16.00	5.12	21.12	30	633.6	---
12	360	1.16	1.18	11.80	2.78	14.58	30	437.4	---
13	390	0.50	0.83	8.30	1.38	9.68	30	290.4	
14	420	0.78	0.64	6.40	0.82	7.22	30	216.6	
15	450	1.60	1.19	11.90	2.83	14.73	30	441.9	---
16	480	0.60	1.10	11.00	2.42	13.42	30	402.6	---
17	510	1.20	0.90	9.00	1.62	10.62	30	318.6	---
18	540	0.38	0.79	7.90	1.25	9.15	30	274.5	---
19	570	0.70	0.54	5.40	0.58	5.98	30	179.4	---
20	600	1.10	0.90	9.00	1.62	10.62	30	318.6	---

Total = 3513.6

cu m

ABSTRACT OF ESTIMATED COST (EX-3)

Item No.	Particulars of items	Quantity	Unit	Rate Rs. P.	per	Cost
						Rs. P.
1	Earthwork in banking ...	3513.6	Cu m	275.00	% cu m	9662.40
Total ...						9662.40
	Add 5% (3% for Contingencies and 2% for workcharged Establishment)					483.12
...						Rs. 10145.52
Total ...					Grand	

EXAMPLE 2:- Estimate the cost of earthwork for a portion of road for 400 meter length from the following data:-

Formation width of the road is 10 meter. Sides slopes are 2:1 in banking $1^{1}/_{2}$: 1 in cutting.

Station	Distance in meter	R.L of Ground	R.L of formation
25	1000	51.00	52.00
26	1040	50.90	I
27	1080	50.50	I
28	1120	50.80	I
29	1160	50.60	Downward gradient
30	1200	50.70	Of 1 in 200
31	1240	51.20	I
32	1280	51.40	I
33	1320	51.30	I
34	1360	51.00	I
35	1400	50.60	I

Longitudinal section of the road and type cross-section are as given in fig. 7-9. The example can, however, be solved without the help of L-section and cross-section.

Fig. 7.9
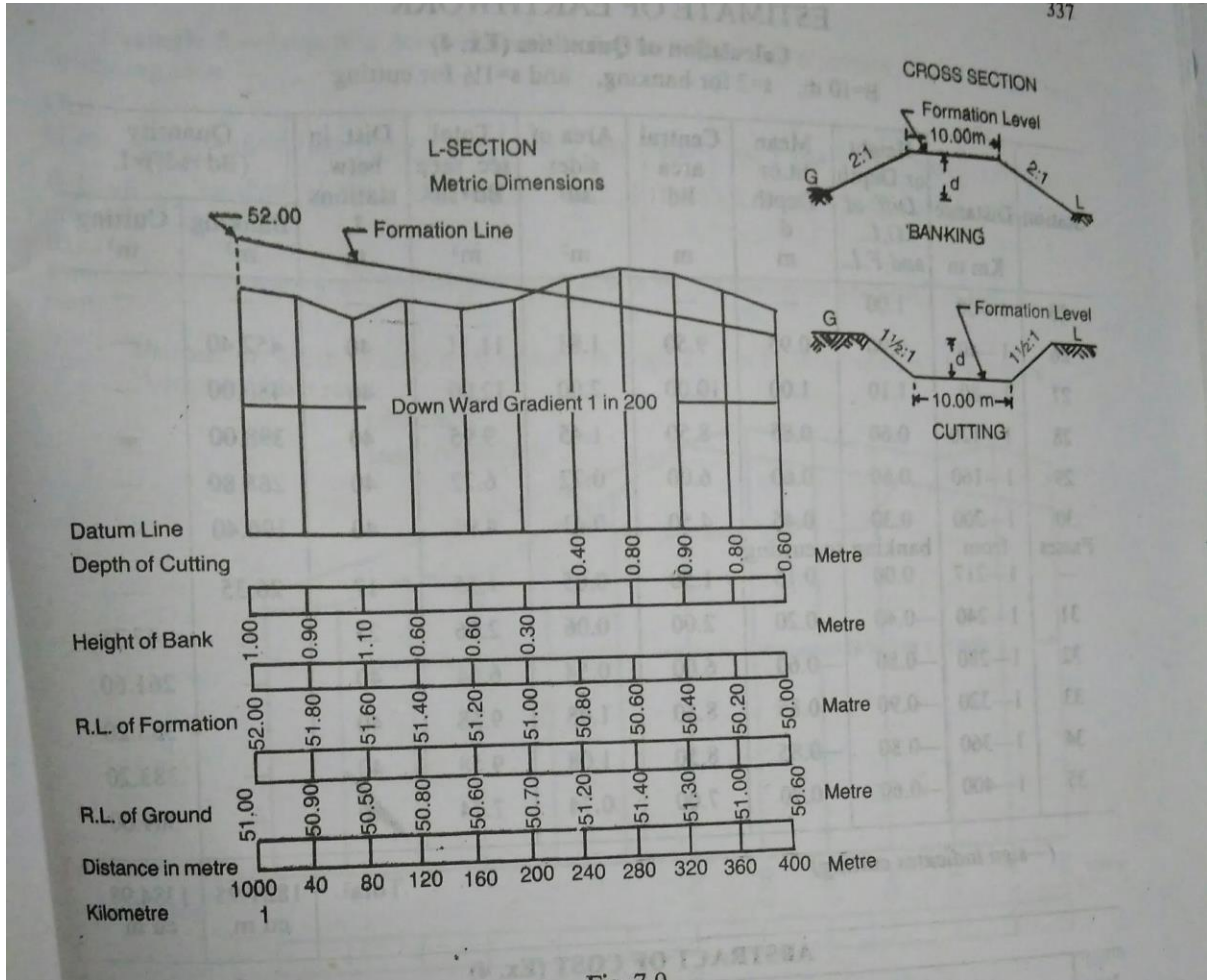
The road passes from banking to cutting in between the station 30 (1200 m) and 31 (1240 m). The distance where it passes through zero, i.e, ground level, may be determined as follows:--

The two traiangles on either side of zero point are symmetrical (fig.7-10) .

$$\frac{x}{.3} = \frac{40-x}{.4}; \ or \ .4x = .3\,(40-x) \qquad or .4x = 12 - .3x. \ or \ .7x12$$

.. $x = \frac{12}{.7} = 17.14$ m $= 17$ m say

Fig. 7.10
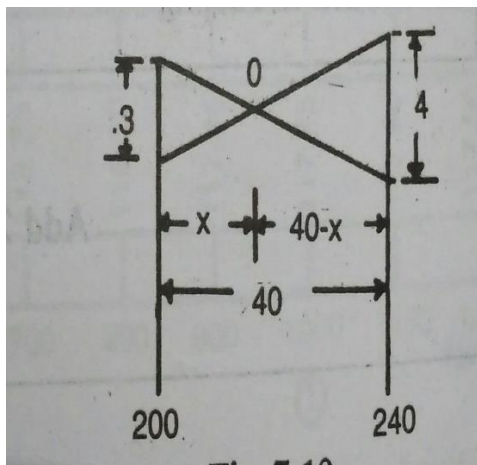
Therefore length of banking portion is 17m, and the length of cutting portion is 40 – 17=23m

ESTIMATE OF EARTHWORK

Calculation of Quantities (Ex.4)

B=10m, s=2 for banking, and s=$1^1/_2$ for cutting

Stations	Distance	Height Or Depth Diff.Of G.L.and F.L	Mean height Or Depth d	Central Area Bd	Area of Sides Sd^2	Total sec.area Bd+sd2	Distance In betw. Stations L	Quantity (Bd+sd2)XL	
	Km m		m	m	m^2	m^2	m	Banking m^3	Cutting m^3
25	1 --- 00	1.00	---	---	---	---	---	---	---
26	1 --- 40	0.90	0.95	9.50	1.81	11.31	40	452.40	---
27	1 --- 80	1.10	1.00	10.00	2.00	12.00	40	480.00	---
28	1 --- 120	0.60	0.85	8.50	1.45	9.95	40	398.00	---
29	1 --- 160	0.60	0.60	6.00	0.72	6.72	40	268.80	---
30	1 --- 200	0.30	0.45	4.50	0.41	4.91	40	196.40	---
Passes	From	banking	To cutting						

---	1 --- 217	0.00	0.15	1.50	0.05	1.55	17	26.35	---
31	1 --- 240	--- 0.40	--- 0.20	2.00	0.06	2.06	23	---	47.38
32	1 --- 280	--- 0.80	--- 0.60	6.00	0.54	6.54	40	---	261.60
33	1 --- 320	--- 0.90	--- 0.85	8.50	1.08	9.58	40	---	383.20
34	1 --- 360	--- 0.80	--- 0.85	8.50	1.08	9.58	40	---	383.20
35	1 --- 400	--- 0.60	--- 0.70	7.00	0.74	7.74	40	---	309.60
(-- Sign indicates cutting)							Total	1821.95 cu m	1384.98 cu m

ABSTRACT OF ESTIMATED COST (EX-3)

Item No	Particulars of items	Quantity	Unit	Rate Rs. P	per	Cost Rs. P.
1	Earthwork in banking ...	1821.95	cu m	275.00	% cu m	5010.36
2	Earthwork in cutting ...	1384.98	cu m	350.00	% cu m	4847.43
Total ...						9857.79
...					Add (3% for Contingencies	295.73
...					Add 2% for workcharged Establishment)	197.16
...					Grand Total	10350.68

ESTIMATE OF SEPTIC TANK FOR 50 USERS

Example-2 Prepare a detailed estimate of a Septic tank with Soak-pit for 50 users together with sanitary fittings of one seat of Latrine from the given drawings (fig-6.2)

Septic tank shall be of 1 class brick masonry 1:4 cement mortar over cement concrete 1:3:6 foundation and base, with R.B . Partition wall and R.C.C slab cover. inside of tank including floor shall be of II class brick masonry with 1:6 cement mortar.

Estimate for water arrangements, Should also be made with a 250 litre G.l overhead tank connecting flushing Cistern of W.C. and a water tap, assuming that there is a pipe water supply within 10 m of the latrine.

Details of Measurement and Calculation of quantities (Ex-1)
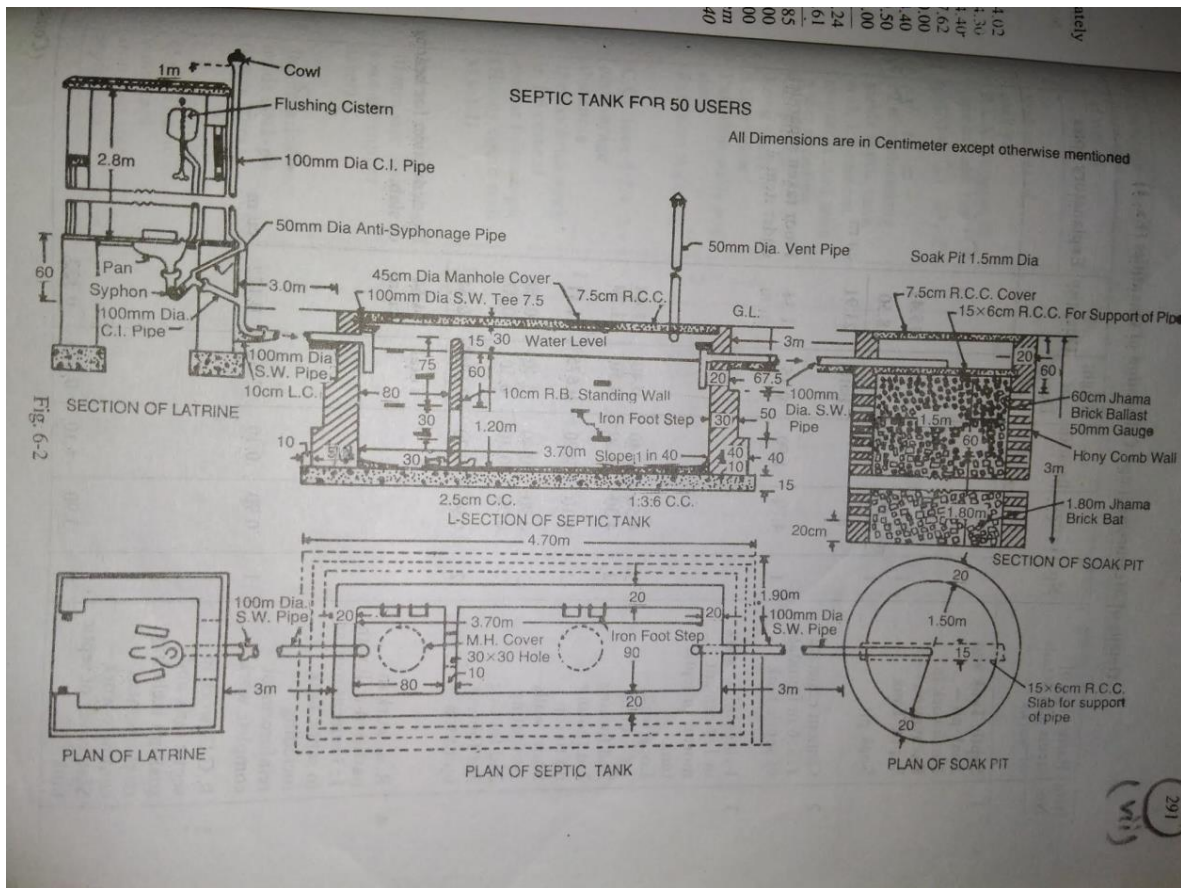
SEPTIC TANK FOR 50 USERS

Item No.	Particulars of item and details of work	No.	Length m	Breath m	Height or Depth M	Quantity	Explanatory notes
1	Septic tank and Soak-Pit — Earthwork in excavation — Septic Tank	1	4.70	1.90	1.725	15.41	
	Soak pit	1	$\pi\text{x}(1.9)^2$	X3.00	—	8.50	
2	Cement Concrete 1:3:6 in Foundation of Septic tank		4		Total	23.91 cu m	
		1	4.70	1.90	0.15	1.34 Cu m	Floor taken separately under item 7.
3	l - class brick workin 1:4 cement mortar in septic tank Long walls						
	1st footing	2	4.50	0.40	0.40	1.44	
	... 2nd footing ...	2	4.30	0.30	0.50	1.29	
	3rd footing Up to top Short walls —	2	4.10	0.20	0.675	1.11	
	1st footing	2	0.90	0.40	0.40	0.29	
	2ndfooting ...	2	0.90	0.30	0.50	0.27	
	3rd footing Up to top ...	2	0.90	0.20	0.675	0.24	
					Total	4.64 Cu m	No deduction for bearing of slab
4	R.B work in partition wall with 1:3 cement mortar in septic tank including Reinforcement						
Item No.	Particulars of item and details of work	No.	Length m	Breadth m	Height or Depth m	Quantity	Explanatory notes
5	complete work ... R.C.C work in septic tank and	1	0.90	0.10	1.35	0.122 Cu m	

	soak pit including reinforcement complete work — slab cover of septic tank	1	3.90	1.10	0.075	0.322	
	slab cover of ... soak pit R.C.C	1	$\dfrac{\pi \times (1.7)^2}{4}$	X	0.075	0.170	
	support of pipe in soak pit ...	1	1.70	0.15	0.06	0.015	
					Total	0.507 cu m	
6	12 mm plastering inside septic tank with 1:2 cement mortar mixed with Water proofing Compound						
	Long walls	2	3.70	—	1.50	11.10	
	Short walls	2	0.90	—	1.50	2.70	
	Partition walls Both Sides	2	0.90	—	1.35	2.43	
	Partition Walls top	1	0.90	0.10	---	0.09	
					Total	16.32 sq m	
7	C.C floor 1:2:4, 5 cm average thickness ...	1	3.70	0.90	—	3.33 sq m	
8	ll class brickwork in 1:6 cement mortar in soak-pit (Honey comb wall as solid) ...	1	πx1.70	X0.20x	3.00	3.20 cu m	Mean circumference.
9	Jhama brick ballast 10 mm size inside Soak-pit (upper layer)	1	$\dfrac{\pi \times 1.5^2}{4}$	X0.60	—	1.06 cu m	
10	Jhama brick bats Inside soak-pit (lower layer) ...	1	$\dfrac{\pi \times 1.5^2}{4}$	X 1.80	—	3.18 cu m	
11	C.l. Manhole cover 45 cm Dia. over Septic tank ...	2	—	—	—	2 Nos.	
12	lron foot steps septic tank	8	—	—	—	8 Nos	Flushing cistern telescopic pipe foot rests. etc,. each may also be taken as separate items.
13	Sanitary works — W.C Indian pattern 50 cm white glazed Pan with siphon and						

	with 13.5 – litre C.l Flushing cistern (E.L.C) with brackets and 32 mm Dia G.l telescopic flush pipe painted two coats and with chain foot rests complete supply and fixing ...	1	—	—	—	1 set
14	S.W (stone –ware) Pipe 100 mm dia. Laid over 10 cm L.C including digging, layer, jointing, testing, etc., complete — Connecting latrine with septic tank ...	1	3.00	—	—	3.00
	Connecting septic tank with soak-pit ...	1	3.00	—	—	3.00
15	S.W Tee 100 mm dia. at the inlet and outlet of septic tank	2	—	—	Total	6.00 m
						2 Nos.
16	C.l Heavy soil Pipe 100 mm dia. connecting latrine Seat, vent pipe incluing fixing with lead jointing ...	1	5.00	—	—	5.00 m
17	C.l Heavy soil Pipe 50 mm dia complete With lead jointing connecting latrine pan with vent pipe...	1	0.60	—	—	0.60
	Vent pipe for septic tank ...	1	3.00	—	—	3.00
					Total	3.60 m
18	C.l Cowl 100 mm dia in latrine ...	1	—	—	—	1 No
19	C.l Cowl 50 mm dia for septic tank vent pipe ...	1	—	—	—	1 No
20	250- litre G.l. Tank of 20 B.W.G. Sheets with 45 cm dia.	1	—	—	—	1 No

	raised hinged cover with locking arrangement and fitted with 15 mm dia. brass ballcock supplying and fixing in position complete ...						
21	15 mm dia G.l pipe with fittings including digging, laying, clamping complete.Connecting G.l tank with water main	1	15.00	—	—	15.00	L= 10.00+0.60+2.80 +1.00+0.60 extra = 15.00 m
	Connecting flushing Cistern from G.I tank	1	2.00	—	—	2.00	
	Connecting water tap from G.l. tank ...	1	4.50	—	—	4.50	Top on outside wall.
					Total	21.50 m	
22	15 mm dia brass stop-cock (one for G.l. tank and one for flushing cistern) supplying and fixing	2	—	—	—	2 Nos.	
23	15 mm dia brass bib cock supplying and fixing	1	—	—	—	1 No.	
24	Brass Ferrule 6 mm dia. supplying and fixing ...	1	—	—	—	1 No.	

ESTIMATE OF 50 MM Dia. TUBE WELL WITH DEEF HAND PUMP

Example:-10 – Prepare an estimate of 50 mm dia. Tube well 100 meter deep with deep well pump from the given drawing (Fig-6.19). The strainer will consist of 3 pieces of 1.50 meter each. The housing pipe consist of 70 mm dia. G.I pipe 6 meter in length.

Assume suitable rates.

Handle

16mm Dia

Connecting Rod

Hand Pump

Mouth

Drain

Top Soil

70mm Dia G.I. Pipe
6 metre Long

Water Table

Sandy Loam

70mm Dia: Gun Metal
Cylinder with Plunger and Valve

Reducing Socket

Fine Sand

Clay and Kankar

Silty Loam

50mm Dia. G.I. Pipe

Coarse Sand

150cm

Strainer 50mm 3Nos.

150cm

Coarse Sand and Gravel

150cm

60cm

Bail Plug

Total depth of tube well = 100m
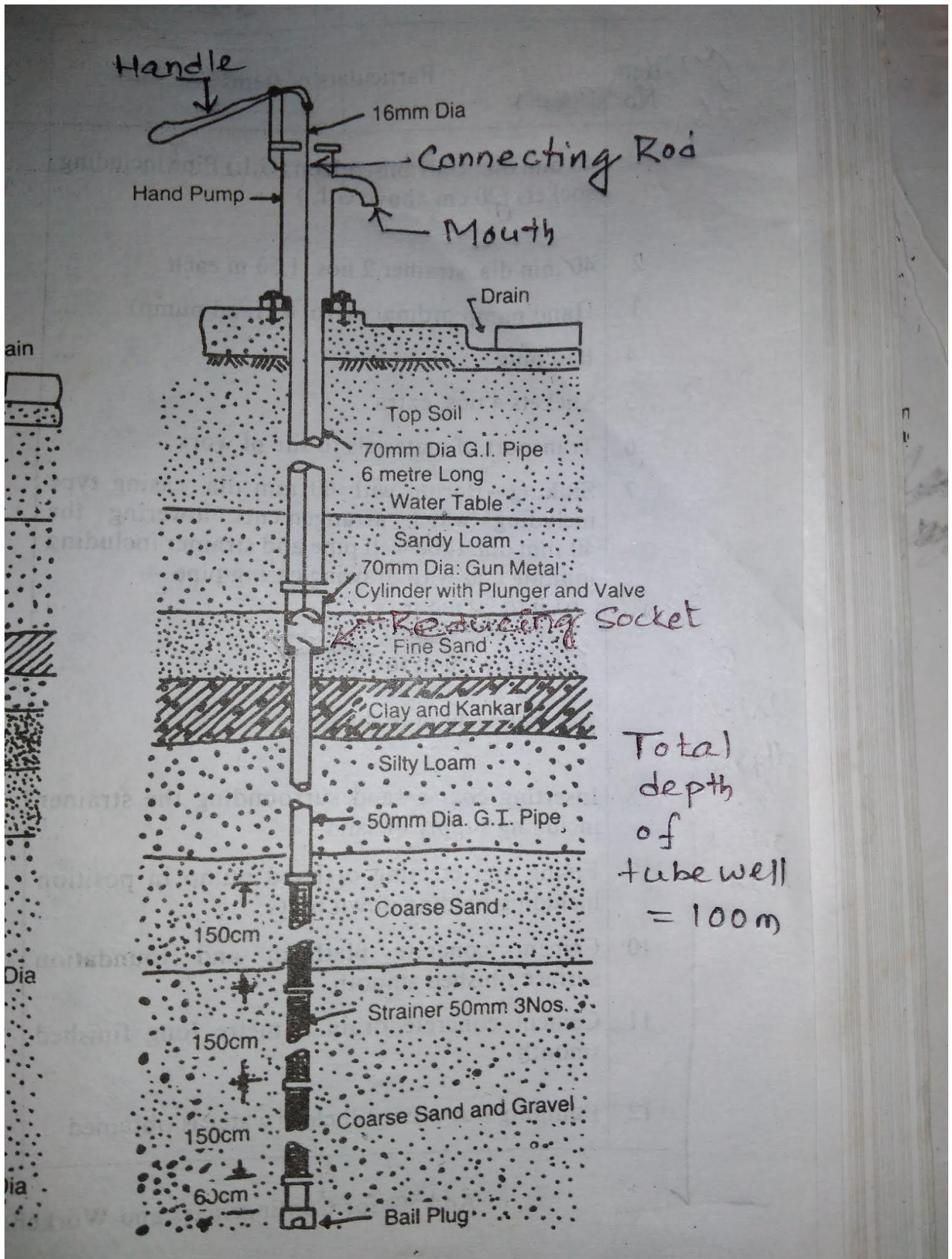
Bill of Quantities and cost (Ex-10)

Item No	Particulars of Items	Quantity	Rate Rs.	Amount Rs.

			P.	P.
1	50 mm dia. Galvanised iron (G.I) pipe	94.00 m	11.50 per r m	1081.00
2	70 mm dia. G.I housing pipe	6.00 m	23.00 per r m	138.00
3	50 mm dia. Strainer 3 nos. 1.50 m each	3 nos.	55.00 each	165.00
4	70 mm dia. Gun metal cylinder with valve and plunger	1 no.	100.00 each	100.00
5	Hand pump with extra length of16 mm dia. Connecting rod	1 no.	45.00 each	45.00
6	Bail plug	1 no.	7.50 each	7.50
7	Sockets 4 nos. extra	4 no.	2.50 each	10.00
8	Transport of materials to site of work	1 job	20.00 L.S	20.00
9	**Sinking –** Boring with 70 mm dia. casing pipe including water arrangements, lowering the 50 mm dia.tube well pipe and strainer including jointing and withdrawing of casing pipe-			
	(i) o to 20 meter	20 m	7.50 per r m	150.00
	(ii) Below 20 m to 35 m	15 m	11.00 per r m	165.00
	(iii) Below 35 m to 50 m	15 m	15.50 per r m	232.50
	(iv) Below 50 m to 65 m	15 m	20.00 per r m	300.00
	(v) Below 65 m to 80 m	15 m	24.00 per r m	360.00
	(vi) Below 80 m to 95 m	15 m	28.50 per r m	427.50
	(vii) Below 95 m to 100 m	5 m	33.00 per r m	165.00
10	Inserting coarse sand surrounding the strainer including supply of sand	1 job	20.00 L.S	20.00
11	Fixing and erecting hand pump in position including holding down bolts	1 job	10.00 L.S	10.00
12	Cement concrete platform and foundation, Surface finished smooth	1 job	30.00 L.S	30.00
13	Cement concrete drain 3 meter long finished smooth …	1 job	9.00 per r m	27.00
14	Plumbing out water till clear water is obtained …	1job	15.00 L.S	15.00
			Total	3468
	…			173.42
	Add 5% for contingencies and workcharged Establishment			3641.92
	…			
			Grand	

40

Total …	

PWD ACCOUNTS WORK

WORKS

Classification of work:

Works are primarily divided into two classes— "Original works" and "Repairs or maintenance."

Original Work: Original works include all new constructions whether of entirely new works or of additions and alterations to existing works which increase the capital cost of a building or work. Repairs to newly purchased or previously abandoned buildings required to render them useable are also original works.

Repairs Work: Repairs or maintenance includes all operations required to maintain in proper condition buildings and works in ordinary use.

Petty Work, Minor Work and Major Work: A petty work is one the cost of which does not exceed Rs. 20,000, a minor work is one the cost of which exceeds Rs. 20,000, but does not exceed Rs. 1,00,000 and a major works is one the cost of which exceeds Rs. 1,00,000.

NOTES—(1) In the case of mixed estimates, if the amount debitable to the "Original Works" portion of the estimate is Rs. 20,000 or less then it should be treated as a petty work and not minor work although the total cost of such mixed work (including repairs) might exceed Rs. 20,000 and the departmental head should be responsible for the execution of such mixed work and for finding funds for the purpose from their own budgets. When the original works portion of a mixed estimate exceeds Rs. 20,000 then it should be treated as a minor work and the whole work including repairs should be entrusted to the Public Works Department for execution. Provision for such works should be made in the Public Works Department budget both under "Original Works" and "Repairs."

(2) When an existing building is to be re-modelled or a portion thereof is to be replaced, then, if a type of construction or material of a more costly nature than that which previously existed is to be adopted or if the proposed reconstruction will result in an increase of accommodation, only one estimate will be prepared but the estimate will be a mixed estimate part of which is chargeable to "Original Works" and part to "repairs." The amount to be charged to "Original Works" will be the total estimated cost of the work minus the original cost (estimated, if necessary) of that portion of the building which has been replaced.

Exception—When in any one estimate, the amount chargeable to "Original Works" is less than Rs. 2,000 the whole of the expenditure may be charged to repairs ; provided that if the work relates to a residential building the amount which is correctly chargeable to "Original Works" is added to capital value and taken into account in calculating the rent.

Repair work

The repair works are classified in under mentioned categories:

- Day to day repairs/service facilities
- Annual repairs

- Special repairs

Day to day repairs

Day to day repairs are carried out by CPWD in all the buildings under its maintenance. The works which are to be attended on day to day basis such as removing chokage of drainage pipes, man holes, restoration of water supply, replacement of blown fuses, repairs to faulty switches, watering of plants, lawn mowing, hedge cutting, sweeping of leaf falls etc. are attended under day to day service facilities. The purpose of this facility is to ensure satisfactory continuous functioning of various services in the buildings. These services are provided after receipt of complaint from the users at the respective Service Centres. Complaints of periodical nature like white washing, painting etc.. which are usually got attended through contractors and cannot be attended on daily basis is transferred to register of periodical repairs.

Annual Repairs

The works of periodical nature like White washing, colour washing. distempering, painting etc. are called Annual Repair works and these are generally undertaken; through system of contracts.

The periodicity of applying white washing and colour washing for a building has been laid down by the Government. The periodicity is two years for white washing and colour washing and three years for painting. In addition, works such as patch repair to plaster, minor repairs to various items of work, replacement of glass panes, replacement of wiring damaged due to accident, replacement of switches, sockets tiles, Gap filling of hedges/perennial beds, Replacement/Replanting of trees, shrubs, painting of tree guards, planting of annual beds and trimming /pruning of plants etc., which are not emergent works and are considered to be of routine type, can be collected and attended to for a group of houses at a time and particular period of financial year, depending upon the exigency. Such works can be done under day to day repair also. The yard stick for annual repairs cover both the above facilities.

SPECIAL REPAIRS

As the building ages, there is deterioration to the various parts of the building and services. Major repairs and replacement of elements become inevitable. It becomes necessary to prevent the structure from deterioration and undue wear and tear as u.ell as to restore it back to its original conditions to the extent possible. The following types of works in general are undertaken under special repairs: -

I. White Washing, Colour washing, distempering etc., after completely scrapping the existing finish and preparing the surface afresh.

II. Painting after removing the existing old paint from various members.

III. Provision of water proofing treatment to the roof. All the existing treatments known are supposed to last satisfactorily only for a periodof about ten years.

IV. Repairs of internal roads and pavements.

V. Repairs/replacement of flooring, skirting, dado and plaster.

VI. Replacement of doors, window frames and shutters. Replacement of door and window fittings.

VII. Replacement of water supply and sanitary installation like water tanks, WC cistern, Wash basins, kitchen sinks, pipes etc..

VIII. Re-grassing of lawns/grass plots within 5-1 0 years.

IX. Renovation of lawn in 5-6 years.

X. Replanting of hedges in 8- 10 years.

XI. Completely uprooting and removing hedges1 shrubbery.

XII. Replanting of
    a) Rose beds in 5-6 years.
    b) Perennial beds in 5-6 years.
    c) Cannal beds in 1-2 years.

XIII. Shifting of any garden feature from one site to another within building.

Quadrennial repair work

Besides annual repair work of white washing and colour washing , every fourth year special repair works are done for through repair as repainting of doors and windows, patch repair of plastering etc. Special repair work every fourth year is known as **Quadrennial Repair.**

Method of execution of works through the contractors

Contract and agreement

When two or more persons have common intention communicated to each other to create some obligation between them there is said to be an agreement. An agreement which is enforceable by law is a "**Contract**."

According to Section 10 of the Indian Contract Act, 1872 only thus agreements are enforceable by law which are made by the free consent of parties competent to contract, for a lawful consideration and with a lawful object and, are not expressly declared to be void. This is subject to any special law according to which a contract should be in writing and attested by witnesses.

The following are the essential ingredients of a contract:-

a) Offer made by one person called the "Promisor".

b) Acceptance of an offer made by the other person called the"Promisee".

c) Doing of an act or abstinence from doing a particular act by promisor for promisee called consideration.

d) The offer and acceptance should relate to something which is not prohibited by law.

e) Offer and acceptance constitute an agreement, which when enforceable by law, become a contract.

f) In order to make a valid and binding agreement, the party entering into such an agreement should be competent to make such agreement.

For the purpose of an agreement, there must be a communication of intention between the parties thereto. Hence in the forms of a Contract there is:

(a) A proposal.

(b) Communication of the proposal.

(c) A communication of the acceptance of the proposal.

The communication of acceptance of the proposal completes the agreement. An offer may lapse for want of acceptance or be revoked before acceptance. Acceptance produces something which cannot be recalled or undone. A contract springs up as soon as the offer is accepted and imposes an obligation upon the person making the offer. It has been opined by the Ministry of Law that before communication of acceptance of an offer the tenderer would

be within his right to withdraw, alter and modify his tender before its acceptance, unless there is a specific promise to keep the offer open for a specific period backed by a valid consideration.

Work order

Small work up to Rs. 2000.00 may be carried out by work order. This is a contract and specifies the approximate quantities of different items of work , details specification of each item of work, time for completion of the whole work , penalty that will be imposed for not fulfilling the terms and conditions etc. Payment is made on the measurement of work done and 10% of the bill amount is deducted from the running bill of the contractor as security money which amount is refunded in the final payment on the satisfactory completion of the work. Debitable agency can be engaged for bad work or for unsatisfactory progress. Cotractors are usually selected by taking quotations. (P.W.Agreement is used in P.W.D and Work Order is used in Irrigation Department)

Item rate contract

It is also knows as **Unit-price contract** or **Schedule contract.** For item rate contracts, contractors are required to quote rates for individual item of work on the basis of schedule of quantities furnished by the department. This schedule indicates full nomenclature of the item as per sanctioned estimate, estimated quantities and therein. While filling up the rates , the contractors are required to express the amount in figures and also to work out the cost against each item. The final total of the amount tendered for the work is also drawn up by them . This type of contract is followed by Railway Department.

Lump sum contract

As its name indicates, is used for work in which contractors are required to quote a lump-sum figure for completing the works in accordance with the given designs, drawings, specification and functional requirements as the case may be. Lump-sum tender can be either for only executing the work as per given design, drawing and specification or it may include element of doing design work and preparation of structural drawings as well which shall be in keeping with the given functional, structural and architectural parameters and subject to approval by the competent authority beforehand.

In cases where work is to be executed as per design and drawing of the department, all the detailed working drawings, both architectural and structural must be prepared before hand and should form part of the tender documents which should also contain complete and detailed specifications of the work. The tender documents must set out complete scope of the work. Only the drawings and the detailed specifications as contained and/or referred to in the tender documents shall form the basis of execution and payment. The extra payment or recovery over and above the accepted rate shall therefore be called for only in the event of authorized deviations from the drawings and specifications (as given and/or referred to in the tender documents) in course of execution and not otherwise.

In cases where the detailed architectural and structural drawings are to be provided by the Contractor, all the architectural and structural data/parameters necessary to work out the cost of the work, details of the functional requirement and complete/detailed specification thereof including preliminary drawings if any, must be finalised before call of tender and the tender documents must

contain all these details so that there is little scope of guess work on the part of the contractor while tendering and chances of dispute at later stage are minimised.

A condition should be stipulated in the tender documents that the work shall be executed as per detailed design and architectural/structural drawings to be prepared by the successful contractor conforming to the given parameters and functional/design requirements as enunciated in the tender documents and submitted to the department within specified time after the award of work. The contractor shall accordingly get the design/drawings approved by the department before taking up execution of the work. In case any modification for any reason is ordered in course of execution, suitable adjustment for extra payment or recovery shall be effected only if such modification in the tender documents or any change from the specified parameters.

Labour contract

In Labour contract the contractors undertakes contract for the labour portion. All material for the construction are arranged and supplied at the site of work by the department or owner, the labour contract engages labour and gets the work done according to specifications. The contract is on item rate basis for labour portion only and contractor is paid for the quantities of work done on measurement of the different item of work at the stipulated rate in the contract agreement. Materials for scaffolding, centering and shuttering and other similar materials are supplied by the department or owner ; contractor may also use his own materials for scaffolding, centering and shuttering, etc. if provided in the agreement. Contractor uses his own tools for working , but plants and machineries are arranged by the dept. Or owner. An agreement with all conditions of contract, rates bill of quantities (BOQ) etc. is prepared before the work is given out to the contractor. This system of contract is not generally adopted in the Govt. Deptt. . Private buildings are however by labour contract system which is less troublesome.

Daily labour

Work may be executed by departmentally by employing daily labour as masons, coolies , bhisties, carpenters, etc. The materials required for the construction as bricks, cement, sand lime, timber, steel etc. and tools and plants required for the operation are, got issued from the store by indent or purchased directly chargeable to the authorised agent as work-supervisor , misty, mate, etc. The attendance of labour is checked and initialled by Assistant Engineer or Sub- Divisional Engineer frequently during their inspections. The labour are paid weekly, fortnightly, monthly or at the completion of work according to the requirement.

Piece work agreement (P.W.A)

P.W.Agreement is that where only rates are agreed upon without reference to the total quantity of work or time , and that involves payment of work done at the stipulated rate. Small work or piece-work up to Rs. 2000.00 may be carried out through contractors by Piece work agreement. The P.W.Agreement contains only the descriptions of different items of works to be done and the rate to be paid for but does not provide the quantities of different items to be executed nor the time within which the work is to be completed. Detailed specification of the different items of work to be done are however included in the P.W.Agreement

and the total cost of the whole work to be done is also mentioned. Contractors have to arranged all materials, labours, etc., required for the execution of work., P.W.Agreement are not contracts in the true sense , there is no penalty clause and no security money and the department may terminate the work at any time they like but a notice specifying the date of termination should be served to the piece worker. Separate agency may also be engaged chargeable to the contractor to complete the work if the contractor does not carry out the work satisfactorily to the specification or delays the work or leaves the work incomplete or used bad materials. Urgent small work are selected by taking quotations. Rates of different item should be within schedule of rates or within sanctioned estimated rates . Payment is made on the measurement of the work actually done.

Scheduled contract or Item Rate contract:

In Schedule contract , the contractor undertakes the execution or construction of a work on the item rate basis. The amount the contractor is to receive depends upon the quantities of various items of work actually done. The contract agreement includes quantities, rates and amounts for various item of work and the total amount of contract (BOQ with rate , amount and total amount), plans and detailed drawings, detailed specifications and deposit of 10% security money; penalty, progress, date of completion and other conditions of contract. The payment to the contractor is made by detailed measurement of different items of works actually done by the contractor. The system is used for all works.

Item Rate Contract may also be a percentage above or below the printed schedule of rates of the department

Cost plus percentage contract

In this system contractor is given certain percentage over the actual cost of the construction as his profit. Contractor arranges materials and labours at his cost and keeps proper account and he is paid by the department or owner the whole cost together with certain percentage, say 10% as his profit as agreed upon beforehand. An agreement is prepared with all conditions of contract in advance . In this case proper control in the purchase of the materials and in labour shall have to be exercised by the department or owner.

Accounts of works Explanation of
various terms Administrative approval

For any work or project required by a department , an approval or sanction of the competent authority of the department, w.r.t the cost and work is necessary at the first instance. The approval authorise the engineering department to take up the work . Administrative approval denotes the formal acceptance by the department concern of the proposal , and after the administrative is given the engineer department (P.W.D) take up the work and prepare detailed designs, plans and estimates and then executes the work. The engineering department prepares approximate estimate and preliminary plans and submits to the department concerned for administrative approval.

Technical sanction

After receipt of administrative approval and expenditure sanction, detailed estimates are required to be prepared for technical sanction. As its name indicates, it amounts to no more than a guarantee that the proposals are

structurally sound and that the estimates are accurately calculated and based on adequate data.

Before an estimate is technically sanctioned, the following shall be available.

(i)   Detailed architectural drawings and specifications

(ii)   Structural drawings for foundations

(iii) Structural drawings of superstructure at least upto slab at level 2

(iv)   Detailed drawings of internal and external services.

Before according technical sanction to detailed estimates, the authority competent to accord such sanction shall ensure that the design and specification etc., of the building are rich enough to provide the desired life to it. In the 'Design and Scope' column of the estimate, it shall be specifically mentioned that, 'Under normal use and maintenance, the building is expected to have an economic life of   years.'

For various types of buildings, the economic life shall be taken as below:

(a) Monumental structures 100 years

(b) RCC framed structures 75 years

(c) Load bearing structures 55 years

(d) Semi permanent structures 30 years

The technical sanction should be given by the competent authority before a work is taken in hand. In case of revised estimates, it is not necessary to wait for the revised administrative approval or the revised expenditure sanction to accord revised technical sanction.

Contingency budget

A contingency budget is money set aside to cover unexpected costs during the construction process. This money is on reserve and not allocated to one area of the work, and simply "insurance" against other costs.

In deterministic methods, contingency is estimated as a predetermined percentage of base cost depending on the project phase. In this technique, you take a percentage of the cost of the project

and calculate the contingency amount.

The estimated costs of the known-unknowns is referred to by cost estimators as cost contingency. Contingency "refers to costs that will probably occur based on past experience, but with some uncertainty regarding the amount. The term is not used as a catchall to cover ignorance.

Tender

To tender is to invite bids for a project or accept a formal offer such as a takeover bid. Tendering usually refers to the process whereby governments and financial institutions invite bids for large projects that must be submitted within a finite deadline.

Before tenders for a work are invited a detailed estimate showing the quantities, rates and amounts of the various items of work and also the specifications to be adopted should be prepared. Before sanctioning, the draft of the detailed estimate, for works having involvement of Architect, should be sent to the Senior Architect to examine it vis-a-vis the specifications of various items provided by him.

In case of works for which tenders are to be invited, tender documents comprising of the following should be prepared and approved by an authority

who is empowered to approve the Notice Inviting Tenders (NIT) before notice inviting tender is issued.

I.   The notice inviting tender in Form PWD 6.

II. The form of tender to be used along with a set of conditions. Particular specifications and special conditions should not be repetitive and in contradiction with each other. Additional condition to be decided by NIT approving authority and he should be responsible for the same.

III. The schedule of quantities of work.

IV. A set of drawings referred to in the schedule of quantities of work.

V.   Specification of the work to be done.

Executive Engineer/Assistant Executive Engineer/Assistant Engineer issuing the tenders should invariably date and initial corrections, conditions and additions in the Schedule of Quantities, Schedule of Material to be issued and specifications and other essential parts of contract documents, and also date and initial on pages of the tender documents irrespective of fact whether they contain or do not contain any corrections or over writings etc. The officer concerned should record the fact in writing at the end of those pages individually.

Preparation of notice inviting tender

All notice calling for tenders should be in the standard form and be serially numbered, a proper register being maintained for the purpose. They should only be issued after the authority competent to accept the tender has approved the NIT papers. The notice inviting tenders should be carefully prepared, the use of symbols % and per thousand in the schedule of quantities accompanying the Notice Inviting Tenders is prohibited and the words hundred and thousand must be written e.g. "Per hundred sq. metre" must be written and not "% sq. metre". The units should thus be more specific.

In case of lump-sum tenders the Divisional Officer should see that detailed drawings and specifications duly authenticated by the competent authority form part of the notice inviting tenders and that the cost of various items forming part of the sanctioned estimate of the work is correctly assessed with reference to the relevant schedule of rates or in the case of non schedule items on the basis of rates supported by detailed analysis therefore, sanctioned by the competent authority.

The NIT papers are very important documents on which call of tenders and subsequent agreements with the contractors are based. It is, therefore, very necessary that each page and the correction slips as also other corrections and modifications made in the NIT papers are numbered and signed by the competent authority in token of approval so that all chances of tampering with such documents are avoided. Mere approval on forwarding letters would not serve the purpose. All corrections in the NITs and pages of the NITs approved by the Superintending Engineer and Chief Engineer should be attested by the Surveyor of Works. Thereafter the documents must be properly sealed to prevent any tempering.

It will be the responsibility of the Divisional Accountant to see that all forms issued to tenderers whether printed or otherwise, are clear, legible and unambiguous. The schedule of quantities attached to tender document other than Form PWD 7 must also contain a column for the "Amount" after the column

"Rate" and the contractor must calculate the amount of each item and enter it in the column. The Contractor must also total these amounts both by sub-head and give a grand total in words and figures both.

It will also be the duty of the Divisional Accountant to ensure that the tenders are issued to only those contractors who satisfy the eligibility criteria for issue of tenders as inserted in the Press Notice. He should properly scrutinise the applications received for issue of tenders keeping in view the eligibility criteria and then put up to the Executive Engineer for a decision.

The NIT for all works for which tenders are invited on PWD form 7 should provide that the Contractor should quote the percentage above or below to two places of decimal only.

The Notice inviting tender should stipulate reasonable time for completion of work. For building works, the Schedule of Contract period should be decided in accordance with Appendix 16 of this Manual by the NIT approving authority.

It should be ensured that a specific reference to the number of correction slips is made while mentioning the schedule of the rates or the CPWD specifications for works at Delhi, e.g. "Schedule of Rates .... for Delhi with correction Slips .... to .... " and "CPWD Specifications for works at Delhi .... with correction Slips .... to .... ". The name should include the year also.

Receiving of quotations

A quotation, or quote, is a document that a supplier submits to a potential client with a proposed price for the supplier's goods or services based on certain conditions. Therefore, a quotation is often required for services but is also commonly used by businesses that sell goods.

Earnest money

According to the practice in Central PWD, earnest money is paid by each tenderer to enable Government to ensure that a tenderer does not refuse to execute the work after it has been awarded to him. In case where a tenderer fails to commence the work awarded to him, the earnest money is absolutely forfeited to the President.

If only a part of the work as shown in the tender is awarded and the contractor does not commence the work, the amount of the earnest money to be forfeited to the Government should be worked out with reference to the estimated cost of the work so awarded.

Rates of Earnest Money

The amount of the earnest money which a contractor should deposit with the tender is regulated by the following scales. In case of petty works costing Rs. 5,000/- or less the Executive Engineer may, at his discretion, dispense with the conditions for calling for earnest money.

(i) For works estimated to cost upto   2% of the estimated cost .
Rs. Twenty five crores

(ii) For works estimated to cost more   Rs. fifty lakhs plus 1% of the excess of than Rs. twenty five crores estimated cost over Rs. twenty five crores.

Rules For Enlistment of Contractors in CPWD, 2001, do not provide for exemption of depositing earnest money with individual tenders by the contractors. Therefore, no exemption of earnest money against lump sum deposit is to be mentioned in the enlistment/revalidation orders.

Security deposit

The security deposit will be collected by deductions from the running bills of the contractors at the rate mentioned below and the earnest money, if deposited in cash at the time of tender, will be treated as part of security deposit. The security deposit will also be accepted in cash or in the form of Government Securities, Fixed Deposit Receipts. Performance security may be accepted as Bank Guarantee of Scheduled Banks and State Bank of India .

A sum @ 10% of the gross amount of the bill shall be deducted from each running bill of the contractor till the sum along with the sum already deposited as earnest money, will amount to Security Deposit of 5% of the tendered value of the work. In addition, the contractor shall be required to deposit an amount equal to 5% of the tendered value of the contract as Performance Security within the period prescribed for commencement of work in the letter of award issued to him.

Advance payment

This means payment made on a running account to a contractor for work done by him but not measured. Advance payment is not generally made to the contractor, but may be made under special case when the work is sufficiently progressed but measurement cannot be taken for certain valid reasons, on the certificates of Assistant Engineer in-charge of work that the value of work done is no case less than the advance payment made or proposed to be made and detailed measurement will be taken as soon as possible.

On account payment

Means a payment made on a running account, to a contractor in respect of work done or supplied made by him and duly measured. Such a payment may or may not be for the full value of work or supplied; if it is subject to the final settlement of running account on the completion of the contract for the work or supplies.

Intermediate payment

The term applied to a disbursement of any kind on a running account not being the final payment. It includes an "Advance payment", a "Secured advance" and an "on account payment" (other than the final payment on a running account) or a combination of these.

Final payment/ Final Bill

This means payment made on running account , made to a contractor on the completion or determination of his contract and in full settlement of the account. The bill on which final payment is made is known as **"Final Bill"**

Running bill

Denotes the account with a contractor when payment for work or supplies is made to him at convenient intervals subject to final settlement of the accounts on the completion or determination of his contract.

Regular and temporary establishment Cash

The term cash includes legal tender coins, notes, cheques payable on demand, remittance transfer receipts and demand drafts. A small supply of revenue stamps (required for a acknowledgement of receipts) may be kept as part of cash balance.

Major & subhead of account

- The main unit of classification in accounts shall be the major head which shall be divided into minor heads, each of which shall have a number of subordinate heads, generally shown as sub-heads. The subheads are further divided into detailed heads. Sometimes major heads may be divided into 'sub-major heads' before their further division into minor heads.

  - The Sectors, Major heads, Minor heads, Sub-heads and Detailed heads together constitute a five tier arrangement of the classification structure of Government Accounts.

  - The Major Heads corresponds to 'Functions' of the Government.

  - Minor Heads subordinate to the Major Head shall identify the 'Programme' undertaken to achieve the objectives of the function.

  - The sub head below the Minor Head represents various schemes or activities under the programme. Detailed Head is termed as object classification.

  - The detailed classification of account heads in Government Accounts and the order n which the Major and Minor Head shall appear in all the account records shall be such as prescribed by the Central Government from time to time on the advice of C&AG of India.

  - The 'List of Major and Minor Heads of Account of Union and States contains the classification prescribed in this regard.

  - The classification prescribed should be strictly followed.

Temporary advance

An accountable advance that substitutes for credit or other payment arrangements that would ordinarily be used under similar circumstances (e.g., advancing cash to a construction contractor for materials received on site instead of only paying for work that has already been erected, inspected, accepted, and invoiced).

Issue rate

Issue rate denotes cost per unit fixed on the article of stock for the purpose of calculating the amount creditable to the subhead concerned of stack account when issued from stock. An issue rate is fixed for each article of stock on the basis of actual cost plus other expenses including storage charges.

Storage charges

This means expenditure incurred on store materials after acquisition of stores, on work-charged establishment employed on handling and keeping initial accounts, the custody of stock and maintenance of store godown or yards etc. and added on a percentage basis of the cost , so as to form part of issue rate.

Supervision charges

This term is ordinarily applied to the charges which are levied, in addition to book value and storage charge(issue rate) , in respect to stock material sold or transferred and are intended to cover such item of expenditure incurred on the stores as do not enter in their book value and are not included in storages. When the stock materials are sold or transferred a certain percentage , about 10% is charged over issue rate as supervision charges which is meant for expenditure on regular establishment.

Suspense account

A suspense account is a general ledger account in which amounts are temporarily recorded. The suspense account is used because the appropriate general ledger account could not be determined at the time that the transaction was recorded.

Debit and credit

Debit means expenditure and credit means receipts. When an amount is to be debited to a work means that the amount is to be shown as expenditure on the work. Similarly when an amount is to be credited to a work it means that the amount is to be shown as receipt under the work.

Book transfer

A **book transfer** is the **transfer** of the legal right of ownership of an asset, without physically shifting the asset to the new owner. The most common use of the concept is when a bank **transfers** funds from the account of the payer to the account of the payee when both accounts are with the same bank.

Voucher and related accounts

A voucher is an accounting document representing an internal intent to make a payment to an external entity, such as a vendor or service provider.   A voucher is produced usually after receiving a vendor invoice, after the invoice is successfully matched to a purchase order.

   They are:

- Debit or Payment voucher.
- Credit or Receipt voucher.
- Non-cash or Transfer Voucher.
- Supporting Voucher.

Measurement book use & maintenance, procedure of marking entries of measurement of work and supply of materials, labour employed, standard measurement books and common irregularity

The measurement book is the basis of all accounts of quantities whether of works done by Contractors or by Labourers employed departmentally or materials received. It should be so written that the transactions are readily traceable.

These books should be considered as very important accounts records and maintained very carefully and accurately as these may have to be produced as evidence in a court of law, if and when required.

All the Measurement Books belonging to a Division, should be numbered serially. A register should be maintained in form CPWA 92 showing the serial number of each book, on receipt, Sub-Division to which it is issued, the date of

issue, date of its return to the Divisional Office and date of its record after the required review in the Divisional Office has been completed.

A similar register should be maintained in the Sub-Divisional Office showing the names of person i.e. Assistant Engineer/Assistant Executive Engineer and Junior Engineer whom the measurement books are issued.

The Books, no longer to be used in the Sub-Division or with the Junior Engineer should be with drawn promptly even though not completely written up and re-issued.

The Measurement Books are required to be reviewed by Divisional Accountant under the supervision of Executive Engineer. The Assistant Engineers are required to submit the Measurement Books in use in the Sub- Divisions to the Divisional Office, from time to time, so that at least once a year the entries recorded in each of the Books are subjected to a percentage check. The Divisional Officer should ensure that this annual review is conducted regularly and positively every year.

When an Assistant Engineer or Junior Engineer in charge of the work or stores is transferred he should hand over the Measurement Books issued to him to his successor and these should be shown as received back from him and reissued to the relieving Officer. The transfer should also be recorded in the Measurement Book after the last entry in each book under dated signature of the relieving Officer and relieved Officer. Recording of Measurement

Each set of measurements to be recorded should commence with entries stating:-

(i) In the case of bills for works done:
    a) Full name of work as given in the agreement/Estimate.
    b) Situation of work.
    c) Name of contractor.
    d) Number and date of agreement.
    e) Date of written order to commence work.
    f) Date of actual completion of work.
    g) Date of recording measurements.
    h) Reference to previous measurements.

(ii) In the case of bills for supply of materials:
    a) Name of supplier.
    b) Number and date of supply order/agreement.
    c) Purpose of supply in one of the following forms as applicable to the
    d) case.

(i) Stock (for all supplies for stock purpose).

(ii) "Purchase" for direct issue to the work (full name of the work as given in the estimate may be mentioned).

(iii) "Purchase" for (full name of work as given in estimate) for issue to contractor ....on
    (d) Date of written order to commence the supply.

(e) Date of actual supply; and

(f) Date of recording measurements.

A suitable abstract should then be prepared which should collect in the case of measurement for works done, the total quantities of each distinct item of work relating to each sanctioned sub-head. The measurement books meant for this

purpose contain pages in singleton. Details of quantities, rate and amount of each item for every bill are entered in this Measurement Book in a tabular form.

For recording measurements and also for preparing abstract, the agreement item No. both in words as well as in figure should be given neatly, instead of writing the description of the item in full or in abbreviated form which would not be necessary.

In case of extra/substituted item of work that is not covered in the agreement, the full nomenclature shall be reproduced in the M.B. and the bill form.

The full nomenclature of the items shall be adopted in preparing abstract of final bill in the measurement book and also in the bill form for final bills.

If the measurements are taken in connection with a running contract, a reference to the last set of measurements, if any, should be recorded. If the entire job or contract has been completed, the date of completion should be duly noted in the prescribed place. If the measurements taken are the first set of measurements on a running account, or the first and final measurements, this fact should be suitably noted against the entries in the Measurement Book and in the latter case, the actual date of completion should be noted in the prescribed place.

All measurements should be recorded neatly in the Measurement Book. The signature of the contractor or his authorised representative should be obtained in the measurement book for each set of measurements.

Muster roll : Its preparation & use for making payment of pay & wages

Muster Roll is used for keeping a complete record of attendance, payment made , un-paid wages and work done by daily labour engaged on the execution of works. It is the basic records of payment made to daily labour. After the payment is made ,the Muster Roll is kept as a Voucher.

Muster rolls should be prepared and dealt with in accordance with the following rules :

1. One or more muster rolls should be kept for each work, but muster rolls should never be prepared in duplicate. It is permissible, however, to keep one muster roll for labourers employed upon several small works, in cases in which no harm can result if the total unpaid wages are regarded as relating only to the largest work in the group.

2. Labourers may be paid more than once a month and the period covered by each payment may be determined locally; but separate rolls must be prepared for each period of payment.

3. The daily attendances and absences of labourers and the fines inflicted on them should be recorded daily in part I of the muster roll in such a way as —

   (i)   to facilitate the correct calculation of the net wages of each person for the period of payment;

   (ii)   to render it difficult to tamper with or to make unauthorized additions to or alterations, in entries once made, and

   (iii) to facilitate the correct classification of the cost of labour by works and sub-heads of works where necessary.

4. After a muster roll has been passed by the local officer, payment thereon should be made as expeditiously as possible. Each payment should be made or witnessed by the official of highest standing available, who should certify to the payments individually or by groups, at the same time specifying both in words and in figures, at the foot of the muster roll, the total amount paid on each date. If any items remain unpaid, the details thereof should be recorded in part II of the register of arrears, before the memorandum at the foot of the muster roll is completed by the person who made the payment.

5. Unpaid items should subsequently be carried forward from muster roll to muster roll until they are paid, the payments being recorded and certified in part II in the same way as payments of current items. It is optional, however, with the local officer to adopt any other alternative method of making payments of unpaid wages, provided that a systematic record of items remaining unpaid is maintained on the basis of the original entries made in part II of the muster roll and that suitable precautions are taken to prevent double payments.

6. Wages remaining unpaid for three months should be refunded into Treasury.

7. The payment of daily labour through a contractor instead of by muster roll in the usual way, is objectionable in principle. In a case of great emergency it may sometimes be found impossible to employ labour otherwise than through a contractor. Should it be possible in such a case, to determine the quantities of work done after its completion or at intervals during its progress, it is expedient to pay the contractor, at suitable rates, on the basis of work actually executed. To avoid disputes with the contractors, they should be encouraged to sign the daily reports in token of their acceptance as correct.

   N.B.—The use of the muster roll is not permissible in such cases.

8. When it is necessary to bring labourers and artificers from a distance they may be allowed wages for the number of days occupied in the journey to and from the site of the work, if they join the work with proper despatch. At the discretion of the local officer, bona fide travelling expenses may also be allowed to them. The above charges must be borne by the estimate of the work.

Acquittance Roll : Its preparation & use for making payment of pay & wages Labour & labour report, method of labour payment, use of forms and necessity of submission

The payment of salary to persons of regular establishment working outstation is drawn on the regular pay-bill , but payment is made on a separate receipt form known as **Acquittance Roll ,** after taking duly stamped signature of the person. The Acquittance Roll is a receipt in evidence of payment in a prescribed form having five columns as Item No. , Name , Designation, Net amount payable and Date signature. The Acquittance Roll

is prepared for the total amount as per Establishment Bill are passed the Drawing Officer. After the payment has been made the paying officer returns it after certifying that proper receipt(signature) has been taken from the person entitled to receive payment, which is then attached to the original Establishment Bill as a record of payment.

Classification of stores, receipt / issue statement on standard form, method of preparation of stock account, preparation and submission of returns, verification of stocks, shortage and excess

The stores of the W.D. are divided into the following classes:-

(i) **Stock –** Consumable materials like cement, steel, pipes, paints, spare parts of machinery, P.O.L (Petroleum, Oil, & Lubricants)., tyres, tubes etc. fall in this category.

(ii) **Tools and Plants.-** Such equipments which can be shifted from one work site to another work site as and when required for theconstruction activities fall under this category e.g., spades, pickaxes, vehicles, road rollers, drilling rigs, concrete mixer /vibrator, compressor, jack hammer etc**.**

(iii) **Road materials. –** Metal, moorum, gravel etc. fall in this category.

(iv) **Material charged direct to work. –** Materials, which are accounted for in "Materials at Site Account" fall in this category. Also the machinery which shall be fixed or embedded at one place permanently shall fall in this category e.g. electric motor or pump to be fixed in pump house, electric switches etc.

(v) **Materials charged to office contingencies.–** Stationery, furniture, typewriters, calculators, duplicating machines, copying machines, air conditioners, air coolers, water coolers, office cycle, three wheelers, blankets, warm clothing etc. fall in this category.

Receipt / issue statement on standard form

(i) The indent books should be machine-numbered and kept in stock by the Executive Engineer, In-charge of Central Stores. He issues these indent books stamped with the stamp of his office to the various indenting Divisions and only the indents issued from such books are accepted by the Central Stores Divisions.

(ii) The Executive Engineer in charge of the work is required to send three sets of the specimen signatures of the Junior Engineer and Assistant Engineer in charge of the work at the work site and that of the work Assistant, if any, authorised by the Executive Engineer for receiving stores in the Junior Engineer's absence, duly attested by him to the Central Stores. One set there of will be kept by the Junior Engineer security and the other two sets will be in the Office under the personal custody of the Assistant Engineer (Indents) and Executive Engineer, Central Stores Division.The specimen signature of the EE shall be attested by the outgoing EE. For new Division, it is to be done by the SE.

(iii) The signatures of the contractors or their authorised Agents to whom the materials are required to be issued are attested by the Asstt. Engineerand

the indent is countersigned by the Executive Engineer of the indenting Division.

(**iv**) The Contractors are required to obtain identity photopasses for themselves or their authorised agents, duly attested by the Executive Engineer of the Indenting Divisions and of the Central Stores Division. On the photopass or identity card, the name of the work, for which the contractor or his agent is authorised to draw the materials is mentioned and the contractor or his agent is allowed to draw materials only for that work.

(**v**) While sending the photographs of the Contractors or their authorised agents for drawal of the materials from the Central Stores, the following certificate should be recorded by the Executive Engineer of the Indenting Division :—

Shri .... whose signature and photograph are attested by me is an authorised agent of Messers .... The indent placed on the Central Stores in the name of the above mentioned contractors for the works being carried out by them under this Division may please be honoured and the materials handed over to the authorised representative, i.e. Shri .... His photograph duly attested and his specimen signatures are given underneath.

(**vi**) The authorisation letter with identity cards duly signed and attested should be sent through the authorised Junior Engineer of the Division in a closed cover in the name of the Executive Engineer, Central Stores Division so that there may be no change or tampering with the original identity card etc.

(**vii**)   The indent shall be presented at the Central Stores within 15 days of EE signing the same.

(**viii**)   The EE shall have the option to send advance payment for every indent or a lump sum advance for 2-3 months.

Verification of stocks

E.E. should have store verified throughout his Division at least once a year. It is not necessary that all the stores of a Division or of a Sub-Division should be checked and counted at the same time, but the dates on which articles are checked should be entered in the store returns. Stores should be counted by an officer not below the rank of an S.D.O. E.E. should ensure confidential check of store by selecting articles for check by S.D.O. monthly and obtain the result. In addition A.E. attached in Circle Office shall also carry out checking of stores atleast once in a year or as and when directed bythe S.E. The Office Superintendent of E.-in-C./C.E./S.E, Head Clerk/establishment clerk in E.E.'s office and Sub-Divisional Clerk in S.D.O's office shall maintain the numerical account of furniture of his office. The physical verification will also be conducted by him and results intimated to Head of the Office.

GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING

GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

Railway & Bridge Engineering

For 5th Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)

Prepared By
Sasmita Kumari Das
(Lecturer in civil Engineering)

R&B

CONTENT

SECTION A

RAILWAY ENGINEERING

INTODUCTION

Indian Railways is an Indian state-owned enterprise, owned and operated by the Government of India through the Ministry of Railways. It is one of the world's largest railway networks.

Railways were first introduced to India in the year 1853 from Bombay to Thane , nationalized as one unit, the Indian Railways, becoming one of the largest networks in the world. IR operates both long distance and suburban rail systems on a multi-gauge network of broad, metre and narrow gauges. It also owns locomotive and coach production facilities at several places in India and are assigned codes identifying their gauge, kind of power and type of operation.

On 23 April 2014, Indian Railways introduced a mobile app system to track train schedules.

Rolling stock used on railways in the earliest days evolved from carriages and wagons which ran on highways to carry both people and bulk materials.

As railway experience was gained, the design of rolling stock also evolved. Springing, body structure, wheels and axles all are subject to varying loads and stresses, when comparing slower speeds on rough roads to much faster speeds on railways, with a comparatively smoother ride.

Railway rolling stock generally runs on hard wheels on hard rails. The wheels are not only supported by the rails but are guided by them. The only exception to this is for a small number of metros where rubber tyres have been introduced. In this case the supporting function of the rail may be separated from the guiding function.

In all cases railway rolling stock will transmit vertical, horizontal and longitudinal forces to the track and its supports. Most railways have adopted twin rails and flanged wheels. Forces are transmitted to the rail structure either by direct bearing on the rail top from the wheel tyre, or by bearing laterally through the flange, or by longitudinal friction. Potential 'overturning' forces, caused by centrifugal force on curves, coupled with wind forces on exposed locations are resisted by vertical dead weight and super-elevation or 'cant' on curves.

Advantages of Railways

The railways offer various advantages and for the purpose of convenience, they can be described in following three categories.

1. Economic Advantages
   i)   Employment to people in the form of staff required for smooth working of railways.
   ii)   Encouragement to commercial farming.
   iii) Increase in cost of land thereby increase of national wealth.
   iv)   Industrial development and growth because of mobility of labour and raw materials.

    v)   Stabilization of prices due to easy, speedy & efficient mobility of products & natural resources.

    vi)   Increase in mobility of people and thereby relieving some extent the congestion of bigcities.

    vii)   Transporting food and clothes in times of emergencies like floods and famines etc.

2. Political Advantages

    i)   Easy control of the central administration.

    ii)   Development of a national mentality in the minds of people.

    iii)   Migrating population on a mass scale.

    iv)   Mobilizing troops and war equipment in time of war and emergencies.

    v)   Unity of people of different castes, customs and religions.

3. Social Advantages

    i)   Broadening the social outlook of masses as people can visit all the parts of country and be proud of this great country.

    ii)   Easy access to religious places of importance.

    iii)   Providing convenient and safe mode of transport.

    iv)   Removal of feeling of isolation as the railway has proved to be the most safe, economic and comfortable mode of conveyance.

Classification of Indian Railways

Indian railway systems according to three classes:-

Class I-   Railways with gross annual earnings of over Rs 50 lakhs (Rs 50,00,000).[1]

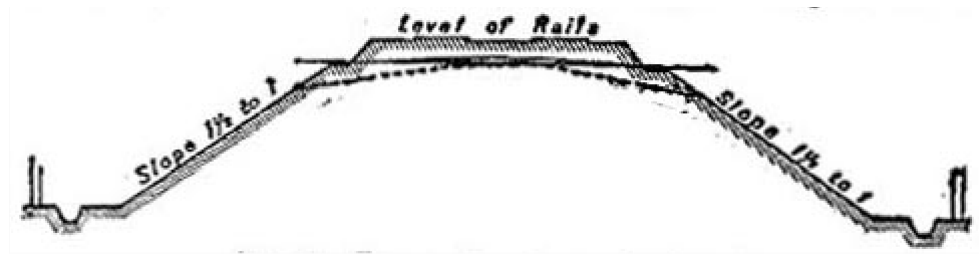Class II - Railways with gross annual earnings of between Rs 10 and 50 lakhs.

Class III - Railways with gross annual earnings of under Rs 10 lakhs.

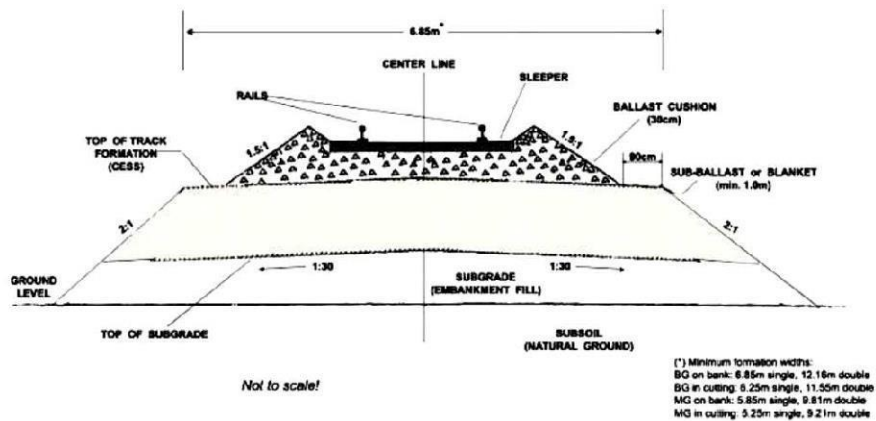# CHAPTER-2

Permanent way
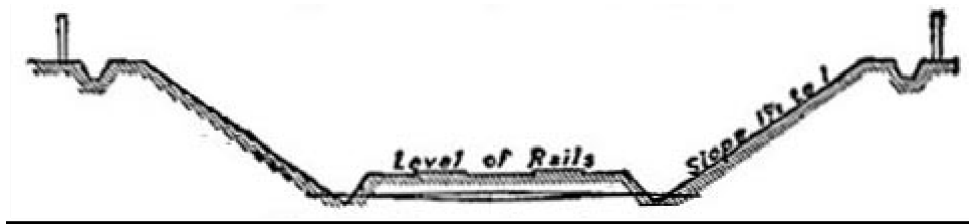
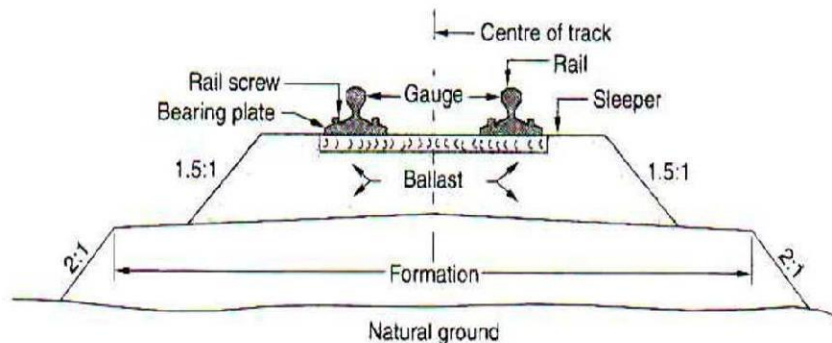

Typical Section of Railway in cutting.

Typical Section of Railway in embankment.

Cross section of a permanent way:

Basic functions of various components of a railway track:

The finished or completed track of a railway line is commonly known as Permanent Way. It essentially consists of following three parts.

1. Rails
2. Sleepers
3. Ballast

The rails are fixed with each other by means of various rail fastenings and they rest on sleepers which are laid at right angles to them. The sleepers in turn rest on ballast which is spread over the formation ground prepared for the railway track.

Requirements of an ideal permanent way

1. The gauge of track should be uniform and there should not be varying gauges.
2. There should be minimum friction between the wheels of rolling stock and therails.
3. Facilities should be provided at various points along the permanent way to repair, replace or renew the damaged portion of the track.
4. The design of the permanent way should be such that the load of the train is uniformly distributed over it.
5. The components of the permanent way should be so selected as to produce a permanent way with a certain degree of elasticity to prevent the shocks due to impact.
6. The gradient provided on the permanent way should be even and uniform.
7. The special attention should be given on the design of permanent way on curves.
8. The overall construction of the permanent way should be such that it requires minimum maintenance.
9. The permanent way should possess high resistance to damage at the time of derailment.
10. The drainage facility should be perfect.
11. The rail joints should be properly designed and maintained.
12. The precautions should be taken to avoid the occurrence of creep.
13. Various components of the permanent way should possess anti-sabotage and anti-theft qualities.

Rail Gauges

In India, the gauge of a railway track is defined as the clear minimum perpendicular distance between the inner faces of the two rails.
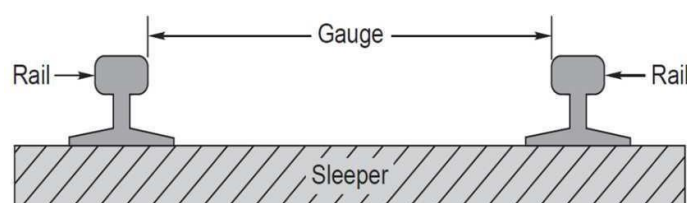
Gauge
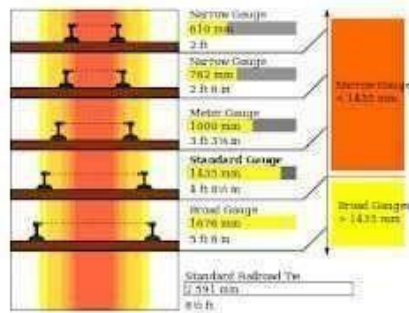
The different gauges used in India can be broadly classified as following four types.

1. Broad Gauge : Width 1676 mm to 1524 mm.
2. Standard Gauge : Width 1435 mm and 1451 mm.
3. Meter Gauge : Width 1067 mm, 1000 mm and 915 mm.
4. Narrow Gauge : Width 762 mm and 610 mm.

Suitability of these gauges under different conditions

1. Traffic condition ◎ If the intensity of traffic on the track is likely to be more, a gauge wider than the standard gauge is suitable.
2. Development of poor areas ◎ The narrow gauges are laid in certain parts of the world to develop a poor area and thus link the poor area with the outside developed world.
3. Cost of track ◎ The cost of railway track is directly proportional to the width of gauge. Hence, if the funds available is not sufficient to construct a standard gauge, a meter gauge or a narrow gauge is preferred rather than to have no railways at all.
4. Speed of movement ◎ The speed of a train is a function of the diameter of wheels which in turn is limited by the gauge. The wheel diameter is usually about 0.75 times the gauge width and thus, the speed of a train is almost proportional to the gauge. If higher speeds are to be attained, the B.G. track is preferred to the M.G. or N.G. track.
5. Nature of Country ◎ In mountainous country, it is advisable to have a narrow gauge of track as it is more flexible and can be laid to a smaller radius on the curves. This is the reason why some important railways, covering thousands of kilometers, are laid with a gauge as narrow as 610mm.
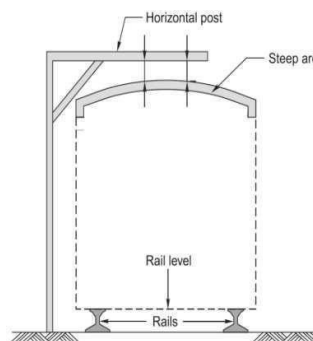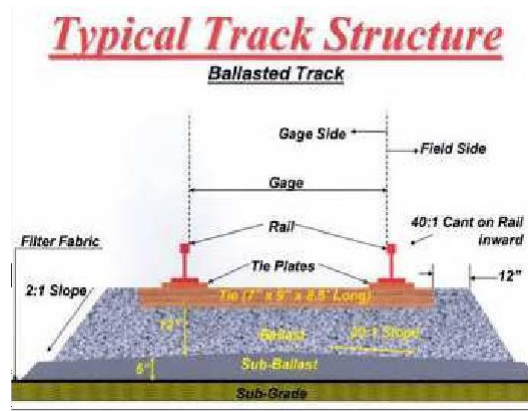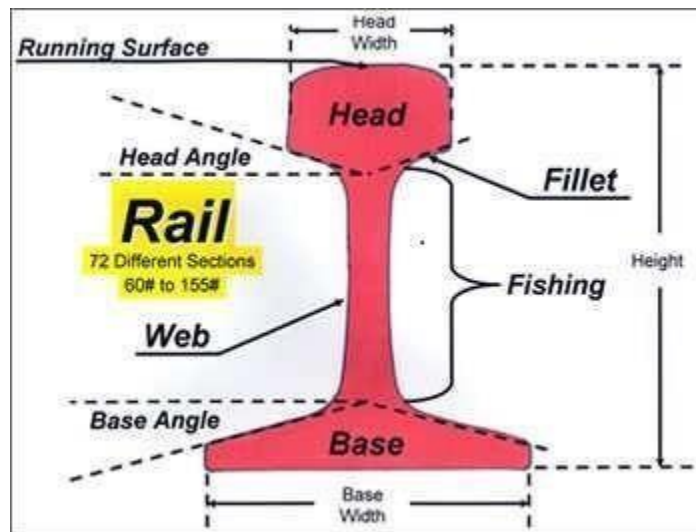

Fig. 2.2 Loading gauge

CHAPTER-3

TRACK MATERIALS

RAILS

Function of Rails

● To transmit the moving loads to the sleepers

● To provide strong, hard and smooth surface for the train journey.

● To bear the stresses developed in the track due to temperature changes and loading patterns.

● To serve as lateral guide to the running wheels.

● To resist breaking forces caused due to stoppage of trains.

Requirements of an Ideal Rail

1. The rail section consists of three components: head, web and foot. It should be designed for optimum nominal weight to provide for the most efficient distribution of metal in its various components.

2. The bottom of head and top of the foot should be given such shapes that fish-plates can easily be fitted.

3. The C.G. of the rail section should be located very near to the centre of height of rail so that maximum tensile and compressive stresses are more or less the same.

4. The depth of head of rail should be sufficient to allow for adequate margin of vertical wear.

5. The rail should possess adequate lateral and vertical stiff nesses.

6. There should be balanced distribution of metal in the head, web and foot of rail so that each of them is able to fulfill its assigned function.

7. The surface of rail table and gauge face of rail should be hard and should be capable of resisting wear.

8. The thickness of web of rail should be sufficient to take safely the load coming on therail.

## TYPES OF RAIL SECTIONS

Double headed rails:

These were the rails which were used in the beginning, which were double headed and consisting of a dumb-bell section. The idea behind using these rails was that when the head was worn out in course of time, the rail can be inverted and reused. But as time passed indentations were formed in the lower table due to which smooth running over the surface at the top was impossible.
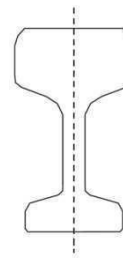
Bull headed rails:

In this type of rail the head was made a little thicker and stronger than the lower part by adding more metal to it, so that it can withstand the stresses.
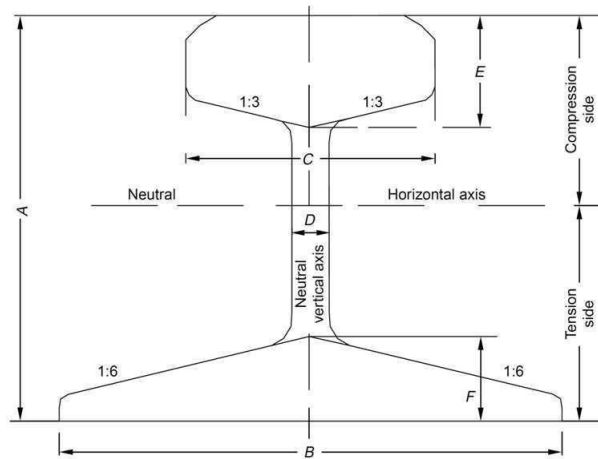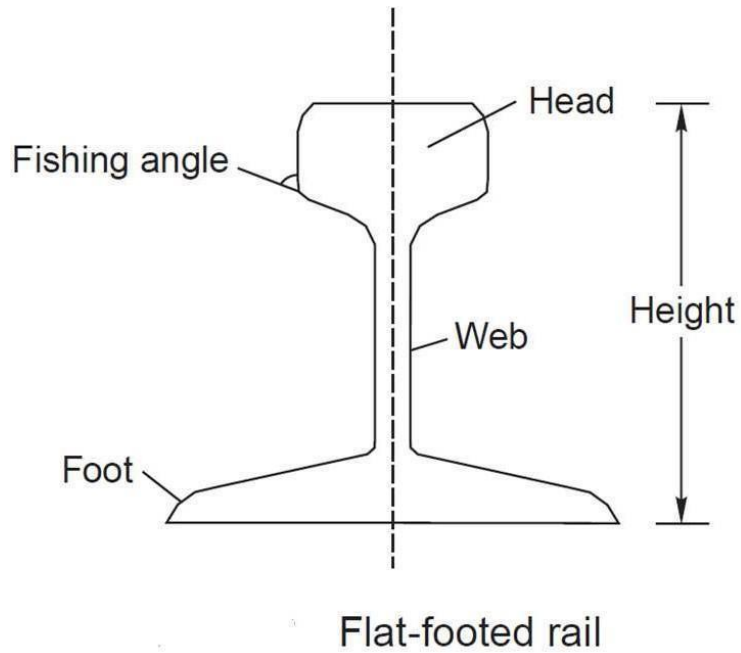
Flat footed rails:

These rails are also called as vignole's rails. Initially the flat footed rails were fixed to the sleepers directly and no chairs and keys were required. Later on due to heavy train loads problems arose which lead to steel bearing plates between the sleeper and the rail. at rail joints and other important places these are the rails which are most commonly used in India.
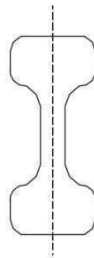
Double headed rail   Bull headed rail

Flat-footed rail

Standard flat-footed rail section

Length of the Rails:

From the consideration of strength of the track maximum possible length is advisable as it will reduce the number of the joints, less number of fittings and fixtures and economical maintenance. But in practice the following factors are considered to decide the length of rails.

i) Ease of transportation

ii) Reasonable cost of manufacture

iii) Ease in loading into the available wagons

iv) Development of temperature stresses

Indian Railways have adopted the following length of rails in practice.

i) For BG tracks = 13 m (42')

ii) For MG & NG tracks = 12 m (39')

Rail Joints

Rail joints are necessary to hold the adjoining ends of the rails in the correct position, both in the horizontal and vertical planes.

Types of Rail joints ⓞ The following are the type of rail joints

Supported rail joints:

When the rail ends rest on a single sleeper it is termed as supported joint. The duplex joint sleeper with other sleepers is an example of the supported joint.

Suspended rail joint:

When rail ends are projected beyond sleepers it is termed as suspended joint. This type of joint is generally used with timber and steel through sleepers.
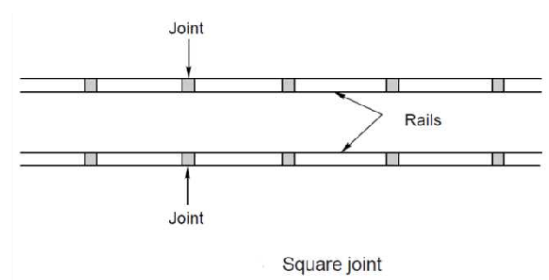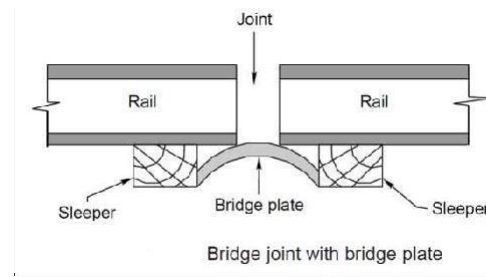
Bridge joints:

When the rail ends are projected beyond sleepers as in the case of suspended joint and they are connected by a flator corrugated plate called as bridge plate it is termed as a bridge joint.

Insulated joint:

When an insulating medium is inserted in a rail joint ot stop the flow of current beyond the track circuited part then that type of joint is called an insulated joint.

Compromise joint:

Where two different rail sections are required to be joined together it is done by means of fish plated which fit both rails and this joint is termed as compromise joint.

Supported rail joint

Suspended joint

Bridge joint with metal flat

Bridge joint with bridge plate

Square joint

Staggered joint
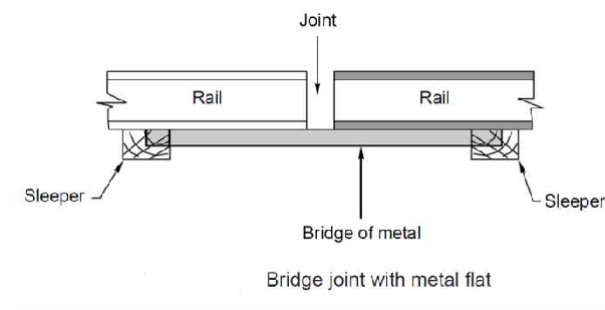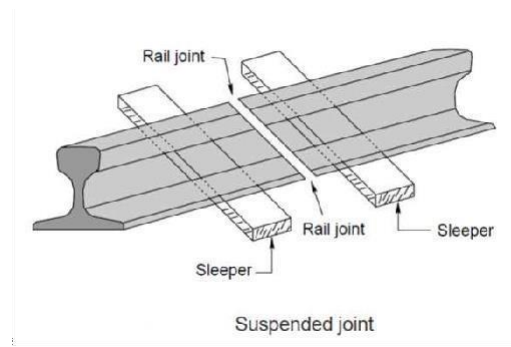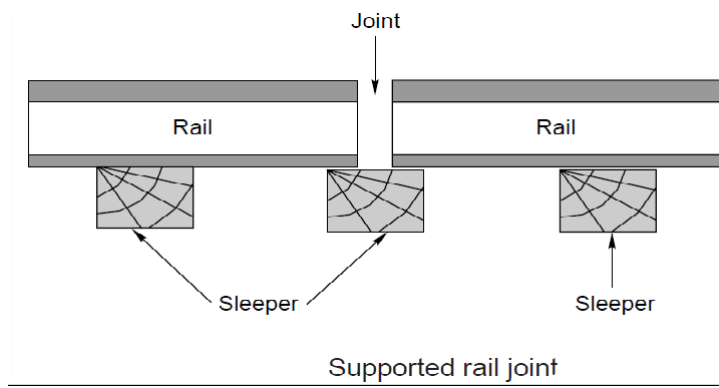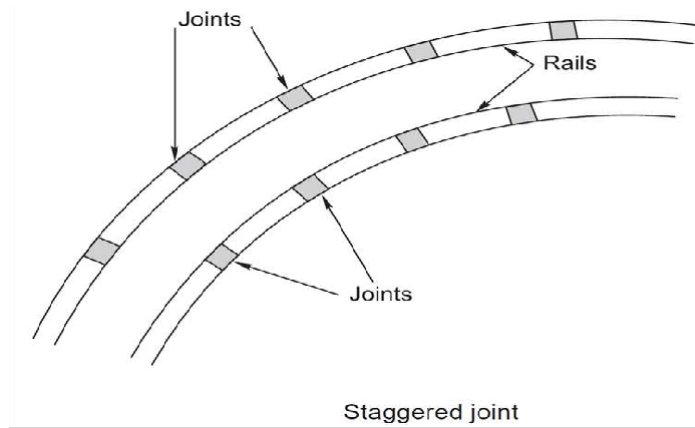
Requirements of an Ideal Rail Joint

i)   Ends to remain true

ii)   Should be strong and stiff

iii)   Provision for expansion

iv)   Perfectly elastic

v)   Ends should not get battered

vi)   Provision for easy removal and replacement

vii)   Economical in cost

Purpose of Welding of Rails

Welded rail joint

Purpose of welding:

- To increase the length of the rails
- To repair the worn out or damaged rails
- To build up worn out points and rails on the sharp curves

Advantages of welding rails:

- Reduces the creep and friction due to increase in length of rail.
- Expansion effect due to reduction in temperature.

- Increase the life of the rails due to decrease in wear.
- It facilitates track circuiting on electrified tracks.
- The cost decrease because as the length increase automatically the number of joints decreases.
- High frequency vibrations due to heavy moving loads are decreased due to the heaviness of the rails.

CREEP

Creep in rails denotes longitudinal movement of rails in the track.
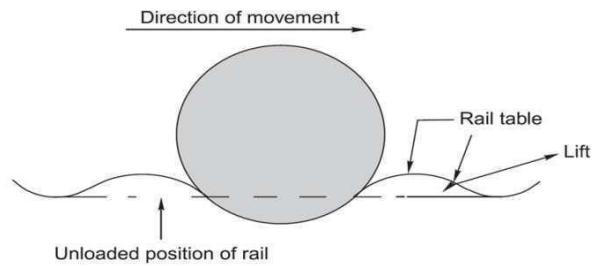
Wave motion theory for development of creep

Percussion theory for development of creep

The following figure shows a type of Fair V-anchor which is used in the track to prevent creep of rails.

Fair V anchor

Direction of Motion

Direction of Creep

Train Starting Point
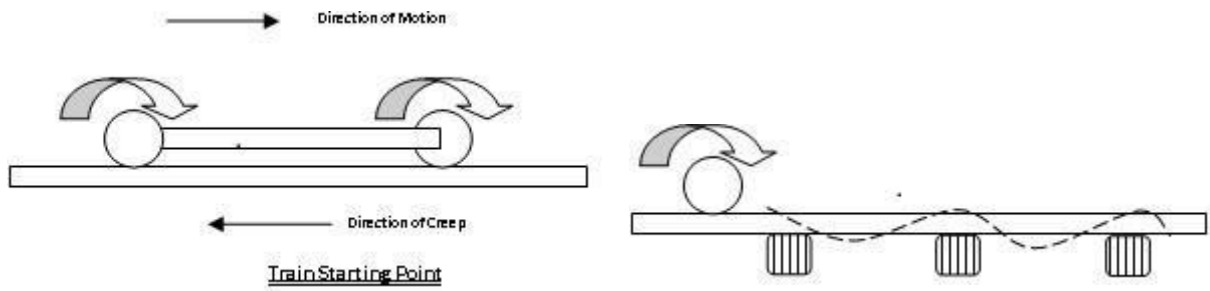
It is defined as the longitudinal movement of rails with respect to sleepers in a track.

Causes of creep:

1. Closing of successive expansion spaces at rail joints in the direction of creep and opening out of joints at the point from where the creep starts.
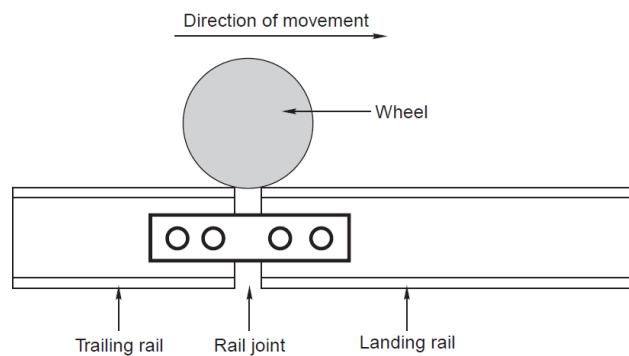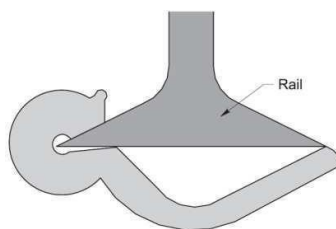2. Marks on flanges and webs of rails made by spike heads by scraping or scratching as the rails slide.

Effects of Creep:

1. Sleepers move out of position which leads to the change in gauge and alignment of the track.
2. Rail joints are opened out of their limit in some case and stresses are set up in fish plates and bols which leads to the breakage of the bolts.
3. Pints and crossings get disturbed.
4. Maintenace and replacement of tracks becomes difficult.
5. Smashing of fish plates and bolts, bending of bars, kinks at joints are other effects of creep.

Sleepers &Ballast

Definition:

Sleepers (known in the US as railroad ties) are used as a base for laying railway tracks. Sleepers were traditionally made from wood but are now usually made from concrete.Ties are laid on top of sand, gravel or heavy crushed stone - called ballast.

Functions of sleepers

The functions of sleepers are as follows

1. To hold the rails in correct gauge i.e. exact gauge in the case of straight and flat curves, loose gauge in the case of sharp curves and tight gauge in the case of diamond crossings.
2. To act as an elastic medium between the ballast and rail to absorb the blows and vibrations of moving loads.
3. They also add to the longitudinal and lateral stability of the permanent track on the whole.
4. They also provide means to rectify track geometry during service life.
5. To support the rails at proper level in straight tracks and at proper super elevation on curves.
6. To distribute the load from the rails to the index area of ballast underlying it or to the girders in the case of bridges.
7. To hold the rails in proper level.

Requirements of Good Sleepers

1. It should be economical
2. It should be minimum possible initial and maintenance costs.
3. The fittings should be such that they can be easily adjusted.
4. Weight should not be too heavy or too light.
5. It should not be easily damaged by anti-Nationals.
6. They should be capable of resisting shocks and vibrations.
7. The insulation of rails should be possible for track circuiting, if required through sleepers.
8. It should be designed that it could not be disturbed from its position easily by the passage of trains.

Classification of Sleepers

Sleepers can be classified into the following types according to the material of construction.

1. Wooden sleepers

2. Metal sleepers

- Cast iron sleepers

CI pot sleeper (dimensions in mm)
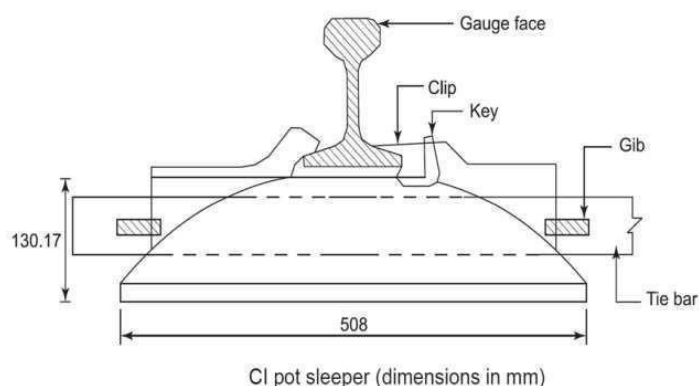
3. Steel sleepers

4. Concrete sleepers

- Reinforce concrete sleepers
- Prestressed concrete sleepers

1. <u>Timber or Wooden Sleepers</u>

Wooden sleepers are regarded as the best as they satisfy almost all the requirements of a good sleeper. The life of timber sleepers depend upon their ability to resist

a. Wear and tear
b. Decay
c. Attack by white ants
d. Quality of the timber used

Advantages:

It is easily available in all parts of India

Fittings are few and simple in design

They are easy to lay, relay, pack, lift and maintain

They are suitable for all types of ballast

They are able to resist the shocks and vibrations of the heavy moving loads

They are economical

Disadvantages:

They are subjected to decay, attack by white ants, warping, cracking and end splitting

It is difficult to maintain the gauge in the case of wooden sleepers.

It is difficult to maintain the alignment in the case of wooden sleepers.

They have got minimum service life (12 to 15 yrs.) as compared to other types of sleepers

2. **Cast Iron sleepers**

Cast iron sleepers are widely used in the world especially in Indian railways. They are available in 2 types, pot type sleepers and plate type sleepers. Pot type sleepers are not suitable for curves sharper than 4 degrees. Cast iron plate type sleepers are more famous in Indian railways.

Advantages:

1. Service life is very long
2. Less liable to corrosion
3. Form good track for light traffic up to 110 kmph as they form rigid track subjected to vibrations under moving loads without any damping
4. Scrap value is high

Disadvantages:

1. Gauge maintenance is difficult as tie bars get bent up
2. Not suitable for circuited track
3. Need large number of fittings
4. Suitable only for stone ballast
5. Heavy traffic and high speeds (>110kmph) will cause loosening of keys and development of high creep

3. Concrete Sleepers

Concrete sleepers are made of a strong homogeneous material, impervious to effects of moisture and unaffected by the chemical attacks. It is moulded easily to size and shape required and it is an ideal material to with stand stresses introduced by fast and heavy traffic.

Advantages

1. The concrete sleepers are quite heavy and thus provide longitudinal, lateral and vertical stability. Because of their weight, these sleepers are more suitable to LWR(Long Welded Rail) tracks.
2. The concrete sleepers result in reduced rail bending stresses.
3. The concrete sleepers reduce the wear of rolling stocks.
4. The concrete sleepers produce less vertical motion and thus provide a comfortable journey due to less noise.
5. The concrete sleepers have flat bottoms. That's why mean modern method of track maintenance i.e. MSP (Measured Shovel Packing) and machine maintenance can be suitably employed.
6. The concrete sleepers are neither inflammable nor subjected to damage by corrosion ortermite.
7. These sleepers have a long useful life of 50 years. It means rail and sleeper renewals can be matched.
8. The concrete sleepers with their fastening system maintain gauge, cross levels, twist, alignment, longitudinal level and unevenness of the track.
9. The concrete sleepers are suitable for track circuiting.
10. The concrete sleepers can be manufactured from local resources.

Disadvantages

1. They are not economical because of high cost of construction.
2. In case of derailments, heavy damage is caused.
3. High standard of maintenance of track is required.
4. The design and construction are both complicated.
5. They are more rigid.
6. They do not have any scrap value.

4. Steel sleepers

ST sleeper with pressed-up lugs
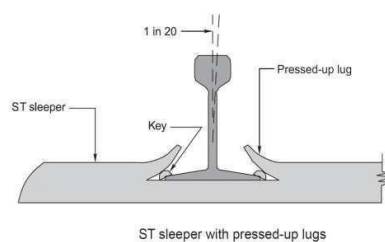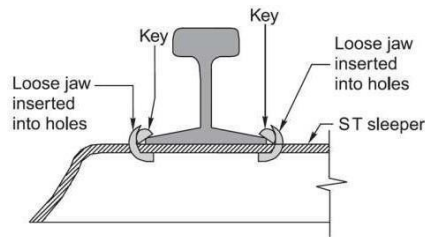
Sleeper with loose jaws inserted into holes

Steel sleepers are lightweight, dimensionally more accurate than wooden or concrete and regarded as an effective technical solution for modern rail networks.

Advantages of steel Sleepers

1. They are manufactured by a simple operation.
2. They can be easily handled as these are light in weight as compared to other types of sleepers. Hence, damages during handling and transporting are less.
3. Less number of fastenings are required and that too simple in nature.
4. The maintenance and adjustment of gauge are easy as compared to the other of sleepers.
5. These sleepers are rolled sections in one piece.
6. Their life is longer than that of other types of sleepers.
7. They provide better lateral rigidity to the track.
8. They are not attacked by vermin.
9. They are not susceptible to fire hazards.
10. Their scrap value is good.

Disadvantages of Steel of Sleepers

1. The steel sleepers possess the following disadvantages:
2. They get easily rusted and corroded.
3. They develop cracks at rail seats or near lugs.
4. Their lugs get broken easily.
5. The steel sleepers do not provide effective track circuiting.
6. The steel sleepers can only be for the type of rails for which theses are manufactured.
7. These develop the tendency to become center bound because of slope at both ends.
8. The overall cost of steel sleepers is more than that of timber sleepers.

Ballast

Definition:

It is a layer of broken stones, gravel or any other such gritty material laid and packed below and around sleepers.

Functions of ballast:

  a. To distribute the loads uniformly over the subgrade.
  b. To provide good drainage for the track structure.
  c. To provide elasticity and resilience to track for getting proper riding comfort.
  d. To held the track structure to line and grade.
  e. To reduce dust.
  f. To prevent growth of brush and weeds.

Requirements of Good Ballast

  1. It should be tough and should not crumble under heavy loads.
  2. It should not make the track dusty or muddy.
  3. It should offer resistance to abrasion and weathering.
  4. It should not produce any chemical reaction with rails and sleepers.
  5. The materials should be easily workable.
  6. It should retain its position and should not be distributed.

Materials used as Ballast

1. Broken Stone ◎ Broken stone is one of the best materials for railway ballast to be used on the railway tracks. Almost all the important railway tracks are provided with broken stone. The stone to be used as railway ballast should be hard, tough nonporous and should not decompose when exposed to air and light. Igneous rocks like quartzite and granite forms the excellent ballast materials. When these are not available then lime stone and sand stone can also be used as good ballast material.

 Advantages of Broken Stone

 1. It holds the track in position

 2. It is good for heavy traffic

 3. It can serve high speeds equally well.

 Disadvantages of Broken Stone

 1. The main disadvantage is that it is expensive in its initial cost.

2. Gravel ◎ Gravel ranks next in its suitability for use as materials for ballast and is used in many countries of the world in very large quantities. Gravel consists of worn fragments of rocks occurring in natural deposits. Gravel or shingle may be obtained from river bed or it may be dug out from gravelpits.

Advantages of Gravel

1. It is cheaper in its cost as it has not to be broken as like stone ballast

2. It has got excellent drainage properties, if properly cleaned

Disadvantages of Gravel

1. It easily rolls down under the vibrations and packing under the sleepers get tense

2. The variation in size is considerable and hence requires screening before use

3. Grovel as obtained from gravel pits, is full of earth and hence requires proper cleaning if proper drainage of the track is to be done.

3. Cinders Or Ashes ◎ The residue from the coal in locomotives or other furnaces is called cinder or ashes. It is one of the universal forms of ballast as it is a byproduct of all the railway which uses coal as a fuel.

Advantages of Cinders or Ashes

1. Handling of the material is not cumbersome this material can be handle easily

2. Cost is very low and hence can also be used for sidings

3. It has got fairly good drainage properties

4. Large quantities of this material can be made available at short notice.

5. In case of emergence such as caused by the destruction of portion of railway track during floods. This material proves to be very useful and is used in the formation repairing as well as for packing of track.

Disadvantages of Cinders or Ashes

1. It is highly corrosive and cannot be used where steel sleepers are fixed

2. The foot of the rails get affected due to use of this type of material as ballast

3. It is very soft and can easily be reduced to powder under vibrations and hence the track becomes very dusty. This is objectionable particularly in dry weather.

4. Sand ◎ Sand is another good materials for railway ballast , coarser sand is to be preferred to finer sand and the best sand is that which contains a quantity of fine gravel varying in size from 1/8 upwards.

Advantages of Sand

1. If the sand is free from earth and vegetation then it has good excellent properties to drain off water immediately

2. It is cheaper if available in nearby locality

3. It produces very silent track and hence are suitable for packing cast iron pot sleepers.

Disadvantages of Sand

1. It gets easily disturbed under vibrations and hence its maintenance is very difficult

2. The sand can be easily washed off or blown away and hence requires frequent renewal.

3. The sand particles may get into the moving parts of the vehicles and produces friction. This leads to heavy wear of vehicles.

5. Kankar ◎ Kankar a lime agglomerate is found in many places in the form of nodules of varying sizes.

Advantages of Kankar

1. Kankar is suitable Materials for ballast when other good material for ballast is not available or if available

uneconomically.

2. Kankar is good for light traffic on metre and narrow gauges

Disadvantages of Kankar

1. It is very soft and can be reduced to powder form easily, hence, making the track dusty.

2. The maintenance of track is very difficult

6. Moorum ⊚ The decomposition of laterite results into the formation of moorum. It has red and sometimes yellow color. The best moorum is that which contains large quantities of small laterite stones.

Advantages of Moorum

1. Moorum is good materials for ballast when other material for ballast is not available.

2. Moorum can be safely used on newly laid track and acts as a soling when broken stones are laid afterwards.

3. Moorum has got good drainage properties

Disadvantages of Moorum

1. Moorum is very soft and reduces to powder and hence to dust form in short time.

2. Maintenance of tracks laid with this material is difficult

7. Brick Ballast Or Brick Bats ⊂⊃ Sometimes the broken pieces of over burnt bricks, called brickbats, are used as materials for ballast.

Advantages of Brick Ballast

1. It has got excellent drainage properties

2. They can be used as good ballast material where suitable material for ballast is either unavailable or uneconomical

Disadvantages of Brick Ballast

1. Brickbats turn down into powder form easily and hence the track becomes dusty

2. Maintenance of the track laid with this material as ballast is very difficult.

3. Rails are often corrugated on the tracks laid with this material as ballast

8. Selected Earth ⊂⊃ Selected earth may be used as material for railway ballast for sidings and also for newly laid tracks.

Track fixtures for BG

Rail joints – Fish plates – Fish bolts- Fang bolts- Hook bolts – Rail chairs and keys – Bearing plates – Blocks – Spikes-Elastic fastenings- Anchors & Anti creepers

Fastening

A rail fastening system is a means of fixing rails to railroad ties. The terms rail anchors, tie plates, chairs and track fasteners are used to refer to parts or all of a rail fastening system. Various types of fastening have been used over the years.

Fish Plates

In rail terminology, a fishplate, splice bar or joint bar is a metal bar that is bolted to the ends of two rails to join them together in a track. The name is derived from fish, a wooden bar with a curved profile used to strengthen a ship's mast. The top and bottom edges are tapered inwards so the device wedges itself between the top and bottom of the rail when it is bolted into place. In rail transport modelling, a fishplate is often a small copper or nickel silver plate that slips onto both rails to provide the functions of maintaining alignment and electrical continuity.

Fig. 1.

Fish as specified.
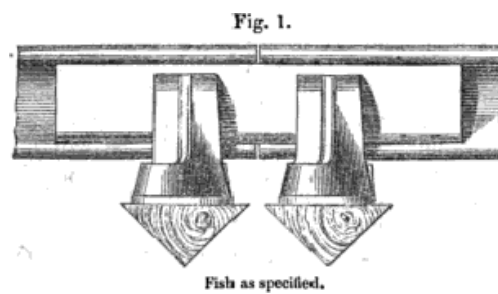
The first railway fishplate

expansion space

running surface

fishplate bolt +

fishplate

spike

+ nut

tie plate

dating nail

www.visualdictionaryonline.com
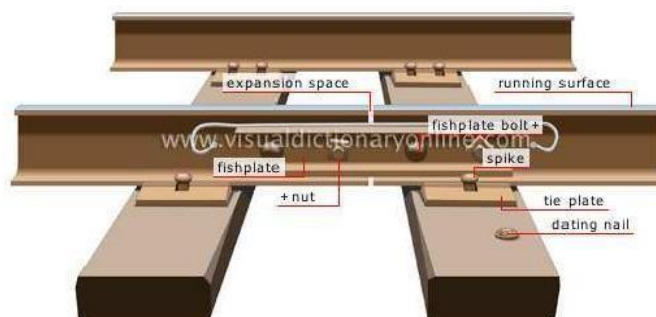
RAIL-TO-RAIL Fastenings
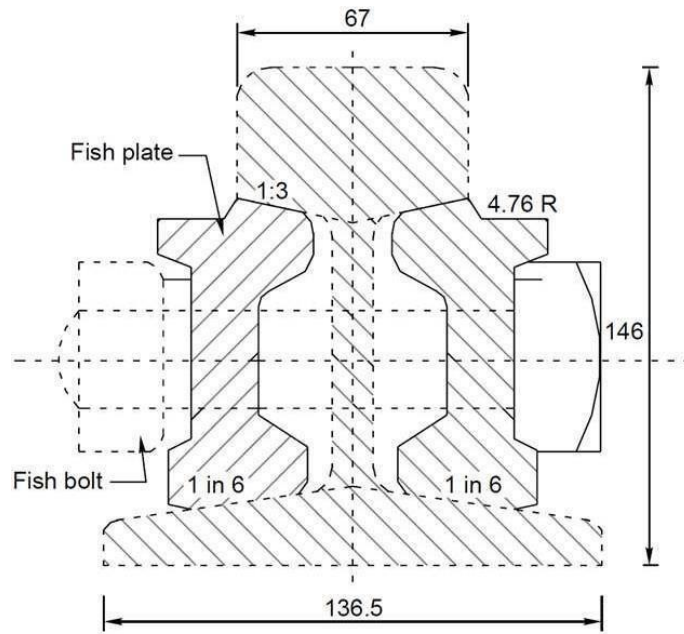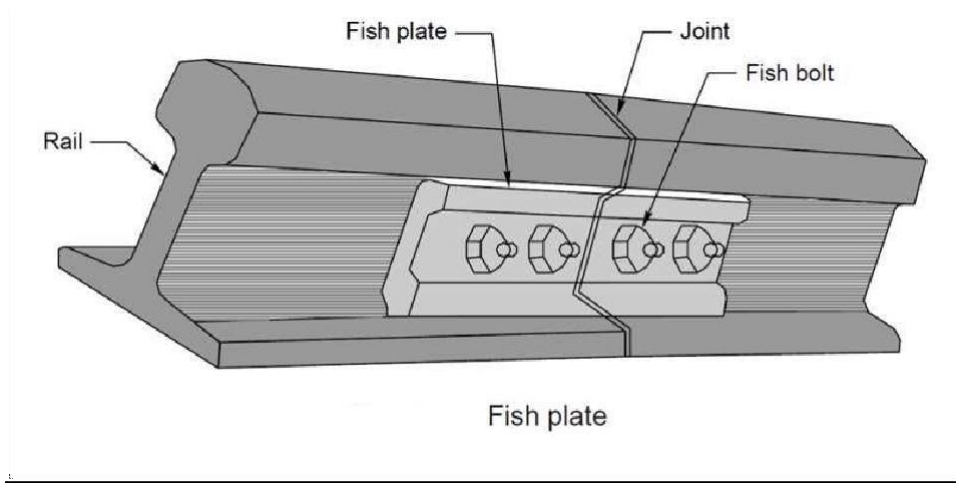
Fish plate

Fish plate for 90 R rails

Combination Fish Plates
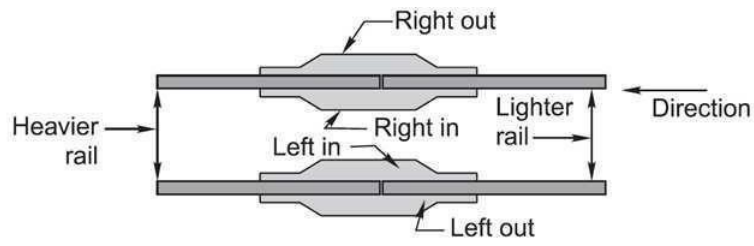
(a) Plan
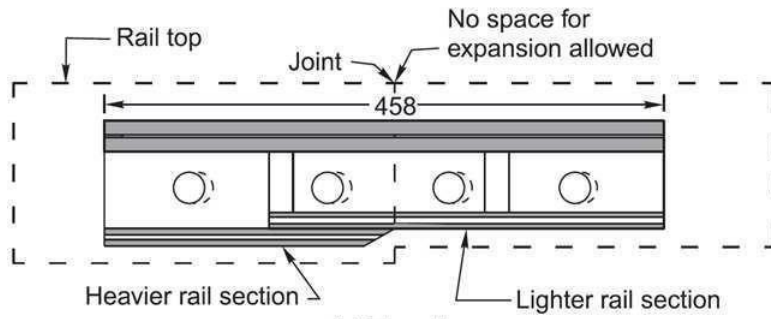
(b) Elevation

Combination fish plate (dimensions in mm)
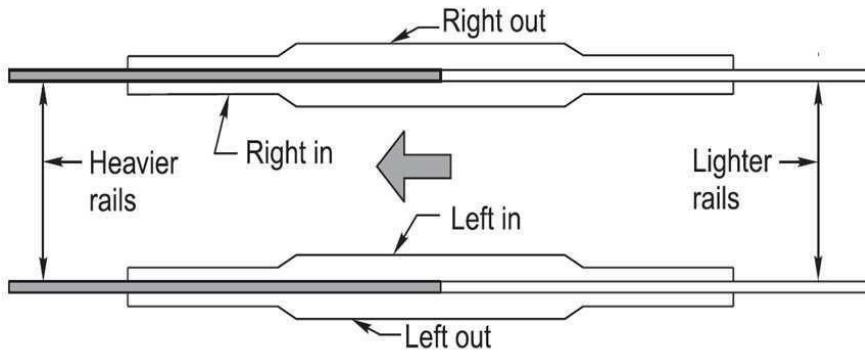
Marking of combination fish plates

Fang Bolts
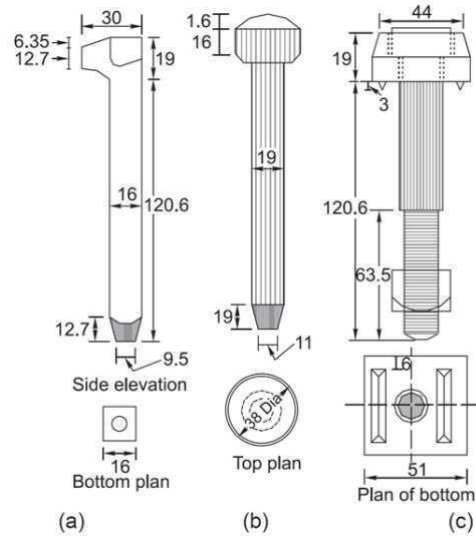
(a) Dog spike, (b) round spike, (c) fang bolt

*Screw Spikes/
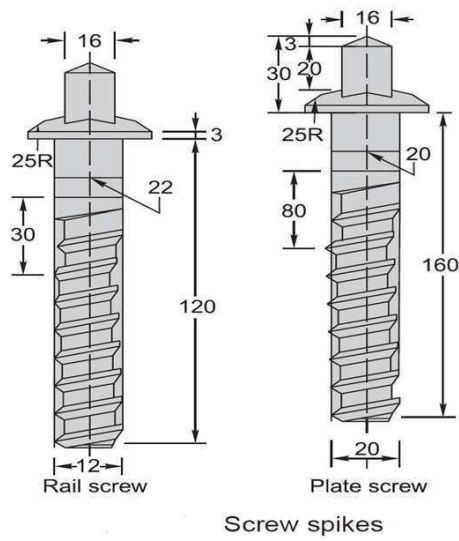
Screw spikes

Bearing Plates
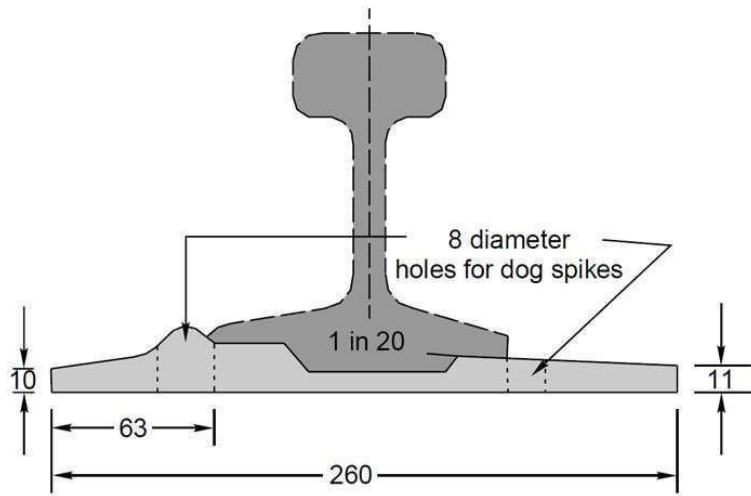
Canted MS bearing plate for 90 R (dimensions in mm)
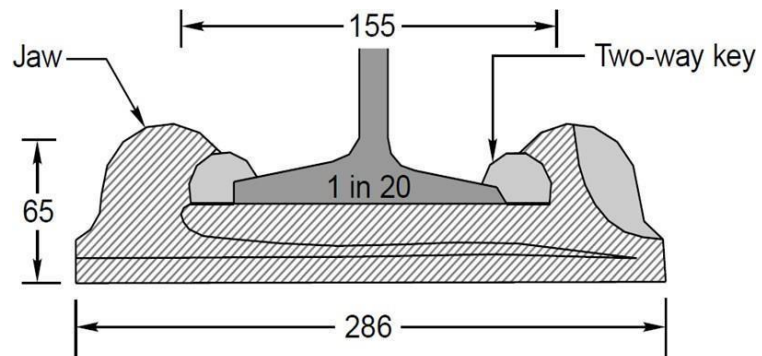
CI anticreep bearing plate
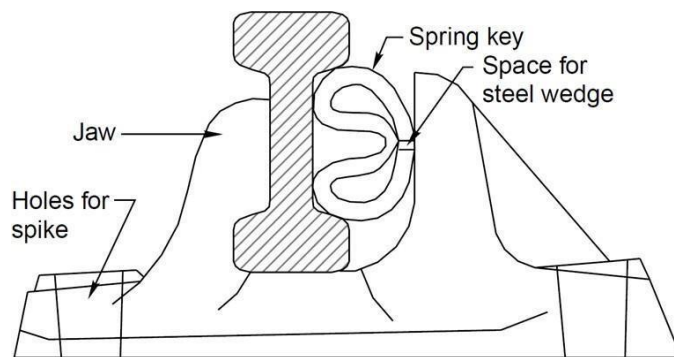
CI bearing plate for BH rail

Fittings of Steel Trough Sleepers
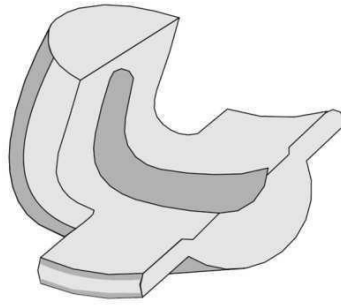
Spring steel loose jaw

STEEL SLEEPERS FITTINGS

(a) Section XX   (b) Elevation   (c) End elevation
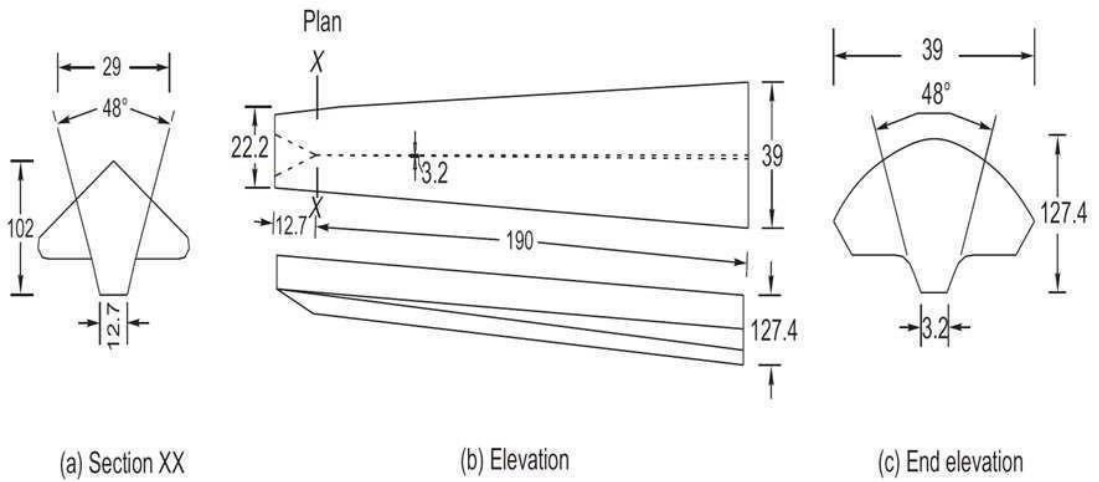
Two-way keys (dimemions in mm)

Section A-B
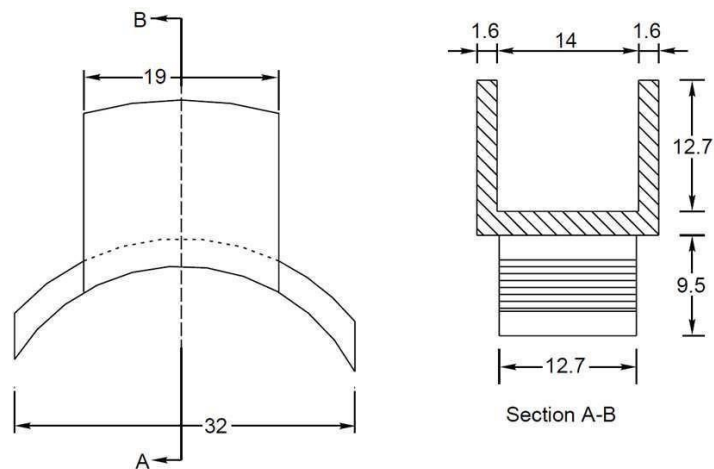
Mota Singh liner (all dimensions are in mm)

CAST IRON SLEEPER FITTINGS

Centre split cotter T-401

Side split cotter T-432

Solid end split cotter T-423

Bent plate cotter T-424
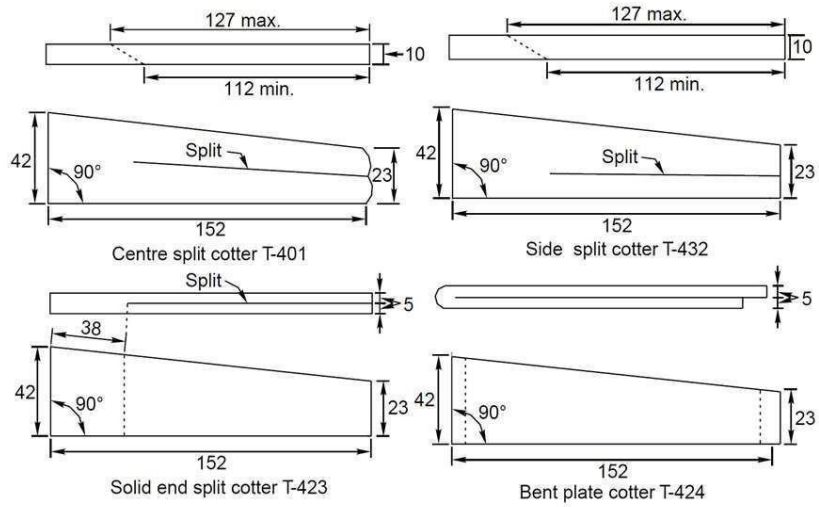
MS cotters (all dimensions are in mm)
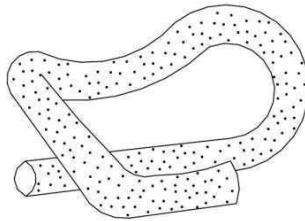
Pandrol clip or elastic rail clip

Isometric view of Pandrol clip
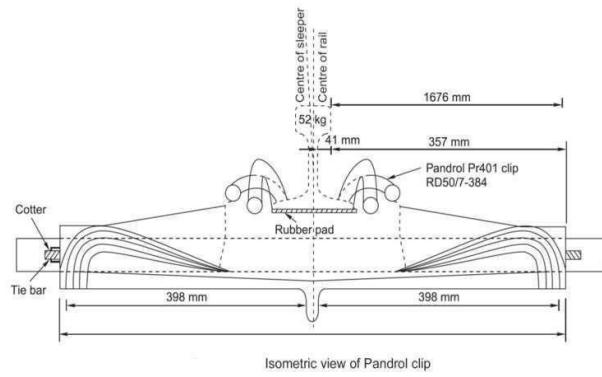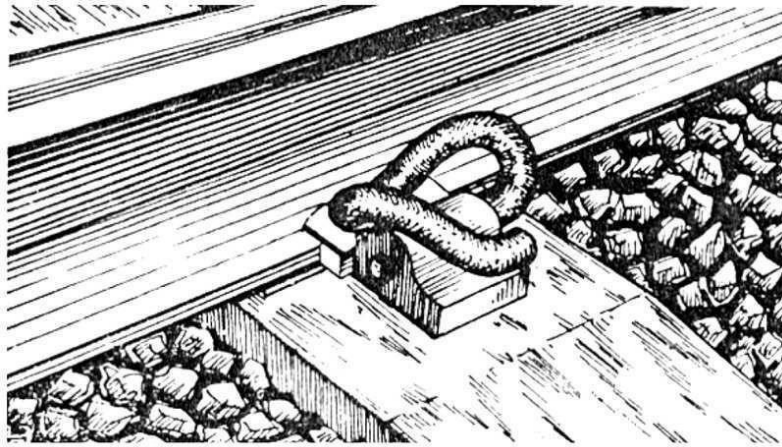
Pandrol clip fixed to the rail seat
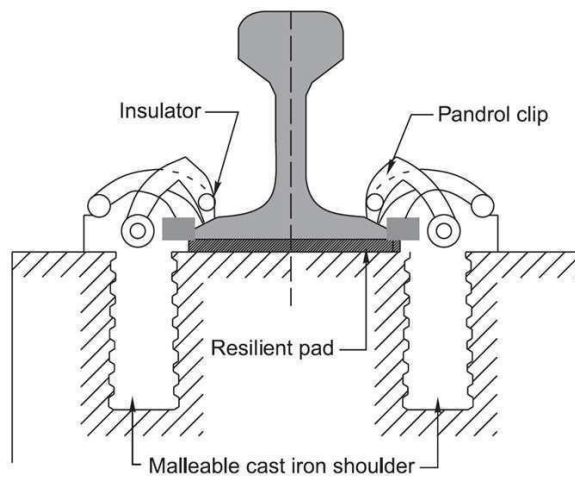
Insulated liners

IRN 202 clip

37.5 mm

165 mm

15.5 mm

(a) Double shank elastic spike

(b) Lock plate

Lock spike

₵ of rail

22 mm-diameter bolt

Spring washer

Spring clip

130 mm

175 mm

175 mm
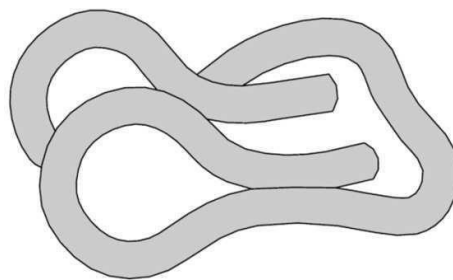
Spring steel clip
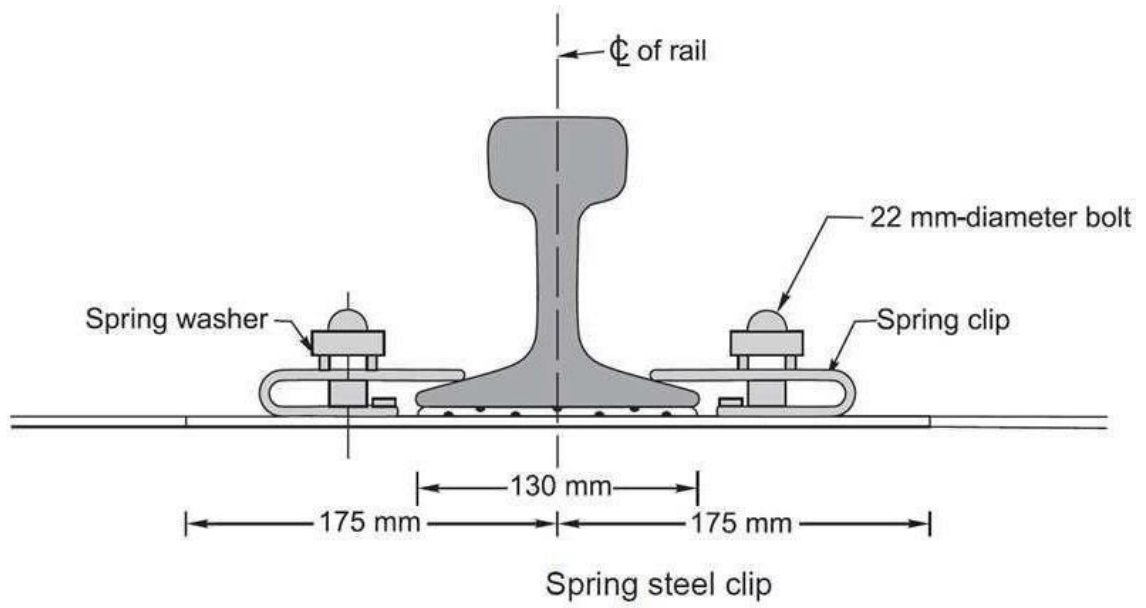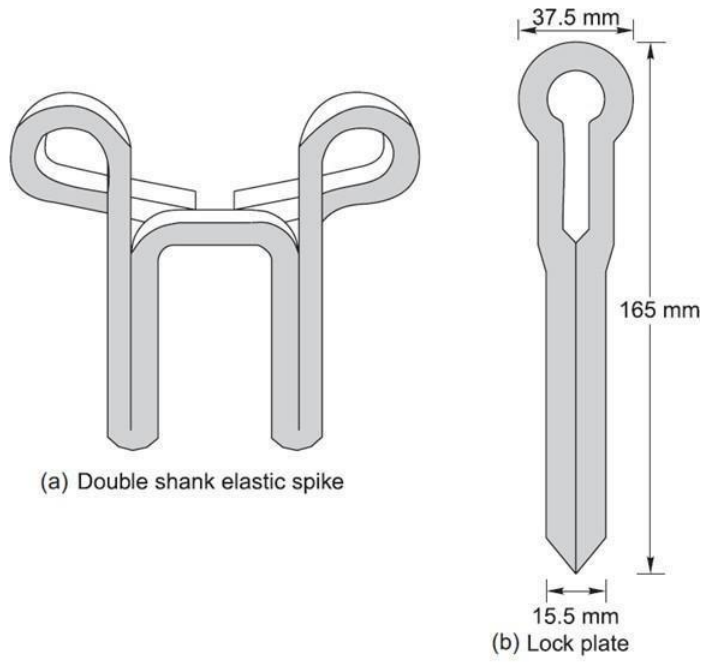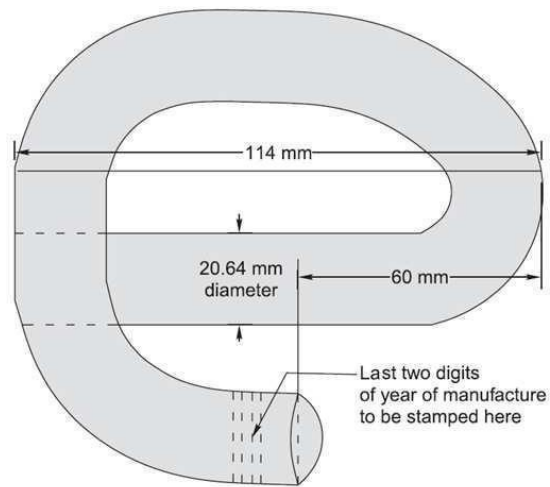
Pandrol clip (ERC rail clip) MK III

New Elastic Fastening (G Clip)
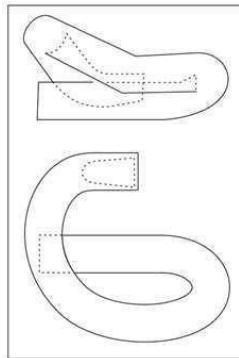
Logwell G clip
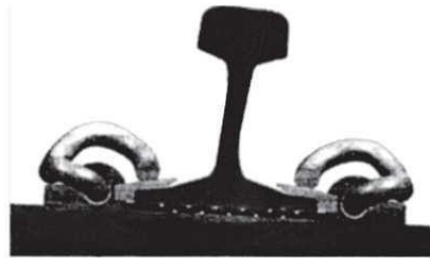
Isometric view of G clip assembly

CHAPTER-4

Geometric for Broad gauge
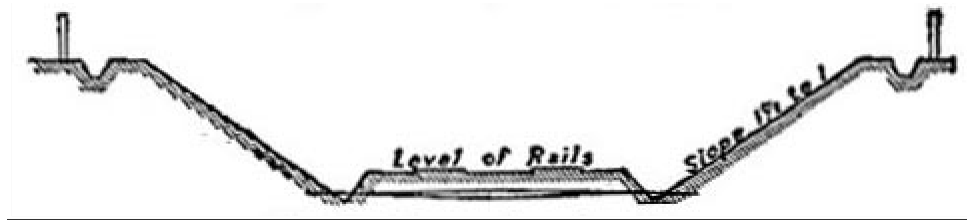
Typical Section of Railway in Cutting.
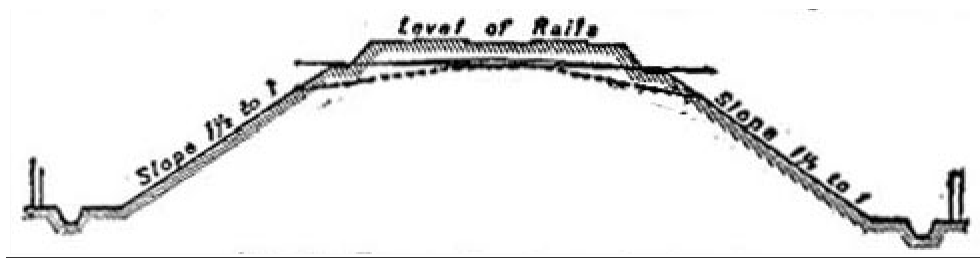
Typical Section of an embankment.

Diagram showing cross section of double track railway alignment including an overhead power supply system
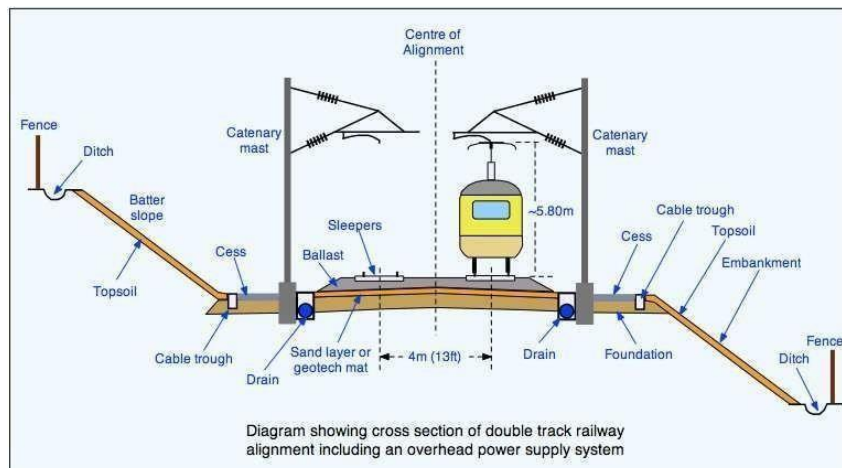
Classification of Railway Land: With a view to determine what the disposition of the land will probably be on the completion of the work for which it had been acquired, the classification given in paragraph 818 etc. should be adopted.

On railways, land is divided into two classes, viz.,

- (i) permanent land width

- (ii) temporary land width

Permanent land width is land which will be required permanently after the railway is open for traffic and the work of construction is complete. Under this head will be included all land to be occupied by the formation of the permanent line of railway with side slopes of banks and cuttings, and the berms connected therewith; catch water drains and borrow pits or such parts of them as it is necessary to retain; the entrances to tunnels and shafts belonging to them; the sites of bridges, and protection or training works; station yards; landing places for railway ferries; ground to be occupied by works belonging to the railway such as gas works, arrangements for water supply, septic tanks, collecting pits, filter beds and pumping installations, & c., ground for the storage manufacture or acquisition of materials; land for sanitary zones, cemeteries, churches, plantations; gardens, and recreation grounds, sites for stations, offices, workshops; dwelling houses and other buildings required for the purposes of the railway, or the accommodation of the staff, with the grounds, yards, roads, & c., appertaining thereto. Under this head will also be included land outside the permanent railway boundary, which will be required for the permanent diversion of roads or rivers, or for other works incidental to the construction of the railway, which are made for public purposes and will not on completion of the works be maintained by the railway authorities.

Temporary land width is land which is acquired for temporary purposes only, and which is disposed of after the work of construction is completed.

Gradients for drainage

Rising and falling gradient
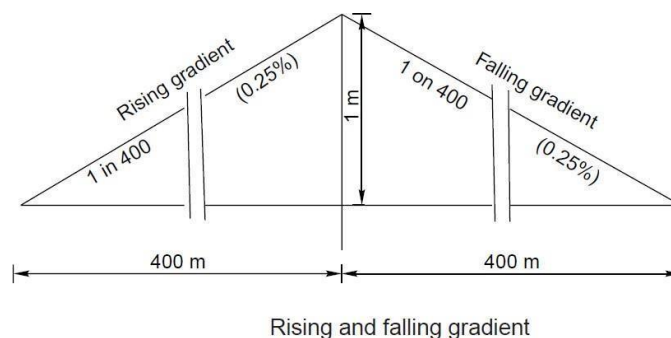
Drainage is defined as interception, collection and disposal of water away from track. Drainage is the most important factor in track maintenance and for stability of banks/cuttings. When water seeps into the formation, it weakens the bonds between the soil particles, softens the soil and results in creation of ballast pockets. On one hand, ingress of water into bank/cutting adds to weight of soil mass trying to slide, thereby increasing propensity for slope-slide, on the other hand, it reduces shear strength of soil, thereby decreasing factor of safety for stability of slope. Therefore, quick disposal of water from formation top/slopes is very essential. Drainage system should be effective in preventing the stagnation of water and allow quick disposal of water. At present, drainage is not being given its due importance in field. Provisions relating to drainage

have been detailed in various guidelines issued by RDSO(Research Design and Standards Organisation) from time to time, however, the present Guidelines highlight the salient features of drainage arrangement in embankment as well as cuttings.

CONVENTIONAL DRAINAGE SYSTEMS

- SURFACE DRAINAGE
- SIDE DRAINS
- CATCH WATER DRAINS
- SUB SURFACE DRAINS

SUPER ELEVATION

(1)   **Cant or super elevation** is the amount by which one rail is raised above the other rail. It is positive when the outer rail on a curved track is raised above inner rail and is negative when the inner rail on a curved track is raised above the outer rail.

(2)   **Equilibrium speed** is the speed at which the centrifugal force developed during the movement of the vehicle on a curved track is exactly balanced by the cant provided.

(3)   **Cant deficiency**- Cant deficiency occurs when a train travels around a curve at a speed higher than the equilibrium speed. It is the difference between the theoretical cant required for such higher speed and actual cant provided.

(4)   **Cant excess** - Cant excess occurs when a train travels around a curve at a speed lower than the equilibrium speed. It is the difference between the actual cant and the theoretical cant required for such a lower speed.

(5)   Maximum permissible speed of the curve-It is the highest speed which may be permitted on a curve taking into consideration the radius of the curvature, actual cant, cant deficiency, cant excess and the length of transition. When the maximum permissible speed on a curve is less than the maximum sectional speed of the section of a line, permanent speed restriction becomes necessary.

(6)   **Cant gradient** and cant deficiency gradient indicate the amount by which cant or deficiency of cant is increased or reduced in a given length of transition e.g., 1 in 1000 means that cant or deficiency of cant of 1 mm. is gained or lost in every 1000mm. of transition length.

(7)   **Rate of change of cant** or rate of change of cant deficiency is the rate at which cant or cant deficiency is increased or reduced per second, at the maximum permissible speed of the vehicle passing over the transition curve, e.g., 35 mm. per second means that a vehicle when traveling at a maximum speed permitted will experience a change in cant or deficiency of cant of 35mm. in each second of travel over the transition.

(8)   Transition curve is an easement curve, in which the change of radius is progressive throughout its length and is usually provided in a shape of a cubic parabola at each end of the circular curve. It affords a gradual increase of curvature from zero at the tangent point to the specified radius of circular are and permits a gradual increase of super elevation, so that the full superelevation is attained simultaneously with the curvature of the circular arc.

Super elevation, Cant deficiency and Cant excess

(1) Super elevation/cant

(a) The equilibrium super elevation/cant necessary for any speed is calculated from the formula

$$C = \frac{GV^2}{127R}$$

Where C is cant/super elevation in mm. G is the gauge of track + width of rail head in mm. R is the radius of the curve in metres.

Necessity of SuperElevation

In order to counteract the effect of centrifugal force the outside rail of the curve may be elevated above the inside rail effectively moving the centre of gravity of the rolling stock laterally toward the inside rail. This procedure is generally referred to as super elevation.

If the combination of lateral displacement of the centre of gravity provided by the super elevation, velocity of the rolling stock and radius of curve is such that resulting force becomes centered between and perpendicular to a line across the running rails the downward pressure on the outside and inside rails of the curve will be the same.

The super elevation that produces this condition for a given velocity and radius of curve is known as the balanced or equilibrium elevation.

Equilibrium superelevation

Negative superelevation
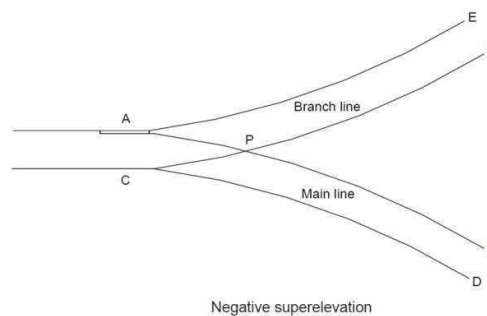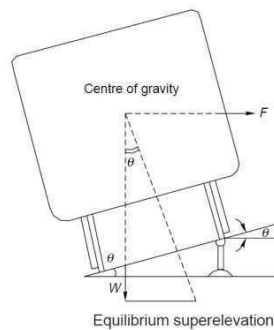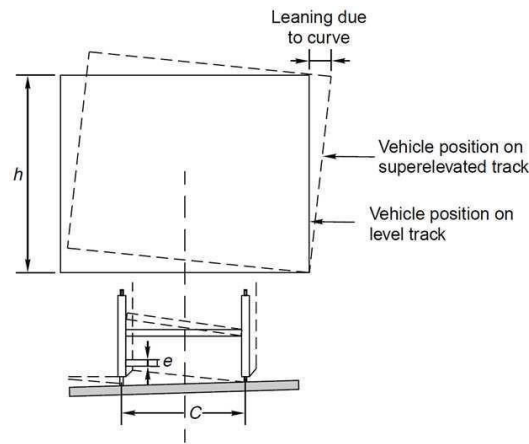
Effect of lean due to superelevation

Limitation of Super elevation

For Mixed Passenger & Freight Routes

Typical early railway operation resulted in rolling stock being operated at less than equilibrium velocity (all wheels equally sharing the rolling stock weight), or coming to a complete stop on curves. Under such circumstances excess super elevation may lead to a downward force sufficient to damage the inside rail of the curve, or cause derailment of rolling stock toward the centre of the curve when draft force is applied to a train. Routine operation of loaded freight trains at low velocity on a curve super elevated to permit operation of higher velocity passenger trains will result in excess wear of the inside rail of the curve by the freight trains.

Thus on these types of routes, super elevation is generally limited to not more than 6 inches.

For High Speed Passenger Routes

Modern high speed passenger routes, do not carry slower speed trains, nor expect trains to stop on curves, so it is possible to operate these routes with higher track super elevation values. Curves on these types of route are also designed to be relatively gentle radius, and are typically in excess of 2000m (2km) or 7000m (7km) depending on the speed limit of the route.

CHAPTER -5.0

POINTS AND CROSSING

DEFINITION: The arrangement which is used to transfer the train from one track to another is known as points and crossings.

Necessity:-

I.   Points and crossing are provided to help transfer railway vehicle from one track toanother.

II.   The track may be parallel to diverging from or converging with each other point and crossing are necessary because the wheels of railway vehicles are provided with inside flange and therefore they require this in special arrangement in order to navigate their way on the rail.

III. The points or switches aid in diverting the vehicles and the crossing provide gaps in the rails so as to help the flanged wheels to roll over them.

IV. A complete set of points and crossings, along with lead rails, is called a *turnout.*

Points or Switches

A pair of tongue rail and stock rail with necessary connection and fitting forms a switch

Crossing: - it is a device introduced at the junction where two rails cross each other to permit the wheel flanges of a railway vehicle to pass from one track to another

Switches

➢ A pair of stock rail, AB and CD made of medium-manganese steel.

➢ A pair of tongue rails, PQ and RS also known as switch rails made withstand wear. The tongue rails are machined to very thin section to obtain a snug fit with the stock rail is called 'toe' and thicker end is called the 'heel'

➢ A no. of slide chairs to support the tongue rail and enable its movement towards or away from the stock rail.

➢ Two or more stretcher bars connecting both the tongue rails close to the toe for the purpose of holding them at a fixed distance from each other.

➢ A gauge ties plate to fix gauges and ensure correct gauge at the points.
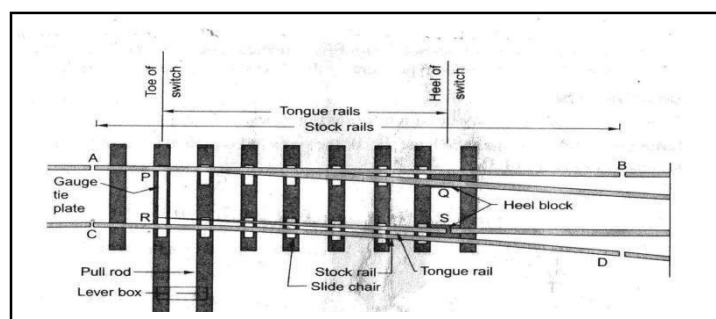
FIGURE: DETAILS OF SWITCH

Types of Switches

Switches are of two types, namely 'stud switches' and 'split switch'.

In 'stud switch' no separate tongue rail is provided and some portion of the track is moved from one side to the other side.

In 'split switch' a pair of stock rail and a pair of tongue rails are present.

Split switches are two types:-

1) **Loose Heel type:-**
    ➢ In this type of split switch, the switch or tongue rail finishes at the heel of the switch to enable movement of the free end of the tongue rail.
    ➢ The fish plates holding the tongue rail may be straight or hightly bent.
    ➢ The tongue rail is fastened to the stock rail with the help of a fishing fit block and fourbolts.
    ➢ All the fish bolts in the lead rail are tightened while those in the tongue rail are kept loose or snug to allow free movement of the tongue.
    ➢ As the discontinuity of the track at the heel is a weakness in the structure, the use of these switches is structures, the use of these switches is not preferred.

(2) **Fixed Heel Type:-**

    In this type of split switch the tongue rail does not end at the heel of the switch, but extends further and is rigidly connected. The movement at the toe of the switch is made possible on account of flexibility of tongue rail.

Crossing:-

A crossing or Frog is a device introduced at the point/junction where two gauge faces / rails cross each other to permit the wheel flanges of a railway vehicle to pass from one track to each other.

Figure :- Details of Crossing
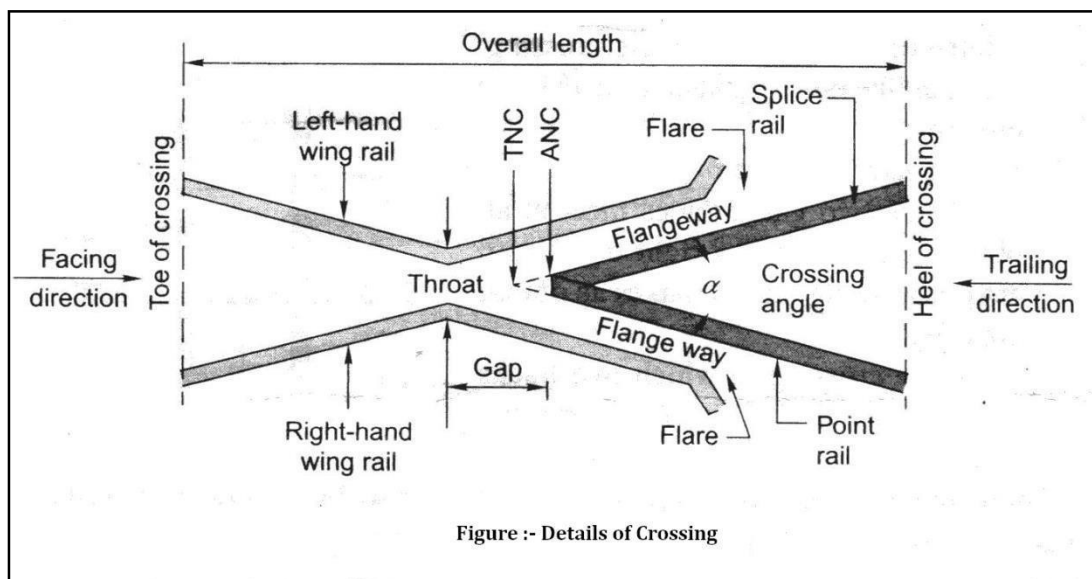
A crossing consists of the following components.

(i)   Two rails, *point rails and splice rails*, which are machined to form a nose. The point rail ends at the nose whereas the splice rail joins it a little behind the nose.

Theoretically, the point rail should end in a point and be made as thin as possible, but a knife edge of point rail would break off under the movement of traffic. The point rail therefore, has its fine end slightly cut off to form a blunt nose, with a thickness of 6mm. The toe of the blunt nose is called the *actual nose of crossing (ANC)* and the theoretical point where the gauge faces form both sides intersect is called the *theoretical nose of crossing (TNC).* The 'V' rail is planed to a depth of 6mm (1/4'') at the nose and runs out in 89mm to stop a wheel running in the facing direction from hitting the nose.
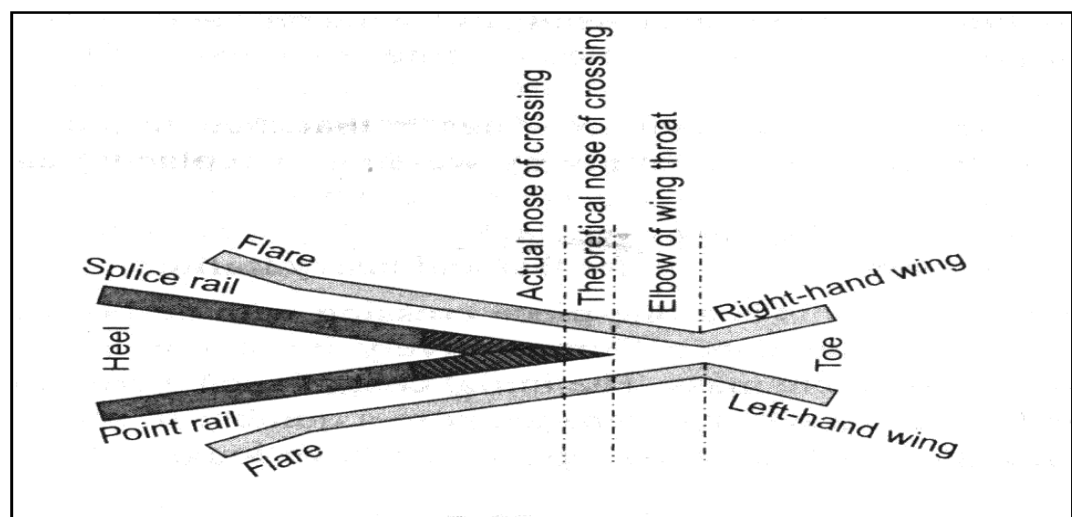
Figure: - Point Rail and Splice Rail

(ii)   Two wing rails consisting of a right hand and a left hand wing rail that converge to from a throat and diverge again on either side of the nose. Wing rails are flared at the ends to facilitate the entry and exit of the flanged wheel in the gap.

(iii)   A pair of check rails to guide the wheel flanges preventing them from moving sideways which would otherwise may result in the wheel hitting the nose of the crossing as it moves in the facing direction.

TYPES OF CROSSING:

A crossing may be of the following types:-

(a) **An acute angle crossing or 'V' crossing** in which the intersection of two gauge faces forms on acute angle, For example when a right rail crosses a left rail, it makes an acute crossing. So unlike rail crossing from an acute crossing.

(b) **An obtuse or diamond crossing** in which the two gauge faces meet at an obtuse angle. When a right on left rail crosses a similar rail, it makes an obtuse crossing.

(c) **A square crossing** in which two tracks crossing at right angles, such crossings are rarely used in actual practice. For manufacturing purposes, crossing is of following types.

According to manufacture, the crossing may be classified as mentioned under:-

(a) Built up crossing:-

In a built-up crossing two wing rails and a V-section consisting of splice and point rails are assembled together by means of bolts and distance blocks to form a crossing.

Advantages:-

(a) Initial cost is low.

(b) Repair can be carried out by welding.

(b) Cast steel crossing:-

This is one piece crossing with no bolts and therefore requiring very little maintenance.

Comparatively

Advantages:-

(a) Longer life.

Disadvantages:-

(a) Initial cost is high.

(b) Repair and maintenance causes a no of problem.

(c) *Combined rail and cast crossing:-.*

is a combination of a built-up and cast steel crossing and consists of a cast steel nose finished to ordinary rail faces to form the two legs of the crossing.

Through it allows the welding of worn-out wing rails, the nose is still liable to fracture suddenly.

CMS Crossing:- Due to increase in traffic and the use of heavier axle loads, the ordinary built-up crossing manufacturer from medium –manganese rails subjected to vary heavy wear and tear, especially in fast lines and suburban sections with electric traction. Past experience has shown that life of such crossings varies six months to two years, depending on their location and service conditions. CMS crossings possess higher strength, offer more resistance to wear and consequently have a longer life.

Advantages:-

➢ Less wear and tear.

➢ Longer life.

➢ Free from bolts as wheel as other components that normally to get loose as a result of movement of traffic.

Spring or Movable Crossing:-

In a spring crossing, one wing rail is movable and held against the V of the crossing with a strong helical spring while the other wing rail is kept fixed. When a vehicle passes on the main track, the

movable wing rail is snug with crossing and the vehicle does not need to negotiate any gap at the crossing. In case the vehicle has to pass over a turnout track, the movable wing is forced out by the wheel flanges and the vehicle has to negotiate a gap as in normal turnout

This type of crossing is useful when there is high-speed traffic on the main track and slow-speed traffic on the turnout track

CHAPTER 6

METHODS OF LAYING & MAINTENANCE OF TRACK

Essential of Track Maintenance:-

1. The gauge should be correct or within the specified limits.
2. There should be no difference in cross levels except on curves, where cross levels vary in order to provide super elevation.
3. Longitudinal levels should be uniform.
4. The alignment should be straight and kink-free.
5. The ballast should be adequate and sleepers should be well packed.
6. The tack drainage should be good and formation should be well trained.

Railway track can be maintained either conventionally by manual labor or by the application of modern methods of track maintenance, such as mechanical tamping or measured shovel packing. The major maintenance operations performed in a calendar year (12-months) are as follows for achieving the above mentioned standards:-

1) **Through Packing-:**
2) **Systematic Overhauling**
3) **Picking up slacks**

1) Through   Packing

Through Packing is carried out in a systematic and sequential manner as described as follows:

➢ **Opening of road:-**

The ballast is dug out on either side of the rail seat for a depth of 50mm (2") below the bottom of the sleeper with the help of a shovel with a wire claw .On the outside, the width of the opening should extend up to the end of the sleeper.

On the inside it should extend from the rail seat to a distance of 450mm (18") in case of BG, 350mm (14") in case of MG, and 250mm (10") in case of NG.

➢ **Examination of rails, sleepers and fastening:-**

The rails, sleepers and fastening to be used are thoroughly examined. Defective sleepers are removed and loose fastening are tightened. Any kinks in rails are removed.

➢ **Squaring of sleepers:-**

(a) To do this one of the rails is taken as the sighting rail and the correct sleeper spacing is marked on it.
(b) The position of the sleeper is checked with reference to the second rail with the help of a T-square.

(c) The sleeper attended to after this defects have been established, which may include their being out of square or at incorrect spacing.

> **Aligning the track:-**

(a) The alignment of the track is normally checked visually, where in the rail is visually assessed form a distance of about four rail lengths or so.

(b) Small errors in the alignment are corrected by slewing the track after loosening the cores at the ends and drawing out sufficient ballast at the ends of the sleeper.

(c) Slewing is carried out by planting crowbar deep into the ballast at an angle not more than 30 form the vertical.

Advantages of Track Maintenance:-

1. If the track is suitably maintained, the life of the track as well as that of the rolling stock increases since there is lesser wear and tear of their components.

2. Regular track maintenance helps in reducing operating costs and fuel consumption.

3. Small maintenance jobs done at the appropriate time, such as tightening a bolt or key, hammering the dog spike, etc., help in avoiding loss of concerned fitting and thus saving on the associated expenditure.

4. When track maintenance is neglected for along time, it may render the track beyond repair, calling for heavy track renewals that entail huge expenses

Gauging:-

The gauge should be checked and an attempt should be made to provide a uniform gauge within permissible tolerance limits.

2. Systematic overhauling:-

The systematic overhauling of the track should normally commence after the completion of one cycle of through packing. It involves the following operations in sequence:-

(a) Shallow screening and making up of ballast section.

(b) Replacing damaged or brother fittings.

(c) Including all items in through packing.

(d) Making up the cess.

3. Picking up stacks:-

Stacks are those points in the track where the running of trains is faculty. Slacks generally occur in the following cases:-

(a) Stretches of yielding formation.

(b) Improperly aligned curves.

(c) Portions of track with poor drainage.

(d) Approaches to level crossing, girder bridges etc.

(e) Section with an inadequate or unclean ballast cushion.

No through packing is done during the raining season and slacks are only picked up in order to keep the track safe and in good running condition.

Duties of a permanent way Inspector (PWI)

The PWI is generally responsible for the following:-

(a) Maintenance and inspection of the track to ensure satisfactory and safe performance.

(b) Efficient execution of all works incidental to track maintenance, including track relayingwork.

(c) Accounts and periodical verification of the stores and tools in his or her charge.

(d) Maintenance of land boundaries between stations and at important stations as may be specified by the administration.

The PWI also carries out inspection of the following facts of a track.

(a) Testing the track.

(b) Inspection of track and gauge.

(c) Level crossing inspection.

(d) Point and crossing inspection.

(e) Curve inspection.

(f) Safety of track.

In addition to the inspections, a PWI also carries out following duties:-

(a) Check the proximity of trees that are likely to damage the track and get themremoved.

(b) Check night patrolling at last once a month by train as well as by trolley.

(c) Takes the necessary safety measures wile executing maintenance work that affects the safety of the track.

(d) Periodically inspects and respective LWR tacks to ensure their safety.

(e) Ensures the cleanliness of station yards.

(f) Keeps proper records of the training out of ballast.

(g) Looks after all establishment work, including the welfare of the staff working under his charge and maintenance their service records.

(h) Ensures the safety of the track during the execution of work that affects thetrack.

SECTION-B

CHAPTER-7

BRIDGE ENGINERRING

DEFINATION

The following definitions of certain important terms used in Bridge Engineering are given below:

1. **Bridge: -** A structure is facilitating a communication route for carrying road traffic or other moving loads over a depression or obstruction such as river, stream, channel, road or railway. The communication route may be a railway track, a tramway, a roadway, footpath, a cycle track or a combination of them.

2. **High Level Bridge or Non-submersible Bridge:** - The Bridge which does not allow the high flood waters to pass over them. All the flood water is allowed to pass through its vents. In other words it carries the roadway above the highest flood level of the channel.

3. **Submersible Bridge: -** A submersible bridge is a structure which allows flood water to pass over bridge submerging the communication route. Its formation level should be so fixed as not to cause interruption to traffic during floods for more than three days at a time not more than six times in a year.

4. **Causeway: -** It is a pucca submersible bridge which allows floods to pass over it. It is provided on less important routes in order to reduce the construction cost of cross drainage structures. It may have vents for low water flow.

5. **Foot Bridge: -** The foot bridge is a bridge exclusively used for carrying pedestrians. Cycles and animals.

6. **Culvert: -** When a small stream crosses a road with linear waterway less than about 6 meters. The cross drainage structure so provided is called culvert.

7. **Deck Bridge:** - These are the bridge whose floorings are supported at top of thesuperstructures.

8. **Through Bridge:-**These are the bridges whose floorings are supported or suspended at the bottom of the superstructures.

9. **Semi-Through Bridges: -** These are the bridges whose floorings are supported at some intermediate level of the superstructure.

10. **Simple Bridges: -** They include all beam, girder or truss bridges whose flooring is supported at some intermediate level of superstructure.

11. **Cantilever Bridges:** - Bridges which are more or less fixed at one end and free at other. It can be used for spans varying from 8 meters to 20 meters.

12. **Continuous Bridges: -** Bridges which continue over two or more spans. They are used for large spans and where unyielding foundations are available.

13. **Arch Bridge: -** These are the bridges which [produce inclined pressures on supports under vertical loads. These bridges can be economically used up to spans about 20 meters. The arches may be in the barrel form or in the form of ribs.

14. **Rigid Frame Bridges: -** In these bridges the horizontal deck slab is made monolithic with the vertical abutments walls. These bridges can be used up to span about 20 meters. Generally this type of bridge is not found economical for spans less than 10 meters.

15. **Square Bridge: -** These are the bridges at right angles to axis of the river.

16. **Skew Bridge: -** These are the bridges not at right angles to axis of the river.

17. **Suspension Bridges: -** These are the bridges which are suspended on cables anchored at ends.

18. **Under-Bridges: -** It is a bridge constructed to enable a road to pass under another work or obstruction.

19. **Over-Bridges: -** it is a bridge constructed to enable one form of land communication over theother.

20. **Class AA Bridges: -** These are bridges designed for I.R.C. class AA loading and checked for class A loading. They are provided within certain municipal limits, in certain existing or contemplated industrial area, in other specified areas, and along certain specified highways.

21. **Class A Bridges: -** These are permanent bridges designed for I.R.C. class Aloading.

22. **Class B Bridges: -** These are permanent bridges designed for I.R.C. class Bloading.

23. **Viaduct: -** It is a long continuous structure which carries a road or railways like Bridge over a dry valley composed of series of span over trestle bents instead of solid piers.

24. **Apron: -** It is a layer of concrete, masonry stone etc. placed like flooring at the entrance or out of a culvert to prevent scour.

25. **Piers: -** They are the intermediate supports of a bridge superstructure and may be solid or opentype.

26. **Abutments: -** They are the end supports of the superstructure.

27. **Curtain Wall: -** It is a thin wall used as a protection against scouring action astream.

28. **Effective Span: -** The centre to centre distance between any two adjacent supports is called as the effective span of a bridge.

29. **Clear Span: -** The clear distance between any two adjacent supports of a bridge is called clear Span.

30. **Economic Span: -** the span, for which the total cost of bridge structure is minimum is known as economic span.

31. **Afflux: -** due to construction of the Bridge there is a contraction in waterway. This results in rise of water level above its normal level while passing under the Bridge. This rise is known asafflux.

32. **Free Board: -** Free Board at any point is the difference between the highest flood level after allowing for afflux, if any, and the formation level of road embankment on the approaches or top level of guide bunds at that point.

33. **Headroom: -** Headroom is the vertical distance between the highest points of a vehicle or vessel and the lowest points of any points of any projecting member of a Bridge.

34. **Length of the Bridge: -** The length of a Bridge structure will be taken as the overall length measured along the center line of the Bridge from the end to end of the Bridgedeck.

35. **Linear Waterway: -** The linear waterway of a Bridge shall be the length available in the bridge between extreme edges of water surface at the highest flood level, measured at right angles to the abutment faces.

36. **Low Water Level (L.W.L.): -** The low water level is the of water surface obtained generally in the dry season.

37. **Ordinary Flood Level (O.F.L.):-** It is average level of a high flood which is expected to occur normally every year.

38. **Highest Flood Level (H.F.L.):-** It is the level of highest flood, every recorded or the calculated level for the highest possible flood.

39. **Effective Linear Waterway: -** Effective linear waterway is the total width of waterway of a bridge minus the effective width of obstruction. For calculating the effective linear waterways, the width of mean obstruction due to each pier shall be taken as mean submerged width of the pier at its foundation up to maximum scour level. The obstruction at ends due to abutments or pitched slopes should be ignored.

COMPONENTS OF A BRIDGE

The bridge structure is divided mainly into two components:

1) Substructure

➢ The function of substructure is similar to that of foundations, columns and walls etc. of a building. Thus the substructure supports the superstructure and distributes the load into the soil below through foundation.

➢ The substructure consists of foundation piers and abutment piers, foundation for the piers, abutments, wing walls, and approaches.

➢ The above all supports the superstructure of the bridge.

2) Superstructure

➢ The superstructure of a bridge is analogues to a single storey building roof and substructure to that of walls, columns and foundations supporting it.

➢ Superstructure consists of structural members carrying a communication route

➢ It consists of handrails, parapets, roadways, girders, arches, wall trusses over which the road is supported.

➢ It is that part of the bridge over which the traffic moves safely.

Figure 1   Components of a Typical Bridge.
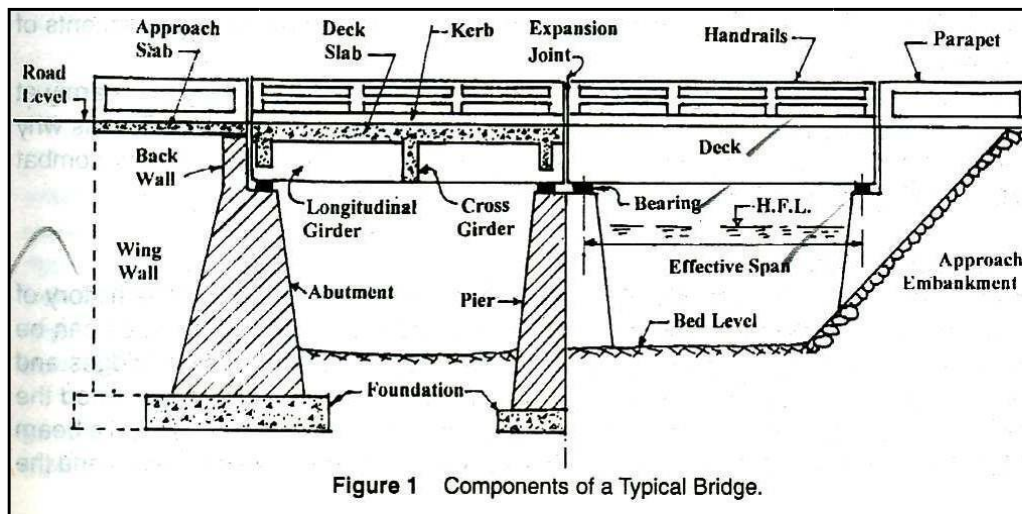
Classification of a Bridge: -

The bridges may be classified depending upon the following factors:-

(a) Their functions or purpose as railway, highway, Foot Bridge, aqueduct etc.

(b) Their material of construction used as timber, masonry, R.C.C. Steel, pre stress concrete etc.

(c) Nature or life span such as temporary, permanent bridge etc.

(d) Their relative position of floor such as Deck Bridge, through bridge, semi through etc.

(e) Type of super-structure such as arched girder, truss, suspension bridge etc.

(f) Loadings: - Road Bridges and culverts have been classified by I.R.C. into class AA, Class A, Class B
    bridges according to the loadings they are designed to carry.

(g) Span Length:- Under this category the bridges can be classified as

   - Culverts (Span Less than 8m) i.e. BOX Type, Hume Pipe Type,

   - Minor Bridge (Span length = 8 to 30m) i.e. BOX type, Girder Type

   - Major Bridge (Span Length =above than 30m)

(h) Degree of Redundancy: - Under this the bridges can be classified as determinate & indeterminate bridges.

(i) Types of Connection: - Under this category the steel bridges can be classified as pinned connected,

riveted or welded bridges.
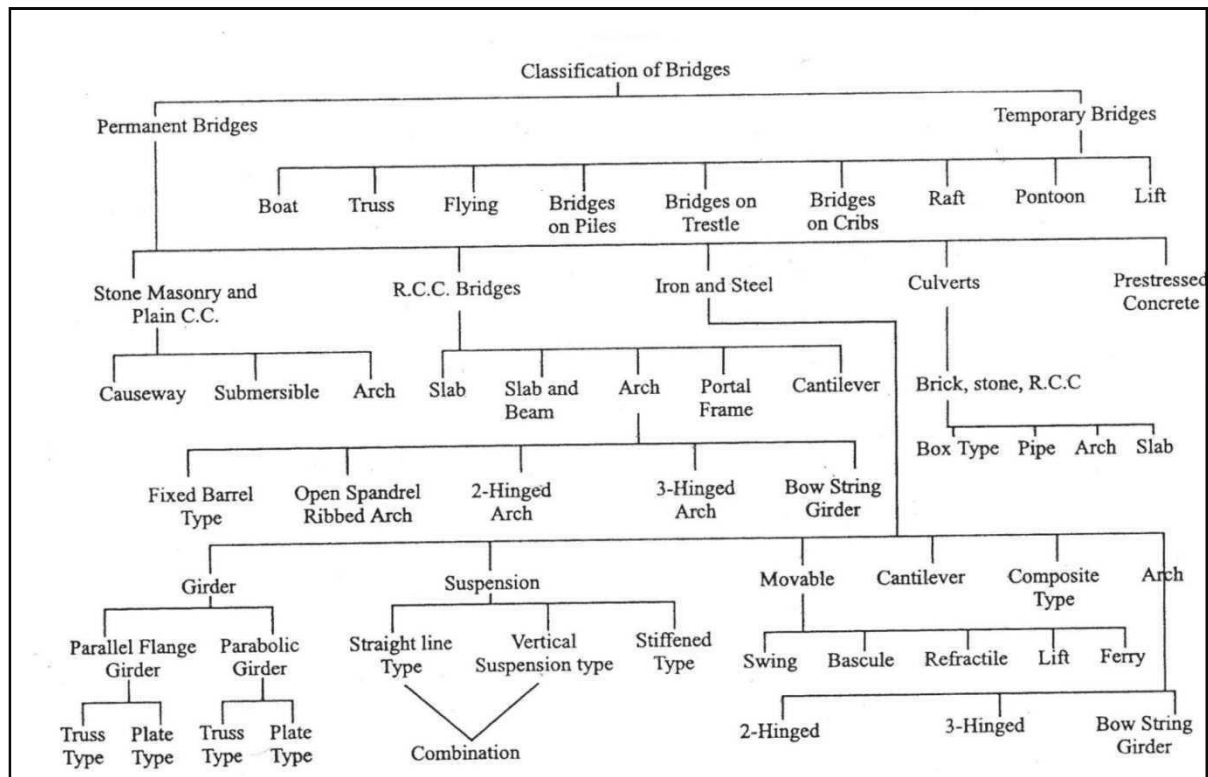
Classification of Bridge

REQUIRMENTS OF AN IDEAL BRIDGE:-

An ideal bridge meets the following requirements to fulfill the three criteria of efficiency, effectiveness and equity

➢ It serves the intended function with utmost safety and convenience

➢ It is aesthetically sound

➢ It is economical

The site characteristic of an Ideal Bridge has been discussed below:

1. The stream at the bridge side should be well defined and as narrow as possible.

2. There should be a straight reach of stream at bridge site

3. The site should have firm, permanent, straight and high banks.

4. The flow of water in the stream at the bridge site should be in steady regime condition. It should be free from whirls and cross-current.

5. There should be no confluence of large tributaries in the vicinity of bridge site

6. It should be reliable to have straight approach roads and square alignment, i.e. right-angledcrossing

7. There should be minimum obstruction of a natural waterway so as to have minimum afflux

8. In order to achieve economy there should be easy availability of labour, construction material and transport facility in the vicinity of bridge site.

9. In order to have minimum foundation cost, the bridge site should be such that no excessive work is to be carried inside the water

10. At bridge site it should be possible to provide secure and economical approaches.

11. In case of curved alignment the bridge should not be on the curve, but preferably on the tangent since otherwise there is a greater likelihood of accident as well as an added centrifugal force which increases the load effect on the structure and will require modification of design.

12. There should be no adverse environmental input.

13. The bridge site should be such that adequate vertical height and waterway is available

14. Underneath the bridge, river should be used for navigational purpose.

In actual practice the determination of best possible site for any proposed bridge is truly an economic problem. The various factors which should be carefully examined before setting final layout of a bridge are as follows:

i. Grade on alignment,

ii. Geographical Conditions,

iii. Government requirements,

iv. Commercial influences ,

v. Adjacent property consideration,

vi. General features of the bridge structure,

vii. Future trends for enlargement,

viii. Time Consideration,

ix. Foundation Considerations,

x. Construction facilities available,

xi. Erection Consideration,

xii. Aesthetics,

xiii. Maintenance and repairs,

xiv. Environmental Impact

CHAPTER-8

Bridge Alignment:-

Depending upon the angle which the bridge makes with the axis of the river, the alignment can be of two types:

a) Square Alignments: - In this the bridge is at right angle to the axis of the river.

b) Skew Alignments: - In this the bridge is at some angle to the axis of the river which is not a right

angle.

Note: - As far as possible, it is always desirable to provide the square alignment. The skew alignment suffers from the following disadvantages:-

(i) A great skill is required for the construction of skew Bridges. Maintenance of such type of Bridges is also difficult.

(ii) The water-pressure on piers in case of skew alignment is also excessive because of non-uniform flow of water underneath the bridge superstructure.

(iii) The foundation of skew bridge is more susceptible to scour action.

Flood Discharge: -

One of the essential data for the bridge design is fair assessment of the maximum flow which could be expected to occur at the bridges site during the design period of the bridge. The conventional practice in India for determination of flood discharge is to use a few convenient formulae or past records.

Note: - This faulty determination of flood discharge which led to failure of many hydraulic structures.

As per I.R.C. recommendation the maximum discharge which a bridge on a natural stream should be designed to pass should be determined by the following methods:-

(a) From the rainfall and other characteristics of the catchment.

(i) By use of an empirical formula applied to that region, or

(ii) By a rational method, provided it is possible to evaluate for the region concerned the various

factors employed in the method.

(b) From the hydraulic characteristics of the stream such as cross-sectional area, and slope of the stream allowing for velocity of flow.

(c) From the records available, if any, of discharges observed on the stream at the site of the bridge, or at any other site in its vicinity.

Empirical Methods for Estimation of Flood Discharge:-

In these methods area of a basin or a catchment is considered mainly. All other factors which influence peak flow are merged in a constant.

A general equation may be followed in the form:-

$$Q = C.M^n$$

Here, Q= Peak Flow or rate of maximum discharge

C= a constant for the catchment

M= area of catchment, and '**n**' is an index

The constant for catchment is arrived at, after taking the following factors into account:

(A) Basin Characteristics

a) Area

b) Shape

c) Slope

(B) Strom Characteristics

a) Intensity

b) Duration

c) Distribution

Limitations:

1. These methods do not take frequency of flood into consideration.

2. These methods cannot be applied universally.

3. Fixing of constant is very difficult and exact theory cannot be put forth for its selection.

1) **Dicken's Formula**

$$Q = C. M^{3/4}$$

Here, Q= Discharge in cum/sec

C= a constant

M= area of catchment in sq.km.

According to the area of catchment and amount of rainfall C varies from 11.02 to 22.04.

2) **Ryve's formula**

$$Q = C. M^{2/3}$$

This formula is used only in Southern India.

Here,   Q= Discharge in cum/sec

C= 6.74 for area within 24 km from coast, 8.45 for areas within 24-161 km from coast, 10.1 for limited hilly areas
In worst case C goes up to 40.5

M= area of catchment in sq .km.

3) **Inglis Formula**

This formula is used only in Maharashtra state and here three different cases are taken into consideration.

(a) For small areas only (It is also applicable for fan-shaped catchment)

$$Q = 123.2 \sqrt{M}$$

(b) For areas between 160 to 1000 square km.

$$Q = 123.2 \sqrt{M} - 2.62(M - 259)$$

 (c) For all type of catchment

$$Q = 123.2 M / \sqrt{(M + 10.36)}$$

In all equations, M= area of catchment in sq .km.

4) **Nawab Jang Bahadur's Formula:-**

$$Q = C (M / 2.59) (a - b. \log M)$$

Here, a, b, and C are constant.

$$a = 0.993 \text{ and } b = 1/14$$

$$C = 59.5 \text{ for North India or,}$$

$$= 48.1 \text{ for South India}$$

5) **Creager's Formula:-**

$$q = C. M^{n}$$

Here,   q=the peak flow per sq. km of a basin

M= area of catchment in sq. km. and '*n*' is some index

By multiplying both sides of the above equation area of the basin M, we get

$$Q = C. M^{n+1}$$

Where Q is peak value

Equation given by Creager, Justin and Hinds is

$$Q = 46. \, CM^{(0.894M-0.048)}$$

6) Khosla's Formula:-

It is a rational formula; it is based on the equation

P= R + L or R= P - L

Here, R is run off, P is rainfall and L is losses.

$L = 4.82 \, T_m$, where L is in mm and Tm is in centigrade   {in C.G.S. System}

$\therefore$ R= P - $4.82T_m$. This formula is for $T_m = 40°c$, for lesser values of T_m the L

value varies. It is useful for river valley project.

7) Besson's Formula:-

This formula is very rational and can be used in any case:

$Q_m = (P_m \, X \, Q_r) / (P_r)$

Here, Q_m = Peak flow expected

Q_r = Some observed peak flow

P_r = Observed rainfall

P_m = expected rainfall

Rational Methods for Estimation of Flood Discharge:-

This method is applicable for determination of flood discharge for small culverts only. In order to arrive at a rational approach, a relationship has been established between rainfall and runoff under various circumstances. The size of flood depends upon the following factors.

(i) Climate or Rainfall Factors. This includes

(a) Intensity (b) Distribution and (c) Duration of Rainfall

(ii) Catchment Area Factors. This includes:

(a) Catchment Area (b) its slope (c) its shape (d) porosity of soil

(e) Vegetable cover (f) initial state of wetness

WATERWAY

The area through which the water flows under a bridge superstructure is known as the waterway of the bridge. The linear measurement of this area along the bridge is known as the linear waterway. This linear waterway is equal to the sum of all the clear spans. This may be called as artificial linear waterway.

Due to the construction of a bridge the natural waterway gets contracted thereby increasing the velocity of flow under a bridge. This increased velocity results into heading up of water on the upstream of the river or stream, known as Afflux.

Economic Span: - the economic span of a bridge is the one which reduces the overall cost of a bridge to be minimum. The overall cost of a bridge depends upon the following factors

    a. Cost of material and its nature.

    b. Availability of skilled labour

    c. Span Length.

    d. Nature of stream to be bridged.

    e. Climatic and other conditions.

Notes:-

It is not in the hand of engineers to bring down the cost of living index or price of the materials like cement, steel, timber, etc. but they can help in bringing down the cost of bridges by evolving economical designs.

Considering only variable items, the cost of superstructure increases and that of sub-structure decreases with an increase in the span length. Thus most economic span length is that which stultifies the following:-

The cost of Super Structure = The cost of the Sub-Structure

AFFLUX

When a bridge is constructed, the structure such as abutments and piers cause the reduction of natural waterway area. The contraction of stream is desirable because it leads to tangible saving in the cost especially for alluvial stream whose natural surface width is too large than required for stability. Therefore, to carry the maximum flood discharge, the velocity under a bridge increases. This increased velocity gives rise to sudden heading up of water on the upstream side of the stream. The phenomenon of heading up of water on the upstream side of the stream is known as "**AFFLUX**"

Greater the afflux greater will be the velocity under the downstream side of the bridge and greater will be the depth of scour and consequently greater will be depth of foundations required.

Afflux is calculated by one of the following formula

    (A) Marriman's Formula

$$h_a = (V^2 / 2g) \{(A/C.a)^2 - (A/A_1)\}$$

Here, h_a = Afflux in meters

      V= Velocity of approach in meters per second

      A= Natural Waterway area at the site

      a=Contracted area in square meters

      A_1=The enlarged area at upstream of the bridge in square meters

C= Coefficient of Discharge = 0.75+0.35 (a/A) - 0.1(a/A) ^{2} approximately

(A) Molesworth's Formula

$$h_a = \frac{V^2}{17.9} + 0.015 \ \{(A/a)^2 - 1\}$$

Here, V, A and a have the same meaning as in the Marriman's Formula

CLEARANCE

To avoid any possibility of traffic striking any structural part clearance diagram are specified. The horizontal clearance should be the clear width and vertical clearance of the clear height, available for the passage of vehicular traffic as shown in the clearance diagram in the figure below.
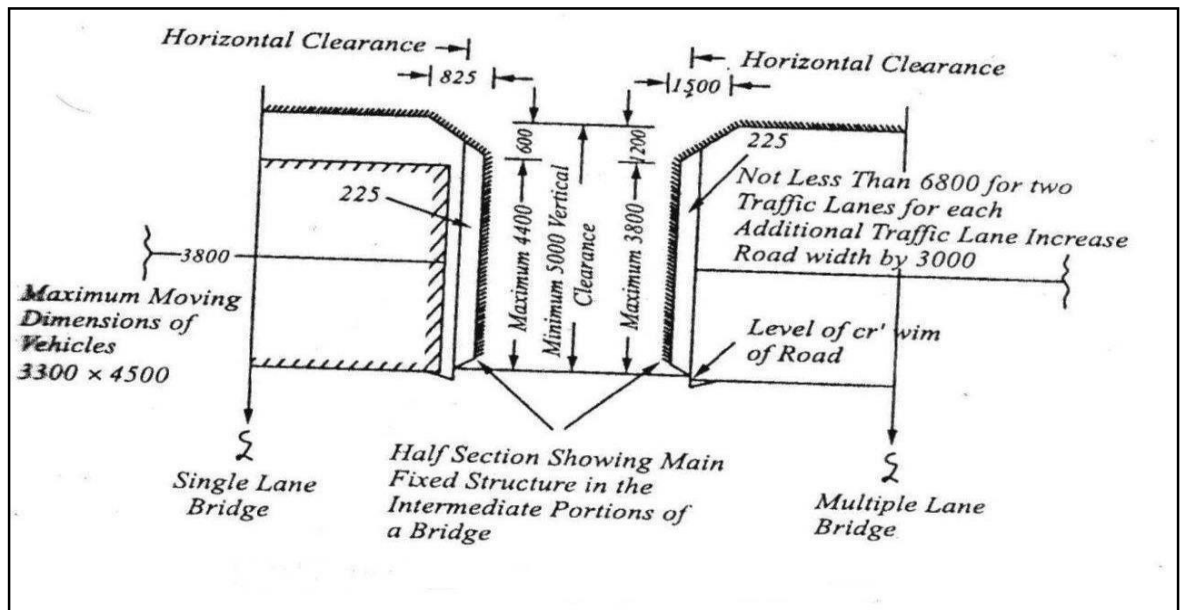
Clearance Diagram for Road Bridges

Note: - For a bridge constructed on a horizontal curve with super elevated road surfaces, the horizontal clearance should be increased on the side of inner kerb by an amount equal to 5m multiplied by the super elevation. The minimum vertical clearance should be measured from the super elevated level of roadway.

FREE BOARD

Free board is the vertical distance between the designed high flood level, allowing for the afflux, if any, and level of the crown of the bridge at its lowest point.

It is essential to provide the free board in all types of bridges for the following reasons:-

➢ Free Board is required to allow floating debris, fallen tree trunks and approaches waves to pass under the bridge.

➢ Free board is also required to allow for the afflux during the maximum flood discharge due to contraction of waterway.

➢ Free board is required to allow the vessels to cross the bridges in case of navigable rivers. The value of free-broad depends upon the types of the bridge.

Collection of Bridge Design Data: -

For a complete and proper appreciation of the bridge project the engineer in charge of the investigation should carry out studies regarding its financial, economic, social and physical feasibility. The detailed information to be collected may cover loading to be used for design based on the present and anticipated future traffic, hydraulic data based on stream characteristics, geological data, subsoil data, climatic data, alternative sites, aesthetics, cost etc.

The following drawings containing information as indicated should be prepared

1. INDEX MAP
2. CONTURE SURVEY PLAN
3. SITE PLAN
4. CROSS-SECTION
5. LONGITUDINAL SECTION
6. CATCHEMENT AREA MAP
7. SOIL PROFILE

Design data for major bridge:-

A- General data:-

(i) Name of the road and its classification.

(ii) Name of the stream.

(iii) Location of nearest G.T.S. bench mark and its reduced level.

(iv) Chainage at centre line of the stream.

(v) Existing arrangement for crossing the stream.

a) During Monsoon   b) During dry season

(vi) Liability of the site to earthquake disturbance

B- Catchment Area and Run off Data:-

(i) Catchment Area

(a) Hilly Area   b) In plains

(ii) Maximum recorded intensity and frequency of rainfall in catchment.

(iii) Rainfall in centimeters per year in a region.

(iv) Length of catchment in kilometres.

(v) Width of catchment in kilometres.

(vi) Longitudinal slope of catchment.

(vii) Cross slope of catchment.

(viii) The nature of catchment and its shape.

C- Data Regarding Nature of Stream

D- Data regarding Alignment & Approaches
E- Superstructure Data
F- Foundation Data
G- Data of existing structure
H- Miscellaneous Data

Sub-Surface Investigation:-

Sub-Surface investigation is essential for to know the properties of the bridge site soil. The field and laboratory investigations required to obtain the necessary soil data for the design are called soil exploration.

The principal requirements of a complete investigation can be summarized as follows:-

1. Nature of the soil deposits up to sufficient depth.
2. Depth, thickness and composition of each soil stratum.
3. The location of ground water.
4. Depth to rock and composition of rock.
5. The engineering properties of soil and rock strata that affect the design of the structure.

In exploration programme, the extent of distribution of different soils both in the horizontal and vertical directions can be determined by the following methods:

1. By use of open pits.
2. By making bore holes and taking out samples.
3. By Soundings.
4. By use of geophysical methods.

Equipments for laboratory Work:-

The disturbed soil sample as taken from bed level to scour level at every one meter interval or at depths wherever strata changes are tested to determine the following properties:-

1. Liquid Limit, Plastic Limit and Plasticity Index,
2. Organic Content,
3. Harmful Salts,
4. Sieve Analysis,
5. Silt Factor.

The undisturbed soil samples as taken below the scour level to a level where the pressure is about 5% of the pressure at the base are tested to determine

1. Particle size analysis,
2. Values of cohesion and angle of internal friction by shear test,
3. Compression index and pre-consolidation pressure by consolidation test,
4. Density specific gravity and moisture content.

The equipment used for the above tests are as per relevant IS specifications.

Advantage of Sub-Surface Investigation:-

There are manifold advantages of carefully planned investigation programme. These can be summarized as below:-

1. A suitable and economical solution can be worked out.
2. The construction schedule can be properly planned.
3. The extent and nature of difficulties likely to be met with can be determined.
4. The rate and amount of settlements can be determined.
5. The variation in the water –table, of the presence of artesian pressures can be found out.

CHAPTER 9

9.1 Depth of Scour:-

DEPTH OF SCOUR (D) is the depth of the eroded bed of the river, measured from the water level for the discharge considered. Well-laid foundation is mostly provided in road and railway bridges in India over large and medium-sized rivers. The age-old Lacey–Inglis method issued for estimation of the design scour depth around bridge elements such as pier, abutment, guide bank, spur and groyene. Codal provisions are seen to produce too large a scour depth around bridge elements resulting in bridge sub-structures that lead to increased construction costs. Limitations that exist in the codes of practice are illustrated in this paper using examples. The methods recently developed for estimation of the scour are described. New railway and road bridges are required to be built in large numbers in the near future across several rivers to strengthen such infrastructure in the country. It is strongly felt that provisions in the existing codes of practice for determination of design scour depth require immediate review. The present paper provides a critical note on the practices followed in India for estimating the design scour depth.

Indian practices on estimation of design scour depth

1. Lacey–Inglis method
2. Comments on Lacey's method

- The probable maximum depth of scour for design of foundations and training and protection works shall be estimated considering local conditions.

- Wherever possible and especially for flashy rivers and those with beds of gravel or boulders, sounding for purpose of determining the depth of scour shall be taken in the vicinity of the site proposed for the bridge. Such soundings are best taken during or immediately after a flood before the scour holes have had time to silt up appreciably. In calculating design depth of scour, allowance shall be made in the observed depth for increased scour resulting from:

(i) The design discharge being greater than the flood discharge observed.

(ii) The increase in velocity due to the constriction of waterway caused by construction of thebridge.

(iii) The increase in scour in the proximity of piers and abutments.

- 4.6.3 In the case of natural channels flowing in alluvial beds where the width of waterway provided is not less than Lacey's regime width, the normal depth or Scour (D) below the foundation design discharge (Qf) level may be estimated from Lacey's formula as indicated below

$$D = 0.473 \ (Q^f / f)^{1/3}$$

Where D is depth in metres Qf is in cumecs and 'f' is Lacey's silt factor for representative sample of bed material obtained from scour zone.

- Where due to constriction of waterway, the width is less than Lacey's regime width for Qf or where it is narrow and deep as in the case of incised rivers and has sandy bed, the normal depth of scour may be estimated by the following formula:

$$D = 1.338 \ (Q_f^2 / f)^{1/2}$$

Where 'Q$_f$' is the discharge intensity in cubic metre per second per metre width and f is silt factor
The silt factor 'f' shall be determined for representative samples of bed material collected from scour zone using the formula : $f = 1.76 \sqrt{m}$ where m is weighted mean diameter of the bed material particles in mm. Values of 'f' for different types of bed material commonly met with are given below :

Type of bed	Material Weighted mean dia of particle(mm)	Value of 'f'
(i)Coarse silt	0.04	0.35
(ii) Fine sand	0.08 0.15	0.50 0.68
(iii) Medium sand	0.3 0.5	0.96 1.24
(iv) Coarse sand	0.7 1.0 2.0	1.47 1.76 2.49

The depth calculated (vide clause 4.6.3 and 4.6.4 above) shall be increased as indicated below, to obtain maximum depth of scour for design of foundations, protection works and training works:-

Nature of the river	Depth of scour
In a straight reach	1.25D
At the moderate bend conditions e.g. along apron of guide bund	1.5D
At a severe bend	1.75D
At a right angle bend or at nose of piers	2.0D
In severe swirls e.g. against mole head of a guide bund.	2.5 to 2.75D

In case of clayey beds, wherever possible, maximum depth of scour shall be assessed from actual observations.

9.2 Types of Bridge
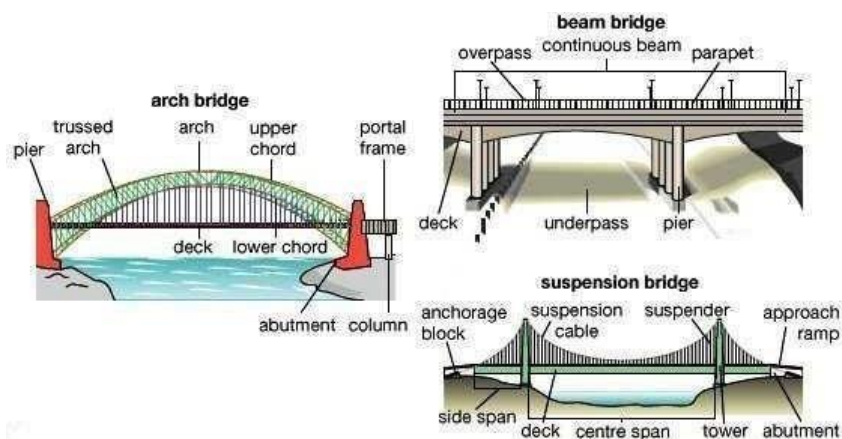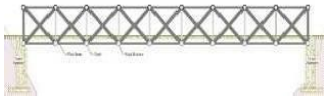
Types of Bridge

Arch bridge	
Arch bridge (concrete)	
Through arch bridge	
Beam bridge	
Log bridge (beam bridge)	
Cavity wall Viaduct	
Bowstring arch	
Box girder bridge	

Cable-stayed bridge	
Cantilever bridge	
Cantilever spar cable-stayed bridge	
Clapper bridge	
Covered bridge	
Girder bridge	
Continuous span girder bridge	
Moon bridge	
Movable bridge	

Pigtail bridge	
Plate girder bridge	
Pontoon bridge	
Roving bridge	
Segmental bridge	
Self-anchored suspension bridge	
Side-spar cable-stayed bridge	
Simple suspension bridge (Inca rope bridge))	
Step-stone bridge	
Stressed ribbon bridge	
Suspension bridge	

[Transporter bridge](#)	
[Trestle](#)	
[Truss arch bridge](#)	
[Truss bridge](#)	
[Vierendeel bridge](#)	
[Brown truss](#)	
[Covered bridge](#)	
[Lattice truss bridge](#) (Town lattice truss	
[Tubular bridge](#)	

Bridge Foundation:-

Definition:- A foundation is the part of the structure which is in direct contact with the ground. It transfers the load of the structure to the soil below. Before deciding upon its size, we must ensure that:

    (i)   The bearing pressure at the base does not exceed the allowable soil pressure.

    (ii)   The settlement of foundation is within reasonable limits

    (iii)   Differential settlement is to limited as not to cause any damage to the structure.

Broadly, foundation may be classified under two categories i.e.

1. Shallow foundation
2. Deep Foundation

Shallow Foundation:-According to Trezaghi, a foundation is said to be shallow if its depth is equal or less than its width.

Deep Foundation:- According to Trezaghi, a foundation is said to be deep , the depth is greater than its width and it cannot be prepared by open excavation.

Types of Bridge Foundation:-

The selection of foundation type suitable for a particular site depends on the following considerations:-

1) Nature of Subsoil

2) Nature and extent of difficulties, e.g. presence of boulder, buried tree trunks, etc. Likely to be met with, and

3) Availability of expertise and equipment.

Depending upon their nature and depth, bridge foundation can be categories as follows:

   i.   **Open Foundation,**
  ii.   **Raft Foundation,**
 iii.   **Pile Foundation,**
  iv.   **WellFoundation,**

1(i)Open Foundation in

Bridges:-

1. An open foundation or spread foundation is a type of foundation and can be laid using open excavation by allowing natural slopes on all sides.
2. This type of foundation is practicable for a depth of about 5m and is normally convenient above the water table.
3. The base of the pier or abutment is enlarged or spread to provide individual support.
4. Since spread foundations are constructed in open excavation, therefore, they are termed as open foundation.
5. This type of foundation is provided for bridges of moderate height built on sufficiently form day ground.
6. The piers in such cases are usually made with slight batter and provided with footings widened at bottom. Where the ground is not stiff the bearing surface is further extended by a wide layer of concreter at bottom (see the figure).

(ii) Raft Foundation:-

1. A raft foundation or mat is a combined footing that covers the entire area beneath a bridge and supports all the piers and abutments.
2. When the allowable soil pressure is low, or bridge loads are heavy, the use of spread footing would cover more one-half of the area, and it may prove more economical to use raft foundation
3. They are also used where the soil mass contains compressible lenses so that the differential settlement would be difficult to control.
4. The raft tends to bridge over the erratic deposits and eliminates the differential settlement.

5. Raft foundation is also used to reduce the settlement above highly compressible soils by making the weight of bridge and raft may undergo large settlement without causing harmful differential settlement. For this reason, almost double settlement of that permitted for footings is acceptable for rafts.

6. Usually when hard soil is not available within 1.5 to 2.5 m a raft foundation is adopted.

7. The raft is composed of reinforced concrete beams a relatively thin slab underneath, figure

(iii) Pile foundation in Bridges:

1. The pile foundation is constructions for the foundation of abridge pier or abutment supported on piers.

2. A pile is an element of construction composed of timber, concrete or steel or combination ofthem.

3. Pile foundation may be defined as a column support type of foundation which may be cast-in-situ or precast.

4. The piles may be place separately or they may be placed in form of a cluster throughout the length of the pier or abutment.

5. This type of construction is adopted when the loose soil extends to great depth.

6. The load of the bridge is transmitted by the piles to hard stratum below or it is resisted by the friction developed on the sides of piles.

Classification of piles:-

Piles are broadly classified into two categories:-.

i- Classification based on the function

ii- Classification based on the materials and composition

Classification based on the function

➢ Bearing Pile.

➢ Friction Pile.

➢ Screw Pile.

➢ Compaction Pile.

➢ Uplift Pile.

➢ Batter Pile.

➢ Sheet Pile.

Classification based on the function

➢ Cement concrete piles.

➢ Timber Piles.

➢ Steel Piles.

➢ Sand Piles.

➢ Composite Piles.

(iv) Well Foundation in bridges

a) Well foundations are commonly used for transferring heavy loads to deep strata in river or sea bed for bridges, transmission towers and harbour structures. The situation where well foundations are resorted are as below as) Wherever consideration of scour or bearing capacity require foundation to be taken to depth of more than 5 M below ground level open foundation becomes uneconomical. Heavy excavation and dewatering problem coupled with effort involve in retaining the soil makes the open foundation costlier in comparison to other type of foundation.

b) Soil becomes loose due to excavation around the open foundation and hence susceptible to scouring. This is avoided in well foundation which is sunk by dredging inside of the well.

c) From bearing pressure considerations, a well foundation can always be left hollow thereby considerably reducing bearing pressure transmitted to the foundation material. This is very important in soils of poor bearing capacity, particularly in clayey soils. In other type of foundation, the soil displaced is occupied by solid masonry/concrete which are heavier than the soil displaced and hence this does not give any relief in respect of adjusting bearing capacity. However in case of well foundation this is easily achieved because of cellular space left inside the well.

Caisson:-
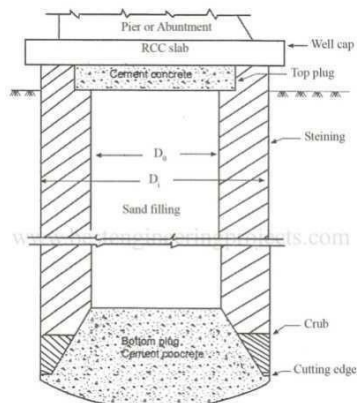
Fig 1 Parts of a Well Foundation

1. Cutting Edge
2. Curb
3. Steining
4. Bottom Plug
5. Filling
6. Top Plug
7. Vert. Reinforcement
8. Horizontal Reinforcement
9. Vert. Section.
10. Horizontal Section
11. Well Cap
12. Sub-structure
13. Superstructure
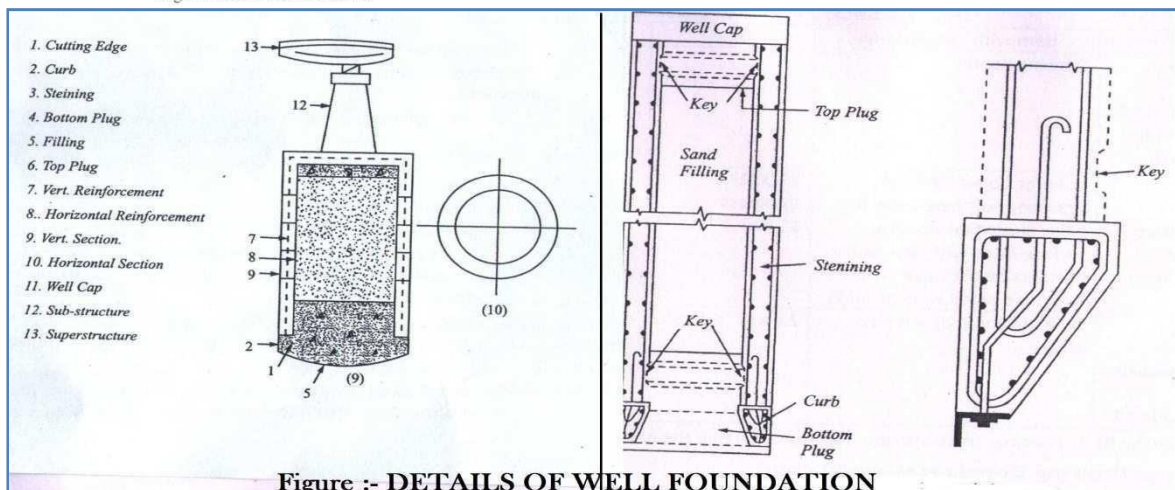
Figure :- DETAILS OF WELL FOUNDATION

Caisson: - The caisson is a structure used for the purpose of placing as foundation in correct position under water. The term caisson is derived from the French word 'caisse' meaning a box. It is a member with hollow portion, which after installing in palace by any means is filled with concrete or other material. Caissons are prepared in sandy soils the caissons can be divided in the following three groups

        a. Box Caissons

        b. Open Caissons or Wells

        c. Pneumatic Caissons

Well components and their functions:

- *Cutting edge:* - It provides a comparatively sharp edge to cut the soil below during sinking operation. It is usually consists of a mild steel equal angle of side 150mm.

- *Curb: -* It has a two-fold purpose. During sinking it acts as an extension of cutting edge and also provided support to the well steining and bottom plug while after sinking it transfers the load to the soil below. It is made up of reinforced concrete using controlled concrete of grade M200.

- *Steining:-* It is the main body of the well. It is serves dual purpose. It acts as a cofferdam during sinking and structural member to transfer the load to the soil below afterwards. The steining may consist of brick masonry or reinforced concrete. The thickness of steining should not be less than 4.5 cm not less than that given by equation.

$$t= K \left\{ (H/100) + (D/10) \right\}$$

Here,   t= minimum concrete steining thickness.

      H= well depth below bed

      D= External diameter of Well

      K= a constant which is 1.0 for sandy strata.

- Bottom Plug: - Its main function is to transfers load from the steining to the soil below.

- Sand Plug: - Its utility is doubtful. It is supposed to afford some relief to the steining by transforming directly a portion of load from well cap to bottomplug.

- Top Plug: - The opinion is divided about the top plug. It, at least, serves as a shuttering for laying well cap.

- Reinforcement: – It provides requisite strength to the structure during sinking and service.

➢ Well Cap: - It is needed to transfer the loads and moments from the pier to the well or wells below. The shape of well cap is similar to that of the well with a cantilevering of about 15cm. Whenever 2 or 3 wells of small diameter are needed to support the sub-structure , the well cap designed as a slab resting over the well or wells with partial fixity at the edges of the wells.

➢ Depth of Well Foundation:- As per I.R.C. Bridge Code (Part-III), the depth of well foundation is to be decided on the following consideration

1. The minimum depth of foundation below the H.F.L. should be 1.33D, Where D is the anticipated max. Depth of scour below H.F.L. Depth should provided proper grip according to some rational formula.

2. The max bearing pressure on the subsoil under the foundation   resulting form any combination of the loads and forces except wind and seismic forces should not exceed the safe bearing capacity of the subsoil, after taking into account the effect of scour.

    With wind and seismic forces in addition, the max. Bearing pressure should not exceed the safe bearing capacity of the subsoil by more than 25%.

3. While calculating max. Baring pressure on the foundation bearing layer resulting from the worst combination of direct forces and overturning moments, the effect of a passive resistance of the earth on sides of the foundation structure may be taken into account below the max, depth of the scour only.

4. The effect of skin friction may be allowed on the portions below the max, depth of scour. Accordingly for deciding the depth of well foundation, we require correct estimation of the following.

    1. Max. Sour depth.
    2. Safe bearing capacity.
    3. Skin friction.
    4. Lateral earth support below max. Scour level.

It is always desirable to fix the level of a well foundation on a sandy strata with adequate bearing capacity. Whenever a thin stratum of clay occurring between two layers of sand is met with, in that case well must be pierced through the clayey strata. If at all foundation has to be laid on clayey layer it should be ensured that the clay is stiff.

Design loads and Forces. The forces acting o n a bridge structure, to be considered fo the design of a well foundation, are as follows:

Vertical

(i)   Dead load,

(ii)   Live load,

(iii)   Buoyancy.

Horizontal

(i)   Wind force.

(ii)   Force due to water currents.

(iii)   Longitudinal forces caused by the tractive effort of vehicle or by braking effect of vehicles.

(iv)   Longitudinal force on account of resistance of the bearing against movement due to variations of temperature.

(v)   Seismic force.

(vi)   Earth pressure.

(vii)   Centrifugal force.

The I.R.C. Bridge code II stipulates the magnitude of above loads and forces. The magnitude, direction and point of application of all the above forces can be resolved into two horizontal forces, P and Q and a single vertical force W under the worst possible combinations.

Chapter 10

10.1 Piers:-

       Piers provide vertical supports for spans at intermediate points and perform two main functions: transferring superstructure vertical loads to the foundations and resisting horizontal forces acting on the bridge. Although piers are traditionally designed to resist vertical loads, it is becoming more and more common to design piers to resist high lateral loads caused by seismic events. Even in some low seismic areas, designers are paying more attention to the ductility aspect of the design. Piers are predominantly constructed using reinforced concrete. Steel, to a lesser degree, is also used for piers. Steel tubes filled with concrete (composite) columns have gained more attention recently.
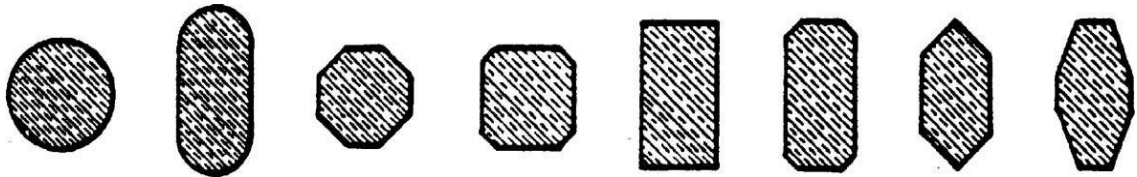
FIGURE: 1 : Typical cross-section shapes of piers for overcrossings or viaducts on land.

       Pier is usually used as a general term for any type of substructure located between horizontal spans and foundations. However, from time to time, it is also used particularly for a solid wall in order to distinguish it from columns or bents. From a structural point of view, a column is a member that resists the lateral force mainly by flexure action whereas a pier is a member that resists the lateral force mainly by a shear mechanism. A pier that consists of multiple columns is often called a bent.
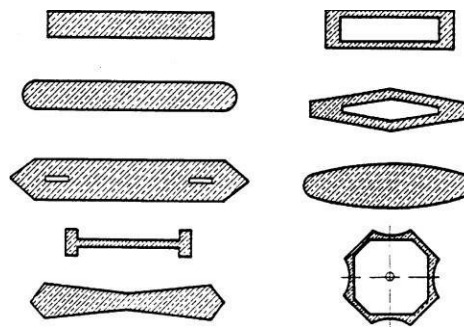
FIGURE:-2 Typical cross-section shapes of piers for river and waterway crossings.

       There are several ways of defining pier types. One is by its structural connectivity to the superstructure: monolithic or cantilevered. Another is by its sectional shape: solid or hollow; round, octagonal, hexagonal, or rectangular. It can also be distinguished by its framing configuration: single or multiple columns bent; hammerhead or pier wall. Selection of the type of piers for a bridge should be based on functional, structural, and geometric requirements. Aesthetics is also a very important factor of selection since modern highway bridges are part of a city's landscape. Figure-1 shows a collection of typical cross section shapes for overcrossings and viaducts on land and Figure-2 shows some typical cross section shapes for piers of river and waterway crossings. Often, pier types are mandated by government agencies or owners. Many state departments of transportation in the United States have their own standard column shapes.

Broadly piers are classified under following two categories:-

    I.   Solid Piers.

    II.   Open Piers.

__Solid wall piers__, as shown in Figures 3-a and 4, are often used at water crossings since they can be constructed to proportions that are both slender and streamlined. These features lend themselves well for providing minimal resistance to flood flows.

__Hammerhead piers__, as shown in Figure 3-b, are often found in urban areas where space limitation is a concern. They are used to support steel girder or precast prestressed concrete superstructures. They are aesthetically appealing. They generally occupy less space, thereby providing more room for the traffic underneath. Standards for the use of hammerhead piers are often maintained by individual transportation departments. A column bent pier consists of a cap beam and supporting columns forming a frame.

__Column bent piers,__ as shown in Figure 3-c and Figure 27.5, can either be used to support a steel girder superstructure or be used as an integral pier where the cast-in-place construction technique is used. The columns can be either circular or rectangular in cross section. They are by far the most popular forms of piers in the modern highway system.

__A pile extension pier__ consists of a drilled shaft as the foundation and the circular column extended from the shaft to form the substructure. An obvious advantage of this type of pier is that it occupies minimal amount of space. Widening an existing bridge in some instances may require pile extensions because limited space precludes the use of other types of foundations.

(a) Solid wall pier   (b) Hammerhead pier   (c) Rigid frame pier
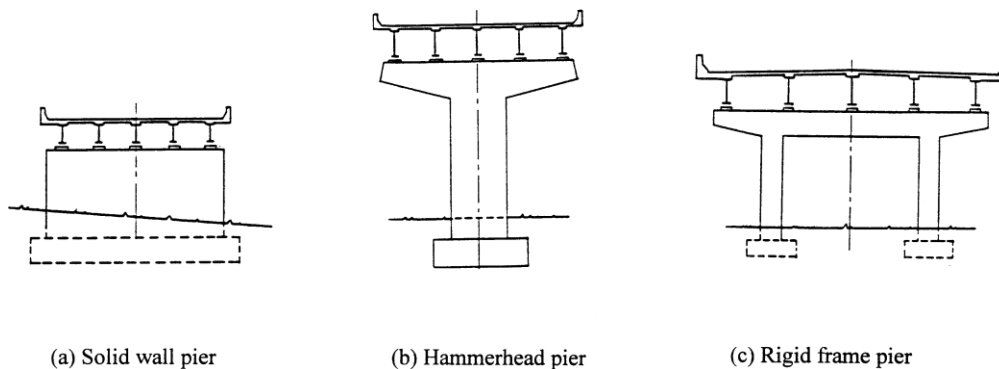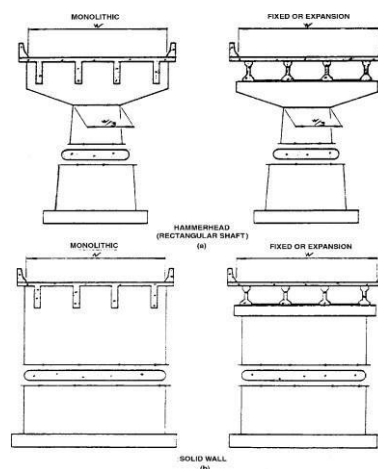
FIGURE-3

FIGURE-4
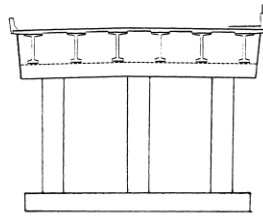
(a) Bent for precast girders   (b) Bent for cast-in-place girders
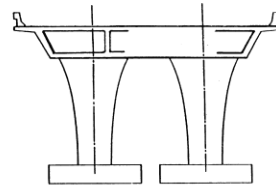
10.2 *Abutments:-*

They are the end supports of the superstructure, retaining earth on their back. They are built either with masonry, stone or brick work or ordinary mass concrete or reinforced concrete. The top surface of the abutment is made flat when the superstructure is of trusses or girders or semi-circular arch. In case of segmental or elliptical arch type of superstructure, the abutment top is made skew. Weep holes are provided at different levels through the body of the abutment to drain of the retained earth.

The salient features of bridge abutments are listed below.

(a) Height. The height of the abutments is kept equal to that of the piers.

(b) Abutment batter. The water face of the abutment is usually kept vertical or could be given a batter of 1 in 12 to 1 in 24 as of piers. The face retaining earth is given a batter of 1 in 6 or may be stepped down.

(c) Abutment Width. The top width of the abutment should provide enough space for the bridge seat and for the construction of a dwarf wall to retain earth up to the approach level.

(d) Length of Abutment. The length of abutment is kept at least equal to the width of the bridge.

(e) Abutment cap. The design is similar to that of pier cap.

Abutments can be spill-through or closed. The spill through abutment generally has a substantial berm to help restrain embankment settlement at the approach of the structure.

Approach embankment settlement can also be accommodated by approach slabs to eliminate bumps at the bridge ends, closed abutments partially or completely retain the approach embankments from spilling under the span, and Bridges of several spans require expansion at the abutments. Therefore they are no usually required to resist the longitudinal forces that develop.

Broadly, abutments are classified under the following categories.

   1. Abutments with wing walls

   2. Abutments without wing walls

   Abutments with wing walls

      (a) Straight Wing walls

      (b) Splayed Wing walls

      (c) Return Wing Walls

   Abutments without wing walls

      (a) Buried Abutments

      (b) Box Abutments

      (c) Tee Abutments

      (d) Arch Abutments

Figure:- Buried Abutment

Figure:- BOX ABUTMENT

Figure:-TEE ABUTMENT

Figure:- Arch Abutments
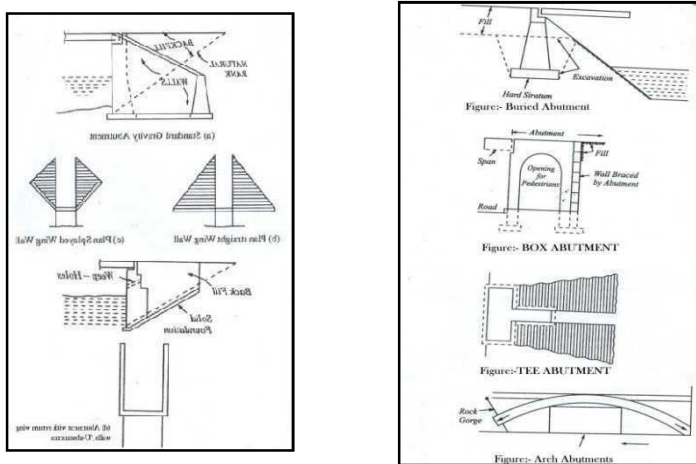
FIGURES: - ABUTMENTS

Buried Abutments: - This type of abutments is generally built prior to the placing of the fill. Since it is filled on both sides the earth pressure is low. Superstructure erection can be begin before placement of fill .

Box Abutments: - This employs a short span of bridge built integral with columns to act as a frame and resist earth pressure of the approaches. It is most often used overpass work where the short span may be employed for pedestrian passage (see figure).

Tee Abutments: - This type looks like T in plan and has now become absolute (see figure)

Arch Abutments: - This type of abutment is used where arches are employed because of their economy in certain conditions. The high inclined skewback thrusts are difficult to handle unless the abutment can be seated in rock. Therefore, they are often used for span over gorges. (see figure)

10.3 WING WALLS:

In a bridge, the wing walls are adjacent to the abutments and act as retaining walls. They are generally constructed of the same material as those of abutments. The wing walls can either be attached to the abutment or be independent of it. Wing walls are provided at both ends of the abutments to retain the earth filling of the approaches. Their design period depends upon the nature of the embankment and does not depend upon the type or parts of the bridge.

The soil and fill supporting the roadway and approach embankment are retained by the wing walls, which can be at a right angle to the abutment or splayed at different angles. The wing walls are generally constructed at the same time and of the same materials as the abutments.

Classification of wing walls

Wing walls can be classified according to their position in plan with respect to banks and abutments. The classification is as follows:

1. Straight Wing walls: They are used for small bridges, on drains with low banks and for railway bridges in cities (weep holes are provided).

2. Splayed Wing walls: These are used for bridges across rivers. They provide smooth entry and exit to the water. The splay is usually 45°. Their top width is 0.5 m, face batter 1 in 12 and back batter 1 in 6, weep holes are provided.

3. Return Wing walls: They are used where banks are high and hard or firm. Their top width is 1.5 m and face is vertical and back battered 1 in 4. Scour can be a problem for wing walls and abutments both, as the water in the stream erodes the supporting soil.

CHAPTER-11

PERMANENT BRIDGES

Masonry Bridges:-

Bridge unit the spandrel, which supports the bridge roadway. The spandrel is made from gravel or crushed stone backing held in by lateral (side) walls made of concrete masonry or stonework or in the form of an open main load-bearing structures are made of natural stone, brick, or concrete blocks. Such a bridge is always arched, with massive supports. The main load-bearing element of a masonry bridge is the arch, over which is structure of small arches resting on crosswalk. The advantages of a masonry bridge are its architectural attractiveness and its durability. Masonry bridges are known that have been in use for more than 1,000 years. The basic short comings that limit the use of masonry bridges are their complexity and labor intensiveness of construction.. Their simplicity, economy and ease with which pleasing appearance can be obtained make them suitable for this purpose.
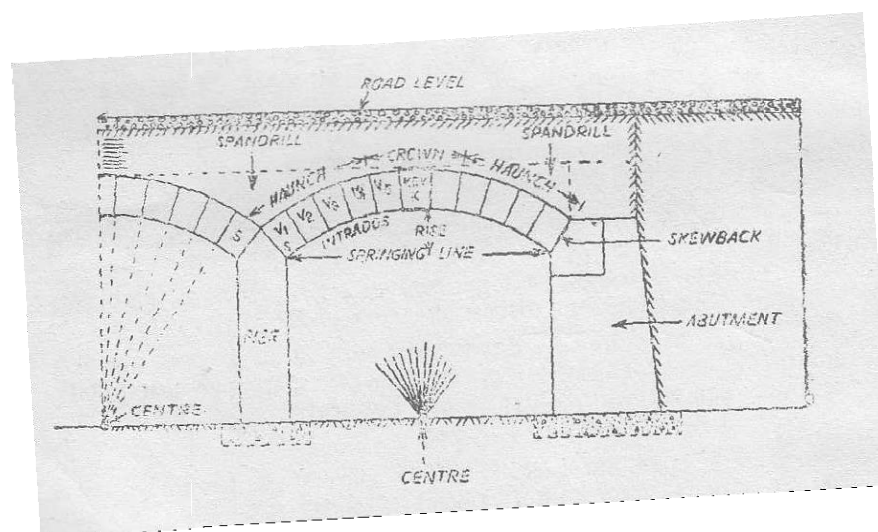
Classification of steel bridges

Steel bridges are classified according to

- the type of traffic carried
- the type of main structural system
- the position of the carriage way relative to the main structural system

These are briefly discussed in this section.

Classification based on type of traffic carried

Bridges are classified as

- Highway or road bridges
- Railway or rail bridges
- Road - cum - rail bridges

Classification based on the main structural system

Many different types of structural systems are used in bridges depending upon the span, carriageway width and types of traffic. Classification, according to makeup of main load carrying system, is as follows:

(i) Girder bridges - Flexure or bending between vertical supports is the main structural action in this type. Girder bridges may be either solid web girders or truss girders or box girders. Plate girder bridges are adopted for simply supported spans less than 50 m and box girders for continuous spans up to 250 m. Cross sections of a typical plate girder and box girder bridges are shown in Fig.7.2 (a) and Fig. 7.2(b) respectively. Truss bridges [See Fig.7. 2(c)] are suitable for the span range of 30 m to 375 m. Cantilever bridges have been built with success with main spans of 300 m to 550 m. . They may be further, sub-divided into simple spans, continuous spans and suspended-and-cantilevered spans, as illustrated in Fig.7. 3.
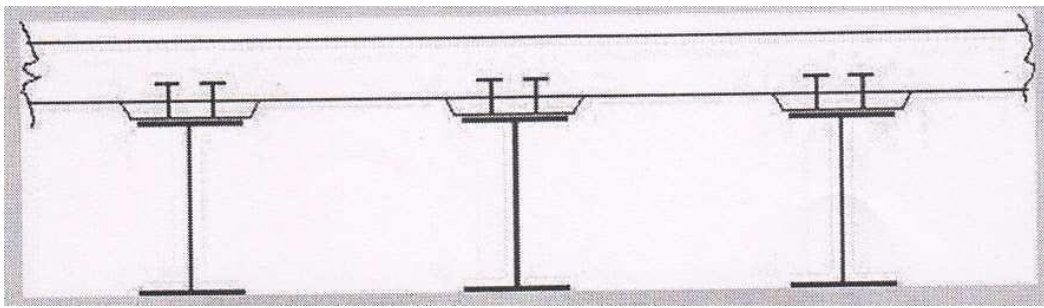
Fig.7.2 (a) Plate girder bridge section
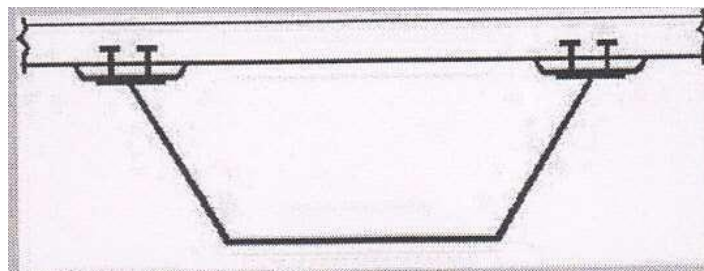
Fig.7.2 (b) Box girder bridge section
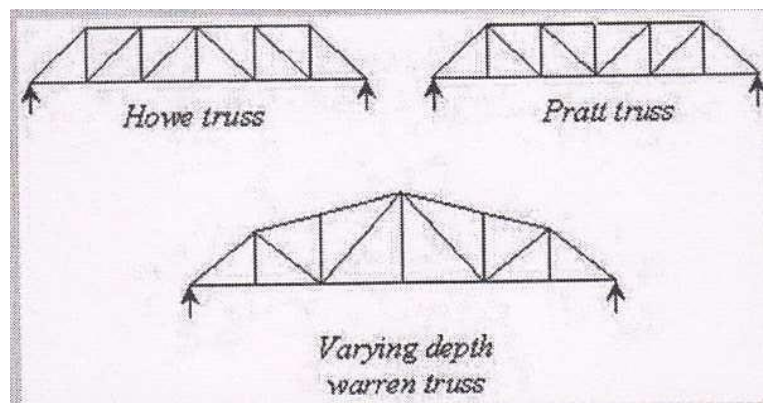
Fig.7.2 (c) Some of the trusses used in steel bridges
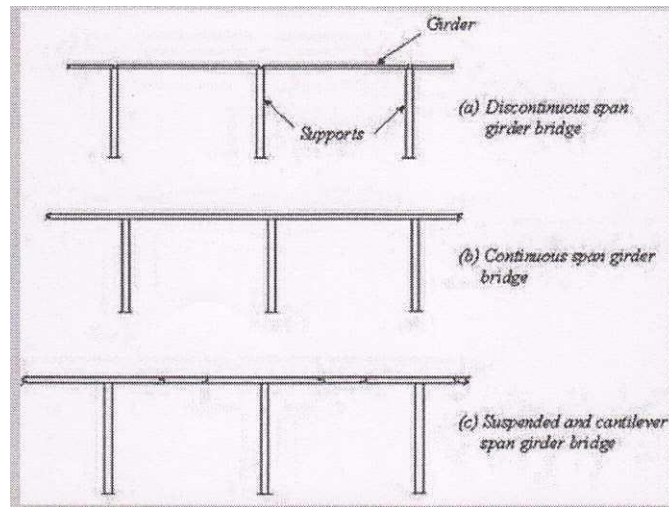
Fig.7.3 Typical girder bridges

(ii) Rigid frame bridges - In this type, the longitudinal girders are made structurally continuous with the vertical or inclined supporting member by means of moment carrying joints [Fig.7.4]. Flexure with some axial force is the main forces in the members in this type. Rigid frame bridges are suitable in the span range of 25 m to 200 m.
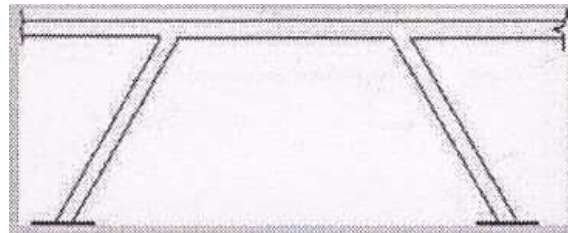
Fig.7.4 Typical rigid frame bridge

(iii) Arch bridges

The loads are transferred to the foundations by arches acting as the main structural element. Axial compression in arch rib is the main force, combined with some bending. Arch bridges are competitive in span range of 200 m to 500 m.
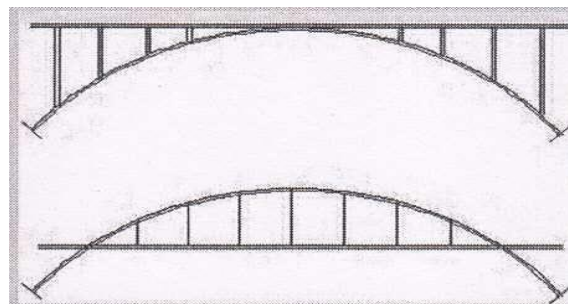
(iv) Cable stayed bridges - Cables in the vertical or near vertical planes support the main longitudinal girders. These cables are hung from one or more tall towers, and are usually anchored at the bottom to the

girders. Cable stayed bridges are economical when the span is about 150 m to 700 m. Layout of cable stayed bridges are shown in Fig. 7.6.
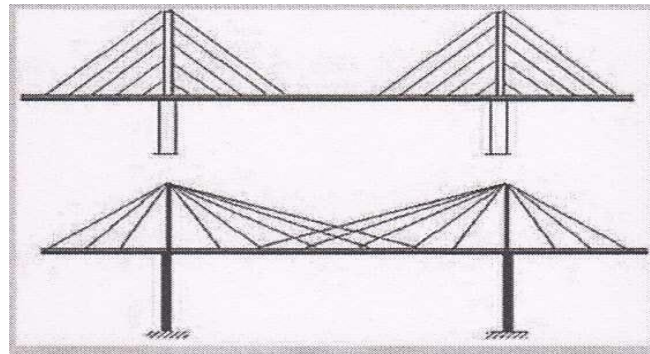
Fig.7.6 Layout of cable stayed bridges

(v) Suspension bridges - The bridge deck is suspended from cables stretched over the gap to be bridged, anchored to the ground at two ends and passing over tall towers erected at or near the two edges of the gap. Currently, the suspension bridge is best solution for long span bridges. Fig. shows a typical suspension bridge. Fig. 7.8 shows normal span range of different bridge types.
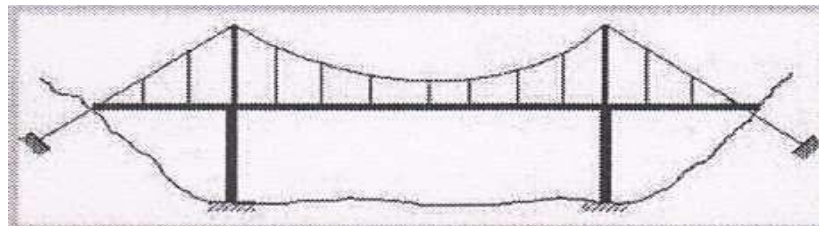
Fig.7.7 Suspension bridge

Classification based on the position of carriageway

The bridges may be of the "deck type", "through type" or "semi-through type". These are described below with respect to truss bridges:

(i) Deck type bridge -The carriageway rests on the top of the main load carrying members. In the deck type plate girder bridge, the roadway or railway is placed on the top flanges. In the deck type truss girder bridge, the roadway or railway is placed at the top chord level as shown in Fig. 7.9(a).
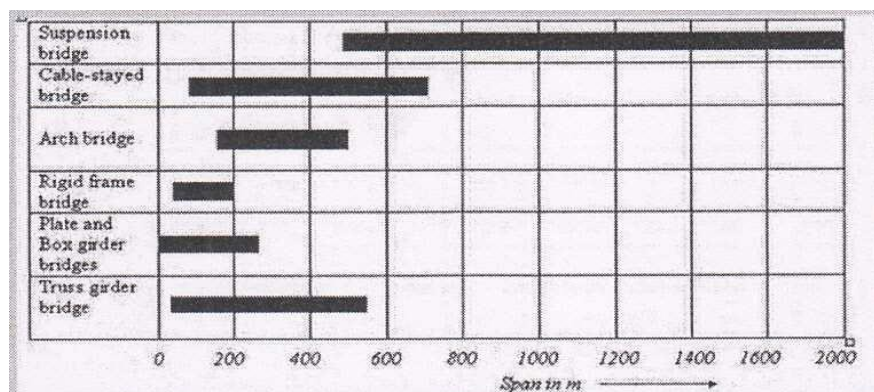
Fig.7.8 Normal span ranges of bridge system

(a) Deck type truss bridge

(b) Through type truss bridge

(c) Semi through type truss bridge
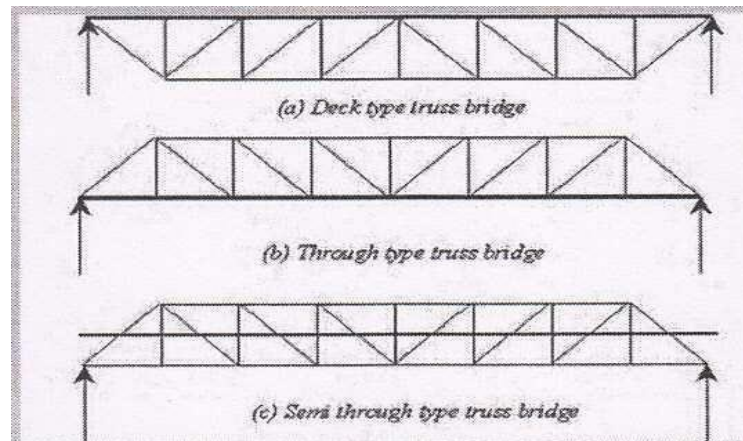
TYPES OF CONCRETE BRIDGES

Arch Bridges

Arch bridges derive their strength from the fact that vertical loads on the arch generate compressive forces in the arch ring, which is constructed of materials well able to withstand these forces. The compressive forces in the arch ring result in inclined thrusts at the abutments, and it is essential that arch abutments are well founded or buttressed to resist the vertical and horizontal components of these thrusts. If the supports spread apart the arch falls down. Traditionally, arch bridges were constructed of stone, brick or mass concrete since these materials are very strong in compression and the arch could be configured so that tensile stresses did not develop. Modern concrete arch bridges utilize prestressing or reinforcing to resist the tensile stresses which can develop in slender arch rings.

Reinforced Slab Bridges

For short spans, a solid reinforced concrete slab, generally cast in-situ rather than precast, is the simplest design. It is also cost-effective, since the flat, level soffit means that false work and formwork are also simple. Reinforcement, too, is uncomplicated. With larger spans, the reinforced slab has to be thicker to carry the extra stresses under load. This extra weight of the slab itself then becomes a problem, which can be solved in one of two ways. The first is to use prestressing techniques and the second is to reduce the deadweight of the slab by including 'voids', often expanded polystyrene cylinders. Up to about 25m span, such voided slabs are more economical than prestressed slabs.

Beam and Slab Bridge

Beam and slab bridges are probably the most common form of concrete bridge in the UK today, thanks to the success of standard precast prestressed concrete beams developed originally by the Prestressed Concrete Development Group (Cement & Concrete Association) supplemented later by alternative designs by others, culminating in the Y-beam introduced by the Prestressed Concrete Association in the late 1980s.

They have the virtue of simplicity, economy, wide availability of the standard sections, and speed of erection.The precast beams are placed on the supporting piers or abutments, usually on rubber bearings which are maintenance free. An in-situ reinforced concrete deck slab is then cast on permanent shuttering which spans between the beams.

The precast beams can be joined together at the supports to form continuous beams which are structurally more efficient. However, this is not normally done because the costs involved are not justified by the increased efficiency.

Simply supported concrete beams and slab bridges are now giving way to integral bridges which offer the advantages of less cost and lower maintenance due to the elimination of expansion joints and bearings.

Techniques of construction vary according to the actual design and situation of the bridge, there being three main types:

1. Incrementally launched
2. Span-by-span
3. Balanced cantilever

Incrementally launched

As the name suggests, the incrementally launched technique creates the bridge section by section, pushing the structure outwards from the abutment towards the pier. The practical limit on span for the technique is around 75m.

Span-by-span

The span-by-span method is used for multi-span viaducts, where the individual span can be up to 60m.

These bridges are usually constructed in-situ with the false work moved forward span by span, but can be built of precast sections, put together as single spans and dropped into place, span by span.

Balanced cantilever

In the early 1950's, the German engineer Ulrich Finsterwalder developed a way of erecting prestressed concrete cantilevers segment by segment with each additional unit being prestressed to those already in position. This avoids the need for false work and the system has since been developed.

Whether created in-situ or using precast segments, the balanced cantilever is one of the most dramatic ways of building a bridge. Work starts with the construction of the abutments and piers. Then, from each pier, the bridge is constructed in both directions simultaneously. In this way, each pier remains stable - hence 'balanced' - until finally the individual structural elements meet and is connected together. In every case, the segments are progressively tied back to the piers by means of prestressing tendons or bars threaded through each unit.

Integral Bridges

One of the difficulties in designing any structure is deciding where to put the joints. These are necessary to allow movement as the structure expands under the heat of the summer sun and contracts during the cold of winter. Expansion joints in bridges are notoriously prone to leakage. Water laden with road salts can then reach the tops of the piers and the abutments, and this can result in corrosion of all reinforcement. The expansive effects of rust can split concrete apart. In addition, expansion joints and bearings are an additional cost so more and more bridges are being built without either. Such structures, called 'integral bridges', can be constructed with all types of concrete deck. They are constructed with their decks connected directly to the supporting piers and abutments and with no provision in the form of bearings or expansion joints for thermal movement. Thermal movement of the deck is accommodated by flexure of the supporting piers and horizontal movements of the abutments, with elastic compression of the surrounding soil.

Already used for lengths up to 60m, the integral bridge is becoming increasingly popular as engineers and designers find other ways of dealing with thermal movement.

Cable-Stayed Bridges

For really large spans, one solution is the cable-stayed bridge. These types of bridges first developed in west Germany. They consists of cables provided above the deck and are connected to the towers. The deck is either supported by a number cables meeting in a bunch at the tower or by joining at different levels on the tower. The multiple cables would facilitate smaller distance between points of supports for the deck girders. This results in reduction of structure depth. The cables can arrange in one plane or two planes. The two plane system requires additional widths to accommodate the towers and deck anchorages. Singly plane system requires less width of deck.   Where all elements are concrete, the design consists of supporting towers carrying cables which   support the bridge from both sides of the tower. Most cable-stayed bridges are built using a form of cantilever construction which can be either in-situ or precast.

The cable stayed bridges are similar to suspension bridges except that there are no suspenders in the cable stayed bridges and the cables are directly stretched from the towers to connect with decking. No special anchorage is required for the cables as incase of suspension bridges because the anchorage at one end is done in the girder and at the other on top of tower. The cable-stayed bridges have been found economical for up to span 300m. However due to cantilever effect their deflection is rather high and hence they are not preferred for very long span in railways.

Suspension Bridges

Concrete plays an important part in the construction of a suspension bridge suspension bridgs are ideal solution for bridging gaps in hilly areas because of their construction technology and capacity of spanning large gaps. There will be massive foundations, usually embedded in the ground, that support the weight and cable anchorages .The cable takes shape of catenary between two points of suspension. The flooring of bridge supported by the cable by virtue of tension developed in its cross section. The vertical members are known as **suspenders** are provided to transfer load from bridge floor to suspension cable. There will also be the abutments, again probably

in mass concrete, providing the vital strength and ability to resist the enormous forces, and in addition, the slender superstructures carrying the upper ends of the supporting cables are also generally made from reinforced concrete.

Typical deck, through and semi-through type truss bridges

(ii) Through Type Bridge - The carriageway rests at the bottom level of the main load carrying members. In the through type plate girder bridge, the roadway or railway is placed at the level of bottom flanges. In the through type truss girder bridge, the roadway or railway is placed at the bottom chord level. The bracing of the top flange or lateral support of the top chord under compression is also required.

(iii) Semi through Type Bridge - The deck lies in between the top and the bottom of the main load carrying members. The bracing of the top flange or top chord under compression is not done and part of the load carrying system project above the floor level The lateral restraint in the system is obtained usually by the U-frame action of the verticals and cross beam actingtogether.

Concrete bridges-

They can be divided into the following main classes
(1)Unstiffened suspension Bridges.
(2) Stiffened suspension Bridges.

Un-stiffened suspension Bridges:-Incase of Un-stiffened suspension Bridges the moving load is transferred direct to the cables by each suspender. These are used for light construction such as foot bridges forest train structures, etc where the moving load is negligible and deflection requirements are not controlling. Also the places where span is very long and the ratio dead to moving load intensity is so great to render stiffening unnecessary.
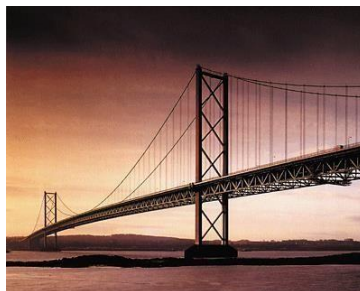
Stiffened suspension Bridges:-In stiffened type suspension Bridges moving loads are transformed to the cables through medium of trusses called **stiffening girders** .The stiffening girder assists the cable to become more rigid and prevent change in shape and gradient of roadway platform. It is therefore adopted for heavy traffic .

IRC Bridge loading:-

The public roads in India are managed and controlled by the Government and hence bridges to be constructed for roads to be designed as per standards set up by standard authorities. For

highway bridges standard specifications are contained in the Indian Road Congress (I.R.C) Bridge code. In India, highway bridges are designed in accordance with IRC bridge code. IRC: 6 - 1966 – Section II gives the specifications for the various loads and stresses to be considered in bridge design. There are three types of standard loadings for which the bridges are designed namely,

   (a) IRC class AA loading,

   (b) IRC class A loading

   (c) IRC class B loading.

IRC class AA loading:-

    IRC class AA loading consists of either a tracked vehicle of 70 tonnes or a wheeled vehicle of 40 tonnes with dimensions as shown in Fig. The units in the figure are mm for length and tonnes for load. Normally, bridges on national highways and state highways are designed for these loadings. Bridges designed for class AA should be checked for IRC class A loading also, since under certain conditions, larger stresses may be obtained under class A loading. Sometimes class 70 R loading given in the Appendix - I of IRC: 6 - 1966 - Section II can be used for IRC class AA loading. Class 70 Rloading is not discussed further here.

I.R.C. Class *AA* loading—wheeled vehicle
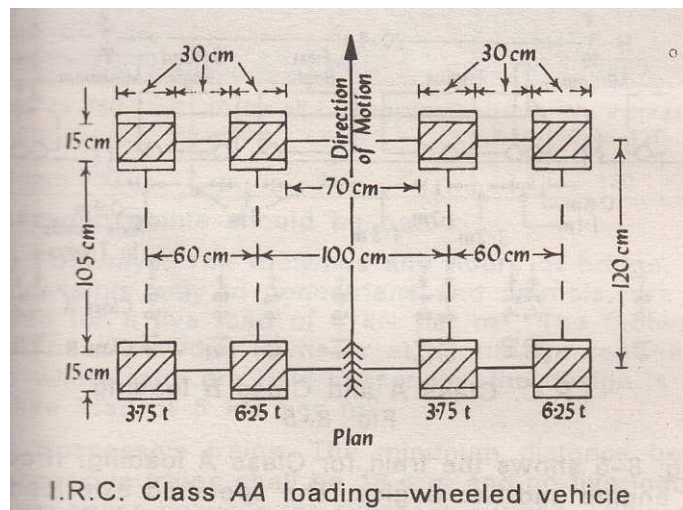
 IRC class A loading:-Class A loading is based on heaviest type commercial vehicle consists of a wheel load train composed of a driving vehicle and two trailers of specified axle spacings. This loading is normally adopted on all roads on which permanent bridges are constructed.

I.R.C. Class A and Class B loading

. **IRC class B loading**:-Class B loading is adopted for temporary structures and for bridges in specified areas. For class A and class B loadings, reader is

referred to IRC: 6 - 1966 – Section II.

CHAPTER-12

CULVERTS AND CAUSE WAYS

Culvert- A culvert is defined as a small bridge constructed over a stream which remains dry most part of the year. It is across drainage work having total length not exceeding 6m between faces of abutment.

Types of Culverts;-

The following are six different type culvert.

      1. Arch culvert

      2.Box culvert

      3.Pipe Culvert

      4.Slab Culvert

Arch culvert:-

An arch culvert consists of abutments wing walls, arch, parapets and the foundation. The construction materials commonly used are brick work or concrete. Floor and curtain wall may or may not be provided depending upon the nature of foundation soil and velocity of flow. A typical arch culvert is shown in figure.

Arch culvert
FiG. 6-1
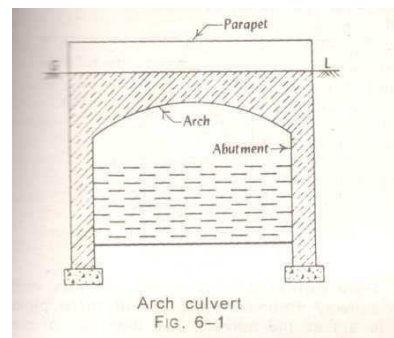
Box culvert:-

In case of box culvert the rectangular boxes are formed of masonry, R.C.C or steel. The R.C.C box culverts are very common and they consist of the following two component

(i) The barrel or box section of sufficient length to accommodate the roadway and the Krebs.

(ii) The wing walls splayed at 45 for retaining the embankment and also guiding the flow of water into and out of the barrel.

An R.C.C. box culvert
FiG. 6-2
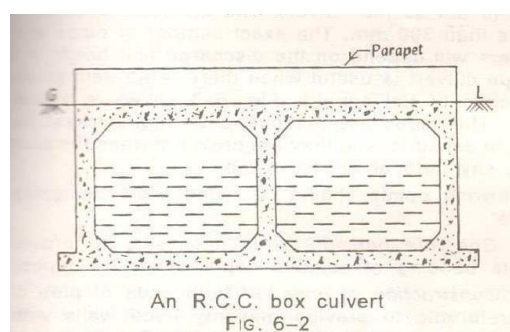
Fig. 6-2 shows an R.C.C box culvert with two openings. Following points should be noted.

(i)Foundation: The box culverts prove to be safe where good foundations are easily available.

(ii)Height: The clear vent height i.e. the vertical distance between top and bottom of the culvert rarely exceeds 3 meters.

(iii) Sap: The box culverts are provided singly or in multiple units with individual span exceed about 6 m or so, it requires thick section which will make the construction uneconomical.

(iv)Top: Depending upon the site conditions, the top level of box may be at the road level or it can even be at a depth below road level with filling of suitable material.

Pipe Culvert:

They are provided when discharge of stream is small or when sufficient headway is not available. Usually one or more pipes of diameter not less than 60cm are placed side by side. Their exact number and diameter depend upon the discharge and height of bank. For easy approach of water splayed type wing walls are provided in fig. 6.3 shows a Hume pipes culvert of single pipe. The pipes can be built of masonry. Stone ware, cement concrete, cast iron or steel. Concrete bedding should also be given below the pipes and earth cushion of sufficient thickness on the top to protect the pipes and their joints. For Economic reason road culverts should have non-pressure heavy duty pipes of type ISI class NP3 conforming to IS:458-1961. As far possible the gradient of the pipe should not be less than 1000.

Pipe culvert
FIG. 6—3
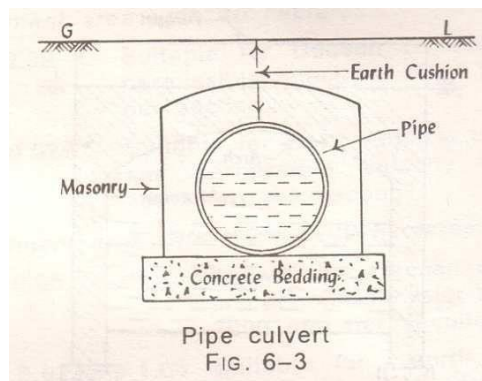
Slab Culvert:

A slab culvert consists of stone slabs or R.C.C slab, suitably support on masonry walls on either side. As shown in fig 6-4. The slab culverts of simply type are suitable up to a maximum span of 2.50 m or so. However the R.C.C culverts of deck slab type can economically be adopted up to spans of about 8 m. However, the thickness of slab and dead weight may sometimes prove to be the limiting factors for deciding the economical span of this type of culverts.
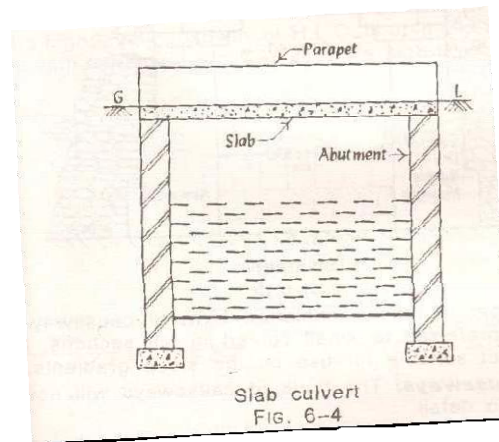
Slab culvert
FIG. 6-4

The construction of slab culverts is relatively simple as the frame work can easily be arranged, reinforcement can be suitably placed and concreting can be done easily. This type of culvert can be used for highway as well as Railway Bridge. Depending upon the span of culvert and site conditions the abutment and wing walls of suitable dimensions may be provided. The parapet or hand rail of at least 750 mm height should be provided on the slab to define the width of culvert.

CAUSEWAYS :

A road causeway is a pucca dip which allows floods to pass over it. It may or may not have opening or vents for low water to flow. If it has vents for low water to flow then it is known as high level causeway or submersible bridge ; otherwise a low level causeway.

TYPES OF CAUSEWAY:

A) **Low level causeway**:

It is also known as Irish Bridge. The beds of small rivers or streams, which remain dry for most part of the year, are generally passable without a bridge. This involves heavy earth works in cutting for bridge approaches .Banks of such types of streams are cut down at an easy slope. For streams of rivers in plains having sandy beds, it is often sufficient to lay bundles of grass over and across the sandy track. The bundles may be of 20 to 25cm in diameter whose ends are secured by longitudinal fascines pegged down by stakes.

Fig. 5.3.1. Low Levels Causeway.

Fig. 5.3.2. Paved Low Level Causeway.
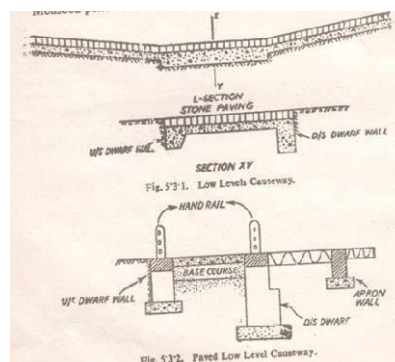
For crossings important from traffic point of view it is essential to lay a metal or pucca paving of stone or brick set in lime mortar on a substantial bed of concrete. To prevent against possible scour and undermining a cut off or dwarf wall usually 60cm deep on the upstream side and 120 to 150cm on downstream side is provided. Fig. 5.3.1 below shows the details of a typical Irish bridge.

The low level causeway could be provided with openings formed by concrete Hume pipes if there is a continuous flow stream during the monsoon periods.

B) **High level Causeway**:

A high level causeway is submersible road bridge designed to be overtopped in floods. Its formation level is fixed in such a way as not to cause interruption to traffic during floods for more than three days at a time not for more than six times in a year. A sufficient numbers of openings are provided to allow the normal flood discharge to pass through them with the required clearance. They are provided with abutments and piers, floors and slabs or arches to form the required number of openings. The slope of the approaches is kept as 1 in 20.When the velocity is high and stream bed is soft the aprons could be of concrete or harder masonry upto a certain distance. Similarly, the road can be formed of a cement concrete slab or stone blocks set in cement mortar. A typical type of high level causeway is shown in Fig.5.3.3.If railing are provided in the bridge, they should be of collapsible type.Temporary causeways   used for an emergency military operations are   formed either by using timber stringers and planking over cribs used as piers or by constructing a culvert using pipes.

Fig. 5·3·3. High Level Causeway.

Reference book:
Railway Engineering
   By: Chandra & Agrawal
A text book of railway engineering
   By:S.C .Sexena & S.P .Arora
Railway Engineering
   By:S.C.Rangawala

GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING

GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

LAND SURVEY -2

For 6th Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)

Prepared By
Swagatika tripathy

(Lecturer in Civil Engineering)

CHAPTER- 1
TACHEOMETRY SURVEY

General

Tachometry is the branch of angular surveying in which the horizontal and vertical distances of points are obtained by optical means as opposed to the ordinary slower process of measurements by tape or chain. The method is very rapid and convenient. Although the accuracy of Tachometry in general compares un-favourably with that of chaining, it is best adapted in obstacles such as steep and broken ground, deep ravines, stretches of water or swamp and so on, which make chaining difficult or impossible.

The primary object of tachometry is the preparation of contoured maps or plans requiring both horizontal as well as vertical control. Also, on surveys of higher accuracy, it provides a check on distances measured with the tape.

Tacheometer:

1. A tacheometer is nothing more than a theodolite fitted with stadia hair.
2. The stadia hairs are kept in the same vertical plane as the horizontal and vertical cross hair.
3. For short distance up to 100 m, ordinary leveling stadia may used.
4. According to measurement process system, it is classified under two categories

    i.e. 1. Stadia hair system

    2. Tangential system

5. The stadia hair system again divided into two categories
    i.e. 1. Fixed hair method
    2. Movable hair method

Fixed hair method:

In this method, the distance between the upper hair and lower hair, i.e. stadia interval i, on the diaphragm of the lens system is fixed. The staff intercept s, therefore, changes according to the distance D and vertical angle θ.

Movable hair method:

In this method, the stadia interval 'i' can be changed. The stadia hairs can be moved vertically up and down by using micrometer screws. The staff intercept s, in this case, is kept fixed. Two vanes (targets) are fixed on the staff at a fixed interval of 2 m or 3 m.

The fixed hair method is the one which is commonly used and, unless otherwise mentioned, stadia method means fixed hair method. Movable hair method is not in common use due to

difficulties in determining the value of i accurately.

Principle of Stadia Method

The stadia method is based on the principle that the ratio of the perpendicular to the base is constant in similar isosceles triangles. In figure (a), let two rays OA and OB be equally inclined to the central ray OC. Let A_2B_2, A_1B_1 and AB be the staff intercepts.

Evidently,

$$\frac{OC_2}{A_2B_2} = \frac{OC_1}{A_1B_1} = \frac{OC}{AB} = \text{constant } k = \frac{1}{2} \cot \frac{\beta}{2}$$

(a)
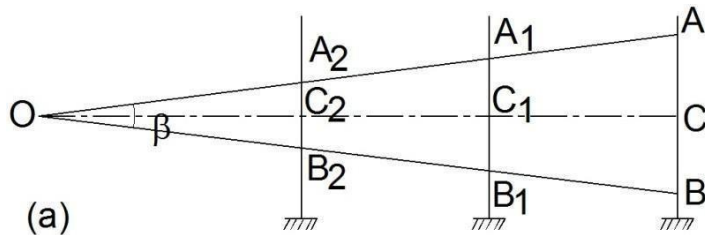
We will derive distance and elevation formulae for fixed hair method assuming line of sight as horizontal and considering an external focusing type telescope. In Figure below, O is the optical centre of the object glass. The three stadia hairs are a, b and c and the corresponding readings on staff are A, B and C. Length of image of AB is ab. The other terms used in this figure are

f = focal length of the object glass,

i = stadia hair interval = ab,

s = staff intercept = AB,

c = distance from O to the vertical axis of the instrument,

d = distance from O to the staff,

d' = distance from O to the plane of the diaphragm, and

D = horizontal distance from the vertical axis to the staff.

4
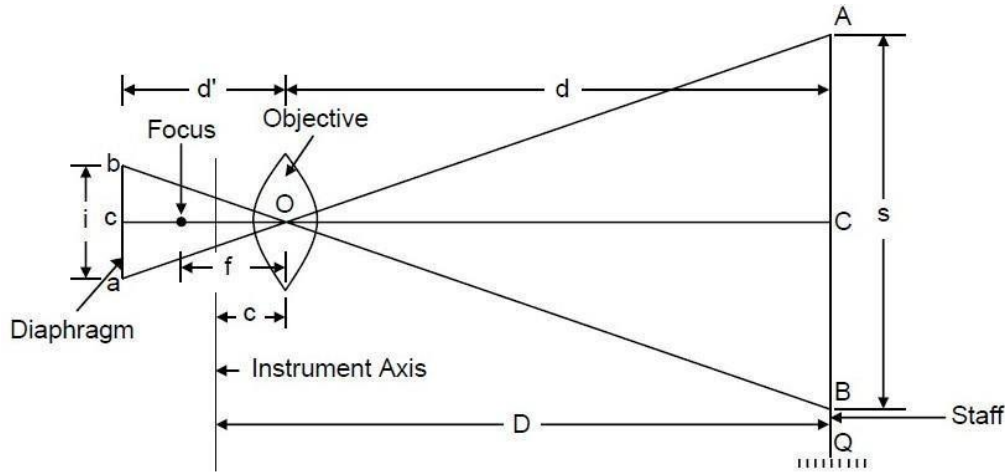
Principle of Stadia Method

From similar Δ, *AOB* and *aOb*, we get

$$\frac{d}{d'} = \frac{s}{i}$$

And from lens formula,

$$\frac{1}{f} = \frac{1}{d'} + \frac{1}{d}$$

Combining the two equations, we get

$$d = \frac{fs}{i} + f$$

Adding *c* to both the sides

$$D = \frac{fs}{i} + (f + c)$$

Or $D = Ks + C$

where the constant *K* is equal to (*f* /*i*). It is called **multiplying constant** of the tacheometer and is generally kept as 100. The constant *C* is equal to (*f* + *c*). It is called **additive constant** whose value ranges from 30 cm to 50 cm for external focusing telescopes and 10 cm to 20 cm for internal focusing telescopes. For telescopes fitted with anallactic lens, *C* equals zero.

Anallactic Lens

The basic formula for determination of horizontal distance in stadia tacheometry is

$$D = \frac{fs}{i} + (f + c)$$

Or $D = Ks + C$

Due to the presence of the additive constant *C*, *D* is not directly proportional to *s*. This is accomplished by the introduction of an additional convex lens in the telescope, called an *anallactic lens*, placed between the eyepiece and object glass, and at a fixed distance from the latter.

The anallactic lens is provided in external focusing telescope. Its use simplifies the reduction of observations since the additive constant $(f + c)$ is made zero and the multiplying constant k is made 100. However, there is objection to its use also as it increases the absorption of light in the telescope thereby causing reduction in brilliancy of the image. Anallactic lens is not fitted in internal focusing telescopes.

Determination of Tacheometric Constants

The stadia interval factor (K) and the stadia constant (C) are known as tacheometric constants. Before using a tacheometer for surveying work, it is required to determine these constants. These can be computed from field observation by adopting following procedure.

Step 1 : Set up the tacheometer at any station say P on a flat ground.

Step 2 : Select another point say Q about 200 m away. Measure the distance between P and Q accurately with a precise tape. Then, drive pegs at a uniform interval, say 50 m, along PQ. Mark the peg points as 1, 2, 3 and last peg -4 at station Q.

Step 3 : Keep the staff on the peg-1, and obtain the staff intercept say s_1 .

Step 4 : Likewise, obtain the staff intercepts say s_2, when the staff is kept at the peg-2,

Step 5 : Form the simultaneous equations,

$D_1 = K. s_1 + C$ ------------- (i)

and $D_2 = K. s_2 + C$ ------------ (ii)

Solving Equations (i) and (ii), determine the values of K and C say K_1 and C_1 .

Step 6 : Form another set of observations to the pegs 3 & 4, Simultaneous equations can be obtained from the staff intercepts s_3 and s_4 at the peg-3 and point Q respectively. Solving those equations, determine the values of K and C again say K_2 and C_2.

Step 7 : The average of the values obtained in steps (5) and (6), provide the tacheometric constants K and C of the instrument.

Stadia tacheometry

Case 1 When staff held vertical and with line of collimation horizontal
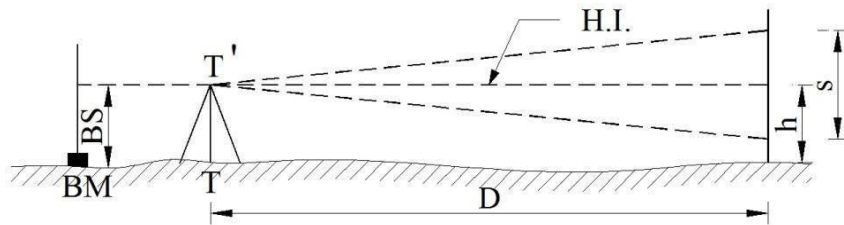
When the line of sight is horizontal , the general tacheometric equation for distance is given by

$$D = \frac{fs}{i} + (f + c)$$

The multiplying constant $\left(\frac{f}{i}\right)$ is 100, and additive constant $(f + c)$ is generally zero.

RL of staff station P = HI – h

Where HI = RL of BM + BS

h = central hair reading

BS = Back sight

HI = height of instrument

Case 2 When staff held vertical and with line of collimation inclined

    (a) *Considering Angle of elevation*

Let
T = Instrument station
T₁ = axis of instrument
P = staff station
A, B, C = position of staff cut by hairs
S = AC = staff intercept
h = central hair reading
V = vertical distance instrument axis and

7
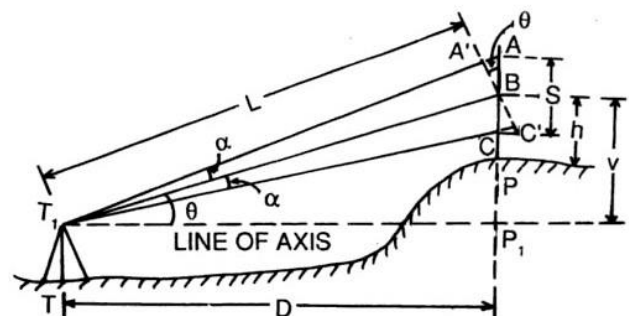
central hair

D = horizontal distance between instrument and staff

L = inclined distance between instrument axis and B

θ = angle of elevation

α = angle made by outer and inner rays with central ray

$A'C'$ is drawn perpendicular to the central ray T_1B

Now , internal distance, $L = \frac{f}{i}(A'C') + (f + c)$

Horizontal distance , $D = L\cos\theta$

$$= \frac{f}{i}(A'C')\cos\theta + (f + c)\cos\theta \qquad (1)$$

Now $A'C'$ is to be expressed in terms of AC (i. e. S)

In Δs ABA' and CBC'

$< ABA' = < CBC' = \theta$

$< AA'B = 90^0 + \alpha$

$< BC'C = 90^0 - \alpha$

The angle α is very small

$< AA'B$ and $< BC'C$ may be taken equal to 90^0

So $A'C' = AC\cos\theta = S\cos\theta$

From equation (1)

$$D = \frac{f}{i}(S\cos\theta)\cos\theta + (f + c)\cos\theta$$

$$D = \frac{f}{i} \times S\cos^2\theta + (f + c)\cos\theta$$

Again $V = L\sin\theta$

$$= \{\frac{f}{i} \times S\cos\theta + (f + c)\}\sin\theta$$

$$= \frac{f}{i} \times S\cos\theta\sin\theta + (f + c)\sin\theta$$

$$V = \frac{f}{i} \times \frac{S \times \sin 2\theta}{2} + (f + c)\sin\theta$$

Also $\qquad V = D\tan\theta$

RL of staff station P = RL of axis of instrument $+ V - h$

(b)   Considering Angle of depression

In this case also the expressions for D and V are same. That is
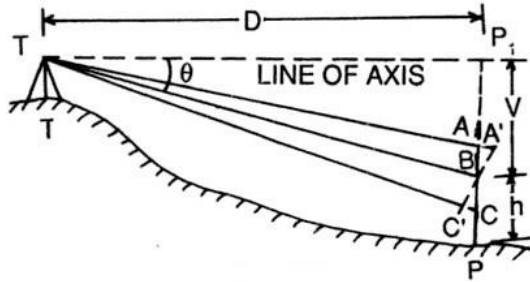
$$D = \frac{f}{i} \times S \cos^2\theta + (f + c) \cos\theta$$

$$V = \frac{f}{i} \times \frac{S \times \sin 2\theta}{2} + (f + c) \sin\theta$$

RL of staff station P = RL of axis of instrument − V − h

Problem

A tacheometer was set up at a station C and the following readings were obtained on a staff vertically held.

Inst. station	Staff station	Vertical angle	Hair readings			Remarks
C	BM	−5° 20′	1.500,	1.800,	2.450	RL of BM =
C	D	+8° 12′	0.750,	1.500,	2.250	750.50 m

Calculate the horizontal distance CD and RL of D, when the constants of instrument are 100 and 0.15 .
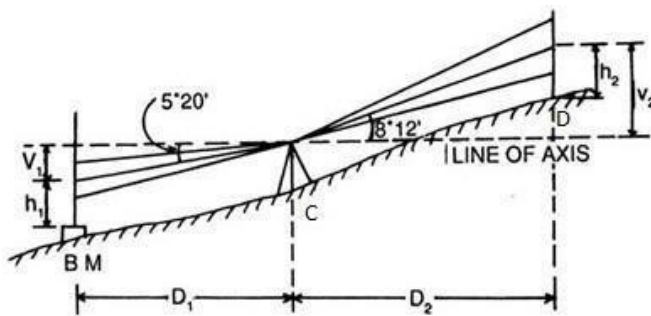
Solution

When the staff is held vertically, the horizontal and vertical distances are given by the relations

$$D = \frac{f}{i} \times S \cos^2\theta + (f + c) \cos\theta$$

$$V = \frac{f}{i} \times \frac{S \times \sin 2\theta}{2} + (f + c) \sin \theta$$

Here $\frac{f}{i} = 100$ and $(f + c) = 0.15$

In the first observation, $S_1 = 2.450 - 1.150 = 1.300$ m

$\theta_1 = 5^0\ 20'$ (depression)

$$V_1 = 100 \times 1.300 \times \frac{\sin 10^0\ 40'}{2} + 0.15 \times \sin 5^0\ 20' = 12.045 \text{ m}$$

In the second observation, $S_2 = 2.250 - 0.750 = 1.500$ m

$\theta_2 = 8^0\ 12'$ (elevation)

$$V_2 = 100 \times 1.500 \times \frac{\sin 16^0\ 24'}{2} + 0.15 \times \sin 8^0\ 12' = 21.197 \text{ m}$$

$D_2 = 100 \times 1.50 \times \cos^2 8^0\ 12' + 0.15 \times \cos 8^0\ 12' = 147.097$ m

RL of axis of instrument $= $ RL of BM $+ h_1 + V_1$

$= 750.500 + 1.800 + 12.045 = 764.345$ m

RL of D $=$ RL of axis of instrument $+ V_2 - h_2$

$= 764.345 + 21197 - 1.500 = 784.042$ m

So, the distance CD $= 147.097$ m and RL of D $= 784.042$ m

CHAPTER-2
CURVES

Introduction:

Curves are required to be introduced where it is necessary to change the direction of motion from one straight section of a highway or a railway to another. These are provided due to the nature of terrain or other avoidable reasons to enable smooth passage of vehicles.

CLASSIFICATION OF CURVES

For survey purposes, curves are classified as horizontal or vertical, depending on whether they are introduced in the horizontal or vertical plane.
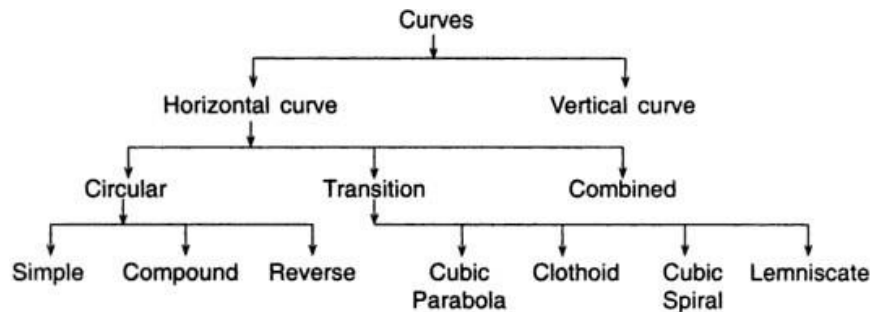
Horizontal Curves

Horizontal curves can be circular or non-circular (transitional) curves. Different types of horizontal curve are shown in figure below.

Simple Circular Curve

When a curve consists of a single arc with a constant radius connecting the two straights or tangents, it is said to be a circular curve.

Compound Curve

When a curve consists of two or more arcs with different radii, it is called a compound curve. Such a curve lies on the same side of a common tangent and the centres of the different arcs lie on the same side of their respective tangents.

Reverse Curve

A reverse curve consists of two arc bending in opposite directions. Their centres lie on opposite sides of the curve. Their radii may be either equal or different, and they have one common tangent.

Transition Curve

A curve of variable radius is known as a transition curve. It is also called a easement curve. Such a curve is provided between a straight and a circular curve, or between branches of a compound or reverse curve to avoid an abrupt change in direction when the alignment

changes. In railways, such curve is used on both sides of a circular curve to minimize superelevation.

(a) Circular Curves

(b) Compound Curve

(c) Reverse Curve

(d) Transition Curve

Type of Horizontal Curves
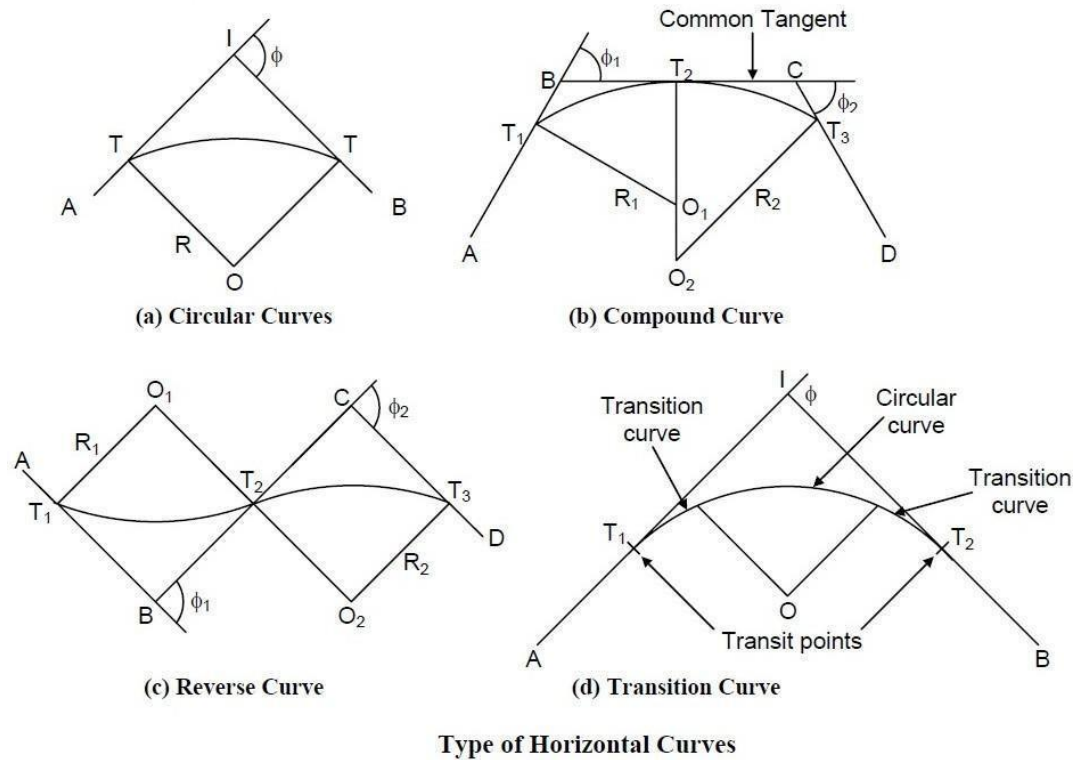
SIMPLE CIRCULAR CURVE

Figure shows a simple circular curve with two straight lines *AI* and *IB* intersect at the point *I*. The curve $T_1 C T_2$ of radius *R* is inserted to make a smooth change of direction from *AI* to *IB*. A simple circular curve has various components whose definitions are given below.
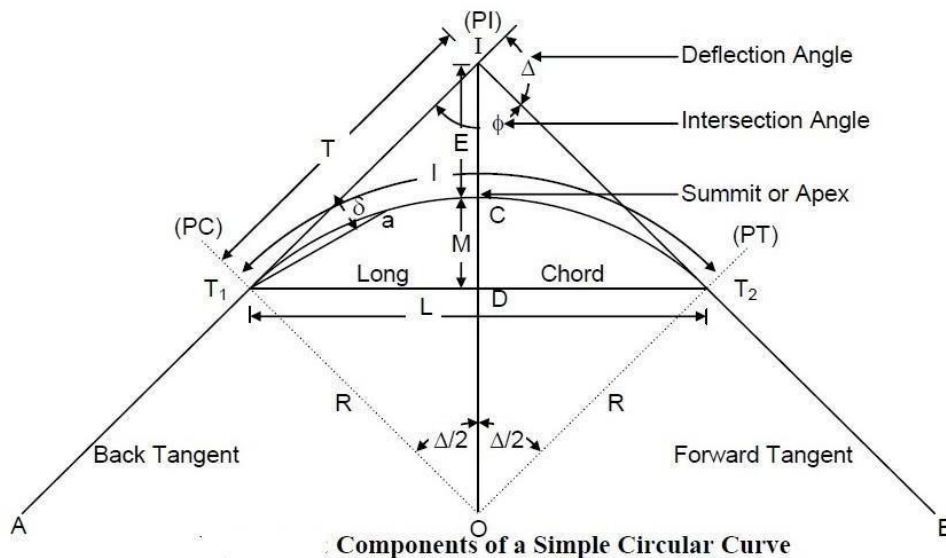
Components of a Simple Circular Curve

Definition of Various Components

Back Tangent

The tangent (AT_1) previous to the curve is called the back tangent or first tangent.

Forward Tangent

The tangent (T_2B) following the curve is called the forward tangent or second tangent.

Point of Intersection

If the two tangents AT_1 and BT_2 are produced, they will meet in a point I called the point of intersection (PI) or vertex.

Point of Curve (PC)

It is the beginning of the curve (T_1) where the alignment changes from a tangent to a curve.

Point of Tangency (PT)

It is the end of the curve (T_2) where the alignment changes from a curve to tangent.

Intersection Angle

The angle between the tangent AT_1 and BT_2 is called the intersection angle (ϕ).

Deflection Angle

The angle Δ through which the forward tangent deflects is called the deflection angle of the curve. It may be either to the left or the right.

Deflection Angle to any Point

The deflection angle δ to any point a on the curve is the angle at PC between the back tangent and the chord $T_1 a$ from PC to point on the curve.

Tangent Distance (T)

It is the distance between PC to PI (also the distance from PI to PT).

External Distance (E)

It is distance from the mid-point of the curve to PI. It is also known as the apex distance.

Length of the Curve (l)

L is the total length of the curve from PC to PT.

Long Chord (L)

It is the chord joining PC to PT.

Mid Ordinate (M)

It is the ordinate from the mid-point of the long chord to the mid-point of the curve. It is also called the versine of the curve.

Normal Chord (C)

A chord between two successive regular stations on a curve is called a normal chord.

Sub-Chord (c)

Sub-chord is any chord shorter than the normal chord. These generally occur at the beginning or at the end of the curve.

Right-hand Curve

If the curve deflects to the right of the direction of the progress of survey, it is called the right-hand curve.

Left-hand Curve

If the curve deflects to the left of the direction of the progress of survey, it is called the left-hand curve.

Elements of Simple Circular Curve

Length of the Curve (*l*)

Length $l = T_1 CT_2 = R \Delta$, where Δ is in radians

$= (\pi R) \Delta/180^{\text{o}}$, where Δ is in degrees.

Tangent Length (*T*)

Tangent length, $T = T_1 I = IT_2$

$= OT_1 \tan \Delta/2 = R \tan \Delta/2$

Length of the Long Chord (*L*)

$L = T_1 T_2 = 2 \, OT_1 \sin \Delta/2$

$= 2 R \sin \Delta/2$

Apex Distance or External Distance (*E*)

$E = CI = IO - CO$

$= R \sec \Delta/2 - R$

$= R (\sec \Delta/2 - 1)$

$= R \operatorname{exsec} \Delta/2$

Mid-ordinate (*M*)

$M = CD = CO - DO$

$= R - R \cos \Delta/2$

$= R (1 - \cos \Delta/2) = R \operatorname{versin} \Delta/2$

Problem :

Two tangents intersect at a chainage of 1250.50 m having deflection angle of 60°. If the radius of the curve to be laid out is 375 m, calculate the Length of the curve, Tangent distance, Length of the long chord, Apex distance, Mid-ordinate, Degree of curve and Chainage of P.C. and P.T.

Solution :

Length of the curve, $l = (\pi R) \Delta/180^{o}$, where Δ is in degrees.

$$= \pi \times 375 \times 60^{o}/180^{o}$$

$$= 392.69 \text{ m}$$

Tangent Length, $T = R \tan \Delta/2$

$$= 375 \times \tan 60^{o}/2$$

$$= 216.50 \text{ m}$$

Length of the long chord, $L = 2 R \sin \Delta/2$

$$= 2 \times 375 \times \sin 60^{o}/2$$

$$= 375.00 \text{ m}$$

Apex distance, $E = R (\sec \Delta/2 - 1)$

$$= 375 \times (\sec 60^{o}/2 - 1)$$

$$= 58.01 \text{ m}$$

Mid-ordinate, $M = R (1 - \cos \Delta/2)$

$$= 375 \times (1 - \cos 60^{o}/2)$$

$$= 50.24 \text{ m}$$

Degree of Arc, $D_a^{o} = 1718.9/R$

$$= 1718.9/375$$

$$= 4^{o}.58$$

Chainage of PC = Chainage of $I - T$

$$= 1250.50 - 216.50$$

$$= 1034.00 \text{ m}$$

Chainage of PT = Chainage of $I + l$

$$= 1250.50 + 392.69$$

$$= 1634.19 \text{ m}$$

Designation of Curve

The *sharpness* of the curve is designated either by its *radius* or by its *degree of curvature*. The degree of curvature has several slightly different definitions. According to the *arc definition* generally used in highway practice, the degree of the curve (D^o_a) is defined as the central angle of the curve that is subtended by an arc AB of 30 m length .

If the degree of curve (D^o_a) is taken in degrees, for a curve of radius R meter, then

$$D^o_a: 30 = 360^o : 2\pi R$$

or $$D^o_a = 10800/2\pi R$$

$$= 1718.9/R \text{ (approximate)}$$
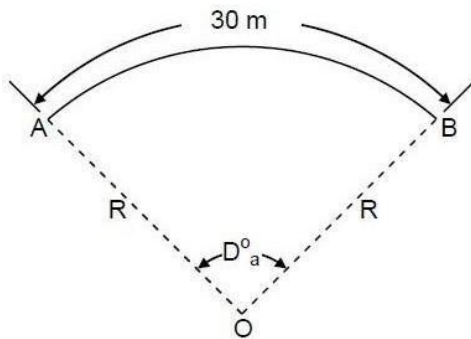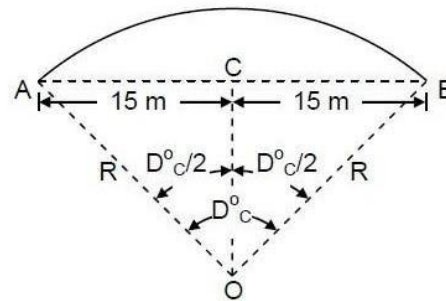
(a) Arc Definition   (b) Chord Definition

Degree of Curve

According to the *chord definition* generally used in railway practice, the degree of the curve (D^o_c) is defined as the central angle of the curve that is subtended by its chord AB of 30 m length.

$$\sin(D^o_c/2) = AC/AO$$

$$= 15/R$$

$$R = 15/\sin (D^o_c /2)$$

Radius of curvature varies inversely as the degree of curve. A sharp curve has a larger degree of curve whereas a flat curve has a smaller degree of curve.

SETTING OUT SIMPLE CIRCULAR CURVE

A circular curve can be set out in the field by linear method and angular method. These are described below.

    (a) Linear method is also called chain and tape method. In this method, only tape and chains are used and no angular measurement is carried out.

    (b) In angular method or Instrumental method, a theodolite, tacheometer or a total station instrument is used for angular measurement.

Linear Method

Listed below are some of the linear methods of setting out simple circular curve followed by their description :

    (a) Offsets from the long chord

    (b) Successive bisection of chord

    (c) Offsets from the tangents

    (d) Offsets from the chords produced

Offsets from the Long Chord

The method is suitable for setting out circular curves of small radius, such as those at road intersections in a city or in boundary walls. In Figure below, the offset O_{xa} to the point a on the curve is the perpendicular distance of point a from the long chord $T_1 T_2$, at a distance x_a from D along the long chord. Considering the origin at D, O_{xa} is the y-coordinate of point a.
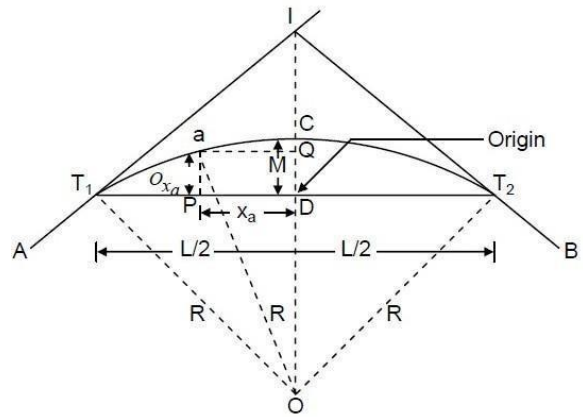
Offsets from the Long Chord

From $\Delta\,OT_1D$,

$$(DO)^2 = (T_1O)^2 - (T_1D)^2$$

Or $$(OC - DC)^2 = (T_1O)^2 - (T_1D)^2$$

Or $$(R - M)^2 = R^2 - \left(\frac{L}{2}\right)^2$$

Or $$M = R - \sqrt{R^2 - \left(\frac{L}{2}\right)^2}$$

Draw a line Qa parallel to DT_1 cutting DC at Q

From $\Delta\,O\,a\,Q$

$$OQ = \sqrt{(Oa)^2 - (Qa)^2} = \sqrt{R^2 - x_a^2}$$

$$OQ = OD + DQ = OD + O_{x_a}$$

$$OQ = OD + O_{x_a} = \sqrt{R^2 - x_a^2}$$

17

$$O_{xa} = \sqrt{R^2 - x_a^2} - OD$$

$$O_{xa} = \sqrt{R^2 - x_a^2} - (R - M)$$

$$O_{x_a} = \sqrt{R^2 - x_a^2} - \sqrt{R^2 - \left(\frac{L}{2}\right)^2}$$

In general $O_x = \sqrt{R^2 - x^2} - \sqrt{R^2 - \left(\frac{L}{2}\right)^2}$

The long chord is divided into equal parts of suitable length. The offset O_{x_a} corresponding to the distances x_a from D are calculated for different points on the long chord. These offsets are measured perpendicular to the long chord with the help of an optical square and points are located. Joining these points will produce the desired curve. The points on the right side of *CD* are set out by symmetry.

Successive Bisection of Chords

The method being approximate is suitable for small curves. It involves the location of points on the curve by bisecting the chords and erecting perpendiculars at the midpoint of the chords.
In Figure , $T_1 T_2$ is the long chord and D is its midpoint. C is the point of intersection of the perpendicular line at D, with the curve. *Dc* is the mid-ordinate, which is equal to

$$M = R\left(1 - \cos\left(\frac{\Delta}{2}\right)\right) = R - \sqrt{R^2 - \left(\frac{L}{2}\right)^2}$$

At *D,* a perpendicular offset equal to *M* is erected and the position *C* is located. Now consider the chords $T_1 C$ and $T_2 C$, locate their midpoints d_1 and d_2 respectively. Erect two perpendiculars at d_1 and d_2 and measure the offsets equal to $d_1 c_1$ and $d_2 c_2$, respectively. The offsets $d_1 c_1$ and $d_2 c_2$ are computed from the following formula :

$$d_1c_1 = d_2c_2 = R\left(1 - \cos\left(\frac{\Delta}{2}\right)\right)$$

Now, by the successive bisection of these chords, more points can be located in a similar manner.
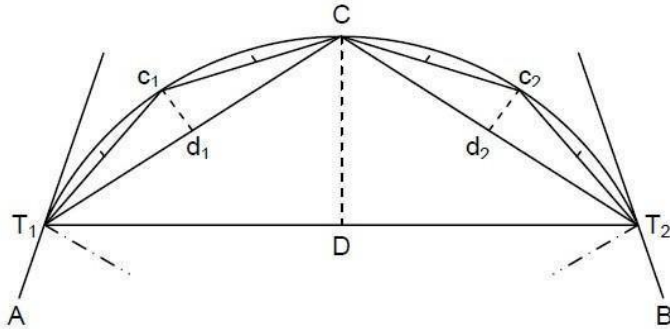
Successive Bisection of Chords

After locating T_1 and T_2, the midpoint D of $T_1 T_2$ is obtained, by measuring $T_1 T_2$. The perpendicular offset DC is set out at D with an optical square and point C is located. Measure $T_1 C$, and $T_2 C$, and locate their midpoints d_1 and d_2. The perpendicular offsets $d_1 c_1$ and $d_2 c_2$ are set out at d_1 and d_2, and the points c_1 and c_2 are established on the curve. The process is continued till sufficient numbers of points on the curve are fixed.

Offsets from the Tangents

This method is used when the deflection angle and the radius of curvature both are comparatively small. In this method, the curve is set out by measuring offsets from the tangent. The offsets from the tangent can be either perpendicular or radial to the tangent.

Perpendicular Offsets Method

Let the point a be on the curve and the perpendicular offset from the tangent T_1 to it at P be O_{x_a}. Let the distance of P from T_1 be x_a. Draw a line Qa perpendicular to $T_1 O$, intersecting OT_1 at Q.

From $\Delta\, QaO$

$$OQ = \sqrt{(Oa)^2 - (Qa)^2}$$

$$R - O_{x_a} = \sqrt{R^2 - x_a^2}$$

$$R - O_{x_a} = R - \sqrt{R^2 - x_a^2}$$

In general $O_x = R - \sqrt{R^2 - x^2}$
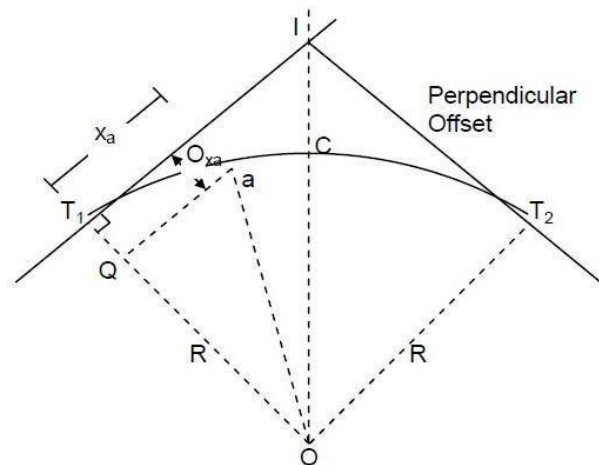
Perpendicular Offsets

Before setting out a curve, a table of offsets for different values of x (e.g., 10 m, 20 m, 30 m, etc.) is made. Then from T_1 the distances x_1, x_2, x_3 etc., are measured along the tangent and the corresponding offsets are measured on the perpendiculars to the tangent with the help of an optical square.

Since the offsets of points equidistant from T_1 and T_2, are equal, the same table is used for offsets from both the tangents.

Radial Offsets Method

Let the radial offset to the point a on the curve be O_{x_a} from the point P at a distance of x_a from T_1.

From $\Delta\ OPT_1$

$$OP = \sqrt{(OT_1)^2 + (T_1P)^2}$$

$$R + O_{x_a} = \sqrt{R^2 + x_a^2}$$

$$O_{x_a} = \sqrt{R^2 + x_a^2} - R$$

In general $O_x = \sqrt{R^2 + x^2} - R$

Radial Offsets
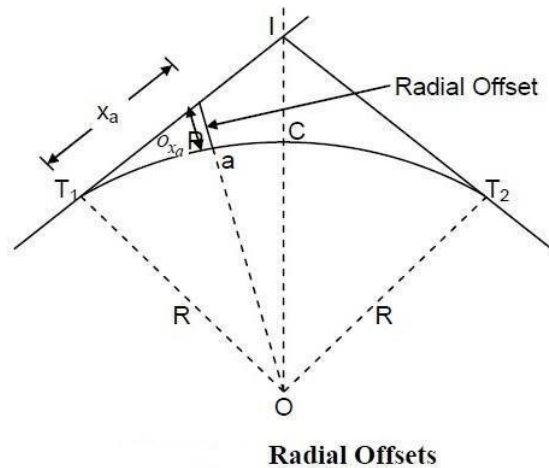
Offsets from the Chord Produced

The method has the advantage that not all the land between the tangents points T_1 and T_2 need be accessible. However to have reasonable accuracy the length of the chord chosen should not exceed R/20. The method has a drawback that error in locating is carried forward to other points. This method is based on the premise that for small chords, the chord length is small and approximately equal to the arc length.

For setting out the curve, it is divided into a number of chords normally 20 to 30 m in length. For the continuous chainage required along the curve, the two sub-chords are taken, one at the beginning and the other at the end of the curve. The first sub-chord length is such that a full number of chainage is obtained on the curve near T_1 and the second sub-chord length near T_2.

From the property of a circle, if the angle $\angle FT_1a = \delta_1$

The angle at the centre $\angle$

$T_1Oa = 2\delta_1$

$C_1 = chord\, T_1a \approx arc\, T_1a$

$\qquad\qquad = 2\delta_1 R$

Or $\qquad \delta_1 = \dfrac{C_1}{2R}$

The first offset $O_1 = C_1\delta_1$

$$O_1 = C_1\dfrac{C_1}{2R} = \dfrac{C_1^2}{2R}$$
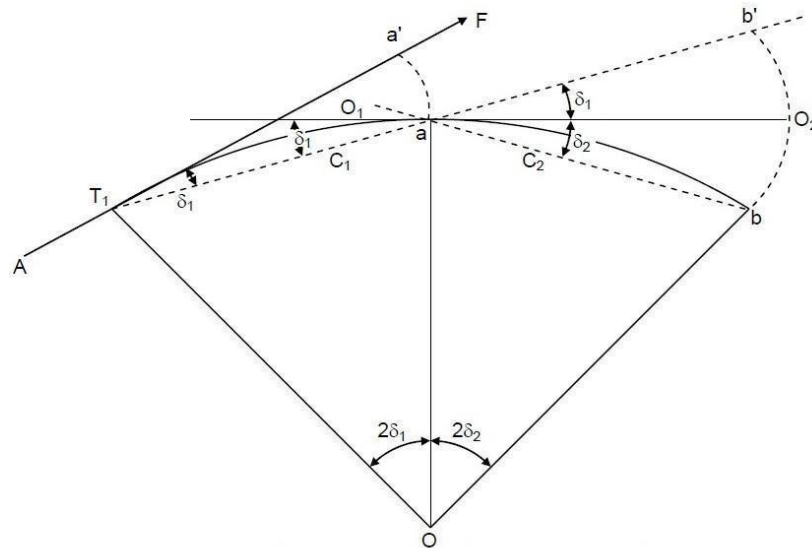
Offsets from the Chord Produced

The first chord C is called the sub-chord. The length of the sub-chord is so adjusted that the chord length when added to the chainage of T makes the chainage of point a as full chain. Subsequent chord lengths C_2, C_3, C_4 .... are full chains. T_1a is then produced to b' such that a full chain $ab' = C_2$, a full chain.

The second offset

$O_2 = C_2(\delta_1 + \delta_2)$

$= C_2\left(\dfrac{C_1}{2R} + \dfrac{C_2}{2R}\right)$

$= \dfrac{C_2}{2R}(C_1 + C_2)$

Similarly $\qquad O_3 = \dfrac{C_3}{2R}(C_2 + C_3)$

The last offset $O_n = \dfrac{C_n}{2R}(C_{n-1} + C_n)$

where C_{n-1} is a full chain and C_n is the last sub-chord which is normally less than one chain length.

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Angular Method

Following are some of the angular method used to set out a simple circular curve :

    (a) Tape and theodolite method

    (b) Two theodolite method

(c) Tachometric method

(d) Total station Method

Tape and Theodolite Method

In this method, a tape is used for making linear measurements and a theodolite is used for making angular measurements. The curve can be set out by the following procedures :

Rankine's Method

The method is known as Rankine's method of tangential angle or the deflection angle method. The method is accurate and is used in railways and highways.

Let T_1ab be a part of a circular curve with T_1, the initial tangent point. Thus, T_1a is the first sub-chord which is normally less than one chain length.

From the property of a circle

$$C_1 = 2\delta_1 R$$

$$\delta_1 = \frac{C_1}{2R}\text{radian}$$

$$= \frac{C_1}{2R}\frac{180^0}{\pi}$$

$$= \frac{C_1}{2R}\frac{180 \times 60}{\pi}\text{minutes}$$

$$= 1718.87\frac{C_1}{R}\text{minutes}$$
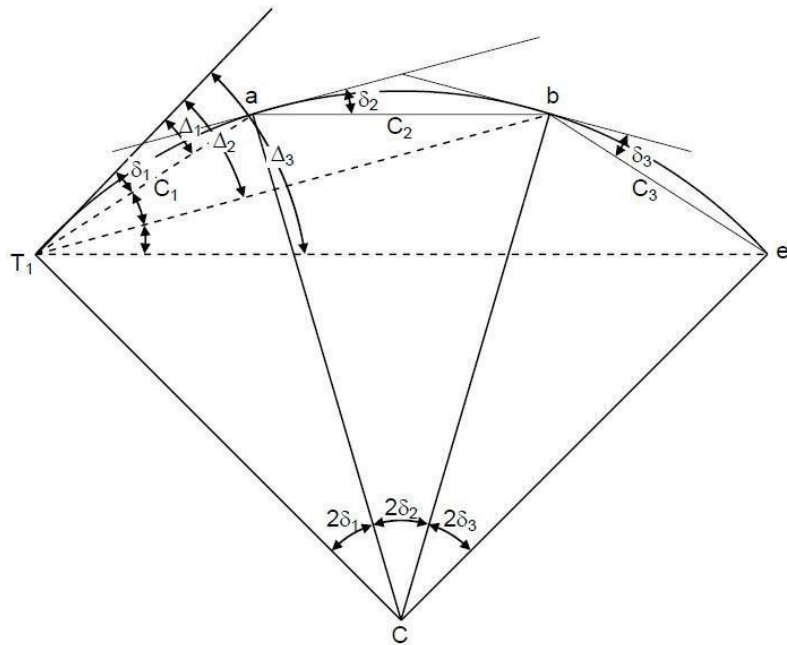
Rankine's Method

Therefore to locate the point a with the help of a theodolite and tape, the instrument is set at T_1 and the line of sight is put at an angle of $\delta_1 = \Delta_1$ as computed above. Then with the help of a tape and ranging rod, the tape is put along the line of sight and distance C_1 is then measured to locate point a along the line of sight.

Similarly,

$$\delta_2 = 1718.87\frac{C_2}{R}\text{minutes}$$

22

Since the theodolite remains at T_1, b is sighted from T by measuring $\delta_1 + \delta_2 = \Delta_2$ from the tangent line. The point b is located with the help of a tape and ranging rod. The tape with the ranging rod is so adjusted that the tape measures $ab = C_2$ and the ranging rod lies along the line of sight $T_1 b$

Similarly,

$\Delta_3 = \delta_1 + \delta_2 + \delta_3 = \Delta_2 + \delta_3$

$\Delta_n = \delta_1 + \delta_2 + \delta_3 + \text{... ... ...} + \delta_n = \Delta_{n-1} + \delta_n$

In practice, C_1 is the first sub-chord and C_n the last sub-chord.

$C_2 = C_3 = \cdots = C_{n-1}$ are full chain lengths. As a check the deflection angle Δ_n for the last point T_2 is equal to $\dfrac{\Delta}{2}$ where Δ is the angle of intersection.

Field Problems in Setting Out the Circular Curves

The following are some of the field problems in setting out the circular curves.

    (a) Point of curve inaccessible.

    (b) Point of tangency inaccessible.

    (c) Point of intersection inaccessible.

    (d) Curve tangential to three lines.

    (e) Both point of commencement and point of intersection inaccessible

Point of intersection inaccessible

If the point of intersection P.I. is inaccessible then to set out a curve, the following procedure is followed : First locate points P and Q on IT_1 and IT_2 respectively, then measure angles α and β with the theodolite and length PQ with a tape .

Then $\dfrac{IP}{\sin\beta} = \dfrac{PQ}{\sin\Delta}$

Or $IP = \dfrac{PQ\sin\beta}{\sin\Delta}$

Similarly

Point of Intersection Inaccessible
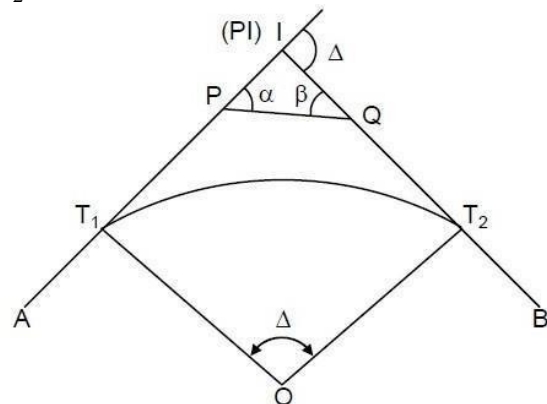
$$IQ = \frac{PQ\sin\alpha}{\sin\Delta}$$

Calculate $PT_1 = IT_1 - IP$

$QT_2 = IT_2 - IQ$

Thus, T_1 and T_2 can be located from P and Q respectively and the curve can be plotted from T_1.

TRANSITION CURVE

A transition or easement curve is a curve of a varying radius introduced between a straight and a circular curve, or between branches of a compound curve or reverse curve. The introduction of a transition curve between the straight and the circular arc, as indicated in Figure below, permits the gradual elevation of the outer edge or gradual introduction of *cant or super-elevation* (raising the outer edge over the inner). At the same time, it also permits gradual change of direction from straight to the circular curve and vice-versa.

On a straight track, its two edges are at the same level. On a circular arc the outer edge is elevated depending on the radius of the curve and the speed to the vehicles expected, to avoid over turning of the vehicles due to centrifugal force acting on them while moving on circular path. Also, there is an abrupt change in direction when the alignment changes from straight to circular curve and vice-versa.
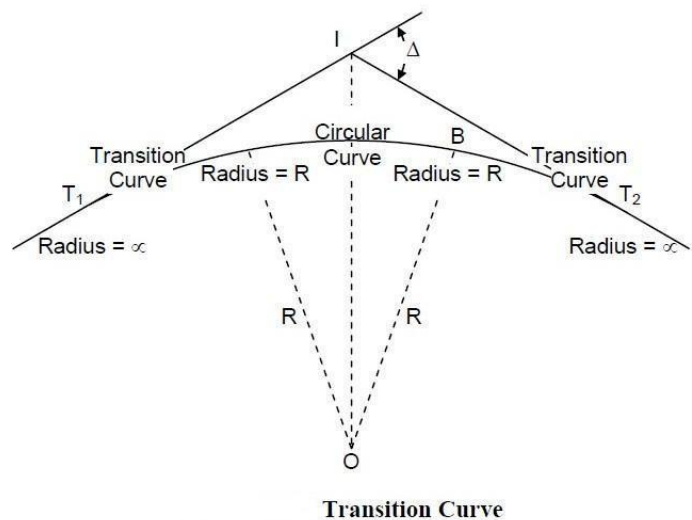
Transition Curve

In railways, such a curve is provided on both sides of a circular curve to minimise super-elevation. Excessive super-elevation may cause wear and tear of the rail section and discomfort to passengers.

Advantages of a Transition Curve

The introduction of a transition curve between a straight and a circular curve has the following advantages :

(a) The chances of overturning of the vehicles and the derailment of trains are reduced considerably.

(b) It provides comfort to the passengers on vehicles while negotiating a curve.

(c) The super-elevation is introduced gradually in proportion to the rate of change of curvature.

(d) It permits higher speeds at curves.

(e) It reduces the wear on the running gears

Characteristics of a Transition Curve

(a) It should be tangential to the straight.

(b) It should meet the circular curve tangentially.

(c) Its curvature should be zero at the origin on tangent.

(d) Its curvature should be equal to that of the circular curve at the junction with the circular curve.

(e) The rate of change of curvature from zero to the radius of the circular curve should be the same as that of increase of cant or super-elevation.

(f) The length of the transition curve should be such that full cant or super-elevation is attained at the junction with the circular curve.

CHAPTER-3

SETTING OUT WORKS

Using different method of surveying, data are obtained about the ground features that are represented in maps and plans of the ground. Such drawings are prepared of various elements of the project.

Marking the outlines of excavation on the ground for the guidance of the contractor and labour is defined as **setting out of works** .

- On completion of the estimates from the approved plan excavation for the foundation is required to be made on the ground.
- In order to minimizes the cost of digging foundation trenches out lines of excavation stakes should accurately marked.

(i)Buildings:

To set out the building following materials are required

(a) A foundation plan is prepared marking the centre lines and excavation width to facilitate setting out on ground.
(b) Depending upon the method used, the instruments are selected.
(c) For important buildings, a theodolite, tape, a cord to stretch, and marking powder, such as lime

There are two methods of setting out of building

a) Circumscribing rectangle method
b) Centre line method

(a) Circumscribing rectangle method :

The procedure for the circumscribing rectangle method is as follows:

- Established control points A and B near the structure but at sufficiently large distance such that are not disturbed during the excavation.
- Erect perpendiculars at A and B to get points C and D of sufficient length so that they are away from the excavation limits. This can be done with a tape alone using the 3-4-5 principle, or more accurately using a prismatic compass or theodolite.
- Check the rectangle set out by measuring the diagonals AC and BD, which should equal to their calculated lengths.
- Correct any error proceeding further.
- The centre lines of the four walls of the building from the rectangle EFGH, Locate the four corners from the respective corners of ABCD.

- Check the lengths of this sides and the diagonals of EFGH.
- Set temporary stakes at points E, F, G, and H and mark these points with a nail.
- Tie cord between the nails and mark the centre line with dry lime powder.
- Mark the inner and outer boundaries of the foundation widths in the same way.
- Again mark the four corners of these rectangles with temporary stakes. Note that these stakes will go away once the excavation starts.
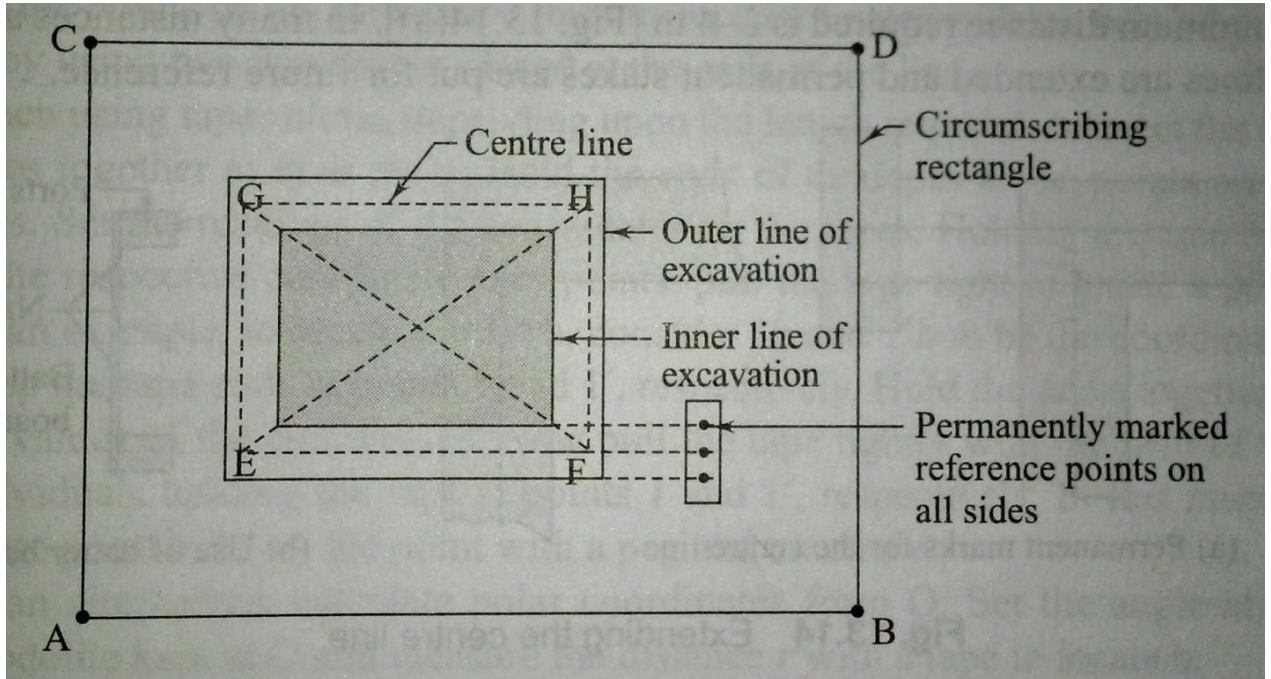
Fig. Circumscribing rectangle method

(b) Centre lines method :

- In the centre lines of the four walls of the building can be used as a reference rectangle.
- The corners of the centre line are accurately set out with respect to the control points near the site.

- In many instances all the three lines are extended and permanent stakes are put for future reference.
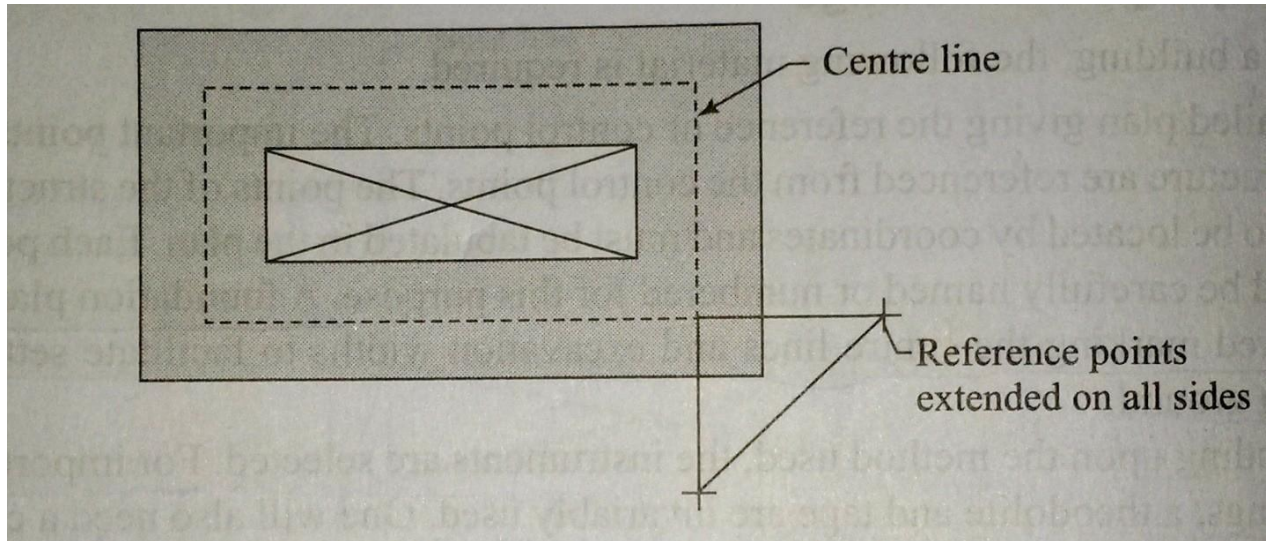
Fig. Centre line method

Setting Out Culverts

For setting out of Culvert foundations, abutments, and wing coordinates of a number of points on the lines are required. For this purpose, the origin is selected at the intersection of the centre lines of a waterway or a road or railway line passing over it.

The following procedure is adopted.

- From the foundation plan of the culvert and the roadways, locate the two centre lines A0A' and B0B' where 0 being the origin as shown in fig.
- Locate these centre lines, from the control points available, near the sites coordinates.
- Check and verify that these two lines are at right angles.
- Drive a peg at 0 and mark it carefully.
- Set up a theodolite at 0, centre and level it.
- Set up a number of points along both the lines. Assume that 1, 2, 3, 4, etc. are the points along A0A' and 1', 2', 3', 4', etc, are the points along B0B'.
- Mark the points with pegs and arrows such that a cord tied along the arrows defines the lines and the points marked on them.
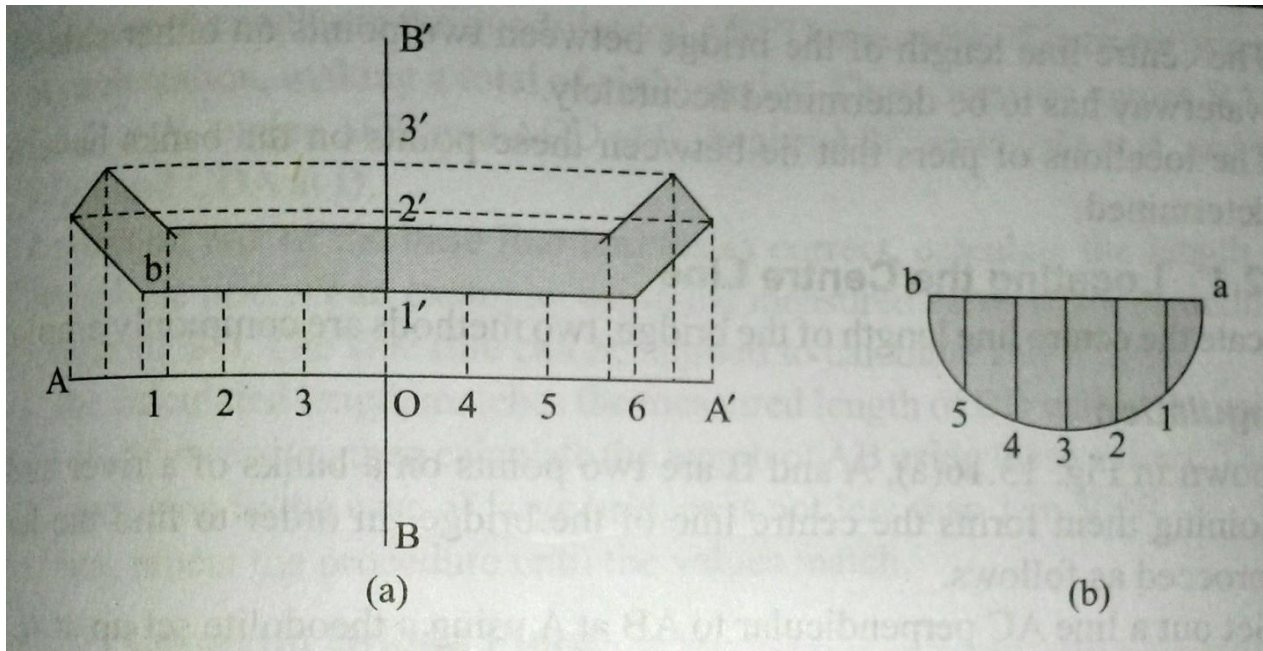
Setting out a culvert

- Set the points on the foundation lines using coordinates with the help of a tape or by using two theodolite placed at the ends of the lines.

- When using tapes alone, depending upon the length required, connect the two tapes together at their rings. Hold the ends of the tapes at the points on the axes. Set the readings of the coordinates on the tapes. Holding the tape ends at the respective coordinate axes points, pull the tight to locate a point. As an example, to locate a point b, consider 1b and 1'b to be the coordinates. Keep the tape ends at points 1 and 1' respectively, Hold the tapes together at the values of the coordinates. Now pull the tape tightly with help of two individuals holding the tape at points 1 and 1', respectively. In this manner, locate point b. mark the point with a peg and an arrow.

- As an alternative, calculate polar coordinates from 0. Set the angle at the theodolite kept at 0 and measure the distance 'r' with a tape to locate b.

- Once all the points a, b, c, d, etc, are located, tie a string around the points located.

- Make a mark along the string with dry lime powder or make a line by nicking.

- In case any of the walls have a curved outline, locate the end points first. Then locate the curved boundary by using coordinates from the chord of the curve. As shown in fig….

CHAPTER-4

9.1 Micro-optic Theodolites :-

Micro-optic theodolites can read angles to an accurancy of 10" or even less. The principle is illustrated in Fig.9.1. The special features of such theodolites are as follows.

(a) Conventional metal circles are replaced by glass circles on which the graduations are etched by photographic methods. The graduations can be made finer and sharper by this technique. Both the horizontal and vertical circles are made of glass and generally graduated to 10.

(b) Light passing through the circle at the point of the reading is taken through a set of prisms to the field of view of the observer. For passing light through glass circles, sunlight is reflected through a reflecting prism and passed through the circle. In case night operation is required, the battery-operated light provided in the instrument can be used.

(c) Both the horizontal and vertical circles are seen at the same in the field of view. This is an advantage, as the readings of both the circles can be taken at the same time. Some manufacturers make a switching arrangement so that the horizontal or vertical circle reading can be seen along with the micrometer reading.

(d) The optical micrometer is used to read fractions of the main scale division. Depending upon the reading system, angles can be read up to 10' or less.

(e) The circles are generally graduated to 10' or 20' of the arc. The micrometer can be read after coinciding the index with the nearest main scale division. The fractions are then read from the micrometer scale, which is also seen in the field of view.

(f) A small, separate reading telescope is provided besides the main telescope. It eliminates the need to move while bisecting an object and taking the reading.

(g) In most instruments, diametrically opposite ends of the circle are brought together in the field of view.
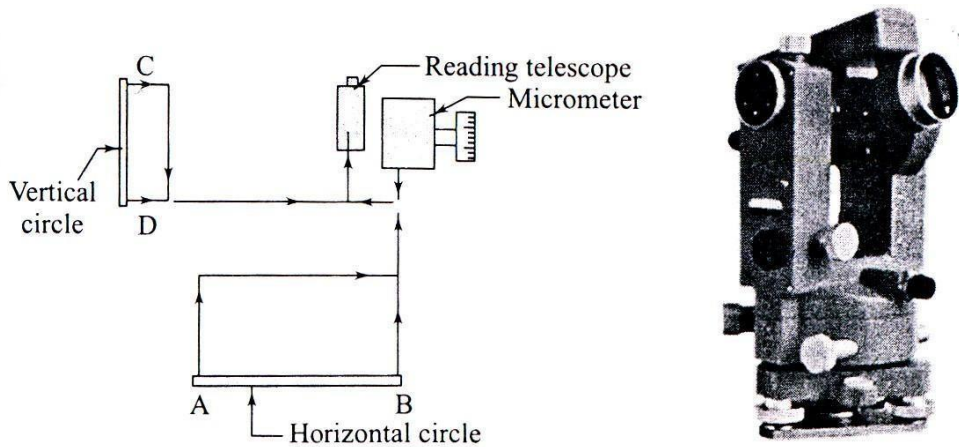
Fig.9.1

Digital theodolites:-

Digital theodolites are very fine instruments for angle and distance measurements. The instruments are light weight and are simplar to electronic theodolites in construction.

The instrument is set up over a station as in the case of normal theodolites. They will have extendable tripod legs which can be adjusted for comfortable viewing. The centering and leveling operations are done with a circular vial for coarse setting one has to press only a measure button to get the readings of angles and distances. Some models also have a laser pointer for easy alignment in critical cases and for staking out operations. With the arrival of total stations, these theodolites have less demand though they are cheaper compared to a total station.

The following are typical features in a digital theodolite:

- Angle measurement – by absolute encoding glass circle; Diameter – 71 mm

- Horizontal angle- 2 sides; vertical angle – one side; Minimum reading – 1"/5"

- Telescope – Magnification – 30x; Length – 152 mm; objective lens – 45 mm
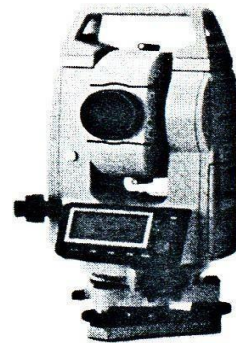
Fig.9.2

- Field of view - $1^0 30'$ Minimum focus distance – 1m

- Stadia values: Multiplying constant – 100; additive constant-0

- Laser pointer – coaxial with telescope; 633 nm class II laser; Method – focusing for alignment and stake out operations.

- Display on both sides; 7-segment LCD unit

- Display and reticle illuminated

- Compensator- tilt sensor; vertical tilt sensitivity + 3'

- Optical plummet – magnification – 3x; field of view - 3^0; focusing from 0.5 m ti infinity

- Level sensitivity – Plate vial – 40"/2mm; circular vial – 10'/2mm

- Power supply – 4 AA size batteries; Operating times – Theodolite only – 140 hours

- Laser only – 80 hours; Theodolite + laser – 45 hours

- Weight – 4.2 kg.

(ii) Electronic Distance Meter (EDM):

EDM equipment can be classified based upon the type of wave used, into M (microwave) DM and EO (electro-optical) DM equipment. The first type uses low-frequency short radio waves while the second type uses high-frequency light waves. They cal also be classified based upon the range as follows.

(a) Short-range equipment such as teleprompters and mekenometers with a range of up to 3 km.

(b) Medium-range equipment such as geodimeters with a range of up to 25 km. The range is about 5 km during the day and can go upto 25 km at night.

(c) High – range equipment with a range of up to 150 km. Tellurometers and distomats come under this category.

The accuracy varies with the range. Short-range equipment has an accuracy of ± (0.2 mm) + 1 mm/km. Medium-range equipment has an accuracy of ± (5 mm + 1 mm/km) while high-range equipment ha san accuracy of ± (10 m + 3 mm/km). Distomats have replaced other forms of equipment due to their compact design, ease of operation, and precision.

All types of equipment using electromagnetic waves perform the following functions.

(a) Generation of two waveforms for carrier and measurement functions.

(b) Modulation and demodulation of waves.

(c) Measurement of phase difference.

(d) Computation and display of distance or the results of measurement.

8.2 Total Stations

One of the recent developments in surveying equipment is the integration of distance- and angle-measuring components in one piece of equipment. A total station is the integration of an electronic theodolite with the EDM equipment. Many companies market total stations. Though the technology details used by different manufacturers may be different, they all have common features, which will be discussed below.

A digital theodolite is combined with one of the many forms of EDM equipment to obtain a very versatile instrument that can perform the required functions very easily.

Digital Theodolite:

The electronic or digital theodolite was discussed in Chapter 4. We will just recapitulate some salient points. These instruments have glass circles, which are encoded in the incremental or absolute mode. These are read by an optical scanning system and the reading is converted into angles and displayed or stored by the instrument. All the instruments are provided with an optical plummet for centering and a compensator system (single-axis or dual-axis) to take care of the tilt of the and the displayed angles and distances are previously corrected for such minor errors. The user can choose the required accuracy of angular measurement. These theodolites are normally operated by a rechargeable battery pack. The charged batteries can work for 40-80 hours. Some instruments need a prisms. Even reflecting tapes are used. A digital theodolite comes with the following facilities.

(a) Zero-setting

(b) Bidirectional measurement

(c) Precision setting

(d) Horizontal and vertical angles

(e) Slant distance and horizontal distance

(f)   Difference in elevations

(g)   Entry and display of data

(h)   Display and storage of result

(i)   Data management system and data transfer facility

A total station has all the above facilities and in addition measures horizontal distance using a built-in EDM module. Total stations come with a lot more facilities of data storage and manipulation. The following are the salient features of a total station.

Angle measurement:- Horizontal and vertical angles are measured to an accuracy of 1"-5". The angles are displayed on the display unit of the console. Many instruments have console units on both sides of the instrument.
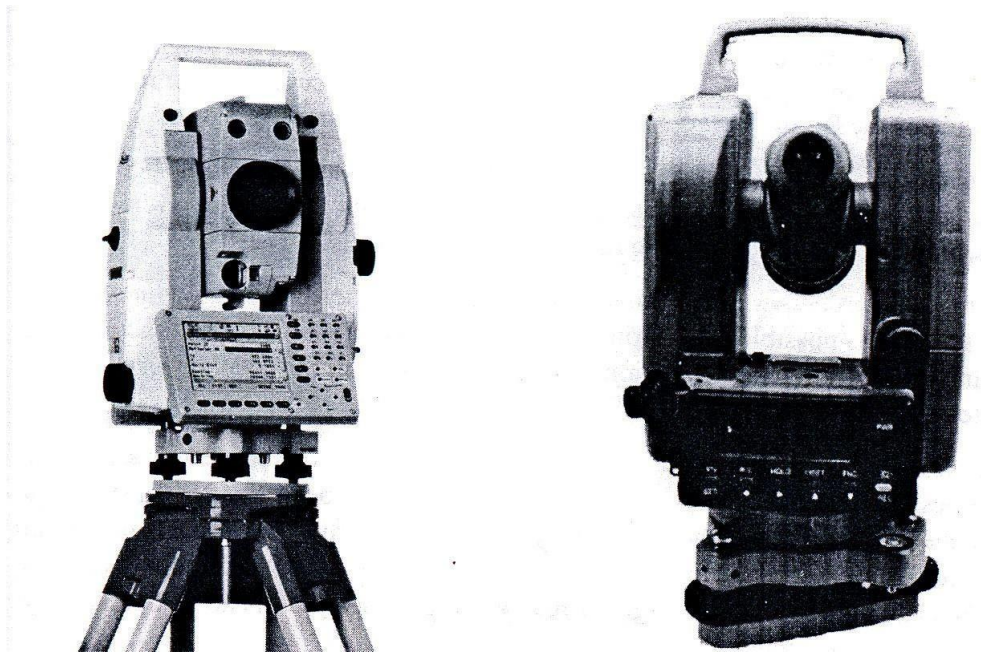
Fig.9.3

Distance measurement:- This is done with an EDM module functioning coaxially with the telescope tube. The distance measured is the slant distance if the stations are at different elevations. Reflecting multiple prisms are commonly used as targets, even though reflectorless distance measurement has also been made possible. The instrument uses the vertical angle measured by the theodolite and calculates the horizontal distance measurement can be done in different modes such as standard or coarse mode, precision mode, and fast mode, and fast mode. The precision and time taken vary depending upon the mode.

Microprocessor and software:- The onboard software in total stations can per form many functions. The processor is per-programmed, and in some cases can be programmed by the user to perform many useful functions with the measured data. The details may very with the manufacturers but some of the common features are as follows.

Automatic target recognition:- Most of the modern total stations have the facility of automatic target recognition (ART). In ATR, the telescope has to be roughly pointed towards the target while the measurement key is pressed. The instrument automatically points to the target before measurement. The instruments have motorized endless drives to facilitate ATR.

Reflectorless distance measurement:-Until recently, total stations had to be used with special multiple prisms as targets for EDM. The new versions of total stations can measure distances without a prism target. This means that distances to points where a target cannot be erected can now be measured easily without any extra survey effort. This has been made possible by a red laser, which can direct to a point on any surface.

Computation of reduced levels:- The reduced levels are measured from slope distance and vertical angle. Data input enables the user to input the height of instrument, height of target prism, and the RL of the station occupid. The instrument calculates the RL of the target station and displays the same.

Orientation:- The instrument automatically orients to any direction specified by the user. If the coordinates of two points are input, the horizontal circle will be oriented to measure the bearing of the line automatically.

Automated processes:- Automatic computation of coordinates of points, areas, offsets, etc. is possible with a total station. More and more on-board functions are being incorporated in total stations. Setting out points on the ground using coordinates or directions is possible.

Wireless keyboard and remote unit:- Many new total stations some with a separate wireless keyboard. The input of data to the station becomes very easy with a handheld keyboard. Another development is the availability of a remote unit so that the person at the prism can operate the total station for almost all the functions. As there is no need to bisect a target or read the angle, the system can be operated by one person positioned near the target.

Data management system:- Total stations have a very efficient data management system. Data transfer to data recorders, computers, or flash cards is possible. The in-built memory can store up to 10,000 blocks of data.

Graphic display:- Many new instrument have extremely powerful graphic display programmes. With large display panels, the data can be plotted and displayed.

Working with total station:-

Total stations are manufactured by many leading manufacturers of Survey equipment. Leica geosolutions, Topcon, Pentax, Nikon tripod data systems, Stonex are some of the major manufacturers of total stations. While specific details may vary with the manufacturers, some features are common to all of them.

A total station, as mentioned earlier, is a versatile equipment for surveying operations. The equipment details and operations can be understood by referring to the user manual provided with the equipment.

8.3 Aerial Surveying

The procedure for aerial surveying includes reconnaissance of the area, establishing ground controls, flight planning, photography, and then paperwork including computation and plotting.

Reconnaissance is undertaken to study the important features of the ground for reference purposes. Ground control is required in order to obtain a set of points known position based on

which other points are located and plotted. The number of ground control points depends upon the extent of area covered, scale of the map to be prepared, flight plan, and the process of preparing the maps. A minimum of three control points must appear in each photograph. These points are established by triangulation or precise traversing.

Flight control is achieved by flight planning, which takes into account the extent of the area, type of camera and its focal length, scale of the photographs, altitude speed of aircraft, and the overlaps of the photograph. The area covered by each photograph. Time interval between exposures and the number of photographs quired are decided based upon such flight planning.

Stereoscopes:-

There are many types of stereoscopes- mirror stereoscope, lens stereoscopes, scanning mirror and zoom stereoscopes. Lens and mirror stereoscopes are handy and commonly used.

Mirror stereoscope :-The schematic diagram of the mirror stereoscope is shown in fig.9.4 (a). The mirror stereoscope consists of two viewing eyepieces. A stereoscopic pair of photographs is placed at a distance from the stereoscope. The photographs are adjusted so that one photograph is seen through one eyepiece. The instrument has four mirrors, two mirrors attached to each eyepiece. As the viewer looks through the stereoscope, he/she sees the image of the same object (the overlapping part) on the two photographs and this gives a stereoscopic view by fusion. The terrain is seen in relief due to this.
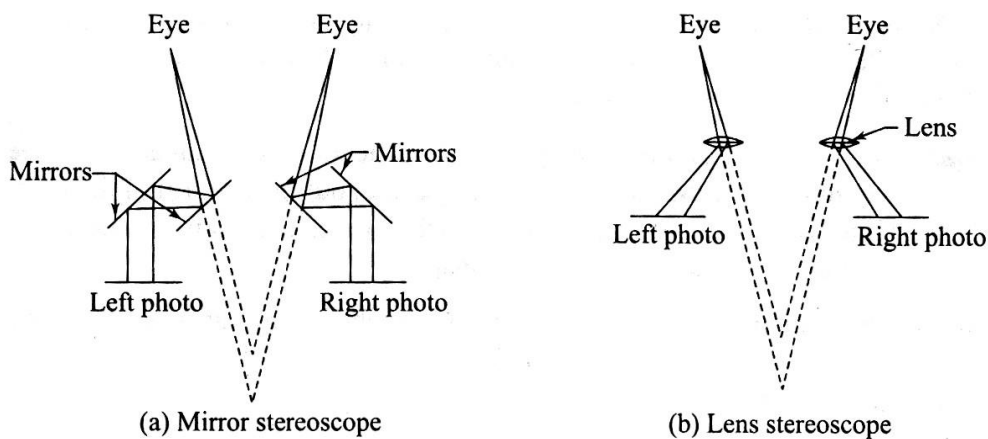
Fig.9.4

Lens stereoscope:- A lens stereoscope has two eyepieces through which the observer sees the photographs, providing the experience of stereoscopic or spatial view. The lenses help to magnify the image as seen by each eye. The distance between the eyepieces is adjustable and can be set by the observer as per requirement. This distance is approximately equal to the distance between human eyes. The lenses tend to magnify the object and its height. Lens stereoscopes are more compact than mirror stereoscopes.

Photo-interpretation:-

Photo-interpretation is the key to effective use of photographs. It refers to the accurate identification of the features seen in photographs. Objects seen in photographs are often not easy to recognize, and it takes some amount of skill on the part of the interpreter to correctly identify the objects and judge their significance. It is more difficult to identify objects in vertical photographs than in tilted photographs owing to the familiarity of view in oblique photographs. Colour photographs are easier to interpret than black and white photographs due to tonal variations. A stereoscopic pair is easier to interpret due to the depth available in the photographs when seen through a stereoscope. Considerable amount of practice and experience is required to correctly interpret photographs.

Interpretation of aerial photographs is required extensively in developmental project design and execution. It has been successfully applied in a variety of fields. The success of project planning depends on the effective and efficient interpretation of photographs by engineers and others and others. A good deal of patience and ingenuity is required to interpret photographs.

General Features of Photographic Images:-

The knowledge of some of the basic characteristics of the image in aerial photographs helps helps one to interpret these images. Photo-interpretation requires large-scale photographs. The success of the interpretation depends upon the experience of the person in addition to the conditions under which the photographs should be studied in the correct orientation with respect to the light conditions at the time of photography. Some of the basic features of photographs that help in identification are discussed here.

Size:- The size of an object in the photograph is sometimes helpful in interpretation. Knowing the photograph scale, it is possible to have an idea about the size knowing the correct size, one may not confuse among objects having similar shapes such as a river, road, canal, or drain.

Shape:- The shape of an object is helpful in identification. Regular shaped objects are generally man-made. Shape relates to the general outline or form of the object. A railway line and a roadway can be distinguished from their form. Objects of the same size can be distinguished from their shape.

Texture:- it is simply the variation in tone of the photograph. It is produced by a combination of factors such as size, shape, tone pattern, and shadow. Vegetation and other ground features can be distinguished by the tonal changes.

Pattern:- It is the spatial arrangement of objects in a particular set. A habit can be easily distinguished by the arrangement of roads, houses, etc. because of the pattern.

Shadow:- The shadow of an object formed during photograph is sometimes helpful in identification, as it shows the outline of the object.

Tone:- It is produced by the amount of light reflected back by the object to the camera. If the particular tones associated with specific objects are known, it is easy to identify them.

Location:- The location of an object in the photographs helps in identifying the object itself. Knowing the objects or areas surrounding the object, one can identify the main object. Refer Chapter 24 for more on visual image processing.

Applications and Advantages of Aerial Surveying:-

As has been discussed in the preceding sections, aerial surveying finds many applications in map preparation and map revision for large areas. Modern plotting machines and mostly automated operations have simplified the process of preparing maps from aerial photographs. Aerial surveying also finds extensive application in urban planning and development, transportation network design and calculations, disaster management, forestry, mining operations, reservoirs, agriculture, etc.

With advancements in technology, aerial photography has given way to aerial image processing. High-resolution digital image (soft image) can be made and processed using software to prepare excellent maps. All forms of rectification and corrections can be done automatically before converting the data into a map. Digital photogrammetric equipment and software have developed sufficiently to facilitate the preparation of very accurate maps.

Aerial Photogrammetry:-

Terrestrial photogrammetry virtually went out of use with the advent of aerial surveying techniques. Aerial photogrammetry makes use of cameras fitted in an aircraft to photograph an area from an overhead position. The principle of stereoscopic vision is used in studying and interpreting aerial photographs. Therefore overlapping photographs are taken in the direction of flight as well as in the lateral direction as the aircraft flies along a parallel path. It must be understood that while a map is an orthographic projection by projecting points perpendicular to the plane a photograph is a perspective projection, as all the light rays for forming the image pass through a point.

Basic Terminology:-

An aerial photograph is a record of the ground features at a point in time. Aircraft fitted with cameras moves along predetermined paths and takes photographs at planned intervals. The following are the basic terminology used to describe aerial photography.

Altitude: - Height of the aircraft above the ground.

Flying height: - Height of the aircraft above a chosen datum.

Exposure station:- Position of the aircraft at the time of exposure of the film. It is essentially the position of the optical centre of the camera lens when film I exposed.

Air base: - Distance between two consecutive exposure stations.

Tilt and tip: - Tilt is inclination of the optical axis of the camera about the line of flight. In ϕ is the tilt. Tip is the inclination of the camera axis about line perpendicular to the line of flight.
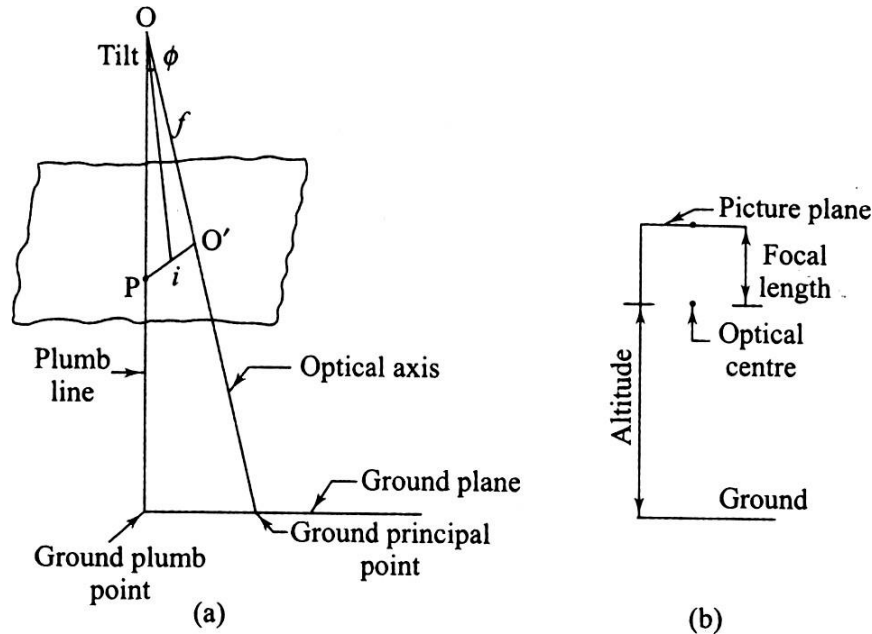
Fig.9.5

Picture plane:- Plane that contains the image at the time of camera exposure.

Ground plane: - Horizontal surface from which heights can be measured and which can be used as a datum surface.

Principal point:- Point of intersection of the optical axis of the camera with the photographic plane. O is the optical centre and O' is the principal point. When the optical axis is extended downwards, the point of intersection with the surface is known as the *principal ground point.*

Isocentre: - Point on the photograph at which the bisector of the angle of tilt meets the photographic plane. 'i' is the isocentre, at a distance of $f \cos \phi$ along the principal line, where f is the focal length of the camera.

Plumb points: - The points at which the vertical line through the optical centre meets the photographic plane and the ground surface. The plumb point on the ground surface is also known as ground nadir point. The plumb point on the photograph is known as nadir point.

Homologous points: - Points on the ground and their representations in the photo graph in perspective projection.

9.4 Remote Sensing

Remote sensing, as the name implies refers to collecting data from a remote location without being in physical contact with the object. Remote sensing is not as uncommon as we may think. We have many remote sensing activities in day-to-day life. When we see an object and recognize its colour as red, we are using the concept of remote sensing. Similarly, our sense of smell also helps us to use remote sensing. Some of the common methods of remote sensing are described below.

Active and passive system of remote sensing:

In an active system of remote sensing, the sensing equipment emits radiation, which is reflected back from the object. Radar is a typical example of such a system. Radar equipment transmits radiation and the reflected radiation is analysed to determine the distance and presence of any object in the ranging area.

In a passive system of remote sensing, the instrument does not generate and emit radiation. The radiation reflected from an external source is made available to the object. We use the passive system exhaustively in the form of the sun's radiation. Taking a photograph using light from the sun is an example. Photographic cameras, still or motion picture, and television cameras use the passive system of remote sensing.

Applications/Uses of Remote Sensing

Remote sensing has applications in a wide spectrum of areas. Remote sensing can be used for taking sound decision for planning many human development activities. It is also possible to take preventive action as in the case of forest fire and natural disasters, Weather forecasting is another important application. Some of the application areas are given below.

Land use and land cover analysis:- Perhaps one of the prime uses of satellite remote sensing is in the study of land use and cover. Land cover through vegetation and specific crop areas can be studied using remote sensing data. Forest cover is an important aspect, which has been studied: the depletion of forest areas has been identified with the help of remote sensing. It is also possible to study crop diseases over large areas.

Mineral exploration:- It will be possible to use satellite data ana discover the presence of valuable minerals and ores that are vital to economic development. Non-renewable energy resources, such as fossil fuels, can be identified using remote sensing data.

Environmental studies:- Global weather phenomena are a major area for study using remote sensing data. Global warming and ozone layer depletion can be continuously monitored using remote sensing. Similarly, oceanographic studi4es also provide valuable information about the various characteristics of oceans around the world. Assessing water resources, their extent and depletion, snow cover studies, etc have proved to be valuable.

Archaeology:- Archaeological studies can make use of remote sensing data. The underlying old settlements can be recognized from remote sensing data and appropriate action can be taken to excavate and study the various aspects of old civilizations.

Disaster management: - This is another important application area of remote sensing. It has been possible to predict earthquake hazards by detecting unusual movements in the earth's crust. Floods .landslides, forest fires, etc can be detected on time and appropriate action can be taken for preventive action in disaster management.

Geomorphology: - Geological studies can provide valuable data on faults, tectonic movements, rock type identification, etc using remote sensing data.

Topography and cartography: - This is another application related directly to surveying. Remote sensing can be used to accurately locate points with reference to ground surveys are difficult or time consuming. This data can be used to prepare maps or revise existing maps.

Other applications: - Remote sensing data is now being used to study troop movements, etc for defense purposes. Other applications include urban planning studies, traffic studies, and assessment of earth's resources for various purposes, and so on.

Image Interpretation:-

Image interpretation is the process of extracting useful information from remote sensing data. Both qualitative and quantitative information can be extracted from maps .Earlier, the data was in analog form which is generally interpreted by humans Today, the data is generally is digital form which can be interpreted by humans of processed by computers. The correct interpretation

of remote sensing data is very important if it is to be useful for the various purposes for which it has been obtained.

Visual Image Processing:-

The remote sensing data can come in either of the two forms – raw data or processes after certain corrections. Visual images can be monochromatic or grey scale images or colour composites or colour photographs. The objective of visual interpretation is to obtain qualitative information about objects seen in the image. This includes finding their size, location, and relationship with other objects the way our eye perceives an object is different from the way remote sensing data is obtained. first, the image is taken from an aerial platform- an aircraft or a satellite. The view from above will be quite different from the view seen from the ground. Second, the sensors used for imaging record radiations from many parts of the electromagnetic spectrum including the visible band. This makes the imagery look different from what we see otherwise. Third, resolution obtained and scale of the image may be quite unfamiliar to the eye. Finally, the ground relief feature may not be evident in two-dimensional photograph or image. Stereoscopes are used to view photo pairs having common imagery to get a feeling of depth.

The following three processes are involved in image interpretation:

(1) Image reading is the first step in image interpretation and involves identifying objects in the image by their size, shape, pattern, etc.

(ii) Measurement from images is the extraction of information such as length, width, height, and other parameters like density or temperature from data keys as reference.

(iii) image analysis is the understanding of the information extracted and comparing with ground reality or the status of the features as existing at the time of imaging.

Visual interpretation as it is done using photographs has to be supported by ground investigation for correctness of the interpretation. This becomes very necessary as the image may have many features which are not immediately understandable by the interpreter. Multiple images in multiple scales and multi- spectral images have to be interpreted and verified before reaching any conclusion.

Elements of visual Interpretation:-

Some key elements that assist the interpreter in studying and extracting information about the objects in the image are the following:

Location: - It refers to the information about the objects in the image in terms of any of the coordinate systems used such as latitude, longitude, and elevation. If some points are available in the image with known coordinates, then the coordinates for other points or objects can be obtained by measuring distances from the known points. Actual ground surveys can also be performed using easier methods that use GPS or by traditional methods of surveying that use total station to get coordinates. Computer processing of the image after rectification can also be employed to get information about coordinates.

Size: - The size of an object seen in an image depends upon the scale of the image. Knowing the scale of the image, the length, width, perimeter or area can be used to extract information about the subject. The absolute size of an object along with its relative size is also important in distinguishing between features having the same shape. The size can help distinguish between objects of the same shape such as a building or a football field.

Shape:- The shape of an object is distinguishable in the image and can help the interpreter to identify the object. Objects of regular shapes such as rectangles square, circle or oval are generally man-made structures. Irregular boundaries of an object generally mean that the object is of natural origin such as forest area or a lake. Since the imaging is done from above, it is necessary to know how an object looks from the top.

Shadow:- Shadows are generally not desirable in images as they change the nature of the image that would have been seen otherwise. However, shadows help in finding the heights of tall structures like towers and multi-storey buildings. Shadows are created due to low sun angles. In addition to aiding in ascertaining the height of objects, shadows also provide a profile view of objects which is helpful in identification.

Tone: - It is the relative brightness or colour intensity of the image. A black and white photograph is a grey tone image with brightness ranging from black to white. The remote sensing sensor receives and displays a band of the spectrum of electromagnetic radiation and this is displayed as continuous shades of grey which gives different tones in the image. Tones are useful

features of interpretation because different objects give unique tonal qualities due to their reflectance. Tonal differences can occur due to different bands in multi-spectral images. Experience and a clear eye help to distinguish the tonal variation.

Color :- Color images are obtained from colour films. Colour photographs or images hold a lot more information than black and white grey tone images. From the natural colour of the image in the film many features like vegetation can be identified. Colour can change depending upon the type of film and filters used. Colour corrections can be done to images to give true colours of the objects.

Texture:- It can be defined as the characteristic placement and variations in definite patterns for objects in the grey tone image. Textures are classified as smooth or coarse. This is due to the visual impression created by the tonal changes. Coarse textures are due to sudden changes due to abrupt changes in tone in small patches giving a mottled appearance. Smooth texture comes from very little changes in tone. Texture helps to identify objects in an image due to characteristic textures of objectse, especially vegetation and forest trees.

Pattern: - It refers to the randomness or regularity of similar objects in the image. The pattern seen in the image is helpful in identification. Arrangement of trees in a forest is random, while trees in a orchard are placed in an orderly way. Same is true of houses in a neighborhood or buildings in a developed area. Such patterns can be identified and the objects recognized from the pattern.

Elevation: - As mentioned in Chapter 22, stereoscopes are used in association with photo pairs to have a view of the difference in elevation of objects. The overlapping areas of the images in photo pairs are useful in finding the elevations of points and also to have an idea about the relative heights of different objects seen in the image.

Interpretation keys: - These are used to help in visual interpretation of images. The keys are prepared by experienced interpreters who from past experience and ground verification prepare keys based on major elements of identification. Keys can be prepared for specific uses such as forestry, urban studies, network studies, and so on.

GANDHI ACADEMY OF TECHNOLOGY AND ENGINEERING

GOLANTHARA, BERHAMPUR

LECTURE NOTES

ON

Construction Management

For 6th Semester

(As per Syllabus prescribed by SCTE&VT, Odisha)

Prepared By

Sasmita Kumari Das

(Lecturer in civil Engineering)

1 Introduction To Construction Management

1.1 Aims and objectives of construction management.

1.2 Functions of construction management.

1.3 The construction team componentsowner,engineer,architect,contractor-their functions and interrelationship and jurisdiction.

1.4 Resources for construction management-men,machines,materials,money

2 Constructional Planning

2.1 Importance of Construction Planning

2.2 Developing work breakdown structure for construction work

2.3 Construction Planning stages-Pre-tender stage, Post-tender stage.

2.4 Construction scheduling by Bar charts-preparation of Bar Charts for simple construction works. 2.5 Preparation of schedules for labour materials,machinery, finance for small works

2.6 Limitation of Bar charts

2.7 Construction scheduling by network techniques-defination of terms ,PERT and CPM techniques, advantages and disadvantages of two techniques, network analysis, estimation of time and critical path, application of PERT and CPM techniques in sample construction works.

3 Materials and Stores Management

3.1 Classification of Stores-storage of stock.

3.2 Issue of materials-indent , invoice, bin card

4 Construction Site Management

4.1 Job Lay out-Objectives, Review plans, specifications, Lay out of equipments.

4.2 Location of equipment, organizing labour at site.

4.3 Job lay out for different construction sites.

4.4 Principle of storing material at site.

5 Construction Organization:

5.1 Introduction – Characteristics, Structure, importance.

5.2 Organization types-line and staff, functions and their characteristics

5.3 Principles of organization- meaning and significance of terms- control, authority, responsibility, job & task.

5.4 Leadership-necessity, styles of leadership, role of leader

5.5 Human relations-relations with subordinates, peers, Supervisors, characteristics of group behavior, mob psychology, handling of grievances, absenteeism, labour welfare.

5.6 Conflicts in organization-genesis of conflicts, types-intrapersonal, interpersonal, intergroup, resolving conflicts.

6 Construction Labour and Labour Management:

6.1 Preparing Labour schedule

6.2 Essential steps for optimum labour output

6.3 Labour characteristics

6.4 Wages & their payment

6.5 Labour incentives

6.6 Motivation- Classification of motives, different approaches to motivation.

7 Equipment Management

7.1 Preparing the equipment schedule

7.2 Identification of different alternative equipment

7.3 Importance of Owning & operating costs in making decisions for hiring & purchase of equipment 7.4 Inspection and testing of equipment

7.5 Equipment maintenance

8 Quality Control

8.1 Concept of quality in construction

8.2 Quality Standards- during construction, after construction, destructive &non destructive methods.

9 Monitoring Progress

9.1 Programme and progress of work

9.2 Work study

9.3 Analysis and control of physical and financial progress corrective measures.

10 Safety Management In Construction

10.1 Importance of safety

10.2 causes and effects of accidents in construction works

10.3 Safety measures in worksites for excavation, scaffolding, formwork, fabrication and erection, demolition.

10.4 Development of safety consciousness

10.5 Safety legislation- Workman's compensation act, contract labour act.

11 Role of Vulnerability Atlas of India in construction projects

11.1 Introduction to Vulnerability Atlas of India, Concepts of natural hazards and disasters and vulnerability profile of India. Definition of disaster related terms.

11.2 Earthquake hazard and vulnerability, Magnitude and intensity scales of earthquake, seismic zones, earthquake hazard maps, types of structures and damage classification, effects in housing and resistant measures.

11.3 Wind / Cyclone hazard and vulnerability, wind speed and pressures, wind hazard and cyclone occurrence maps, storm surveys and cyclone resistant measures.

11.4 Flood hazard and vulnerability, Flood hazard and Flood prone areas of the country, General protection of habitants and flood resistant construction.

11.5 Landslides, Tsunamis and Thunderstorm hazards and vulnerability, Landslide & Thunderstorm incidence maps, Measures against Tsunami hazards.

11.6 Housing vulnerability risk tables and usage of vulnerability atlas of India, Inclusion of vulnerability atlas in Tender documents.

CHAPTER-1 INTRODUCTION

• Aims and objective of construction management

• Functions of construction management

• The construction team components

• Resources for construction management

CONCEPT OF MANAGEMENT

• The term management has different senses of use. Sometimes it is used in the sense of an organisation in which different class of people work together to provide qualitative and economical product by the use of human beings and other resources like machine, money and material.

• Or somtimes it may be defined as the process consisting of planning, organising, activiting and controlling the performance to determine and accomplish the objective by the use of men. machines, materials and money.

1.1. Aim & Objective of constructionManagment.

The following are the main objectives of the construction management.

- The work should be completed within estimated budgect and specified time

- There should be the motivation to working people to give their level best their capacities to complete the work.

- There should be qualified and trained staff to sup? the work properly.

- The execution of work should be done as per specification.

- The execution of work should be done as most economically.

- The working quality and workmanship should be good.

- There should be a proper plan of work and it should be organised properly.

- There should be an awareness of creating an organisation that works as a team.

- The workers should have been provided with safe and satisfactory.

FUNCTIONS OF CONSTRUCTION MANAGEMENT

The following are the functions of construction management v Planning

- Organising.
- Staffing.
- Directing
- Controlling
- Co-ordinating
- Communicating.

PLANNING

• Time needed to complete the whole construction project

• Type, quantity and exact time for delivery of materials of costruction.

• Type, number and duration of use of different machines and equipments.

• Category of staff i.e.,Managers, skilled and unskilled workers required.

• Type of uncertainties likely to cause delays such as weather conditions, shortage of supply, labour unrest and sub-judice land matter etc.

- WHAT TO DO

- WHEN TO DO
- HOW TO DO
- WHO TO DO

<u>ORGANISING</u>

• After the planning is in place, a manager be needs to organize her team and materials according to her plan.

• This process involves To identify the work to perform To classify or group the work To assign these group of activities to individuals To delegate authority and fix responsibility STAFFING

• Staffing is filling the position in the organisation structure for defining recruitments.

• It is a very important responsibility to select right person for right jobs in a construction organisation.

• Staffing is not only about the recruitment but also their training and developing activities.

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• It is a very important responsibility to select right person for right jobs in a construction organisation. • Staffing is not only about the recruitment but also their training and developing activities. DIRECTING

• A manager needs to do more than just plan, organize, and staff her team to achieve a goal.

• She must also lead.

• Leading involves motivating, communicating, guiding, and encouraging.

• It requires the manager to coach, assist, and problem solve with employees.

CONTROLLING

• After the other elements are in place, a manager's job is not finished. He needs to continuously check results against goals and take any corrective actions necessary to make sure that his area's plans remain on track.

• Controlling is an important action for ensuring effective and efficient working.

• It reviews the work plan to check and rectify the deviation.

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CO-ORDINATING

• It means developing harmony between employees and group of employees for smooth and efficient functioning of construction work.

• In large organisation the work is divided into different departments. So there is a great importance for good coordination. COMMUNICATING

• Communication is the process of transmitting receiving and understanding the ideas by others for the purpose of effective desired results

• There are various methods of communication like verbal return others reports instruction result .

• Ineffective communication leads to confusion misunderstanding Etc.

OWNER

❖ The owner of a construction project may be an individual, group of indivduals or public body. The owner finances the project and also recoginses the need for a project.

❖ in view of all aspects the owner has the power to take major decisions regarding managerial financial and administrative aspects

CONTRACTOR

• The contractor executes various types of works and also makes necessary arrangements for labour, machinery, materials, in order to complete the project in the limited scheduled time.

• In some projects, the contractor may appoint sub-contractor. There is a rate or bid between Contractor & owner before starting any project.

RESOURCES FOR CONSTRUCTION MANAGEMENT Money

• Money the first and foremost recruitment for any project and it should be arranged before starting any construction project for smooth implementation of a project

• If the financial resources are insufficient than the project will not be completed within the limited scheduled time period Material

• Sufficient quantity of materials required for the completion of any project and should also be available at the site.

• Material required for project rest method before starting the project

• For example- bricks ,cement ,stones ,Timber ,water supply electrical fitting etc Money

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Machine

• Different type of machineries and equipments required for any construction work

• Although the cost of machines are high but reduces the high requirement of manpower

• For example mixers, tractors , cranes ,pumps,generators excavators etc.

Man power

• Successful completion of any project manpower is an important factor

• It may be bahut skilled and unskilled.

• Man power deals with engineers architects supervisors repair technicians skilled or unskilled labour,etc.

Chapter 2 - CONSTRUCTION PLANNING

- Objective of Construction Planning

- Work Breakdown Structure

- Construction Scheduling

- Classification of Construction Scheduling

- Methods of Construction Scheduling

- Bar Chart

- Bar Chart of a Residential Building

IMPORTANCE OF CONSTRUCTION PLANNING

Importance of construction planning are as follows.

- The work may be completed within the scheduled time.

- The work may be executed most economically.

- The work will be both qualitative & quantitative.

- There shall be minimum wastage during construction work.

- The work should be completed as per specification.

- There will be a minimum cost of maintainance of mechinery& equipment.

- There will be optimum use of available resources.

- Controlling of construction activities can be possible.

WORK BREAKDOWN STRUCTURE

- It is the preliminary diagram which showing the breaking down a project into sub-systems and each sub-systems into major components and discrete activities.

- In WBS, top-down approach to planning is adopted. Such an approach ensures that the total project is fully planned and all derivative plan contribute directly to the desired end objectives.

- WBS aids in the identification of objectives and allows the planner to see the total picture of the project.

- WBS is developed by considering the end objective and breaking it into smaller manageable units on the basis of size, duration and responsibility.
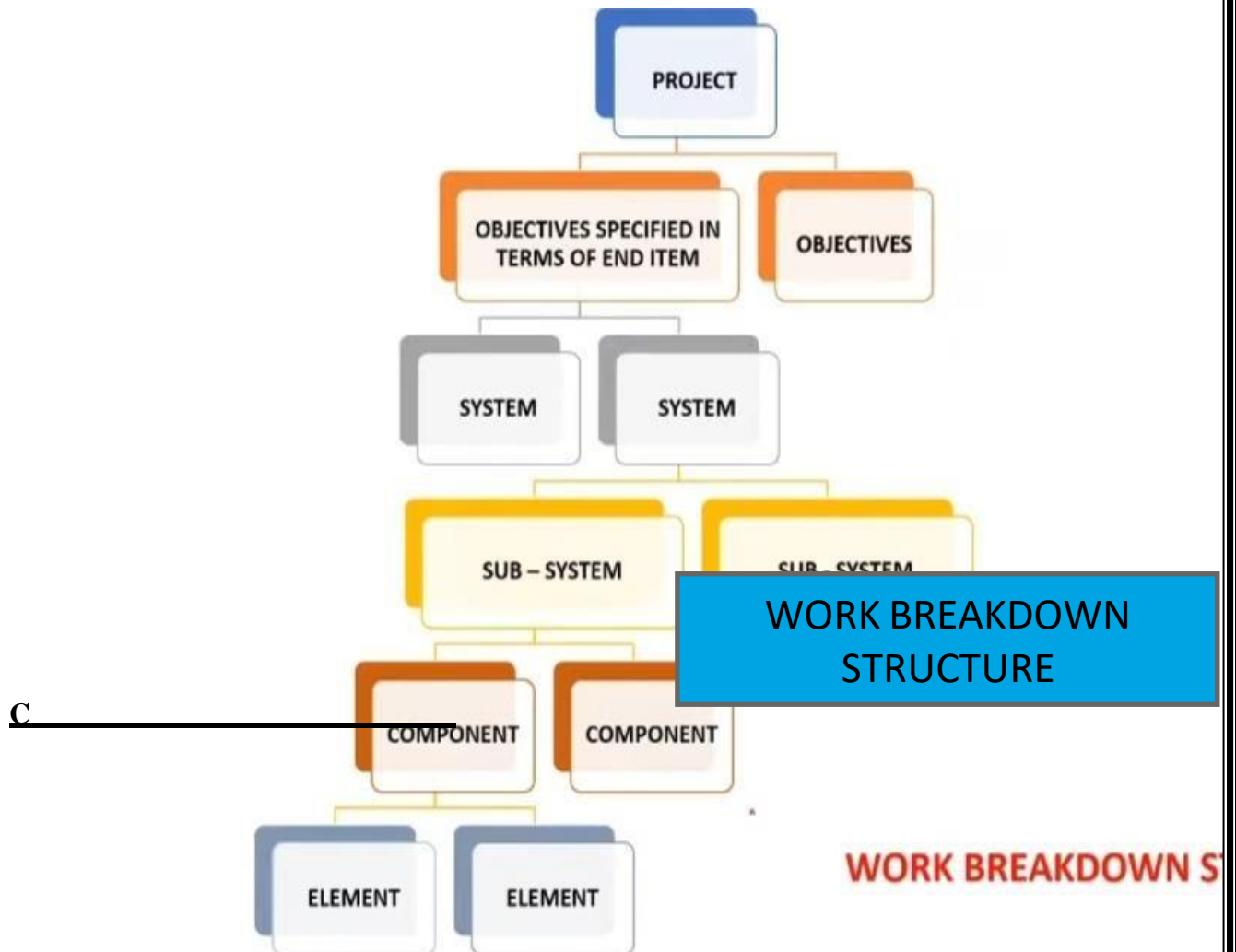
- In other words scheduling is the time table for executing each and every activity with its fixed starting and finishing date

CLASSIFICATION OF SCHEDULING

Schedules can be classified into various groups.such as;

- Material Schedule

- Labour Schedule

- Equipment Schedule

- Financial Scedule

MATERIAL SCEDULE

- this type of schedule is prepared for moving and storing of material in advance before starting of construction schedule acts as a guide for preparing materials schedule.

- This schedule is done to avoid delay in the execution of the work

- The materials should be delivered at site at least one week before its use

- The materials at site should not remain on used for long

- The materials stored at site long before its use it is likely to deteriorated in quality.

- for example cement made its strength by 50% if stored for 6 months and steel may be attacked by corrosion due to long storage at site.

LABOUR SCEDULE

- The labour schedule is prepared for deciding the actual number of skilled and unskilled labour which is required for the construction work

- With the help of this schedule required labour can be arranged in time

- It helps in reducing labour cost.

- Labour schedule is important as it is difficult and costly to arrange skilled labour as and when required.

EQUIPMENT SCHEDULE

- This type of schedule is prepared to decide the type and quantity of equipments as also on which date the equipment will be needed.So that they can be arranged when requirement.

- The aim of this schedule is to derive maximum advantage of the equipment when it is required and remove it from the site when the job is over.

FINANCIAL SCHEDULE

- Financial schedule is prepared to estimate the amount of money that owner or contractor has to spend as finance for the project work.

- In maximum construction project the owner will pay a stated percentage of the value to the contractor for the completion of work in each month. it is about 90% of the cost during each month.

METHOD OF SCHEDULING

Depending upon the size of the project scheduling is done by different methods.

Following are the methods of scheduling.

1. **Bar chart or gantt charts.**

2. **Network analysis (CPM ,PERT)**

BAR CHARTS

- Bar chart is a graphical representation of various activities their duration start and period of a project.

- This method was developed by Henry Gantt around 1900.

- They consists of 2 co-ordinate axis, i.e., horizontal and vertical.

- Horizontal axis is used to represent the time required for the completion of activity and vertical axis is used to represent the activities required for the completion of the project.

- The start and end point of bar represents the time of start and finished time of the activity hence the length of bar represents the duration of activity.

- The bar chart or gantt chart represents the schedule of a project also represent the actual progress

- We can also check the accuracy of work and can compare the actual progress of work with the schedule.
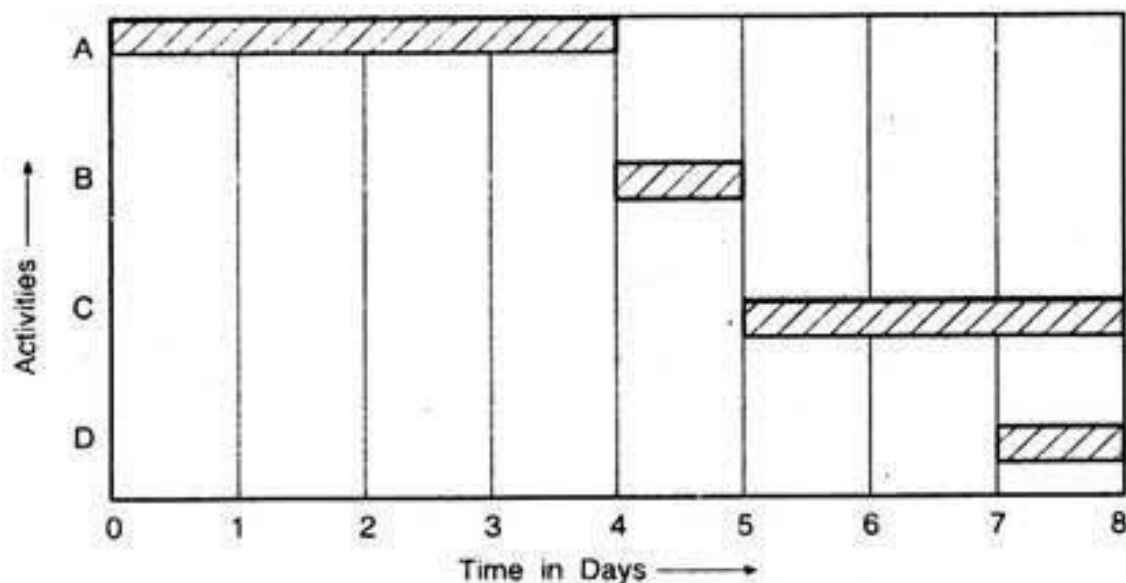
Bar chart of a residential building

- Example:

Activity ID	Activity Description	Dependency	Duration
A	Excavation	–	3
C	Foundation	A	3
D	Column	B, C	2
B	Moving the soil out	A	2
E	Wall	C, D	5
F	Roof	E, D	5

Solution
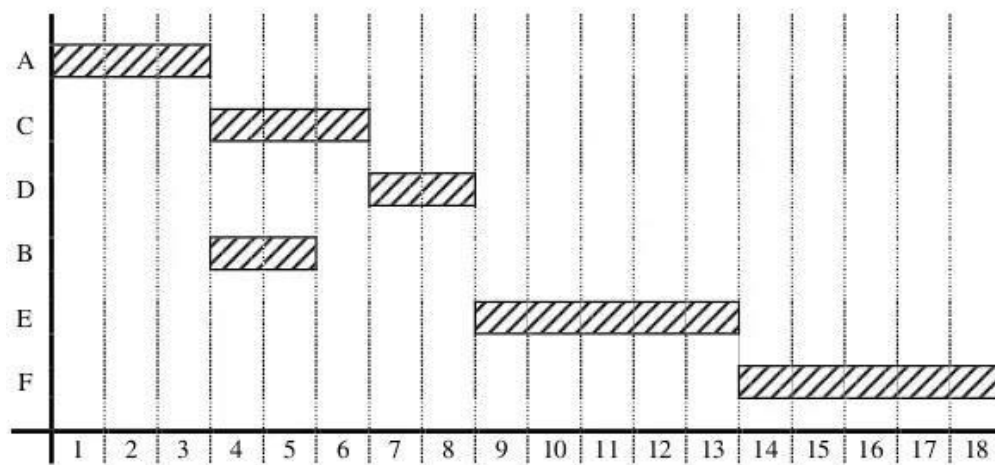
Advantages of Bar Chart

- Very Graphical

- Easy to understand

- Most widely used

Disadvantages of Bar Chart

- Difficult to Update.

- Difficult to find the Critical Path
- Difficult to setup and maintain a large project because it is essentially a manual graphical procedure.

Chapter – 3 Material And Store Management

Contents

• Introduction and Objective

• Classification of store-storage of stock

• Issue of materials(indent , invoice , bin card)

• Stores accounting procedures

• Inspection of stores

• Procedure of write off

INTRODUCTION

For the execution of different kinds of works in public work department,different types of material and equipments required.

it is necessary to maintain a store of various types of construction material at one or more place in a division so that the execution of work will be efficient.

Before start of any work material should be arranged well in time.

So store and material management is an integral function of different sections of the organisation.

It deals with the supply of material and other related activities and aims at minimum expenditure on materials.

Material management deals with the overall activities of materials such as type, amount ,movement, purchase ,location ,timing of various materials which are used in an organisation. So store and material management is an integral function of different sections of the organisation.

Objective of material management

• To select the right quality

• To meet the production requirements

• Selection of suppliers

• Limit the Wastages

 • Product enhancement

• Forecasting

• Standardization Process Objective of material management

• To select the right quality

• To meet the production requirements

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• Standardization Process

Classification of stores

Stores can be divided into four categories according to public work department.

1. Stock

2. Tool and plants

3. Road metals

4. Material charged directly to works.

Stock

The stock is the store which is required for general work and kept under suspense head and finally issued for the work.

The items which are in common use in the construction activity for the execution of different works are kept in stores.Such materials of general use such as cement, timber , bricks ,aggregates,steels, paints etc are kept in store are called as stock.

Reserve stock limit

The maximum amount of materials that can be kept in a stock in a division is fixed and is known as the reserve stock limit.

The limit is fixed by the Government keeping into consideration the normal requirements of stock in the division.

Note

'Stock' is a suspense head of account. When an item of stock is purchased, its cost is debited to the suspense head 'Stock'.

When the item is issued for use in a work, the cost of the item issued is credited to the suspense, head 'Stock' and debited to the final head of the work concerned.

Subhead of stocks

The various materials of similar nature grouped under different heads to facilitate the proper maintenance of stock account are known as sub-head of stock .

The following are some of various sub heads of stocks

1. Small stores (like nails, screws, hinges, bolts, etc.).

2. Building materials (like cement, aggregates, bricks, lime, etc.).

3. Timber (like deodar, chir, plywood, hardboard, etc.).

4. Metals (like mild steel bars, rolled steel sections etc)

5. Fuel (like kerosene, coal, etc.).

6. Painter's stores (like paints, varnishes, etc.).

7. House fittings (like bathroom fittings, pelmets, etc.).

8. Miscellaneous stores (like cord, wood preservatives, fertilisers, etc.)

9. Lands, kilns, etc. (like road metal quarries),

10. Manufacture (i.e. manufacture in Government workshops).

11. Storage (i.e. charges incurred on the storage of articles, such as rent of godowns, payment to work charged store establishment **Issue of materials**

➢ The store keeper can issue the materials to different departments upon the receipt of a withdrawal form with proper authority and it is called as material issue requisition form.
➢ Depending upon the nature and amount of material to be withdrawn from stores the material requisition is prepared in duplicate by the manager.
➢ Both the copies are sent to the store keeper who issues and records the materials distributed.

Indent and Invoice

- The material from the stores are procured by the process of indenting.
- Materials received from the stock on demand in a proper form called indent form.
- Indent form consist in triplicate of counter foil ,indent and invoice.
- The counter foil and indent part of the indent form filled by the Indent officer.
- Then this form with blank invoice sent to the issuing officer in charge of the stock.

• Invoice is an indent having list of articles actually should and giving price and particulars of the articles.

• Then the issuing officer corrects the indent and fills up the invoice. • Then the issuing officer sends it back to the indenting officer to sign the invoice and they return it to him as an acknowledgement.

Rules for preparing indent and invoice

• There should be description of unit of supply and quantity of material

• The cost of materials of the head of account should be specified

• The name of work should be given when the material is issued.

• Full details of department ,division and any other person for which the metro Brijesh should should be given.

Bin Card

• Bin Card is a card which maintains the details of quantities of each type of material received issued and on hand each day .

• The material and other items are kept in appropriate bins, drawers etc. The store keeper maintains the record on a Bin Card.

• A bin or shelf is attached to each bin card.

• Bin cards are made in duplicate

• One is attached to the bin and another is for the store keeper.

Procedure for store accounting

Final head

The cost of acquisition of stores is debited to the particular work for which they are required.This is known as final head of account. **Suspense head**

Suspense head includes the temporary booking of expenditure incurred for the purchasing of materials for the execution of work is debited to the final head of the expenditure is debited to the minor head i.e suspense expenditure.

a. The procedure for store accounting is done separately for various classes of stores suchas stock, tools and plants, road metals and other miscellaneous material.

b. When the stock is placed then the store is debited to suspense head .When the stock material is issued for the execution of a particular work then it is debited to the final head.

c. The supply of tools and plants in the division and its expenditure is debited to the minor head sometimes for general use special items of tools and plants are not required but for a specific work they are debited to that work.

d. For certain road the road metal is required for the construction its cost is debited to the estimate of that road construction and once the road metal is required for the maintenance of the road it is debited to the sub head under minor head.

e. Similarly for other materials if the materials are purchased for general requirement then the cost is debited to the suspense head.

f. The initial account of all receipt and issues is maintained by the section officer.

g. After closing the monthly account section officer forwards its to the sub divisional office.

Physical Verification And Inspection of stores

Necessity

Inspection of stores and its physical verification is essential for fulfillment of following

(i) To ensure the correctness of stock held by comparing them with the balance shown in the store ledger or bin cards.

(ii) To avoid shortage of materials in the stock.

(iii) To check losses in inventory due to pilferage, improper storage or misplacement, deterioration etc.

(iv) To correct and update store records.

(v) To calculate the values of the stock carried for the balance sheet and profit and loss account.

(vi) To calculate the rate of turn-over of an item.

(vii) To ensure maximum economy in stock carrying.

(viii) To effect insurance covers.

Method of Physical Stock Verification

• Annual physical Verification

• Perpetual Inventory and Continuous Stock Taking System.

Annual physical verification

The following procedure is adopted for carrying out the annual physical verification.

(i) By the end of the year, the stores are closed for a few days; no material etc. is issued to any project work/shop in the plant. In case it leads to plant shut down, the activities such as repair and over hauling of equipment and machineries are resorted to.

(ii) A team of stores inspectors or stores verifying officers physically check and count each and every item lying in the entire store. It is tallied with the quantities marked on bin cards and store ledgers.

(iii) Step (ii) above may lead to the formation of a list of surplus and short items. Damaged and obsolete items may also be traced and recorded.

(iv) Inspectors check a number of items every day as per a preplanned schedule and finish the complete work within a few days.

Advantages in the sense that all the items are checked at one time so there is no confusion about any item being left unchecked.

Perpetual inventory and continuous stock taking

Perpetual inventory and continuous stock taking system is a more appropriated method for large plant with huge inventories which records store balances after every receipt and issue and facilitates regular checking.

(i) Under this system, store items are checked continuously throughout the year; a number of items are counted daily or at frequent intervals and compared with the bin cards and stores ledger.

(ii) Discrepancies found if any, owing to in correct entries, breakage, pilferage, over issue, placing of items in the wrong bin etc. are investigated and corrected accordingly.

• This method is less costly

• In this method only few items are required to check every day as compared to annual physical verification.

Procedure for write off

• The articles of tools and plants get worn out by continuous use and become unserviceable. They can be written off only with the approval of the competent authority. A survey report of all the unserviceable articles is prepared on D.F.R. (P.W.)-15 giving full particulars of their value, date of purchase and reasons for their becoming unserviceable.

• The survey report is submitted to the competent authority for approval. As a general practice, the articles which are written off are destroyed in presence of a gazette officer.

• As regards the articles of stock, which get deteriorated, an estimate for the loss of stock is prepared. The tools and plants articles are written off after preparation of survey report.

• DFR- Document Filing and Retrieval Form

Example

Prepare a Write off in respect of following Articles of tools and plants.

• Name of the sub division- Killamaidan

• Name of the Division and Circle-Cuttack

• 10 nos, of metallic tapes 30m purchased on 6.5.2004 for Rs. 5000/ • 04 nos. of brass pad locks 7.5cm size purchased on 2.6.1999 for Rs. 1200/

• 1 time piece (Ajanta Make) purchased for rest house OMP square on 3.10.2006 400/ These articles became unserviceable through fair wear and tear.

CH-4 - CONSTRUCTION SITE MANAGEMENT

CONTENTS
• JOB LAYOUT
• REVIEW PLAN
• FACTORS AFFECTING SELECTION,DESIGN,LAYOUT OF CONSTRUCTION SITE
• LAYOUT OF EQUIPMENT
• LOCATION OF EQUIPMENT
• PRINCIPLES OF STORING MATERIALS AT SITE
• ORGANISING LABOUR AT SITE
• DIFFERENT JOB LAYOUTS

JOB LAYOUT
• Job layout is drawing the prepared plan of construction site by the site engineer in-charge of the project. The arrangements made at the construction site for different camps and the area around it, is known as job layout.

OR

• Job layout is a scaled diagram of the proposed construction site showing all the relevant features such as, Entry point , Exit point Storage areas of materials, Temporary services Contractor's site office Areas for keeping equipments such as mixers Bar bending area , Labour Housing etc.

Objective of preparing job layout

 Following are the objective of job layout.
• It saves time in delivering the construction materials at the site.
• The best method of working may be adopted.
• It helps to complete the work within the minimum use of equipments.
• The maximum output from labour and machines can be taken.
• It provides safety to the workers.
• It helps to avoid damage to the nearby properties due to construction work.
• It plans for the construction materials to be placed as near as possible to the work Following are the objective of job layout.
• It saves time in delivering the construction materials at the site.
• The best method of working may be adopted.
• It helps to complete the work within the minimum use of equipments.
• The maximum output from labour and machines can be taken.
• It provides safety to the workers.
• It helps to avoid damage to the nearby properties due to construction work.
• It plans for the construction materials to be placed as near as possible to the work

REVIEW PLAN

• Before preparing a job layout the details of different plans for the execution of the work should be studied carefully.
• Site plan
• Working drawing
• Specification

Site plan

The siteplan shows
• The boundaries of the site
• The adjacent area of the boundary of the construction site.
• Location of any existing building standing near site.
• Space left around the building to secure verification or free air condition.
• Space left around the building for cleaning and admission of light.
• Position of any natural drains, rivers, Wells located near the site.
• Any other information which are considered to be necessary Site plan Thesiteplan shows

Working drawing

• The working drawing consists of the building plans and other works to be constructed at the site. The working drawing include ;
• Floor plan of the building with covered area ,size of the room, opening of doors &
windows, structural members, staircase ,lifts Etc
• Elevation of all sides are shown.
• Indication of direction of North line in the plan of buildings.
• Indication of rejected persons beyond the permissible building line.
• Locating exactly of the essential services like Water closet ,sink , bath etc
• Showing sectional details drawing of footing thickness of world current slabs with their material.

Specifications

Specification indicates the details of the types and grade of the material to be used in construction work which was signed duly the authority or engineer and shall be available at the working place before start of any work.

Specification is an important document in the construction industry which helps the designer to come and get It is thought and ideas to the other construction team members.

Factors affecting selection ,design& layout at construction site

i. Nature of project
ii. Location of project
iii. Sevices
iv. Availability of material &equipments
v. Availability of manpower
vi. Medical facility
vii. Availability of space
viii.Other miscellaneous factors

I) Nature of the project

The nature of the project plays an important role in its layout process. The camp layout depends on the nature and types of project. For example the layout of camp for a highway construction project will differ from that of a building.

II) Location of project

Location of the project also plays an important role in job layout plans the location project should be properly chosen such that there will be no difficulty for any type of climatic situation and transportation. So transportation facility to the construction site is an important factor for job layout.

IV) Services

There should be proper service of water supply ,sanitation and electricity. If these services are not available then it will be badly affect the job layout.

V) Availablity of Material &Equipments

There should be sufficient availability of materials and equipments at the construction site. If the materials and equipments are not available locally then it will create problem in storage which will affect the shape of job layout.

VI) Medical facility

If the project is for a long time it is essential to have a field medical aid facility for the workers.

VI) Availability of man power

Man power is an important resource in any construction site. The arrangement of manpower at construction site should be made locally otherwise it will be a great difficulty for their shelter . So labour should be arranged locally.

VII) Availability of Space

If less space available at the construction site, then it will be difficult for job layout because the storage should have to be located nearest the regular supply of material & equipment.

Urgent availability of material may not possible as required.

VIII) Other miscellaneous factors

There should be availability of education facilities like schooling for the children of labours and staff ,daily necessities of life and other welfare facilities for the workers. If these facilities are not available then it will also tend to change the layout of the project.

Principles of storing materials at site

The materials should be stored in proper manner at the construction site.The following are the important principles which are to be considered for storing materials.

• Materials should be stored at the construction site so as to prevent mixing of foreign matter.

• Materials should be stored in such a manner as to protect it from any weathering agent like rain , sun and wind.

• Materials which are suspected to get fire easily should be prevented from fire hazards i.e the products like petrolium and explosives should be stored properly.

• Precast beams pieces of timber and slabs which are likely to be affected by the soil or support should be stored with properly adopted measures.

• Materials like cement bags which are easily affected by the contact of the moisture are to be stored with special precautions.

• The material regularly used are to be placed relatively nearer to the place of use.

• There should be proper arrangement of fire extinguisher and fire buckets wherever necessary for the safety measure.

Location and layout of equipments

Why equipments required?

As there is a increased cost of labour, the use of more & more mechanical equipments becomes necessary for construction work very often the available manpower is not sufficient for the completion of construction work with in stipulated time, so it is essential to use mechanical equipments along with the available manpower for the construction activity. So there should be a careful consideration for correct choosing at right equipment. For a

construction project to be completed with in the scheduled time economically, it is essential to choose the correct and well-operated equipments.

For the location of equipment following points are to be considered.

(i) Equipments should be nearer to the construction work.

(ii) Equipmements should be near to the materials.

(iii) The owned equipments may be provided near the entrance so that there will be no requirement of any aditional guard.

(iv) The hired equipments should be placed in suitable places and the vacant place may be left where it can be accomodated.

(v) The maintenance ,repairing and fuel filling of equipment should be arrenged at the construction site

(VI) There should be adequate space available for parking of the transport vehicles like trucks tractors etc.

(VII) temporary sheds should be provided to safeguard the costly equipments from any type of weather condition.

Organising labour at site

Organizing labour properly at the working site is an important responsibility of the supervisory staffs.

The labours are divided into different groups by the supervisor under the guidance of effective leader who has the quality to control the labours.

Proper way of organising of labours results the completion of work within the stipulated time period.

So it is very essential to organize the labours at the construction site.

For example Suppose 10 labours and one supervisor are put for beam casting the division of the labour may be.

(i) For bringing the aggregates, three labourers are put.

(ii) For mixing the ingredients one labour is put.

(iii) Four labourers are put on some other work.

(iv) For compaction purposes two labours are put.

There are some points which are to be considered while organising labour at construction

(i) Rehandling of material unnecessarily should be avoided.

(ii) Supply of material should be sufficient as per requirement of labour.

(iii) Labour supply should be uninterrupted.

(iv) The materials should be taken once for the whole day from the godown. It reduce the frequent movement of labour.

(v) There should be some permanent labours as it is economical.

(vi) Increasing and decreasing of labour should be done as per necessity.

(vii) To avoid wastage of time of labour, minimum facilities should be made available

(viii) Also to save wastage of time of labourers, drinking water facility should be made available at the site.

(ix) A record should be maintained about the progress of the labour.

(x) Record maintain once will help to compare the progress of work with the completion of work at right time at the site.

Preparation of job layout

• The construction plans, specifications, contract documents and other available material describing the job should be studied carefully in order to get the idea of the nature and extent of the work.

• A scaled drawing with a scale of 1 in 100 should be prepared showing the out line of the work or job to be constructed.

• Also the position of entry and exit points as well as the areas of temproray facilities should be marked on it.

• In job layout plan

Moreover following information should be collected from the above study.

• Area needed for accomodation: This area includes the area required for office. stores and residential accomodation for officers, staff and labour.

• Area required for machines, sheds, repair shops and workshops etc.

• Area for security and fine fighting facilities.

• Area required for construction work.

• Area for miscellaneous amenities such as canteen, toilets, dispensary etc.

• Length of period for which area may be available.

Ch – 5 Construction Organisation

CONSTRUCTION ORGANISATION

CHARACTERISTICS AND STRUCTURE OF ORGANISATION

IMPORTANCE OF ORGANISATION

TYPES OF ORGANISATION

LEADERSHIP AND ITS IMPORTANCE

STYLES OF LEADERSHIP

ORGANISATION

For any sucessfulbusiness , a sound organisation is highly essential.

Better the organisation the more is the achievement of the common business objectives.

Organisation is the foundation upon which the business management is dependent.

Organisation is a large group human association united together for the attainment of business objective.

Structure of an organisation

Organisation structure specifies the various job tasks and shows how job tasks are formally devided ;grouped;and co-ordinated.

Organisational structure covers the overall arrangement of an organisation.

It provides an appropriate framework for intra relationship and also indicates the hierarchy of authority and the reporting relationships.

So organisational structure coordinates the relationship between the various positions in the organisation.

There are some elements with which each member of the organisation should be similar with following are the main elements

Members of the organisation should understand about the well defined goal of the organisation.

They should be familiar with the rules ,regulation, policies, procedures of the organisation.

They should know with whom they have to work.

They should understand their duties and responsibilities towards the organisation.

They should understand the delegation of the authority and responsibility.

Importance of organization

- For a successful business, a sound organisation is highly important.

- Organisation enbles a large group of people working effectively together for a common goal.
- Only a sound and well designed organisation can maintain the co-ordination between the managgment and administration.
- Organisational diversification or expansion of organisation can only be possible by a well-planned &welldesigned organisation.
- Effective use of man power can also be possible by a sound organisation.
- A sound organisation makes an optimum use of rawmaterials and resources.
- Wastage and expenditure is less in a sound organisation.
- A sound organisation always stimulates the people for better, creative and innovative ideas.

Types of organisations

There are different types of organise structure have been developed and the following are more common
- Line or military organisation
- Functional organisation
- Line and staff organisation
- Matrix organisation.

Line or Military Organisation

- Line or military organisation is the simplest and earliest form of organisation.
- This system of organisation is based upon the scalar principle.
- According to this principal when the level of authorities arranged in the structure from the chief executive at the top to the workers at the bottom the system is known as scalar principle.
- In this line structure the authority and the responsibility flows directly from the manager to foremen and from foremen to workers.
- In other words authority and responsibility should flow directly in a line vertically from the highest level of the organisation to the lowest level of the organisation.

- Line organisation is also called as military administration or military organisation.
  Advantages
- Because of its simple logic and common sense appeal this type of organisation is most widely used.
- It makes use of specialist to give expert advice to workers .
- It also provides opportunities for promotion and career development.

  Disadvantages
- It is difficult to maintain discipline in the organisation
- It makes the complex industrial relationship
- It is difficult to know who is the 'boss' of whom.
  Line and Staff Organisation
- As the name suggest this type of organisation is the combination of the line and functional organisation.
- Here the line of authority remains the same as it does in the line organisation i.e authority
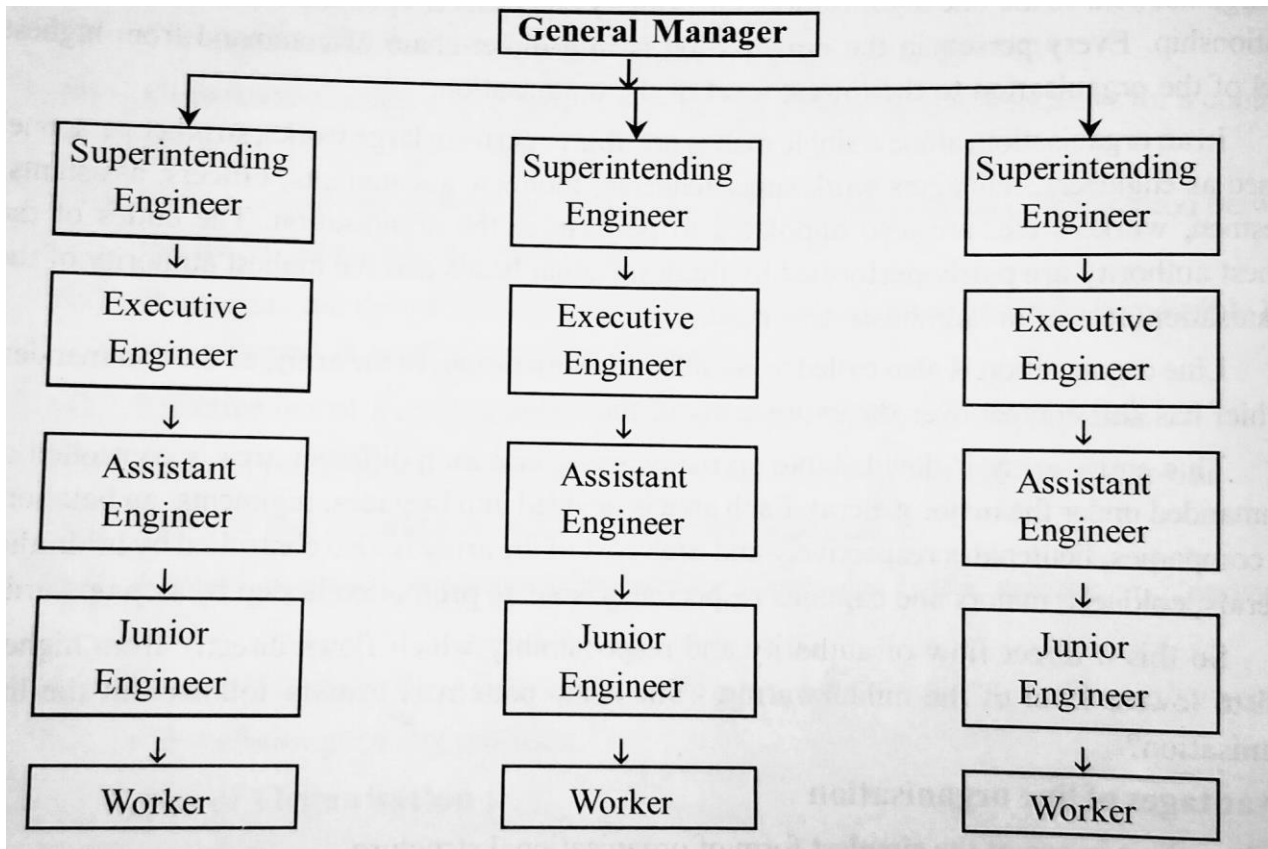
- flows from top to bottom and the line executive perform the major function while staff responsibilities carried out by the functional specialist with their knowledge and experiences.
- The staffs are constituted by the specialists in the organisation who are expert with long experience.

- In this system staffs are are divided into functional staff and line staff as shown in the figure below.
  Advantages
- Line and staff organisation possesses all the advantages of the line and functional organisation
- Discipline is maintained by the line authority.
- It improves quality of product.
- It enables availability a greater variety of jobs.
  Disadvantages
- Due to high salary of the staff executive the product cost will increase.
- There may develop jealousy between staff executives
- Line staffs do not have direct authority to enforce their decision and implement their ideas
  Matrix Organisation.
- Matrix structure is the combination of two departments. One is functional and another is product responsibilities. Product manager is also known as project manager.
- In this system every group of employees have two bosses one is there functional department manager and another is there project manager.
- The matrix structure allows for an efficient use of resources because teams include specialists from various departments.

(Line and Staff Organisation)

Advantages
- It ensures the effective utilisation of the services of the people with highly specialised skills.
- Communication improves by direct contact with different functional specialist.
- Employees can develop new skills
- Disadvantages
- decision making process may be slowed down.
- too much work can cause overload.
- Measuring employee performance might become difficult

LEADERSHIP
- Leadership is the ability of a manager to build up confidence among the subordinates
- Leadership is a process of influence in a group in specific setup circumstances which encourages workers to work willingly to achieve organisation objective.
- when a group of employees in a project have a common goal performance objective then some sort of leadership is essential there to build up confidence among the employees.

Importance of Leadership
➢ It leads the group to a higher level of performance.
➢ It implies a motive power to group efforts.
➢ Leadership acts as an way of influencing,inspiring , taking actions by the authority.
➢ Effective leadership creates a better understanding between the subordinates and the management.

Styles of Leadership
According to the attitude and behaviour patterns, leaders are classified as the following

1. Autocratic or authoritarian style
2. Laissez-faire or free-rein style
3. Democratic or participative style leader
4. Paternalistic style leader.

Autocratic or authoritarian style
- This type of leader is the absolute power with himself.
- Here the subordinates are completely obedient to the leader and the leader also centralised all the powers decision making in himself
- Here the subordinates have to follow the leader's orders,rules and regulation blindly or forcefully without any question.
- There is a threat of penalties and punishment to the subordinates in case of deviation.
- So this types of leader is not a real leader.
- Liassez-faire or free-rein style.
- This type of leadership maintains a good relationship between sinordinates and its leader because under these type of leadership the sinordinates allowed there maximum freedom.
- They are given the capacity to decide their policies and programs with their own style and take their independent decisions.
- But this type of leadership rarely exists because in these type of leadership the subordinates must be required to be competent ,sincere and self disciplined.
- Democratic or participative style.
- This type of leadership is the exactly middle position between two extremes of the autocratic and freerein style of leader.
- By this process of decision making the subordinates are encouraged to make suggestions while taking decisions.
- Subordinates are given chance to explore their potential in strength to complete the challenging responsibilities.
- It creates a friendly working atmosphere and reduces conflicts like strikes ,industrial unrest
- ,employees complaints etc
- Peternalistic style or functional style.
- Under these style of leadership the subordinates become dependent upon the leader.
- Here the sentiments and emotions are given more priority
- And and the leader looks after is subordinates like a father looks after his children and family.
- He is supposed to help, guide and protect his subordinates.

Chapter 6

Construction Labour & labour Ianagement

Introduction: Construction industry is one of the largest industry En india, where about four Crones of workers are employed and most of them are unskilled labourers. In general construction Labourers are Classified as unskilled anal skille of and semi skilled Persons. The labourers employed in construction industry are paid wages on daily basis as the construction work is temporary. Hence the job in Construction industry is also temporary and Workers have no job security. There fore, construc -tion labourers can easily be shifted from one Place to another.

6.1 Labour schedule мйmurt

A labour Schedule can be prepared from the construction Schedule and the objective of this Schedule is to decide, the number of skilled and unskilled labour required for the execution of different operation on different dates.

* With the help of this schedule required labour can be arranged well in time.

It is difficult and costly to arrange skilled labour as and when required It helps in reducing the labour cost.

6.2 Essential steps for Optimury labour output мини in in m + Labour output is a major concern for employers and it is desirable to have higher level of Productivity in any organization. For achieving this, a lot has to be done with the environmentat work Place and the work conditions along with a Series of factors that define the work couture. The employers have to implement wide spread changes in theire set up to improve the out put or

Productivity of their work

A few factores that help to improve the employee Productivity on Labour out put at the work place ane!

1. Accountability.

Every employee needs to be well aware that he is accountability for this actions and decisions and he can neither pass the bulk or pass the

blame to someone else. of This will help him to work more meticulously. Hake cautions rather than reckless decision, and not take advantage of his place, position or relationship with his superions

2. Follow up

→ Every target on milestone set needs to be followed up as well to see if the progress is sufficient and if not, whether any interim measures can be taken before it is too late to salvage a situation.

→It also keeps the employee on track ensuring no Fod consistent efford there time of the Project.

3. Management without micromanagement of course, the pool of employees does need to be Managed, provided direction and given assistance.

But side by side.they must also be trusted, given freedom to operate in their own style and adopt measures which they think are the best to deliver results4. Encourage Motivate reward and recognize

~w 2

The employers Must ensure that on this part he always has the words of encouragement for his

It helps them move forward and do even better and Make the worker feel happy. Innovative way of motivating them spurs them even more. For EX: holidays or conferences paid

for by the company have been found to motivate

employees immensely.

5. Reach out to employees by seeking them

→ Every employee loves to feel he has the ears of the management who will recognize him and Listen to what he says. of me

Display of interpersonal skills in which the boss appears humane and one of them rather than a larger than life, distant figure, helps to have employees warm up to him and feel happy working of him.

6. Demand realistic targets

→ •Employers need to set realistic goals that are within the limits of achievement. → while an aggressive employer may want his people

to out streetch themselves to achieve fare fatched goals, it may also burn them out."

7. Teary Work

→ Team work always helps in increasing workplace Productivity since there is more input in the from of More ideas and minds at work.→ working alone is not always the happiest situation

either especially in the fields.

8. Ensure that people enjoy their work The best performing employee, is the happy employee and the employer has to find ways of Making his people happy.

→ Besides working conditions and the work culture Implemented, he has to devise way of making the work seem challenging and interesting rather than mundane and boring.

9. Break the monotony

and

rotate

→ while employers assign tasks according to an employee's core competence, even with the task they are best at, can make an employee bored and

this work Seery monotonous.

This Monotony can be broken with rotation and giving people new tasks and exposure to other divisions.

10. Courses and Emprovement options.

→ Employees are delighted when they can enhance their skills and get additional larning opportunities Sponsored by the employer. om

This help them learn feel indebted for the? money being spent on them, which also adds to their resume and are obliged to perform better by applying all the knowledge gained in these Courses11. Spend less time on meetings and more an action →The current trend to have more meetings and discussion more time in rather than spending more time in working to achieve results leads to Precious Productive time loss.

→ Meetings for reviews and Sharing of ideas can be Limited and kept short. Employees should have more time to show results.

12. Tool and equipments to raise Productivity.

The workplace should have the best of Machinery devices and equipments that yield error free results in the minimum possible time

→ Efficient electronic equipment with no connectivity ssues and breakdowns will in saving Precious time. help

→ They should take the place of paper work

and yield fast results was an e 6.3 Labour Characteristics!

1. Labour is original and indispensable factor of Production. 2. Labour is an active factor of Production

3. Labour is Perishable than any other ARTH Commodity 4. Labour can not be separated from the labourer

5. Labour is less

6. Labour supply is inelastic

7. A Labour sell has labour and not himself. 8. Laboure has weak barganizing power.

9. Labour is both the beginning and the end of Production.10. Efficiency of labour differs.

1. Labour can not be engaged continuosly in production Like Machine

19. Labour Creates capital: labour is more important in the process of production than capital because Capital is the result of the working of labour.

13. Tt is difficult to calculate the cost of production

of labour

14. Labour has not tangible form: Labour has not reserve price. Labourer can be seen, can be touched but labour can not be seen.

6.4. Wages Payment To Labour:

→ Payment made to labour is generally referred to as wages. It can be time-rated on piece rated. It can be nate per hour, per day, per week, per Month Or per year.

This is the renumeration paid to to workers for the actital work they do.

The wages can be paid to ordinary skilled, unskilled Or semi-skilled workers as daily basis, on week weekly basis.

The wages are both monetary and non-monetary.

→ The monetary wages are money paid to workers as wage. But non-Monetary payment may are known as

fringe benefits.

6.4.1 Types of wages

wages are generally of two types.

iy Time wages.

how to desig

iy Real wages Hos at 40 cortom
Py Time wages:

When payment of wages made to labour is in the form of money for the work done on the basis of Per hour, Per day, per week, per month ore per year it is often called as time wages.

iiyReat wages:

After Satisfying the basic needs of a worker and for improving the standard or uving of a worker wages given in the form of lawny and comfort On extra security, is often known as real wage.

The real wages sepeify the amount of goods and. Services that the money wages will buy.
6.4.2 Method of wage

Wages can be calculated on the basis of the Out Put Errespective of the time taken in completing

14. → Efficiency may bea factor which varies from individual to individual.

The efficient worker may creat more out put than Other so wages can be calculated on the basis of the work irrespective of the time.

Also a good wage payment system establishes a good relationship between worker and employeer. The payment of wages can be classified into two method.

iy Time on day rate system to

iiy piece work on piece rate system. Вран 1. Time on day rate systems

In this Method of wage payment, the worker is Paid a fixed renumeration as per his unit of time which can be rate per hour, perday, per week, Pear month or per year.This is one of the oldest method of wage. payment adopted in india.

As in this method, workers don't get extra benefits except their weekly leave, they have no special intrest to work hard for the Optimum Profit of the organization. Merits!

iy This Method is suitable work can not be. Masured directly.

iiy By this method of wage payment, worker f ensures regular employment and greater security. of service.

y Here Skilled, unskilled and semi. Skilled all The workers get the same wages of one class.

The calculation of wage can be done easily by Method. where measurement of output is not feasible, this

Method is specially useful.

viy As workers have no tendency to show increased Out Put the quality of work is good. ئانون

vii) Also this method can be understood by all class of workers easily. con of vigane

Demerits!

iy As the workers don't get entra benefits encept their weekly leave, they have no such interest to work hand for the organization.

iiy There is no inspiration of competition among the workers and hence there is no chance of extra Profet.

ily By this method, a skill employee becomes intrest -less to Produce more than the unskilled workers.ivy A regular Supervision is required for this the"" work to extract work from the labour. vy The workers are assured of their wages to theire

out put is low.

vix cost control can't be ensured effectively due to varying Production. 27 Piece work one piece rate system.

→ In this system according to the worker's output

their payment is decided. Of course payment is made

at the agreed rate. → In this method, an efficient worker can earn Money by increasing his out put.

→ Here Payment &&Purely based on production on output of workers → Payment is decided at the actual quantur of work

done by the work.

Advantages to

12 23 bon

ix Suitable incentives are given to efficient workers in proportion of their output or production.

iiy There exists a healthy. atmosphere among the employer and employees.

ii) Higher wages are given to worker with higher Output on Production na R Prok 200000

ivy less supervision is required. 14, 2

vy In this system, a good worker can make more 202.00 money by increasing his out Put.

vix By this Method, unefficient and unskilled yees are pointed out. employeesviiy As effort of workers increases, the over all Production of the organization increases similarly there are some demerits of piece-work system of wage payment as given below.

Disadvantage:

by workers are not careful about the quality of work. They have to any how increase the out put of the Organization.

ily over time work causes sickness to health. iiiy Sometimes no work no pay situation arises 107 because during the period of sickness or absence there will be no payment as the output will be there.

ivy It causes a competitive f jealous atmosphere. among the workers of organization.

65 Motivation

6.5

Labour incentives

→ Labour incentives refer to those incentives that Supplement a salary and are given to the employees of a company fore their excellent Performance

Most often that not incentives are what attract the employees to keep working Company and go an extra mile to achieve something they are set to do.

6.6 Motivation:

→ Motivation is an important factor which encourages persons to gives their performance and help them in reaching the enterprise goals. Motivation is one of the most important factor

affecting human behaviour.→It helps the individual towards the fulfilment of

durable objectives. 14 Ex o complene force that is responsible for Starting and keeping a person at work in an it

Organization.

→ Motivation is something that mobilises a person to to work.

Defination

Motivation is an inspiration that simplers and Person to expand energy to achieve a goal on Areward.

→ Motivation is acts as a driving force by conich 1000 521 the human being achieve their goal.

→ In other words Motivation is the stimulation of emotion or desire and an inner state that activates On directs the behaviour towards achiveing the goals.

6.6.1 Classification of Motiverne masque

When a Manager wants to get more this Sub-Ordinates Motivate them for 230 work from then he will have to improving their mance.

This Motivation may be in the form of an anyalim on incentive or bonus no 13th B mo? NS150 OF

There are two types of motivations.

Ey Internal Motivation 201

External Motivation. Ant ofi Internal Motivation

Internal Motivation Motivates people internally and 24 meters to motivation by interest on enjoyment.in doing the task itself.

Internal Motivation emists within the individual rather than any external intuence. ANTS → Need to get an accomplishment of good job, and

the illusion of self- determination and freedom

are the examples of the internal Motivation.

ify External Motivation.

Enternal Motivation comes from outside of the individual.

Common external Motivations are rewards Like money. grades, pay, incentives, threat of punishmement or praise, in

ALSO Motivation can broadly be classified into two broad types as follows: ay Positive Motivation.

when the employees are offered the incentives they try to improve their performance wil willingly.

Positive Motivation or incentive motivation is, ww bask based on reword. - مہلوان

The incentives may be in the form of More pay, Promotion, recognition of Job etcrop get born work, more responsible

Positive Motivation is achieved by the Co-operation of the employees to do andain and they have a feeling of happiness. avit of an nomepinnalby Negative Motivation.

Negative motivation is based on force or fear. → Fean causes the employees to do certain job.

→ It they do not do accordingly then they may be Punished with demotions on lay-offs.

This types of motivation causes anger and funestration because the employees do not work willingly rather they want to avoid the Punishment. 40

6.6.2 Different Approaches to motivation

The Motivation differ from time to time, place, to place, situation to situation and person to Person.

→ so it is difficult to set a specific theory which will be universally accepted. WHOM OLIA

,

w's Need Hierachy theory ish bond f

Maslow's

olay Herzberg's two-factor theory. By Alden for's ERG Ttheoryhemal to try

1. Maslow's Need Hierachy theory This theory includes the hie Abraham Maslow. hierarchy of need by

+ Maslow's theory is one of the most widlydiscursed theories of motivation.

→ Motivation is influenced by the needs of a Person A.H. Maslow an American social scientists has developed the hierarchy of needs consisting of five hierachic Classes

Maslow categoried human needs into five categories

97
ty Basic Physiological Needs

These needs are most essential for the survival and Maintenance of human life,

These needs include satisfaction of the needs of hunger, and shelter, drinking water, clothing, rest," etc. اده

Self actualisation

Top (self-fulfilment) Esteem need/ Ego needs (Prestige, status, self-respect Social needs.

(Affection, friendship, belongings

Safety and security needs

(Protection, onder, stability) Basic Physiological needs. (food, water, air, Shelter)

iix Safety Needs:

Once Physiological needs are stais tied, the human being want the assurance of maintaining a given economic level. 197

These are the needs to be free from Physical dangers, and fear of jobs Property, Sheller, etc. 10

→ Every person would like to be free from worries like loss of job, sickness, old age pension, physical safety like accident and fire.

Social Needs: Once the individual esSatisfid with social needs, they are concerned about the next level.

Being a social being people belong to be accepted all

by others voskedalTherefore the man is intrested in conversation. Sociability, exchange of feelings and grievances,

Companionship and belongingness.

ivy Esteem on Ego Needs! 7863 → These needs are concerned with

self-respect. self-confidence, feeling of being unique recognition etc. Satisfaction of these needs bring confidence, power,

control and prestige achievment, independence

competence, knowledge & → The individual have to learn on acquire these only through his intelligence and hard work.

etc.

&Hanxberg's Two factor Theory:

vy self-actualisation self-actualisation need is the need for self fulfillment of wants of a person considered to be mission of his life.

+ self-fulfillment is the highest need in

Maslow's

hierarchy and then to These needs which help an individual to devlop his potentialities.

self-fulfillment needs gives satisfaction to the person concerned and gives a tendency of capability of doing of self-development

2. Herzberg two-factore

or Theory

→ This theory was devloped by frederick in 1959. y's two factor theory is also known as → Herzberg's Motivation-hygien theory of Motivation.

→ Herzberg and his associates concluded a of need satisfaction of 200 engineers and accountants in an Organisation. StudyThese person were asked to describe a few Previous job experiances in which they felt exceptionally good or enceptionally bad about the jobs.

The satisfaction of some need may not have Positive effect on motivation but their non-satisfac -tion on may act as a negative factor.

These factors operate to build strong motivation and high job satisfaction and their absence affects both satisfaction and Motivation.

ix Hygiene factor: These factor are responsible for reasonable level of satisfaction and are called Maintenance or hygiene factor.

→ The hygiene factor Ex just like hygiene; the Presence will not make the employee healthy but its absence causes a deterioration of health.

There are factors concerned with the company Policy and administration, teennical supervision, winter personal relations with Supervisor, inter personal relations with peers, inter-personal relations with Subordinates, Salary, Job security, Personal Life, work relations with subordinates and status.

These are the maintenance factor conich are necessary to maintain a reasonable level of Satisfaction.

→ These factor are not responsible for growth of motivation in workers but the absence of these factors Creates problem ostaliy Motivation factor

These factors create high motivation and job Satisfaction in their Presence.

The absence of these factors do not cause.

dissatisfaction. → According to Herzbeng, there are six factor

which give positive satisfaction. → These six factors are recognition, advancement. work itself. possibilites of personal growth,

achievment and responsibility.

It is essential to increase these factor for increasing the motivation of employees.

3. ERG Theory:

→ This theory was introduced by Aldenter. T Aldenten's ERG theory is the expanded form

of Maslow's hierarchy of need theory and

Herzbery's two factor theory of motivation. He found some over- lapping between Physiollo - gical need security need and social needs.

He classified the various needs into three main categoriesitten your attoTomogr

iy Existence needs.

y Relatedness needs atom all wa

Growth needs.

iy Existence needs:

infotem of gove

This need of ERG Theory includes both Physiological and safely needs of an individual

in Maslow's Model.These needs include the basic Servival needs of human beings like food, clothing, Shelter and drinking water.

These are the Primary to fulfill this. try to needs, human being

ily Relatedness Needs This need of ERG theory is the combination of the social need and esteem need of Maslow's

Model.

These needs are the emotional needs of the human being for Love, affection, warmth and friendship. These needs give human being ego sats faction

so combining these two needs of maslow relatedness need is derived. iliGroth Needs; plus no 29t

These needs are same as the Maslow's self

actualisation needs.

These needs satisfy the human being for nis personal development and achivement.

When the individual wants to do something Challenging, the samse of achivement gives him a satisfaction.

This need satisfies all desires of the individu -al to increase and develop his potential."
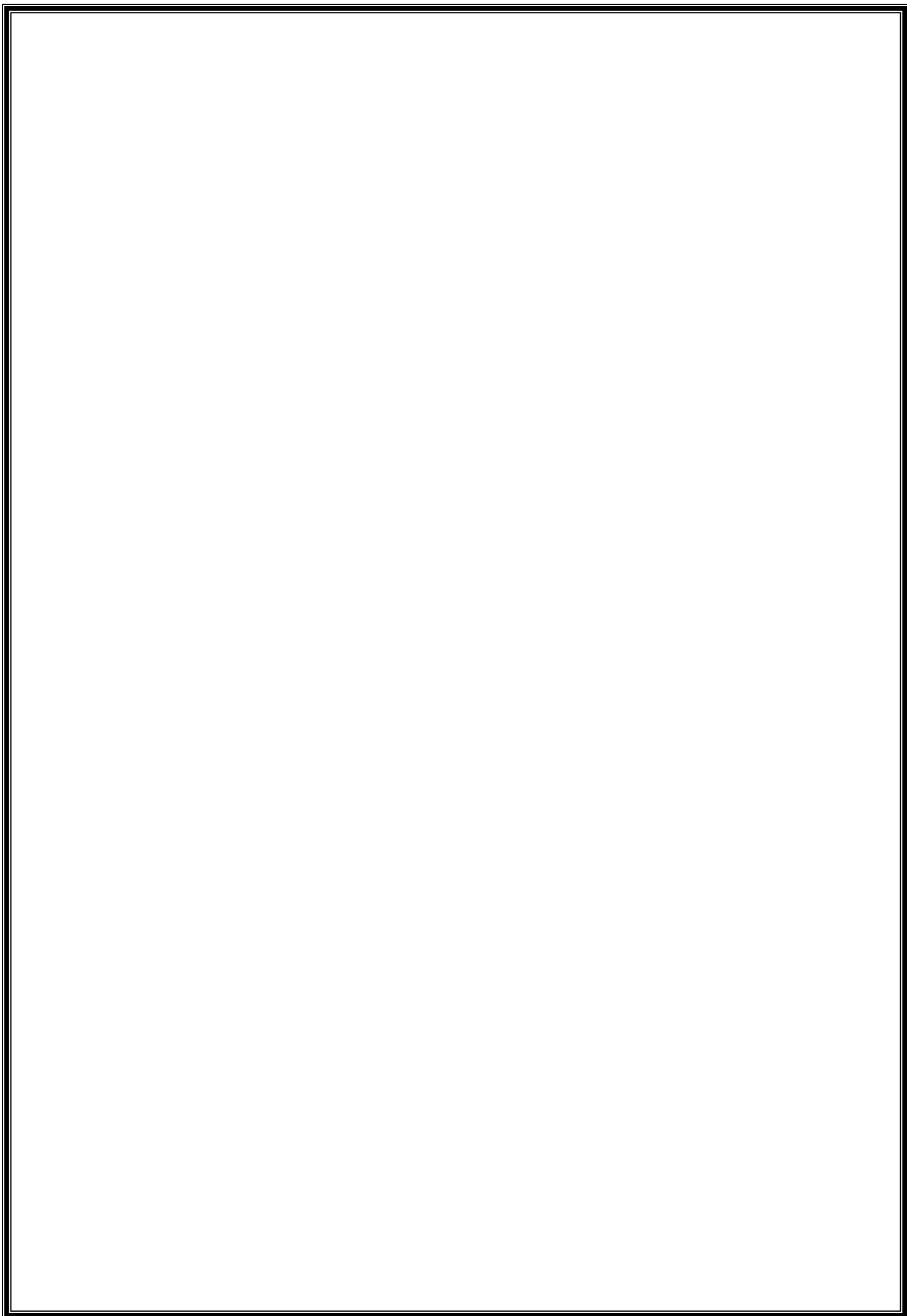
<u>CHAPTER – 7 EQUIPMENT MANAGEMENT</u>

(The cost of equipment in a project varies from 10 to 3% of the total cost on ne project, depending upon the extent of mechanization. In modern fully mechanized projects, the cost of equipment may vary in the range of 25% to 30%.) However, proper planning, selection, procurement, installation, operation, maintenance and equipment replacement policy plays an important role in equipment management for the successful completion of the project. With the growing use of the machinery, it has become necessary for the construction engineers to be thoroughly familiar with the construction application and up keep of the wide range of the modern equipment.

(The aim of the equipment management is to reduce down time, achieve optimum equipment utilization and increase production at minimum cost. There is a need for a rational planning, proper selection and judicious deployment of equipments so as to achieve optimum utilization. Of course, the equipment management integrates and continuously interacts with human, technical, financial and production system in order to achieve top efficiency and cost effectiveness.

7.1 PREPARING EQUIPMENT SCHEDULE

The equipment use schedule has to be prepared before the start of the project in order to decide the type, number and dates on which a particular equipment will be needed so that it is arranged well in advance and brought to the site as and when required. The aim of this schedule is to derive maximum advantage of the equipment when at site and remove it from the site when its job is over. This is most likely to result in economy.

After the purchase of the equipment for a particular project, the owner gets a mark punched or painted on the equipment or machine to identify it from others. The owner should have full knowledge of the equipment purchased by him. Usually following information is sent to the owner:

1. Cost of the equipments.
2. Efficiency of each equipment or machine.
3. Record of their repair.
4. Details of expenditure on repairs. Duration of effective use of the equipment.
5. Details of fuel consumption by the equipment.
6. Details of servicing of the equipment.

With the above information, the efficiency and working capacity of the equipment can be compared with the figures supplied by its manufacturers. This schedule is very useful at the use of purchase of new equipments.

SELECTION OF EQUIPMENTS

Identification and selection of proper equipment is of paramount importance for the speedy and economical completion of a construction project. The problem of selection of a particular equipment or identification of different alternative equipments is a difficult task because of availability of variety of equipments in the market by different manufacturers. For proper selection of equipments considerable field experience in their operation and maintenance is required. Previous records of operation, maintenance and actual output under comparable conditions of similar projects comes to great help in deciding the choice of equipments.

Basically, following two aspects are considered for the selection of construction equipments in a project. While the first aspect deals with the type, size and other particulars of the equipment, the second aspect decides wheather it is to be purchased, hired or to be procured under hire-cum-purchase arrangement. But in all cases, the following factors must be taken into account before having a final choice.

(i) Suitability for job condition

The equipment selected or identified must satisfy the requirements of work, climatic and working conditions

(ii) Size of the equipment

The size of the equipment should be such as to be compatible with other matching units. If the chosen equipment is of larger size, it is likely to remain idle for most of the time or shall work on part loads leading to rise in the cost of production / work. On the other hand, if the equipment selected is of smaller size than that is required, it may not be compatible for matching equipments, in which case, the other equipments will have to either remain idle or to be allowed to work on part loads, which will again be uneconomical.

(iii) Standardisation

It is desirable to have the same type and size of the equipment in a project which will ensure lesser spare parts reserve, better interchangeability of parts, easy understanding of operators and efficient maintenance and repair as the mechanics become adept by handling the same type of equipment.

(iv) Ease of availability in the market

The equipment selected should be easily available in the market but side by side it is also to be ensured that the equipment is of reputed company and likely to be continued to be manufactured in future also. This is essential for future standardization and ensuring spare parts supply as well as for disposing off such equipment after completion of the project.

(v) Availability of spare parts

The availability of spare parts at reasonable price throughout the working life of the equipment is all the more important while selecting a particular type or make of the equipment. It is desirable that the down time of the equipment for want of spare parts should not be more. This is very important in case of imported equipments.

(vi) <u>Versatility of equipments</u>

There are certain type of equipments that are not fully utilized for a particular function. In that case, they should be capable of performing more than one function so that it is not layedidle and has multi-purpose use.

(vii) <u>Availability of know how</u>

The equipment selected should be capable of being handled satisfactorily by the available operators and mechanics. Therefore, a sophisticated equipment that gives excellent performance but difficult to handle and maintain should be avoided.

(viii) <u>Possibility of use in future projects</u>

When selecting an equipment that completes only a part of their useful life in a project, its use in future projects should be kept in view before it becomes obsolate.

(ix) <u>Economic aspect</u>

While selecting any equipment, it should be ensured that the cost of unit production is minimum.

(x) <u>Reliability and support service</u>

The equipment selected for the project must be reliable one. In addition, support service should be available in the area of project where the equipment is to be used. After sales service should be a major criteria for the selection of equipments.

(xi) <u>Operating equipment</u>

The equipment selected should be easy for operation and maintenance, user friendly to the and should have lesser fuel consumption.

Satisfactory past performance While procuring an equipment of new make and model, it is desirable to enquire about its cory performance from other users, who are using the make and model for quite some

Besides these, the other points that are to be taken care of are reputation of the acturer, warranty or guarantee offered, use of standard components in the equipment adequacy of drive mechanism or power of the prime mover. use of However, versatility of the equipment should be given due priority. Multipurpose achine promises extra profit due to following reasons. Allows one machine to do the job of several machines and thus cutting into ownership and operating costs associated with additional plant and labour. It increases the utilization enabling a machine to earn money when it might otherwise be idle.(increases the utilization enabling a machine to earn money when it might otherwisehe idle. Generally, there are provisions of fitting and changing the attachments of such machines with the help of couplers. Thus, a balance between reliability, investment cost and operating

cost should be worked out, since a policy of selecting only the lowest priced equipment may lead to overall higher costs.

OWNING AND OPERATING COSTS

The principles of engineering economics are made use of while selecting and planning of finances for the procurement of construction equipments. Economics of construction equipment mainly deals with the study of working of the equipment and computation of the unit cost of production, which comprises of the following components:

a) **Owning cost by Operating cost**

The cost of possession of an equipment is called the cost of owning while the cost of fael and lubricants for running the equipment is known as operating cost. These two combined so gather when estimated on hourly basis represent the amount by which an equipment should be hired. Of course, it is exclusive of labour cost. There are several methods of determining the probable cost of owning and operating of a construction equipment but there is no guarantee that similar equipment will have similar cost if used under different conditions. Past records of use of equipment may often serve as a guide but it must be adjusted as per the prevailing situation to arrive at the realistic value.

The following factors affect the owning and operating cost.

1. Initial cost of the equipment and its delivery.

2. Severity of the conditions under which it is used.

3. No of hours it is used per year.

4. The care with which it is maintained and repaired.

5. The salvage value of the equipment after its useful period.

6. Useful life or service period of the equipment in years.

When detailed cost records based on past performance of an equipment is not available, it is analyzed from the first principles. The following costs constitute the cost of owning and operating.

1. Investment cost (including interest, insurance and storage)

2. Maintenance and repair cost

3. Depreciation cost

4. Fuel or energy consumption cost

5. Cost of lubrication

1. **Investment cost**

The owner has to invest money in order to own an equipment. This is a kind of fixed cost and is incurred, wheather the equipment is used or not. Investment cost comprises of the following:

a) Interest on the money invested in the procurement of the equipment

b) Various taxes on the equipment

c) Insurance expenses

d) The cost of storage

The chance of earning the interest on the amount spent on the purchase of equipment is a loss to the owner. In addition to this, all type of taxes assessed against the equipment, insurance premium and storage charges have to be paid. Therefore, it is desirable that these costs must be realized during the life of the equipment. Sometimes, a fixed percentage of the original cost of the equipment is charged towards the cost of investment each year equally which is much higher than the actual, because the insurance, taxes etc. are usually paid on the depreciated value of the equipment and the amount of interest charged also should be based on the book value of the equipment instead of its original cost. So, the owner must deduct from its earnings an amount equal to the annual cost of depreciation to get the book value of the equipment and the realistic value of investment cost of a particular year will be some percentage of the book value of that year. But, this process being cumbersome, the average value of the equipment is used in determining the annual cost of the investment. Generally, annual investment cost 10 to 15% of the average annual cost of the equipment.

Maintenance and repair cost

The annual cost of maintenance and repair is based on the experience obtained from the operation of the equipment under average conditions. The actual cost varies with the conditions under which it is used and the care with which it is handled. Of course, it varies with the type and quality of equipment. The annual cost of maintenance and repair may be expressed as a percentage of the annual cost of depreciation or it may be expressed independent of depreciation. But in any case, it should be sufficient to meet all maintenance work including cleaning. washing, checking of component units, instruments, ropes, adjustment of component units as well as routine and major repairs. Normally, annual repair and maintenance cost- 50 to 100% of annual depreciation but 100% of the fair value.

Depreciation cost

Depreciation is the loss in value of the equipment resulting from wear and tear or obsolescence. The owner of the equipment must recover the loss in value of the equipment during its useful life by way of depreciation. There are different methods of determining the cost of depreciation.

Generally, standard equipments are preferred in construction industries. However, special equipments also may be considered provided the economic analysis justifies its selection. So, for all equipments used in the construction, if the equipments do not suffer from the danger of obsolescence, the annual depreciation cost of the equipment may be obtained by Initial value-Salvage value

Annual depreciation Useful life of the equipment (in years) Thus, at the end of the useful life, the value obtained because of the disposed of the equipment may be in terms of salvage value or scarp value. 4. Fuel or energy consumption cost

Construction equipments require fuel in the form of gasoline oil, diesel, electrical energy and lubricating oil, which is considered as operating cost. Although, the amounts consumed depends upon the type of equipment, its rated horse power, location, temperature 615erie pressure and the conditions under which it is used as well as other factors but the working condition is the most significant to estimate the realistic consumption of the fuel per hour. These working conditions of the engine are defined by engine factor i.e. extent to which the engine will operate at full power all the time and the time factor i.e. that the actual time that the engine will operate in one hour.

Then operating factor Engine factor x Time factor.

A common value of operating factor for an engine during a construction equipment may be taken as 0.6 if detailed information regarding its engine factor and time factor are not available.

Cost of lubrication

An engine requires lubricating oil for its smooth functioning and getting more output minimum loss on account of frictional force in the machine. The quantity depends upon the size of the engine, the capacity of the crank case, the condition of piston rings and the number of hours between oil changes. However, it is common practice to change the oil every 100 to 200hrs. The empirical formulae may be used to estimate the quantity of lubricating oil and the factor of 0.6 may be assumed in those formulae when sufficient data is not available for the purpose.

7.4 INSPECTION AND TESTING OF EQUIPMENT

Inspection is taken to mean observation of work environment, work practices, equipment used, work posture or reported hazard which may be generic or it may be specific to assess a particular risk, task or part of occupational health and safety management systems. Testing means use of standardised tests to check the equipment, plant operation, process control, and effectiveness.

The purpose of inspection of an equipment is to identify whether the equipment can be operated, adjusted and maintained safely with any deterioration detected and remedied before it results in the health and safety risk. Not all work equipments need formal inspection to ensure safety and, in many cases, a quick visual check before use may be sufficient.

However, timely inspection is necessary for any equipment where significant risks to health and safety may arise from incorrect installation, reinstallation, deteroration or any other circumstances. Usually, the need for inspection and inspection frequencies are determined through risk assessment.

the need for inspection and inspection frequencies are determined through risk assessment.

The equipment should be inspected if the risk assesment identifies any significant risk (for example, of major injury) to operators and others from the equipment's installation use. The result of the inspection should be recorded and such records must be preserved until the next inspection of the equipment. The records need not be necessarily in writting but if kept in any other from (e.g. on a computer), these should be held securely and made available on request by any enforcing authority.

Equipment that demands inspection should not be used, unless required inspection has taken place. When equipment is transfered or returned, it should be accompanied by the physical evidence of last inspection such as an inspection report or for small equipments, some from of tagging, colour coding or lebelling system.

Where the safety of the equipment depends on the installation conditions, it should be invariably inspected after installation and before the first use and after reassembly at the new site or location at suitable internals, where the equipment may be exposed to conditions causing deterioration liable to result in dangerous situations.

The scope of inspection will depend on the type of equipment, its use and conditions to this exposed. This should be determined through the risk assessment and take full account manufactures recommendation. The inspection should concentrate on those parts which ecessary for the safe operation of the equipment and, in some cases, this may require sting and dismantling. However, not all safety critical features on a particular equipment may pre important same intervals. Rather, an inspection may vary in its extent such as quick chess before use. weekly cheeks and more extensive examinations under taken every few ths or longer, Records need not be maintained for the simple pre-use cheeks. The frequency nection may vary depending on environmental conditions and your own experience, Intervals the inspection may be increased if the inspection history shows negligible deterioration a shortened where experience shows this is necessary to prevent danger.

The equipments should be inspected by competent persons who has sufficient knowledge d experience of it. Of course, the necessary level of competence will vary for inspections, cording to the type of equipment and how/where it is used.

Agencies who conduct the testing of equipments must have required competency and tification in this regard. They may inspect and test an equpment following a report of ancident and check if corrective action has been taken on factors contributing to the incident. Thus, testing and inspection may consist either of a scheduled programme to meet the requirements of legislation and standards or on an adhoc basis which is initiated because of

an sue that arises requiring testing to diagnonise the cause or source and resolve the safety problem.

Persons responsible for coordinating inspection and testing of equipments are generally sponsible for maintainance and security of such records. Inspection and test results should be made available to the manager of the area concerned. It is also important that the inspection and testing results are discussed with the safety committee to ensure all possible solutions. Procedures must be reviewed to ensure relevance, currency and corrective action on nonconformance found during inspection or testing procedures. **EQUIPMENT MAINTENANCE**

Maintenance of an equipment is the operation of keeping its various components in their original form as far as possible with a view to ensure that safety as well as production in operation do not deteriorate. It includes servicing, inspection and adjustment, small repairs in field, major repairs and over hauls in main workshops and proper case of laid up machine. The objectives of maintenance are:

i) To maximize the availability of machinery needed for smooth production.

ii) To minimize down time due to break down of machinery

iii) To ensure longevity of the machinery to avoid high rate of depreciation of capital.

7.5.1. Types of maintenance

There are mainly four types of maintenance: a) Reactive maintenance / Break down maintenance / Corrective based maintenance)

b) Predictive maintenance and Reliability centered maintenance (condition based maintenance)

c) Preventive maintenance/Schedule maintenance (Time based maintenance)

d) Pro-active maintenance (Advance maintenance technique)

7.5.1.17 **Reactive maintenance**

Reactive maintenance is based on the principle of "run it till it breaks" mode of maintenance. No efforts are mode or no actions are taken to maintain the equipment as intended by the designer, either to prevent failure or to ensure that the designed life of the equipment is attained. Of course, reactive maintenance is still the predominant mode of maintenance in the Indian construction scenario, accounting for about 65 to 70% of the maintenance programme.

The advantages of reactive maintenance are:

(1) It has lower initial costs

(ii) It requires fewer maintenance staff

However, the disadvantages of this approach to maintenance are

(i) Cost escalation due to unplanned down time of the equipment.

(ii) Increased labour cost, especially towards over time for untimely repairs and replacement.

(iii) Increase in cost associated with sudden requirement of repair or replacement of equipments.

(iv) May result in possible secondary equipment or process damage from equipment failures.

(v) Leads to in efficient use of staff resources.

7.5.1.2 Predictive maintenance

The predictive maintenance approach aims at detecting the onset of equipment degradation and addressing the problem as soon as they are identified. This allows stressors to be eliminated or controlled, prior to any significant deterioration in the physical state of the component or equipment. It leads to both current and futurefunctional capabilities. Of course, predictive maintenance techniques provide data that define required servicing and inspection periods so that maintenance departments can determine in advance when the equipment must be shut down for over haul. Statistical evidence proves that these programmes, when properly minimize equipment and system breakdowns resulting in a major reduction in tenance and operating costs. It accounts for about 2% of the maintenance programme for diagnostic tools in predictive maintenance programme .

The following are the six major diagnostic tools in the predictive maintenance programme cularly scheduled basis.

wear particle analysis Vibration analysis

Infrared thermography Electrical testing

Ultra sonic/acousticProcess variables / inspections / non-destructive The advantages of predictive maintenance are:

1. Increased component operational life and availability.
2. Allowance for pre-emptive corrective actions
3. Decrease in equipment and/or process downtime
4. Lowering of cost for parts and labour
5. Better product quality
6. Improvement of worker and environmental safety
7. Rise in morale of the workers
8. Increase in energy saving

The disadvantages associated with it are:

1) Increase of investment in diagnostic equipment.
2) Increase in investment of staff training.
3) Non-availability of immediate savings potential by the management.

15.1.3 **Preventive maintenance**

It refers to a series of actions that are performed on either a time-based schedule or a schedule based on that of machine-run time. These actions are designed to detect, preclude or stigate degradation of a system (or its components). The goal of preventive maintenance proach is to minimize system and component degradation and thus sustain or extend the riodic inspection of equipment to uncover conditions leading to production break downs or seful life of the equipment. The basic activities involved in the preventive maintenance are i) mful depreciation and ii) Upkeep of equipment to minimize down time and break down conditions while they are still in a major stage. It accounts for 30% maintenance programme in India. Application of preventive maintenance technology

The preventive maintenance technology is applied in respect of the following:

a) Lubrication

b) Cleaning

c) Replacement

d) Inspection

The advantages of preventive maintenance are as follows

i) Cost effectiveness in capital intensive processes and equipment

ii) Flexibility in the adjustment of maintenance periodicity.

iii) Increase in component life cycle.

iv) Generation of energy savings.

v) Reduction in equipment andor process failures.

vi) Cost saving (around 15%) over that found in a reactive maintenance progremme.

The disadvantages of this approach are

i) Inability to eliminate catastrophic failures.

ii) More labour intensive.

7.5.1.4 **Pro-active maintenance**

Although predictive maintenance uses online condition monitoring to help predict the occurrence of failure, it often fails to identify the root cause of failure. That is where the proactive maintenance is called for. Of course, proactive maintenance relies on information provided by predictive methods to identify problems and isolate the source of failure.

Now a days, the old ideas on the machine maintenance being discarded in major industries throughout the world. Moreover, the cost saving trend is towards a maintenance programme

that targets the root causes of machine wear and failure. Thus, proactive maintenance methods have been able to save quite sizable amount on machine maintenance every year in various industries and construction organizations. Intact, in many companies, it often exceeds annual net profit.

Thus, equipment maintenance is not simply either preventive maintenance or lubrication which is an important aspect or function of maintenance. Also, it is not a hasty rush to repair a broken machine part or replace a faltering bearing, although these are essential maintenance activities. Rather equipment maintenance is a science, an exercise in economics and an art as 11/15 philosophy. Broadly maintenance encompasses all these aspects and too many wholly as are of its components.

maintenance planEquipments

The maintenance plan necessarily embodies the quality of maintenance work. An aspect of maintenance activity is the difficulty in accessing the quality of work ip executed maintenance work may lead to a breakdown. Because of the intervening becomes very difficult to judge wheather the breakdown was attributable to errors or defective parts. In other words, the quality of maintenance must ensure work itself. In order to accomplish this, each individual member of the maintenance possess a sense of responsibility and consider the methods for preparing, executing ading his or her own work. From efficiency point of view, the works planned in advance can be accomplished compared to the work done in case of abrupt failure. With a view to raise the ading his or her efficiency point of view, the works planned in advance can be accomplished compared to the work done in case of abrupt failure. With a view to raise the tenance, quality and efficiency, it is imperative to create a maintenance plan and cpus preparation before it execution. The work ethics of the maintenance staff mat curbing the breakdown. However, the following are the benefits of adopting plan.

The number of operational steps can be repeatedly utilized. Advance planning of human resources can be made for availability of required personnel.

Prevention of errors in procurement of materials, spare parts and sub contractingwork is possible.

Facilitates checking of quality and procurement of better material.

Schedules can be set so that work detail plans are coordinated with production plans. Repair cycles can be identified to take measures in a timely fashion Standardization pattern of repair work can be adopted enabling the work to be done efficiently Makes possible simultaneous devising of repair plans.

- Peoples sense of responsibility are encouraged.
- Large volume of work can be handled more efficiently.
- Precautionary measures for maintenance
- Before carrying out maintenance

- Failure notification should be given to appropriate authority having thorough knowledge of the equipment.

7.5.2 <u>Equipments maintenance plan</u>

The maintenance plan necessarily embodies the quality of maintenance work. An important aspect of maintenance activity is the difficulty in accessing the quality of work de. A poorly executed maintenance work may lead to a breakdown. Because of the intervening time lag, it becomes very difficult to judge wheather the breakdown was attributable to maintenance errors or defective parts. In other words, the quality of maintenance must ensure the quality of work itself. In order to accomplish this, each individual member of the maintenance crew must possess a sense of responsibility and consider the methods for preparing, executing and validating his or her own work.

From efficiency point of view, the works planned in advance can be accomplished more quickly compared to the work done in case of abrupt failure. With a view to raise the level of maintenance, quality and efficiency, it is imperative to create a maintenance plan and make tenacious preparation before it execution. The work ethics of the maintenance staff should aim at curbing the breakdown. However, the following are the benefits of adopting maintenance plan.

(i) The number of operational steps can be repeatedly utilized.

(ii) Advance planning of human resources can be made for availability of required personnel.

(iii) Prevention of errors in procurement of materials, spare parts and sub contracting work is possible.

(iv) Facilitates checking of quality and procurement of better material.

(v) Schedules can be set so that work detail plans are coordinated with production plans.

(vi) Repair cycles can be identified to take measures in a timely fashion.

(vii) Standardization pattern of repair work can be adopted enabling the work to be done efficiently.

(viii) Makes possible simultaneous devising of repair plans.

(ix) Peoples sense of responsibility are encouraged.

(x) Large volume of work can be handled more efficiently.

7.5.2.1 Precautionary measures for maintenance

(A) <u>Before carrying out maintenance</u>

(i) Failure notification should be given to appropriate authority having thorough knowledge of the equipment.

(ii) Warning tags indicating not to operate must be tied to the equipment to avoid inadvertent use.

(iii) It must be cleaned before inspection and maintenance.

(iv) The work place must be kept clean and tidy.

(v) Care must be taken to stop the engine before carrying out inspection and maintenance .

(vi) Establishment of five fighting arrangement and prevention of smoking must be ensured .

(vii) Presence of proper rigging tools and repair tools are to be ascertained before taking up maintenance work.

(B) During maintenance

(i) Only authorized persons should carry out the maintenance of equipments.

(ii) Attachments are to be stored in safe custody andaccess of unauthorized persons are to be restricted.

(iii) The equipment must be placed on the firm level ground while working under the machine.

(iv) When maintenance work is to be carried out with engine in running condition, operator must be seated to stop the machine in case of emergency.

(v) Care must be taken to see that no tools are left as it is inside the machine by mistake.

(vi) It is to be ensured that the machines are provided with personal protective equipments.

(vii) Repair should be under taken immediately as soon as abnormality is reported.

(viii) High level of alertness must be maintained while handling fuel high pressure hoses, high pressure oils and working at high temperature.

7.5.3 General guidelines for maintenance

Do's

(i) Keep the equipment clean and dry.

(ii) Be familiar with operation and maintenance manual of the equipment and be thorough with manufacturer's instruction.

(iii) Pay particular attention to

(iv) Keep all the nuts and bolts tight.

(v) Use only genuine spares.

(vi) Check the level of engine oil and radiator water regularly/daily before starting the equipment.

(vii) Attend periodic preventive maintenance.

(viii) Take steps to keep all meter and safety devices functional.

Do not's

(i) Avoid over loading the engine and equipment.

(ii) Don't run the engine in case black smoke is coming out of the exhaust.

(iii) While cleaning engine parts avoid using cotton waste.

(iv) Don't mix different brands of oils.

(v) Avoid storing fuel, oil in galvanized containers.

(vi) Don't observe economy only in the cost of maintenance.

CHAPTER -8(QUALITY CONTROL)

INTRODUCTION: Quality control in its simplest term, is ensuring quality aspect during manufacturing or production process. The aim of quality control is to ensure construction or production of items for their intended use without defects and variations from prescribed standards within allowable tolerance limits.

In the current concept of quality control, the meaning of quality is closely associated with cost and customer needs or performance standards. So, quality may simply be defined as fitness of purpose at lowest cost and highest performance level.

After all, the quality of a product depends upon the application of materials, men, machine and manufacturing conditions.

Thus, quality control may be broadly defined as a industrial management technique by of which a product of uniform acceptable quality are manufactured.

8.1 CONCEPT OF QUALITY IN CONSTRUCTION :

The basic elements of quality in construction are as follows:

(1) Quality characteristics –

The properties that define the nature of a product for quality control viz. strength, colour, Amension and temperature etc. are called quality characteristic. Eg-Cement concrete which 82 very common construction material now-a-days, the compressive strength, size of aggregate, sater-cement ratio, slump, surface finish etc.

(2) Design quality –

It is a fact that no design can produce absolutely perfect results, whatsoever good design it may be. Thus, the desired standards for characteristics such as strength, dimension etc.

(3) Quality of conformance –

The degree of quality of work found in actual construction work is known as quality of conformance. As in the case of design quality, the degree to which the quality is to be enforced in the field has to be considered along with the cost necessary for quality control.

• Factors affecting the quality of conformance :

The quality of conformance is affected by the following factors:

1. Construction method in the field:

The quality of materials used, skill of the workers and efficiency of machinery and equipments affect the quality of conformance.

2. Field supervision level :

The managerial control exercised in directing the workers to conform to the plans and specifications and the level of supervision enforced affects the quality of conformance.

3. Inspection and quality control procedure :

The inspection and the quality control procedure adopted also greatly influence the quality of conformance.

8.2 Quality Standards- during construction, after Construction, destructive &non destructiveMethods :-

In construction work, the following activities/works need effective quality control.

1.Concrete works

Concrete is a very important construction material possessing high compressive strength, whose quality is influenced to a great extent by its constituent materials, water-cement ratio, size of aggregate and their grading, rate of loading and curing conditions etc. The supervisor

should be well versed with the properties of concrete. So, to obtain a good quality concrete, the engineer-in-charge is required to be present throughout the operation of mixing, placing, compacting and finishing etc.

2.Steel works

Steel is a costly item and constitutes a major item of expenditure in most of the civil works. In RCC structures, it is used as reinforcement to take up tensile stress. Hence its tensile strength, proper binding, binding and placing etc. should be checked carefully while using it. The reinforcing bars should be free from rust, scales, oil, grease and other harmful coatings.

3.Form work

The shape and finished surface of concrete depends upon the form work. The form work must have smooth surface so that the finished concrete may require minimum amount of rendering. When the finished surface is not required to be plastered.

4.Masonry work

The bricks/stones to be used in masonry work should be of specified quality and grade, having requisite strength and water absorption capacity within permissible limits. The dimensions and verticality of masonry works are very important and care should be taken to maintain it.

5.Water proofing

Provision of damp proof coarses at plinth level and water proofing of roof and expansion joints etc. Hence proper care should be taken to have them properly installed.

6.Joinery and Timber work

For wood work, timber of specified quality should be used. The workmanship of wood work should be properly checked and maintained as per specifications. These are important aspects from the quality control point of view.

7.Services works

Water supply, electric fittings, sanitary air conditioning etc. are classified under this category. Therefore, these works need special attention of quality control.

8.3   QUALITY CONTROL METHODS (DESTRUCTIVE)

The following are the important quality control methods

1.Inspection

Inspection is the function to judge the quality of a product. To be more precise, it is the process of measuring the quality of a product or service in terms of established standards. Any defect noticed must be got notified before proceeding to the next stage of construction.

2.Testing

Testing is the examination of the material or product to check its conformance to the specified standards. The testing may be either destructive or non-destructive and can be performed at site or in the laboratory.

3. Sampling

The process of determining the quality of a large group by examining a part of the group that will be statistically representative to the whole group is called sampling. The reliability of the test results of the sample is determined by the reliability number. The reliability of the information obtained increases with the size of the sample used.
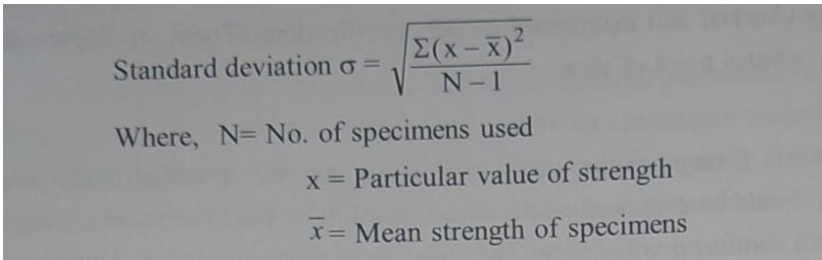
(a)   Reliability Number - The reliability number is taken as the reliability of the test results of the sample. It is expressed as

Reliability number R = 100 –[(No. of defectiveunits)/ (no. Of unitstested)×100]

(b)   Deviation

It is the root mean square of the deviation of all the results and is calculated as follows:
Standard deviation σ =
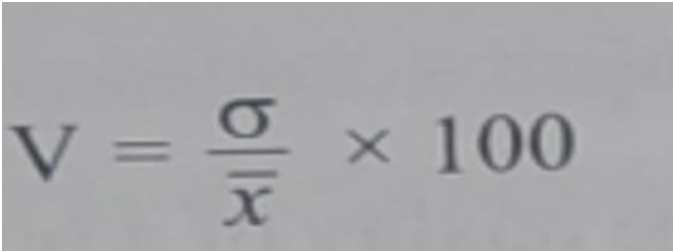
(C) Coefficient of variation

$$\text{Standard deviation } \sigma = \sqrt{\frac{\Sigma(x - \bar{x})^2}{N - 1}}$$

Where, N = No. of specimens used

x = Particular value of strength

$\bar{x}$ = Mean strength of specimens

This is an alternative method of expressing variation of results and is a non-dimensional measure of variation. It is obtained by dividing the standard deviation by the arithmetic mean value.
Co-efficient of variation =

$$V = \frac{\sigma}{\bar{x}} \times 100$$

However, in India, the standard deviation is taken as the most reliable method of quality control. Standard deviation is an absolute measure of dispersion,where as the coefficient of variation is a relative measure of dispersion.

8.4   NON-DESTRUCTIVE METHODS OF QUALITY CONTROL:

The quality control tests or exercises conducted on a structure without causing slightest damage to whole or part of it are known as non-destructive methods. As per ISO-13822, the existing structure may be assessed for load bearing capacity and other structural properties by non- destructive testing and continuous monitoring process. The various stages of testing and monitoring are as under:

1.   Load testing

It is done to test the structure or part thereof by external loading to evaluate its behavior or properties or to certain its load bearing capacity.

2.   Inspection

On site non-destructive examination may be done to establish the present conditions of The structures.

3.   Monitoring

It is an act of acquiring, processing and communicating information about a structure under operational conditions over a period of time with a high level of automation. Monitoring of structures is done continuously or frequently for observing or measurement of structural conditions.

Of course, non-destructive methods have been in use for about four decades and during this period, considerable development has taken place to such an extent that it has become a powerful method for evaluating existing structures with regard to their strength and durability apart from assessment and control of quality. In certain cases, the investigation of crack depth, existence of micro cracks and progressive deterioration are also studied by this method. For example, taking concrete as the material under consideration, some such properties are hardness, resistance to penetration, rebound number, resonant frequency and ability to allow ultrasonic pulse velocity to propagate through it. Further, certain electrical properties of concrete,

(I)Surface hardness test -

These are of indentation type, include the William's testing pistol and impact hammers and are

used only for estimation of concrete strength.

(ii)   test-

The rebound hammer test measures the elastic rebound of concrete and is primarily used for the estimation of concrete strength as well as for comparative investigations.

(iii)   Penetration and pull out tests-

These include the use of Simbi hammer, spit pins, the Windsor probe and the pull out test. They measure the penetration and pull out resistance of concrete and are primarily used for strength estimations. However, they can also be used for comparative studies.

(iv)   Dynamic or vibration tests-

These include resonant frequency and mechanical sonic and ultrasonic pulse velocity methods. These are mainly used to evaluate the durability and uniformity of concrete including estimation of strength and elastic properties.

(v)   Combined methods-

The combined methods involving ultrasonic pulse velocity and rebound hammer may be effectively used to estimate the strength of concrete.

(vi)   Radioactive and nuclear methods-

These include the X-Ray and Gamma-Ray penetration tests for measurement of density and thickness of concrete. Also the neutron scattering and neutron activation method are useFor moisture and cement content determination.

(vii)   Magnetic and electrical methods-

The magnetic methods are primarily concerned with determining the cover thickness of reinforcement in concrete, where as the electrical methods, including, the micro wake absorption technique, have been used to measure moisture content and thickness of concrete.

(viii)   Acoustic emission technique-

These techniques are mostly used to study the initiation and growth of cracks in concrete.
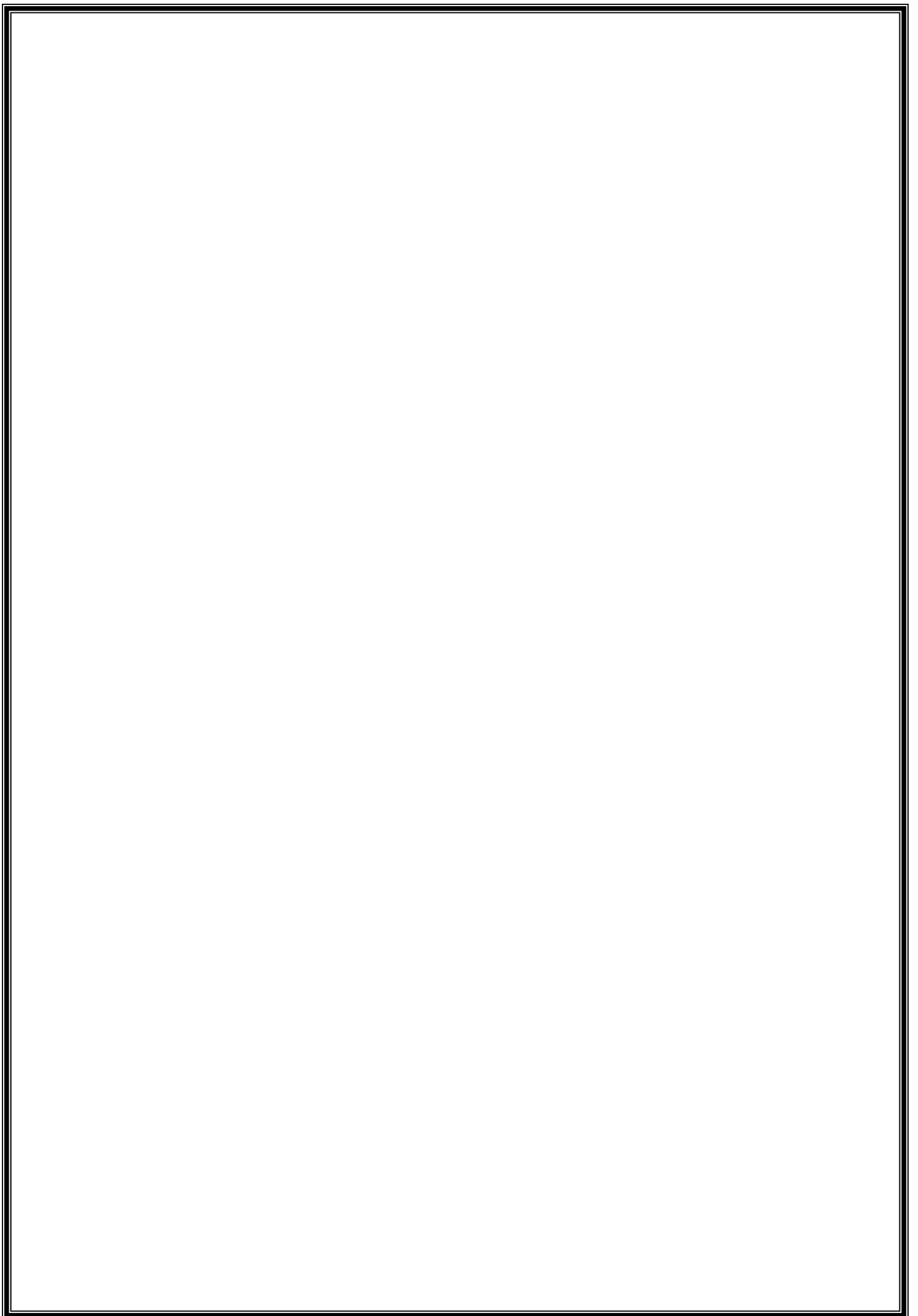
Chapter -10
SAFETY MANAGEMENT IN CONSTRUCTION WORK

10.1 IMPORTANCE OF SAFETY :-

Thus social concern and efforts are being made to adopt safety measures by creating safety conciousness among the wokers. Form a survey of occupational injury and illness accident, it is found that upto 14.5% workers suffer from these injuries. So sufficent care & though preventive measures should be taken for it, so that, accidents of any sort during the construction period can be avoided to some extent. Accident prevention should be considered is an essential activity in all construction sites.Inindia, construction industry covers the largest labour force after agriculture. With the advance of construction industry, the number accidents also is increasing and accident happens all of a sudden enexpectedly.

From economic reasons, the injured worker faces difficulty owing to medical expenses For the injury. It also causes the slow down in progress of work and decrease in productivity. Loss of confidence, and loss of administrative work due to accident.

10.2.1 **CAUSES AND EFFECTS OF ACCIDENT**-

There are many causes of accident in construction industry. These are classified broadly.

1.Physical

Physiological

Psychological

10.2.1.1 PHYSICAL CAUSE:

The following factors may be grouped under physical cause. 1.Causes relating to machines:

(I)Due to obstruction free movement of man & machine is not possible & there may be inadequate working space for the machines.

(ii)   to improper placing or adjusting of machines.

(iii)   Accident may be caused due to unsuitable machines being used for the job.

(iv)   Accident may be caused due to improperly guarded machines

(v)   Accident may be caused due to improperly insulated electric motor on the machine.

2.   Causes Relating to tools &equipments

1) Accident may be caused due to constant use of tools, which has been blunt and worn out.

2)Tools used for the job is being too small.

3)   Sometimes due to brittle nature of tools, it breaks suddenly, accident may be caused.

4)   The tools having handle too short or loose 5. Use of unsuitable tools for the work may be a cause for accident.

3.   Causes relating to materials

1)   Accident may be caused due to careless handling of explosives, petrolium products &Brittle materials.

2)   At the time of use of road materials, there should be careful handling of too hot materials like tar or bitumen.

3)   Accident may be caused due to use of materials being poisonous & dangerous as acid and some salts.

4)   Due to not adopting proper precautions while handling materials emitting foul gases.Eg sewage in the maintenance of sewers.

4.   Causes relating to uniform

1)   uniform should not be loose.

2)   The slippery and loose shoes may be used during the work time. 3)While working on welding job, no protective devices are being used. 4)Sleeves of the shirt being out of bottons.

5.   Causes Relating to Environment

1)   Poor lighting arrangement at the working site.

2)   Poor ventilation & unhygienic conditins at the working place.

3)   Loose electric cables & live conductors carelessly.

4)   Obstacles in the working area.

5)   Floors being slippery.

6)   Use of unstable & unsafe ladder.

10.3   SAFETY MEASURES AT WORK SITES

Prevention of accidents is the prime concern at any construction site both from human life and financial considerations. Irrespective of the nature of construction projects, accidents are likely to take place resulting in physical injury, loss of property or even casualties.

(a)   Excavation

(b)   scaffolding

(C) Form work (d)Fabrication (e)Errection (f)Demolition

10.3.1   Safety measures of excavation:

The following safety measures are required to be adopted at Excavation sites .

(i)   The excavation work should be carried out under the supervision of experienced and competent supervisor having responsibility for enforcement of safety rules and prevention of the use of defective and unsafe appliances.

(ii)   It is essential to have a complete knowledge of under ground structures (such as sewers,Pipe lines, electrical conduits, gas mains etc) before commencement of the excavationWork and proper precautions should be taken to ensure safety of workmen as well as Public.

(iii)   The workers should be advised to make use of safety appliances whenever required and safety helmet must be invariably used by the workers where hazards from falling stores, timber and other materials exist.

10.3.2 Safety measures for scaffolding :

The necessity of scaffolding/staging arises for all types of construction works carried out above ground level viz brick or stone masonry in super structure painting, repair mentainance of structures etc.

(I) ensure safety and stability, every scaffold should be securely supported or suspended and properly strutted or braced. Thus, all scaffolds and working platforms should be securely fastened to the building or structure. They should be braced or guyed properbly. If independent of a building.

(ii) The vertical standards of scaffolds should be embedded into the ground sufficiently deep so that these are capable of withstanding loads. On pucca floors or on black topped pavaments/streets, the standards could be supported in empty tar drums and packed with sand/stone bricks etc.

(iii) member of scaffolds should be free from defects and particularly the wooden Members should be free from dry rot & wet rot etc. The members should be thoroughly Tested for their strength and inspected before these are put to use for the second time or For any subsequent use.

(iv) Also the sizes of different members are properly designed for the loads that are intended to be carried. Lacings should be done securely for connecting the ledgers to standards and putlogs to ledgers. The rope used for tieing should be stout and thick.

10.3.3 Safety measures for formwork :

(i) The material of the formwork should be carefully selected as per relevant Indian standard. The members should be properly designed for adequate strength and the sections are generally worked out in consideration of the loads that are likely to be Taken up.

(ii) The formwork should be sufficiently strong and stiff to resist bending and deformation deflection after concrete is placed. Also, it should be capable of taking up all dead Load, live load and the impact to which it may be subjected to.

(iii) Partially seasoned timber should be used for formwork to minimize the effect of shrinkage, warping, bending or bulging under loads whenever, green timber is to be used, due allowance for bulging and shrinkage should be made while preparing the Surface.

10.3.4 Safety in fabrication and erection:

The following safety measures are adopted during fabrication and erection works.

(I) Periodical checking of all equipments required for fabrication such as gas cutter and welding sets, power hacksaws, drills, grinders etc. should be done to ensure their safe working.

(II) All workers engaged in gas cutting or welding operations should wear safety gloves And aprons and use proper welding.

(III) Moving parts of machineries or equipments should be invariably provided with safety guards.

(IV) Power cables of all equipments/machineries should be properly insulted and protected from damage and cuts against any mishap.

10.3.5 Safety measures for demolition :

The following are the various safety measures to be adopted at the time of any demolition work.

(I)Before the start of demolition process, the manner in which various parts of the structure are supported and the extent of demolition effect on the adjoining structures should be thoroughly studied.

(ii)　A site specific systematic plan for the step by step demolition work should be prepared And strictly followed under the guidance of experienced foreman.

(iii)　The demolition work should not be taken up at night or during stormy weather or heavy rains especially when the structure to be demolished is situated in an inhabited area.

10.4　DEVELOPMET OF SAFETY CONSCIOUSNESS :

Safety programme on each construction project is highly important so as to reduce the total number of accidents. So safety consciousness should be developed in the construction work. Also safety programme should be made an integral part of each construction company.Someimprotant aspects of safety programme of construction projects are as follows.

(I)A sefety committee should be set up to guide the operation of safety programme. Both worker and management should paticipate in it to give their opinion on safety measures.

(ii) The employees should secure full support of the top management, because the employers are not to be expected to maintain an intrest in the safety programme.

(iv)　Before starting construction on a project, the safety director should analyse with foreman and superintendent about the safety programme.

10.4.1 Safety Equipments :

The workers while performing their duties, particularly while proforming difficult or hazardous or dangerous operations, there are some safety accessories which the workers must use. By using these accessories the accident hazards can be reduced to a great extent.

These accessory items are such as:

1.Safety shoes.

2.Goggles

3.Apron

4.Hand gloves

5.Helmets

6.Dresses

7.Gunboot

8.Safetybelts.

9.Portable light & lantern

10.Fire extinguisher etc.

10.5 SAFETY LEGISLATION :

There are some laws or legislations in connection with safety & wellbeing of the workers. Among

them the Workman's Compensation Act & Contract Labour Act are important.

10.5.1 Workman's Compensation Act :

In India the workmen compensation act was passed for the first time in 1923. But it came into force in 1924, and it was amended in 1958, 1976. This act regulates the payment of compensation to a worker who had injured in the course of employment. But before this act the payment of compensation to the injured worker was a lengthy & costly process. In case any accident occurring at the site of work, this act provides the payment of compensation to the workers.

OBJECTIVES:

The main objective of this act is to payment of compensation to the workers who had injured in the couse of employment. Another aim of this act is protect the worker & his family from hardly caused by the accident. Also by result of accident, the worker losses his earning capacity. This act forces the worker to work carefully while dangerous situations and also this act forces the employer to take all the safety measures which are to be provided to the workers.

10.5.2 Contract Labour Act :-

10.5.2.1 Aim of this Act

In order to provide all the facilities and to protect the contract labourers from the contractors engaged in the government and private works, the Government of India passed contract labour act. As this act was enacted in 1970, therefore it is also known as 1970 contract labour act.

10.5.2.3 Contract labourers:

As the know that in India construction industry is the second largest industry. Construction industry plays on important role not only in the development of infrastructure of our country but also it affects our economy. About five crores of labourers are employed under the construction industry. Most of the labourers employed in construction industry are unskilled. They are employed on daily wage basis.They are for carrying the constructional activities. The contractor who engages the labourers for construction of the government and the private works, decide the wages and other facilities to the labourers: When the construction work is over, then they are terminated. In order to seek employment, they are to shift to new construction sites. Thus, these labourers move from one site to another. They are paid wages and have to also work for long hours

GANDHI ACADEMY OF TECHNOLOGY &ENGINEERING GOLANTHARA, BERHAMPUR

Lecture notes on

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CONTENTS

CHAPTER -1

Grades of concrete

Grade of concrete	(Cement: Sand: Aggregates)	Compressive strength (Mpa)
M5	1:5:10	5
M7.5	1:4:8	7.5
M10	1:3:6	10
M15	1:2:4	15
M20	1:1.5:3	20
M25	Design Mix	25
M30	Design Mix	30
M35	Design Mix	35
M40	Design Mix	40
M50	Design Mix	50
M70	Design Mix	70

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Classification of Concrete Grades

The grades of concrete classify the concrete into four types:

- Lean Concrete
- Ordinary grade concrete
- Standard grade concrete
- High strength grade concrete

Advantages of Concrete

- Ingredients of concrete are readily available in most places.
- Unlike natural stones, concrete is free from defects and flaws.
- Concrete can be manufactured to the desired strength with an economy.
- The durability of concrete is very high.
- It can be cast to any desired shape.
- The casting of concrete can be done on the working site which makes it economical.
- The maintenance cost of concrete is almost negligible.
- The deterioration of concrete is not appreciable with age.
- Concrete makes a building fire-safe due to its non-combustible nature.
- Concrete can withstand high temperatures.
- Concrete is resistant to wind and water. Therefore, it is very useful in storm shelters.
- As a soundproofing material cinder concrete could be used.

Disadvantages of Concrete

- Compared to other binding materials, the tensile strength of concrete is relatively low.
- Concrete is less ductile.
- The weight of concrete is high compared to its strength.
- Concrete may contain soluble salts. Soluble salts cause efflorescence.

CHAPTER-2

CEMENT

Cement is a binder, a substance that sets and hardens and can bind other materials together. Cements used in construction can be characterized as being either hydraulic or non-hydraulic, depending upon the ability of the cement to be used in the presence of water. Non-hydraulic cement will not set in wet conditions or underwater, rather it sets as it dries and reacts with carbon dioxide in the air. It can be attacked by some aggressive chemicals after setting. Hydraulic cement is made by replacing some of the cement in a mix with activated aluminum silicates, pozzolana, such as fly ash. The chemical reaction results in hydrates that are not very water-soluble and so are quite durable in water and safe from chemical attack. This allows setting in wet condition or underwater and further protects the hardened material from chemical attack (e.g., Portland cement).

Uses:-

• Cement mortar for Masonry work, plaster and pointing etc.

• Concrete for laying floors, roofs and constructing lintels, beams, weather- shed, stairs, pillars etc.

• Construction of water, wells, tennis courts, septic tanks, lamp posts, telephone cabins etc.

• Making joint for joints, pipes, etc.

• Manufacturing of precast pipes, garden seats, flower posts, etc.

• Preparation of foundation, water tight floors, footpaths, etc.

ORDINARY PORTLAND CEMENT:-

Ordinary Portland cement is the most common type of cement in general use around the world. This cement is made by heating limestone (calcium carbonate) with small quantities of other materials (such as clay) to 1450°C in a kiln, in a process known as calcinations, whereby a molecule of carbon dioxide is liberated from the calcium carbonate to form calcium oxide, or quicklime, which is then blended with the other materials that have been included in the mix. The resulting hard substance, called 'clinker', is then ground with a small amount of gypsum into a powder to make 'Ordinary Portland Cement'(often referred to as OPC). Portland cement is a basic ingredient of concrete, mortar and most non-specialty grout. The most common use for Portland cement is in the production of concrete. Concrete is a composite material consisting of aggregate (gravel and sand), cement, and water. As a construction material, concrete can be cast in almost any shape desired, and once hardened, can become a structural (load bearing) element. Portland cement may be grey or white.

• This type of cement use in construction when there is no exposure to sulphates in the soil or ground water.

• Lime saturation Factor is limited between i.e. 0.66 to 1.02.

• Free lime-cause the Cement to be unsound.

• Percentage of (AL_2O_3/Fe_2O_3) is not less than 0.66.

• Insoluble residue not more than 1.5%.

• Percentage of SO_3 limited by 2.5% when $C_3A < 7\%$ and not more than 3% when $C_3A > 7\%$.

• Loss of ignition -4%(max)

• Percentage of Mg0-5% (max.)

• Fineness -not less than 2250 cm_2/g.

Chemical constituents of cement:-

These are the different constituents which combine to make cement. These are their percentage content in order to give good cement.

Oxide Per cent content
CaO 60–67
SiO_2 17–25
Al_2O_3 3.0–8.0
Fe_2O_3 0.5–6.0
MgO 0.1–4.0
Alkalies(K_2O,Na_2O)
0.4–
1.3
SO_3 1.3–3.0

Hydration of Cement:-

The chemical reaction between cement and water in a proportioning mix is called as hydration of cement. It may be in concrete mix or in the making of mortar in the field work.

SETTING OF CEMENT:-

The action of changing mixed cement from a fluid state to a solid state is called setting of cement and time required for it to set is called setting time of cement. Setting time of cement is same as setting time of concrete.

SETTING TIME OF CEMENT:-

1. Initial Setting Time

Initial Setting Time is defined as the period elapsing between the time when water is added to the cement and the time at which the needle of 1 mm square section fails to pierce the test block to a depth of about 5 mm from the bottom of the mould. A period of 30 minutes is the minimum initial setting time, specified by ISI for ordinary and rapid hardening Portland cements and 60 minutes for low heat cement.

2. Final Setting Time

Final Setting Time is defined as the period elapsing between the time when water is added to cement and the time at which the needle of 1 mm square section with 5 mm diameter attachment makes an impression on the test block. 600 minutes is the maximum time specified for the final set for all the above mentioned Portland cement. IS: 269-1976 specifies the strengths in compression on the standard mortar-cube.

STRUCTURE OF HYDRATED CEMENT:-

The desirable engineering characteristics of hardened concrete —strength, dimensional stability, and durability —are influenced not only by the proportion but also by the properties of the hydrated cement paste, which, in turn, depend on the micro-structural features (i.e., the type, amount, and distribution of solids and voids).

Fresh cement paste is a plastic network of particles of cement in water but, once the cement paste has set, its apparent or gross volume remains approximately constant.

At any stage of hydration, the hardened paste consists of hydrates of the various compounds, referred to collectively as gel, crystals of Ca(OH)2, some minor components, un hydrated cement and the residue of water-filled spaces in the fresh paste.

VARIOUS TESTS ON CEMENT:

Basically two types of tests are under taken for assessing the quality of cement. These are either field test or lab tests. The current section describes these tests in details.

Field test:

There are four field tests may be carried out to as certain roughly the quality of cement. There are four types of field tests to access the colour, physical property, and strength of the cement as described below.

Colour:
- The colour of cement should be uniform.
- It should be typical cement colour i.e. grey colour with a light greenish shade.

Physical properties:
- Cement should feel smooth when touched between fingers.
- If hand is inserted in a bag or heap of cement, it should feel cool.

Presence of lumps:
- Cement should be free from lumps.
- For a moisture content of about 5 to 8%, this increase of volume may be much as 20 to 40 %, depending upon the grading of sand.

Strength:
- A thick paste of cement with water is made on a piece of thick glass and it is kept under water for 24 hours. It should set and not crack.

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Strength:
- A thick paste of cement with water is made on a piece of thick glass and it is kept under water for 24 hours. It should set and not crack.

Laboratory tests:
Six laboratory tests are conducted mainly for assessing the quality of cement. These are: fineness, compressive strength, consistency, setting time, soundness and tensile strength.

Fineness:
- This test is carried out to check proper grinding of cement.
- The fineness of cement particles may be determined either by sieve test or permeability apparatus test.
- In sieve test, the cement weighing 100 gm is taken and it is continuously passed for 15 minutes through standard BIS sieve no. 9.The residue is then weighed and this weight should not be more than 10% of original weight.
- In permeability apparatus test, specific area of cement particles is calculated. This test is better than sieve test. The specific surface acts as a measure of the frequency of particles of average size.

Compressive strength:
- This test is carried out to determine the compressive strength of cement.
- The mortar of cement and sand is prepared in ratio 1:3.
- Water is added to mortar in water cement ratio 0.4.
- The mortar is placed in moulds. The test specimens are in the form of cubes and the moulds are of metals. For 70.6 mm and 76 mm cubes, the cement required is 185gm and 235 gm respectively.
- Then the mortar is compacted in vibrating machine for 2 minutes and the moulds are placed in a damp cabin for 24 hours.
- The specimens are removed from the moulds and they are submerged in clean water for curing.

- The cubes are then tested in compression testing machine at the end of 3days and 7 days. Thus compressive strength was found out.

Consistency:

- The purpose of this test is to determine the percentage of water required for preparing cement pastes for other tests.
- Take 300 gm of cement and add 30 percent by weight or 90 gm of water to it.
- Mix water and cement thoroughly.
- Fill the mould of Vicat apparatus and the gauging time should be 3.75 to 4.25 minutes.
- Vicat apparatus consists of a needle is attached a movable rod with an indicator attached to it.
- There are three attachments: square needle, plunger and needle with annular collar.
- The plunger is attached to the movable rod.
- The plunger is gently lowered on the paste in the mould.
- The settlement of plunger is noted. If the penetration is between 5 mm to 7 mm from the bottom of mould, the water added is correct.
- If not process is repeated with different percentages of water till the desired penetration is obtained.
- The purpose of this test is to determine the percentage of water required for preparing cement pastes for other tests.
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- If not process is repeated with different percentages of water till the desired penetration is obtained.

Setting time:

- This test is used to detect the deterioration of cement due to storage. The test is performed to find out initial setting time and final setting time.
- Cement mixed with water and cement paste is filled in the Vicat mould.
- Square needle is attached to moving rod of Vicat apparatus.
- The needle is quickly released and it is allowed to penetrate the cement paste. In the beginning the needle penetrates completely. The procedure is repeated at regular intervals till the needle does not penetrate completely.(up to 5mm from bottom)
- Initial setting time =<30min for ordinary Portland cement and 60 min for low heat cement.
- The cement paste is prepared as above and it is filled in the Vicat mould.
- The needle with annular collar is attached to the moving rod of the Vicat apparatus.
- The needle is gently released. The time at which the needle makes an impression on test block and the collar fails to do so is noted.
- Final setting time is the difference between the time at which water was added to cement and time as recorded in previous step, and it is =<10hours.

Soundness:

- The purpose of this test is to detect the presence of uncombined lime in the cement.

- The cement paste is prepared.
- The mould is placed and it is filled by cement paste.
- It is covered at top by another glass plate. A small weight is placed at top and the whole assembly is submerged in water for 24 hours.
- The distance between the points of indicator is noted. The mould is again placed in water and heat is applied in such a way that boiling point of water is reached in about 30 minutes. The boiling of water is continued for one hour.
- The mould is removed from water and it is allowed to cool down.

Setting time:

- This test is used to detect the deterioration of cement due to storage. The test is performed to find out initial setting time and final setting time.
- Cement mixed with water and cement paste is filled in the Vicat mould.
- Square needle is attached to moving rod of Vicat apparatus.
- The needle is quickly released and it is allowed to penetrate the cement paste. In the beginning the needle penetrates completely. The procedure is repeated at regular intervals till the needle does not penetrate completely.(up to 5mm from bottom)
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- The mould is removed from water and it is allowed to cool down.
- The distance between the points of indicator is again measured. The difference between the two readings indicates the expansion of cement and it should not exceed 10 mm.
-

Tensile strength:

- This test was formerly used to have an indirect indication of compressive strength of cement.
- The mortar of sand and cement is prepared.
- The water is added to the mortar.
- The mortar is placed in briquette moulds. The mould is filled with mortar and then a small heap of mortar is formed at its top. It is beaten down by a standard spatula till water appears on the surface. Same procedure is repeated for the other face of briquette.
- The briquettes are kept in a damp for 24 hours and carefully removed from the moulds.
- The briquettes are tested in a testing machine at the end of 3 and 7 days and average is found out.

Compressive strength:
- This test is carried out to determine the compressive strength of cement.
- The mortar of cement and sand is prepared in ratio 1:3.
- Water is added to mortar in water cement ratio 0.4.
- The mortar is placed in moulds. The test specimens are in the form of cubes and the moulds are of metals. For 70.6 mm and 76 mm cubes, the cement required is 185gm and 235 gm respectively.
- Then the mortar is compacted in vibrating machine for 2 minutes and the moulds are placed in a damp cabin for 24 hours.
- The specimens are removed from the moulds and they are submerged in clean water for curing.
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- The purpose of this test is to determine the percentage of water required for preparing cement pastes for other tests.
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- Mix water and cement thoroughly.
- Fill the mould of Vicat apparatus and the gauging time should be 3.75 to 4.25 minutes.
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- There are three attachments: square needle, plunger and needle with annular collar.
- The plunger is attached to the movable rod. The plunger is gently lowered on the paste in the mould.
- The settlement of plunger is noted. If the penetration is between 5 mm to 7 mm from the bottom of mould, the water added is correct. If not process is repeated with different percentages of water till the desired penetration is obtained.

Setting time:
- This test is used to detect the deterioration of cement due to storage. The test is performed to find out initial setting time and final setting time.
- Cement mixed with water and cement paste is filled in the Vicat mould.
- Square needle is attached to moving rod of Vicat apparatus.
- The needle is quickly released and it is allowed to penetrate the cement paste. In the beginning the needle penetrates completely. The procedure is repeated at regular intervals till the needle does not penetrate completely.(up to 5mm from bottom)
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- The needle is gently released. The time at which the needle makes an impression on test block and the collar fails to do so is noted.
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Soundness:
- The purpose of this test is to detect the presence of uncombined lime in the cement.
- The cement paste is prepared.
- The mould is placed and it is filled by cement paste.
- It is covered at top by another glass plate. A small weight is placed at top and the whole assembly is submerged in water for 24 hours.

• The distance between the points of indicator is noted. The mould is again placed in water and heat is applied in such a way that boiling point of water is reached in about 30 minutes. The boiling of water is continued for one hour.

• The mould is removed from water and it is allowed to cool down.

• The distance between the points of indicator is again measured. The difference between the two readings indicates the expansion of cement and it should not exceed 10 mm.

Tensile strength:

• This test was formerly used to have an indirect indication of compressive strength of cement.

• The mortar of sand and cement is prepared.

• The water is added to the mortar.

• The mortar is placed in briquette moulds. The mould is filled with mortar and then a small heap of mortar is formed at its top. It is beaten down by a standard spatula till water appears on the surface. Same procedure is repeated for the other face of briquette.

• The briquettes are kept in a damp for 24 hours and carefully removed from the moulds.

• The briquettes are tested in a testing machine at the end of 3 and 7 days and average is found out.

Grade of cement:-

Grade of cement represents the specific 28 days compressive strength. The following three grades are given along with their compressive strengths

- 33 Grade OPC –33 MPa
- 43 Grade OPC – 43 MPa
- 53 Grade OPC – 53 MPa

CHAPTER-3

AGGREGATE

Aggregates are defined as inert, granular, and inorganic materials that normally consist of stone or stone -like solids. Aggregates can be used alone (in road bases and various types of fill) or can be used with cementing materials (such as Portland cement or asphalt cement) to form composite materials or concrete.

Classification of aggregates based on: Grain Size

If you separate aggregates by size, there are two overriding categories:

• Fine aggregates

• Coarse aggregates

The size of fine aggregates is defined as 4.75mm or smaller. That is, aggregates which can be passed through a number 4 sieve, with a mesh size of 4.75mm. Fine aggregates include things such as sand, silt and clay. Crushed stone and crushed gravel might also fall under this category.

Typically, fine aggregates are used to improve workability of a concrete mix.

Coarse aggregates measure above the 4.75mm limit. These are more likely to be natural stone or gravel that has not been crushed or processed. These aggregates will reduce the amount of water needed for a concrete mix, which may also reduce workability but improve its innate strength.

Classification of aggregates based on: Density

There are three weight-based variations of aggregates:

• Lightweight

• Standard

• High density

Different density aggregates will have much different applications. Lightweight and ultra lightweight aggregates are more porous than their heavier counterparts, so they can be put to great use in green roof construction, for example. They are also used in mixes for concrete blocks and pavements, as well as insulation and fireproofing.

High density aggregates are used to form heavyweight concrete. They are used for when high strength, durable concrete structures are required – building foundations or pipe work ballasting, for example.

Classification of aggregates based on: Geographical Origin

Another way to classify aggregates is by their origin. You can do this with two groups:

• Natural – Aggregates taken from natural sources, such as riverbeds, quarries and mines. Sand, gravel, stone and rock are the most common, and these can be fine or coarse.

• Processed – Also called 'artificial aggregates', or 'by-product' aggregates, they are commonly taken from industrial or engineering waste, then treated to form construction aggregates for high quality concrete. Common processed aggregates include industrial slag, as well as burnt clay. Processed aggregates are used for both lightweight and high-density concrete mixes.

Classification of aggregates based on: Shape

Shape is one of the most effective ways of differentiating aggregates. The shape of your chosen aggregates will have a significant effect on the workability of your concrete. Aggregates purchased in batches from a reputable supplier can be consistent in shape, if required, but you can also mix aggregate shapes if you need to.

The different shapes of aggregates are:

• Rounded – Natural aggregates smoothed by weathering, erosion and attrition. Rocks, stone, sand and gravel found in riverbeds are your most common rounded aggregates. Rounded aggregates are the main factor behind workability.

• Irregular – These are also shaped by attrition, but are not fully rounded. These consist of small stones and gravel, and offer reduced workability to rounded aggregates.

• Angular – Used for higher strength concrete, angular aggregates come in the form of crushed rock

and stone. Workability is low, but this can be offset by filling voids with rounded or smaller aggregates.

• Flaky – Defined as aggregates that are thin in comparison to length and width. Increases surface area in a concrete mix.

• Elongated – Also adds more surface area to a mix – meaning more cement paste is needed. Elongated aggregates are longer than they are thick or wide.

• Flaky and elongated – A mix of the previous two – and the least efficient form of aggregate with regards to workability.

Particle shape and Texture:-

Aggregate particle shape and surface texture are important for proper compaction, deformation resistance, and workability. However, the ideal shape for HMA and PCC is different because aggregates serve different purposes in each material. In HMA, since aggregates are relied upon to provide stiffness and strength by interlocking with one another, cubic angular-shaped particles with a rough surface texture are best. However, in PCC, where aggregates are used as an inexpensive highstrength

material to occupy volume, workability is the major issue regarding particle shape. Therefore, in PCC rounded particles are better. Relevant particle shape/texture characteristics are discussed below. Rounded particles create less particle-to-particle interlock than angular particles and thus provide better workability and easier compaction. However, in HMA less interlock is generally a disadvantage as rounded aggregate will continue to compact, shove and rut after construction. Thus angular particles are desirable for HMA (despite their poorer workability), while rounded particles are desirable for PCC because of their better workability.

Flat or Elongated Particles

These particles tend to impede compaction or break during compaction and thus, may decrease strength.

Smooth-Surfaced Particles

These particles have a lower surface-to-volume ratio than rough-surfaced particles and thus may be easier to coat with binder. However, in HMA asphalt tends to bond more effectively with roughsurfaced particles, and in PCC rough-surfaced particles provide more area to which the cement paste can bond. Thus, rough-surface particles are desirable for both HMA and PCC.

ENGINEERING PROPERTIES OF AGGREGATES:-

Aggregates are used in concrete to provide economy in the cost of concrete. Aggregates act as filler only. These do not react with cement and water.

But there are properties or characteristics of aggregate which influence the properties of resulting concrete mix. These are as follow.

1. Composition
2. Size & Shape
3. Surface Texture
4. Specific Gravity
5. Bulk Density
6. Voids
7. Porosity & Absorption
8. Bulking of Sand
9. Fineness Modulus of Aggregate
10. Surface Index of Aggregate
11. Deleterious Material
12. Crushing Value of Aggregate
13. Impact Value of Aggregate

14. Abrasion Value of Aggregate

1. COMPOSITION

Aggregates consisting of materials that can react with alkalies in cement and cause excessive expansion, cracking and deterioration of concrete mix should never be used. Therefore it is required to test aggregates to know whether there is presence of any such constituents in aggregate or not.

2. SIZE & SHAPE

The size and shape of the aggregate particles greatly influence the quantity of cement required in concrete mix and hence ultimately economy of concrete. For the preparation of economical concrete mix on should use largest coarse aggregates feasible for the structure. IS-456 suggests following recommendation to decide the maximum size of coarse aggregate to be used in P.C.C & R.C.C mix. Maximum size of aggregate should be less than

- One-fourth of the minimum dimension of the concrete member.
- One-fifth of the minimum dimension of the reinforced concrete member.
- The minimum clear spacing between reinforced bars or 5 mm less than the minimum cover between the reinforced bars and form, whichever is smaller for heavily reinforced concrete members such as the ribs of the main bars.

Remember that the size & shape of aggregate particles influence the properties of freshly mixed concrete more as compared to those of hardened concrete.

3. SURFACE TEXTURE

The development of hard bond strength between aggregate particles and cement paste depends upon the surface texture, surface roughness and surface porosity of the aggregate particles.

If the surface is rough but porous, maximum bond strength develops. In porous surface aggregates, the bond strength increases due to setting of cement paste in the pores.

4. SPECIFIC GRAVITY

The ratio of weight of oven dried aggregates maintained for 24 hours at a temperature of 100 to 110_0C, to the weight of equal volume of water displaced by saturated dry surface aggregate is known as specific gravity of aggregates.

Specific gravities are primarily of two types.

- Apparent specific gravity
- Bulk specific gravity

Specific gravity is a mean to decide the suitability of the aggregate. Low specific gravity generally indicates porous, weak and absorptive materials, whereas high specific gravity indicates materials of good quality. Specific gravity of major aggregates falls within the range of 2.6 to 2.9.

Specific gravity values are also used while designing concrete mix.

5. BULK DENSITY

It is defined as the weight of the aggregate required to fill a container of unit volume. It is generally expressed in kg/litre.

Bulk density of aggregates depends upon the following 3 factors.

- Degree of compaction
- Grading of aggregates
- Shape of aggregate particles

6. VOIDS

The empty spaces between the aggregate particles are known as voids. The volume of void equals the difference between the gross volume of the aggregate mass and the volume occupied by the particles alone.

7. POROSITY & ABSORPTION

The minute holes formed in rocks during solidification of the molten magma, due to air bubbles, are known as pores. Rocks containing pores are called porous rocks.

Water absorption may be defined as the difference between the weight of very dry aggregates and the weight of the saturated aggregates with surface dry conditions.

Depending upon the amount of moisture content in aggregates, it can exist in any of the 4 conditions.
- Very dry aggregate (having no moisture)
- Dry aggregate (contain some moisture in its pores)
- Saturated surface dry aggregate (pores completely filled with moisture but no moisture on surface)
- Moist or wet aggregates (pores are filled with moisture and also having moisture on surface)

8. BULKING OF SAND

It can be defined as in increase in the bulk volume of the quantity of sand (i.e. fine aggregate) in a moist condition over the volume of the same quantity of dry or completely saturated sand. The ratio of the volume of moist sand due to the volume of sand when dry, is called bulking factor.

Fine sands bulk more than coarse sand

When water is added to dry and loose sand, a thin film of water is formed around the sand particles. Interlocking of air in between the sand particles and the film of water tends to push the particles apart due to surface tension and thus increase the volume. But in case of fully saturated sand the water films are broken and the volume becomes equal to that of dry sand.

9. FINENESS MODULUS

Fineness modulus is an empirical factor obtained by adding the cumulative percentages of aggregate retained on each of the standard sieves ranging from 80 mm to 150 micron and dividing this sum by 100.

Fineness modulus is generally used to get an idea of how coarse or fine the aggregate is. More fineness modulus value indicates that the aggregate is coarser and small value of fineness modulus indicates that the aggregate is finer.

10. SPECIFIC SURFACE OF AGGREGATE

The surface area per unit weight of the material is termed as specific surface. This is an indirect measure of the aggregate grading. Specific surface increases with the reduction in the size of aggregate particle. The specific surface area of the fine aggregate is very much more than that of coarse aggregate.

11. DELETERIOUS MATERIALS

Aggregates should not contain any harmful material in such a quantity so as to affect the strength and durability of the concrete. Such harmful materials are called deleterious materials. Deleterious materials may cause one of the following effects
- To interfere hydration of cement
- To prevent development of proper bond
- To reduce strength and durability
- To modify setting times

Deleterious materials generally found in aggregates, may be grouped as under
- Organic impurities
- Clay , silt & dust
- Salt contamination

12. CRUSHING VALUE

The aggregates crushing value gives a relative measure of resistance of an aggregate to crushing under gradually applied compressive load. The aggregate crushing strength value is a useful factor to know the behaviour of aggregates when subjected to compressive loads.

13. IMPACT VALUE

The aggregate impact value gives a relative measure of the resistance of an aggregate to sudden shock or impact. The impact value of an aggregate is sometime used as an alternative to its crushing value.

14. ABRASION VALUE OF AGGREGATES

The abrasion value gives a relative measure of resistance of an aggregate to wear when it is rotated in a cylinder along with some abrasive charge.

SIEVE ANALYSIS:-

Sieve analysis is a technique used to determine the particle size distribution of a powder. This method is performed by sifting a powder sample through a stack of wire mesh sieves, separating it into discrete size ranges. A sieve shaker is used to vibrate the sieve stack for a specific period of time.

Sieve analysis is important for analyzing materials because particle size distribution can affect a wide range of properties, such as the strength of concrete, the solubility of a mixture, surface area properties and even their taste.

FINENESS MODULUS:-

The *Fineness Modulus* (FM) is an empirical figure obtained by adding the total percentage of the sample of an aggregate retained on each of a specified series of sieves, and dividing the sum by 100. Fineness modulus of sand (fine aggregate) is an index number which represents the mean size of the particles in sand. It is calculated by performing sieve analysis with standard sieves. The cumulative percentage retained on each sieve is added and subtracted by 100 gives the value of fineness modulus. Fineness modulus is generally used to get an idea of how coarse or fine the aggregate is. More fineness modulus value indicates that the aggregate is coarser and small value of fineness modulus indicates that the aggregate is finer.

THE GRADING CURVE:-

The grading curve graphically represents the proportion of different grain sizes which the aggregate is composed of and which form part of the shortcrete mix. It provides useful information to find out: Whether the distribution of the different aggregate sizes is suitable for pumping.

GRADING OF AGGREGATES:-

Grading of aggregates is determining the average grain size of the aggregates before they are used in construction. This is applied to both coarse and fine aggregates. The aggregate sample is sieved through a set of sieves and weights retained on each sieve in percentage terms are summed up.

GAP GRADING AGGREGATE:-

Gap grading is defined as a grading in which one or more intermediate size fractions are absent. On a grading curve, it represents a horizontal line over the range of sizes that are absent.

MAXIMUM AGGREGATE SIZE:-

Typically, coarse aggregate sizes are larger than 4.75 mm (5 mm in British code), while fine aggregates form the portion below 4.75 mm. A maximum size up to 40 mm is used for coarse aggregate in most structural applications, while for mass concreting purposes such as dams, sizes up to 150 mm may be used.

ADMIXTURE

Admixture is a compound which is used in order to increase or decrease the initial and final setting time of cement.

Admixture and its types

Chemical admixtures are the ingredients in concrete other than Portland cement, water, and aggregate those are added to the mix immediately before or during mixing. Producers use admixtures primarily to reduce the cost of concrete construction; to modify the properties of hardened concrete; to ensure the quality of concrete during mixing, transporting, placing, and curing; and to overcome certain emergencies during concrete operations.

It is of various types-

Water-reducing admixtures usually reduce the required water content for a concrete mixture by about 5 to 10 percent. Consequently, concrete containing a water-reducing admixture needs less water to reach

a required slump than untreated concrete. The treated concrete can have a lower water-cement ratio. This usually indicates that a higher strength concrete can be produced without increasing the amount of cement. Recent advancements in admixture technology have led to the development of mid-range water reducers. These admixtures reduce water content by at least 8 percent and tend to be more stable over a wider range of temperatures. Mid-range water reducers provide more consistent setting times than standard water reducers.

Retarding admixtures, which slow the setting rate of concrete, are used to counteract the accelerating effect of hot weather on concrete setting. High temperatures often cause an increased rate of hardening which makes placing and finishing difficult. Retarders keep concrete workable during placement and delay the initial set of concrete. Most retarders also function as water reducers and may entrain some air in concrete.

Accelerating admixtures increase the rate of early strength development; reduce the time required for proper curing and protection, and speed up the start of finishing operations. Accelerating admixtures are especially useful for modifying the properties of concrete in cold weather.

Super plasticizers, also known as plasticizers or high-range water reducers (HRWR), reduce water content by 12 to 30 percent and can be added to concrete with a low-to-normal slump and watercement ratio to make high-slump flowing concrete. Flowing concrete is a highly fluid but workable concrete that can be placed with little or no vibration or compaction. The effect of super plasticizers lasts only 30 to 60 minutes, depending on the brand and dosage rate, and is followed by a rapid loss in workability. As a result of the slump loss, super plasticizers are usually added to concrete at the jobsite.

Corrosion-inhibiting admixtures fall into the specialty admixture category and are used to slow corrosion of reinforcing steel in concrete. Corrosion inhibitors can be used as a defensive strategy for concrete structures, such as marine facilities, highway bridges, and parking garages, that will be exposed to high concentrations of chloride. Other specialty admixtures include shrinkage-reducing admixtures and alkali-silica reactivity inhibitors. The shrinkage reducers are used to control drying shrinkage and minimize cracking, while ASR inhibitors control durability problems associated with alkali-silica reactivity.

DOSASES:-

Super plasticizers are commonly known as High Range Water Reducers because it permits low water cement ratio as well as the workability also affected. In very recent decades, super plasticizers creates milestone in the advancement of chemical admixtures for Portland cement concrete. The dramatic effect of super plasticizer (SP) on properties of fresh and hardened concrete has studied and the properties of concrete inspected are compressive strength and slump test. To determine the optimum dosage for the admixture, an experimental investigation conducted and the effect of over dosage of the SP admixture experimented, together with one control mixed. The viscosity of grout and hence the workability of concrete influenced by the dosage of super plasticizer. From dosages of admixture, the difference between concrete mixes comes, which used at amounts 400 ml/100 kg, 600 ml/100 kg, 800 ml/100 kg, 1000 ml/100 kg and 1200 ml/100 kg of cement were prepared. By dosage 1.0% of SP, compressive strength is improved and after 28 days curing it is 57 N/mm2 , which is higher than that of control specimen.

CHAPTER-4

FRESH CONCRETE

Workability is the property of concrete which determines the amount of internal work necessary to produce full compaction. It is a measure with which concrete can be handled from the mixer stage to its final fully compacted stage.

FACTORS AFFECTING WORKABILITY OF CONCRETE:-

01. Water Content of the Concrete Mix:

Water content will have important influences on the workability in given volume of concrete. The higher the water content per cubic meter of concrete, the higher will be the fluidity of concrete, which affect the workability.

Water requirement is mainly associated with absorption by aggregates surface& filling up the voids between aggregates.

• The strength of the concrete may get reduced.

• More quantity of water comes out from the surface of concrete resulting into bleeding.

• Cement slurry also escapes through the joints of formwork resulting into the loss of cement from concrete.

02. The Size of Aggregates:

Workability is mainly governed by the maximum size of aggregates. Water and paste require, will be not less if a chosen size of aggregates for concrete is bigger. Consequently, for a given quantity of water content & paste, bigger size aggregate will give higher workability.

Note: On the site, the maximum size of aggregate to be used will depend upon the many factors such as the handling, mixing and placing equipment, the thickness of section and quantity of reinforcement. Later two are very important.

03. The shape of Aggregates:

Angular, flaky & elongated aggregate reduces the workability of concrete.

Rounded or sub-rounded aggregates increase the workability due to the reduction of surface area for a given volume or weight. Therefore, an excess paste is available to give better lubricating effect.

Rounded shape aggregate has less frictional resistance and gives a high workability as compared to angular, flaky or elongated aggregates.

Note: River sand & gravel provide greater workability to concrete than crushed sand.

04. Surface Texture of Aggregates:

The roughly textured aggregates have more surface area than smoothly rounded aggregates of the same volume. Smooth rounded or glassy aggregates will give better workability than roughly textured aggregates. A reduction of inter particle frictional resistance offered by smooth aggregates also contributes to higher workability.

05. The Porosity of Aggregates:

Porous and non-saturated aggregate will require more water than non-absorbent aggregates. For the same degree of workability, latter will require less water. Overall, this factor is only of secondary importance.

06. Grading of Aggregates:

Grading of aggregates has the greatest influence on workability. The better the grading of aggregates, the less is the amount of void in concrete so well-graded aggregates should be used. When total voids are less in concrete, the excess paste is available to give better lubricating effect.

With excess amount of concrete paste present in the mixture, it becomes cohesive and fatty that prevents segregation of particles & least amount of compacting efforts is required to compact the concrete.

For a given workability, there is one value of coarse aggregate / Fine aggregate ratio, which needs the lower water content.

07. Uses of Concrete Admixtures:

This is one of the commonly used methods to enhance the workability of concrete. Concrete admixtures such as plasticizer and super plasticizers greatly improve the workability.

Air entraining agents are also used to increase the workability. Air entraining agents creates a large number of very tiny air bubbles. These bubbles get distributed throughout the mass of concrete and act as rollers and increase the workability.

Mineral admixtures like Pozzolanic materials are also used to improve the workability of concrete.

08. Ambient Temperature:

In hot weather, if temperature increases, the evaporation rate of mixing water also increases and hence fluid viscosity increases, too. This phenomenon affects the flowability of concrete and due to fast hydration of concrete; it will gain strength earlier which decreases the workability of fresh concrete.

Measurement of Workability:-

The followings tests for workability of concrete gives a measure of workability, which is applicable specifically concerning some particular methods. They bear no relationship to any of the common methods of placing and compacting concrete. So, the test results are only relative and should not be given any absolute measurement. We need to understand that each test has their importance, and as such there is no unique test to measure the workability of concrete in total. The significant advantage is the simplicity of the procedure with an ability to detect variation in the uniformity of a mix of given nominal proportion.

Types of tests for workability:-

- Slump Test
- Compacting Factor Test
- Flow Test
- Vee-Bee Consistometer Test

SLUMP TEST:-

The slump test is a means of assessing the consistency of fresh concrete. It is used, indirectly, as a means of checking that the correct amount of water has been added to the mix.

The steel slump cone is placed on a solid, impermeable, level base and filled with the fresh concrete in three equal layers. Each layer is rodded 25 times to ensure compaction. The third layer is finished off level with the top of the cone. The cone is carefully lifted up, leaving a heap of concrete that settles or 'slumps' slightly. The upturned slump cone is placed on the base to act as a reference, and the difference in level between its top and the top of the concrete is measured and recorded to the nearest 10mm to give the slump of the concrete.

When the cone is removed, the slump may take one of three forms. In a true slump the concrete simply subsides, keeping more or less to shape. In a shear slump the top portion of the concrete shears off and slips sideways. In a collapse slump the concrete collapses completely. Only a true slump is of any use in the test. If a shear or collapse slump is achieved, a fresh sample should be taken and the test repeated. A collapse slump will generally mean that the mix is too wet or that it is a high workability mix, for which the flow test (see separate entry) is more appropriate.

- Compacting Factor Test:-

The compacting factor test is designed primarily for use in the laboratory but it can also be used in the field. It is more precise and sensitive than the slump test and is particularly useful for concrete mixes of very low workability as are normally used when concrete is to be compacted by vibration. Such dry concrete are insensitive to slump test.

The degree of compaction, called the compacting factor is measured by the density ratio i.e., the ratio of the density actually achieved in the test to density of same concrete fully compacted. The sample of concrete to be tested is placed in the upp r hopper up to the brim. The trap-door is opened so that the

concrete falls into the low r hopper. Then the trap-door of the lower hopper is opened and the concrete is allowed to fall into the cylinder. In the case of a dry-mix, it is likely that the concrete may not fall on opening the trap-door. In such a case, a slight poking by a rod may be required to set the concrete in motion. The excess concrete remaining above the top level of the cylinder is then cut off with the help of plane blades supplied with the apparatus. The outside of the cylinder is wiped clean. The concrete is filled up exactly up to the top level of the cylinder. It is weighed to the nearest 10 grams. This weight is known as ―Weight of partially compacted concrete.

The cylinder is emptied and then refilled with the concrete from the same sample in layers approximately 5 cm deep. The layers are heavily rammed or preferably vibrated so as to obtain full compaction. The top surface of the fully compacted concrete is then carefully struck off level with the top of the cylinder and weighed to the nearest 10 gm. This weight is known as Weight of fully compacted concrete.

The compaction factor =Weight of partially compacted concrete Weight of fully compacted concrete

FLOW TEST

This is a laboratory test, which gives an indication of the quality of concrete with respect to consistency, cohesiveness a d the proneness to segregation. In this test, a standard mass of concrete is subjected to joint. The spread or the flow of the concrete is measured and this flow is related to workability.

It can be seen that the apparatus consists of flow table, about 76 cm. in diameter over which concentric circles are marked. A mould made from smooth metal casting in the form of a frustum of a cone is used with the following internal dimensions. The base is 25 cm. in diameter, upper surface 17 cm. in diameter, and height of the cone is 12 cm. The table top is cleaned of all gritty material and is wetted. The mould is kept on the centre of the table, firmly held and is filled in two layers. Each layer is rodded 25 times with a tamping rod 1.6 cm in diameter and 61 cm long rounded at the lower tamping end. After the top layer is rodded evenly, the excess of concrete which has over flowed the mould is removed. The mould is lifted vertically upward and the concrete stands on its own without support. The table is then raised and dropped 12.5 mm 15 times in about 15 seconds. The diameter of the spread concrete is measured in about 6 directions to the nearest 5 mm and the average spread is noted. The flow of concrete is the percentage increase in the average diameter of the spread concrete over the base diameter of the mould.

Flow per cent =Spread diameter • in cm – 25 X 100

25

The value could range anything from 0 to 150 per cent. A close look at the pattern of spread of concrete can also give a good indication of the characteristics of concrete such as tendency for segregation.

• VEE-BEE CONSISTOMETER TEST

This is a good laboratory test to measure indirectly the workability of concrete. This test metal cylindrical pot of the con istomeconsists of a vibrating table, a metal pot, a sheet metal cone, a standard iron rod. Slump test as described earlier is performed, placing the slump cone inside the sheetter. The glass disc attached to the swivel arm is turned and placed on the top of the concrete in the pot. The electrical vibrator is then switched on and simultaneously a stop watch started.

The vibration is continued till such a time as the conical shape of the concrete disappears and the concrete assumes a cylindrical shape. This can be judged by observing the glass disc from the top for disappearance of transparency. Immediately when the concrete fully assumes a cylindrical shape, the stop watch is switched off. The time required for the shape of concrete to change from slump cone shape to cylindrical shape in seconds is known as Vee Bee Degree. This method is very suitable for very dry concrete whose slump value cannot be measured by Slump Test but the vibration is too vigorous for concrete with a slump greater than about 50 mm.

SETTING TIME OF CEMENT:-

The concrete setting time mostly depends upon the w/c ratio, temperature conditions, type of cement, use of mineral admixture, use of plasticizer, in particular, retarding plasticizer. The significance of setting parameter of concrete is more important for site engineers than setting time of cement. For keeping the concrete we use retarding plasticizers, which increases setting time and the duration up to which concrete remains in the plastic condition is of special interest.

The concrete setting time is determined by using a penetrometer test.

The test procedure involves,

1. Taking a sufficient quantity of fresh concrete mix sample and sieves it through 4.75 mm sieve.

2. The mortar sample passed through the sieve is collected.

3. This mortar is then compacted by rodding, tapping, rocking or by vibrating.

4. Level the surface and keep it covered to prevent the loss of moisture.

5. Remove bleeding water, if any, using a pipette. Insert a needle of appropriate size, depending upon the degree of the setting of the mortar in the following manner.

6. Bring the bearing surface of the needle in contact with the mortar surface.

7. Gradually and uniformly apply a vertical force downwards on the apparatus until the needle penetrates to a depth of 25 ± 1.5 on, as indicated by the scribe mark.

8. The time is taken to penetrate 25 mm depth could be about 10 seconds.

9. Record the force required to produce 25 mm penetration and the time of inserting from the to water is added to the cement.

10. Calculate the penetration resistance by dividing the recorded force by the bearing area of the needle. This is the penetration resistance.

- Plot a graph of penetration resistance as ordinate and elapsed time as abscissa.
- Test conducted must determine 6 penetration resistances.
- Continue the tests until one penetration resistance of at least 27.6 MPa is reached.
- Plot these penetration resistance values on the graph and connect each point.
- Now draw a horizontal line from penetration resistance equal to 3.5 MPa.
- The point of intersection of this with the smooth curve is read on the x-axis which gives the initial setting time.
- Similarly, a horizontal line is drawn from the penetration resistance of 27.6 MPa and point it cuts the smooth curve is read on the x-axis which gives the final set.

Effect of time and temp in workability:-

Temperature decreases the setting time by increasing hydration rate and that increase the early age strength of the concrete. This is an advantage that less time will be required before removing of form works on site, but this decrease the use of proper placement of concrete in the initial stages.

It indicates that the temperature has a negative effect on the workability of concrete as well as strength up to some extent. Temperature decreases the setting time by increasing hydration rate and that increase the early age strength of the concrete.

This is an advantage that less time will be required before removing of form works on site, but this decrease the use of proper placement of concrete in the initial stages. And if concrete is not properly laid, then strength distribution will not remain the same throughout the cross-section.

SEGREGATION:-

The tendency of separation of coarse aggregates grains from te concrete mass is called segregation.

BLEEDING:-

The tendency of water to rise to the surface of freshly laid concrete is known as bleeding.

MIXING AND VIBRATION OF CONCRETE:

Mixing is the uniform incorporation of the ingredients within the concrete mix and vibration usually means the mechanical process to assist in the removal of any entrapped air. The air entrapment causes a honeycomb effect which weakens the concrete,

There are calculations and processes for concrete to allow for movement, which often translates to vibration due to friction or the dissimilarity of materials, a serious concern as it would be the cause for structural fatigue and failure.

The stages of concrete production are:

1. Batching or measurement of materials
2. Mixing
3. Transporting
4. Placing
5. Compacting
6. Curing
7. Finishing

Batching of Materials

For good quality concrete a proper and accurate quantity of all the ingredients should be used.

The aggregates, cement and water should be measured wit h an accuracy of 3 per cent of batch quantity and the admixtures by 5 per cent of the batch quantity. There are two prevalent methods of batching materials, the volume batching and the weigh batching. The factors affecting the choice of batching method are the size of job, required production rate, and required standards of batching performance. For most important works weigh batching is recommended.

a) Volume Batching
b) Weigh Batching

Mixing

1. Hand Mixing
2. Machine Mixing

a) Tilting Mixers
b) Non-tilting Mixer
c) Reversing Drum Mixer
d) Pan-type or Stirring Mixer
e) Transit Mixer

Charging the Mixer and Mixing Time

The order of feeding the ingredients into the mixer is as follows:

About 25 per cent of water required for mixing is first introduced into the mixer drum to prevent any sticking of cement on the blades and bottom of the drum. Then the ingredients are discharged through the skip. In the skip the sequence of loading should be to add first half the coarse aggregate then half the fine aggregate and over this total cement and then the balance aggregates. After discharging the ingredients into the drum the balance water is introduced. The mixing time is counted from the instant complete water is fed into the mixer.

The speed of the mixers is generally 15 to 20 rpm. For proper mixing, the number of revolutions per minute required by the drum are 25 to 30. Time of mixing also depends on capacity of mixer.

A poor quality of concrete is obtained if the mixing time is reduced. On the other hand if the mixing time is increased it is uneconomical. However, it is found that if the mixing time is increased to 2 minutes the compressive strength of concrete produced is enhanced and beyond this time the improvement in compressive strength is insignificant. A prolonged mixing may cause segregation. Also, due to longer mixing periods the water may get absorbed by the aggregates or evaporate resulting in loss of workability and strength.

Transporting

Concrete should be transported to the place of deposition at the earliest without the loss of homogeneity obtained at the time of mixing. A maximum of 2 hours from the time of mixing is permitted if trucks with agitator and 1 hour if trucks without agitators are used for transporting

concrete. Also it should be ensured that segregation does not take place during transportation and placement. The methods adopted for transporting concrete depend upon the size and importance of the job, the distance of the deposition place from the mixing place, and the nature of the terrain. Some of the methods of transporting concrete are as below:

a. Mortar Pan
b. Wheel Barrow
c. Chutes
d. Dumper
e. Bucket and Ropeway
f. Belt conveyor
g. Skip and Hoist
h. Pumping

Placing:

To achieve quality concrete it should be placed with utmost care securing the homogeneity achieved during mixing and the avoidance of segregation in transporting. Research has shown that a delayed placing of concrete results in a gain in ultimate compressive strength provided the concrete can be adequately compacted. For dry mixes in hot weather delay of half to one hour is allowed whereas for wet mixes in cold weather it may be several hours.

Compaction

After concrete is placed at the desired location, the next step in the process of concrete production is its compaction. Compaction consolidates fresh concrete within the moulds or frameworks and around embedded parts and reinforcement steel. Considerable quantity of air is entrapped in concrete during its production and there is possible partial segregation also. Both of these adversely affect the quality of concrete. Compaction of the concrete is the process to get rid of the entrapped air and voids, elimination of segregation occurred and to form a homogeneous dense mass. It has been found that 5 per cent voids in hardened concrete reduce the strength by over 30 per cent and 10 per cent voids reduce the strength by over 50 per cent. Therefore, the density and consequently the strength and durability of concrete largely depend upon the degree of compaction. For maximum strength driest possible concrete should be compacted 100 per cent.

The compaction of concrete can be achieved by the following methods.

1. Hand Compaction
2. Compaction by Vibration
a. Needle Vibrator:
b. Formwork Vibrator
3. Compaction by Spinning
4. Compaction by Jolting
5. Compaction by Rolling

Curing

Cement gains strength and hardness because of the chemical action between cement and water. This chemical reaction requires moisture, favourable temperature and time referred to as the curing period. Curing of freshly placed concrete is very important for optimum strength and durability. The major part of the strength in the initial period is contributed by the clinker compound C_3S and partly by C_2S, and is completed in about three weeks. The later strength contributed by C_2S is gradual and takes long time. As such sufficient water should be made available to concrete to allow it to gain full strength. *The process of keeping concrete damp for this purpose is known as curing*. The object is to prevent the loss of moisture from concrete due to evaporation or any other reason, supply additional moisture or heat and moisture to accelerate the gain of strength. Curing must be done for at least three weeks and in no case for less than ten days.

Approximately 14 litres of water is required to hydrate each bag of cement. Soon after the concrete is placed, the increase in strength is very rapid (3 to 7 days) and continues slowly thereafter for an indefinite period. Concrete moist cured for 7 days is about 50 per cent stronger than that which is exposed to dry air for the entire period. If the concrete is kept damp for one month, the strength is about double than that of concrete exposed only to dry air.

Methods of Curing:

Concrete may be kept moist by a number of ways. The methods consist in either supplying additional moisture to concrete during early hardening period by ponding, spraying, sprinkling, etc. or by preventing loss of moisture from concrete by sealing the surface of concrete by membrane formed by curing compounds. Following are some of the prevalent methods of curing.

1. Water Curing
2. Steam Curing
3. Curing by Infra Red Radiation:
4. Electrical Curing
5. Chemical Curing:

Finishing:

Concrete is basically used because of its high compressive strength. However, the finish of the ultimate product is not that pleasant. In past couple of decades efforts have been made to develop surface finishes to give a better appearance to concrete surfaces and are as follows.

1. Formwork Finishes
2. Surface Treatments
3. Applied Finishes

QUALITY OF MIXING WATER:-

The common specifications regarding quality of mixing water is water should be fit for drinking. Such water should have inorganic solid less than 1000 ppm. This content lead to a solid quantity 0.05% of mass of cement when w/c ratio is provided 0.5 resulting small effect on strength.

CHAPTER-5

HARDENED CONCRETE

WATER CEMENT RATIO

The water–cement ratio is the ratio of the weight of water to the weight of cement used in a concrete mix. A lower ratio leads to higher strength and durability, but may make the mix difficult to work with and form. Workability can be resolved with the use of plasticizers or super-plasticizers. A maximum of 0.5 ratio when concrete is exposed to freezing and thawing in a moist condition or to de-icing chemicals, and a maximum of 0.45 ratio for concrete in a severe or very severe sulphate condition.

However, a mix with a ratio of 0.35 may not mix thoroughly, and may not flow well enough to be placed. More water is therefore used than is technically necessary to react with cement. Water–cement ratios of 0.45 to 0.60 are more typically used. For higher-strength concrete, lower ratios are used, along with a plasticizer to increase flow ability.

Nature of strength of concrete:-

The strength can be defined as the ability to resist force. With-regard to concrete for structural purposes it can be defined as the unit force required to cause rupture. Strength is a good index of most of the other properties of practical importance. In general stronger concretes are stiffer, more water tight and more resistant to weathering etc.

Rupture of concrete may be caused by applied tensile stress, shearing stress or by compressive stress or a combination of two of the above stresses. Concrete being a brittle material is much weaker in tension and shear than compression and failures of concrete specimens under compressive load are essentially shear failures on oblique planes as shown in fig.

It is called as shear or cone failure. As the resistance to failure is due to both cohesion and internal friction, the angle of rupture is not 45° (plane of maximum shear stress), but is a function of the angle of internal friction. It can be shown mathematically that the angle ϕ which the plane of failure makes with the axis of loading is equal to $(45° - \phi/2)$ as shown in fig.

The angle of internal friction ϕ of concrete being of the order of 20°, the angle of inclination of the cone of failure in the conventional test specimen is approximately 35° as shown in fig.

Maturity of concrete:-

Concrete maturity is an index value that represents the progression of concrete curing. It is based on an equation that takes into account concrete temperature, time, and strength gain. Concrete maturity is an accurate way to determine real-time strength values of curing concrete.

Tension Vs. Compression of Concrete:-

Concrete has enormous compressive strength, the ability to withstand heavy weights or forces on it. It also gains strength as it ages. Concrete will solidify in a few hours and harden or set in a few days, but continues to gain strength for at least 28 days. Some very thick concrete structures, like dams, will continue to gain strength for months or years.

Concrete has almost no tensile strength, the ability to withstand pressing or stretching. Put a board between two supports and press down on the centre. It will bend. The top of the board is under compression, the bottom which bends is under tension. Concrete can resist the compression, but will break under the tension. Concrete cracks in roads and slabs are largely due to tension; different weights in different areas produce tensile forces.

The tension to compression ratio for concrete is about 10 to 15 percent. That is, it can withstand about 10 times the pushing force or compression of the pulling force or tension. Both strengths increase with age, but the ratio is steady. Portland cement concrete less than a year old has compression strength of 1,000 pounds per square inch (psi) and tension strength of 200 psi. Concrete more than a year old has compression psi of 2,000 pounds and tension psi of 400.

Factors affecting strength of concrete:
Concrete strength is affected by many factors, such as quality of raw materials, water/cement ratio, coarse/fine aggregate ratio, age of concrete, compaction of concrete, temperature, relative humidity and curing of concrete.

Relation between tensile and compressive strength of concrete:-
The theoretical compressive strength of concrete is eight times larger than its tensile strength. This implies a fixed relation between the compressive and tensile strength of concrete. In fact there is a close relation but not a direct proportionality. The ratio of tensile to compressive strength is lower for higher compressive strengths.

This may be due to the following two reasons:
(a) Formation of inferior quality gel due to improper curing.
(b) Development of more shrinkage cracks due to improper curing. The uses of pozzolanic materials have shown the increase in tensile strength.

Curing of Concrete:-
Curing of Concrete is a method by which the concrete is protected against loss of moisture required for hydration and kept within the recommended temperature range. Curing will increase the strength and decrease the permeability of hardened concrete.

TESTING OF HARDENED CONCRETE

The compressive strength of concrete is considered the basic character of the concrete. Consequently, it is known as the characteristic compressive strength of concrete (fck) which is defined as that value below which not more than five percent of test results are expected to fall based on IS: 456-2000. In this definition the test results are based on 150 mm cube cured in water under temp. of 27 ± 2°C for 28 days and tested in the most saturated condition under direct compression.

Other strength like, direct tensile stress, flexural stress, shear stress and bond stress also are directly proportional to the compressive stress. Higher is the compressive stress, higher is other stresses also. Not only stresses, other properties for example modulus of elasticity, abrasion and impact resistances, durability are also taken to be related to the compressive strength, hence, the compressive strength is an index of overall quality of concrete.

Factors Affecting Compressive Strength:-
Among the materials and mix variables, water -cement ratio is the most important parameter governing the compressive strength. Besides W/C ratio, following factors also affect the compressive strength.
I. The characteristics of cement.
II. The characteristics and properties of aggregates.
III. The degree of compaction
IV. The efficiency of curing
V. Age at the time of testing.
VI. Conditions of testing.

TENSILE STRENGTH:
• Tensile strength of concrete under direct tension is very small and generally neglected in normal design practice. Although the value ranges from 8 to 12% of its compressive strength. An average value 10% is the proper choice. The direct tension method suffers the problem like holding the specimen properly in the testing machine and the application of uniaxial tensile load not being free of eccentricity.

• The tensile strength can be calculated indirectly by loading a concrete cylinder to the compressive force along the two opposite ends (with its axis horizontal)

• Due to uniform tensile stress acting horizontally along the length of cylinder, the cylinder splits into two halves. The magnitude of this tensile stress (acting in a direction perpendicular to the

line of action of applied compression) is given by

The indirect tensile stress is known as splitting tensile strength.

FLEXURAL STRENGTH

• The maximum tensile stress resisted by the plain concrete in flexure (bending) is called flexural strength (or modulus of rupture) expressed in N/mm 2 or kg/m 2.

• The most common plain concrete subjected to flexure is a highway/runway pavement. The strength of pavement concrete is evaluated by means of bending on beam specimen.

• The flexural strength (modulus of rupture) is determined by testing standard test specimens of 150 mm x 150 mm x 700 mm over a span of 600 mm or 100 mm x 100 mm x 500 mm over a span of 400 min. under symmetrical two point loading.

SHEAR STRENGTH:

• Shear strength is the capacity of concrete to resist the sliding of the section over the adjacent section. A good amount of shear strength capacity is possessed by concrete depending upon the grade of concrete and percentage of tensile reinforcement in the section.

• It is difficult to obtain shear strength of concrete but I.S. code suggests the value for different grade of concrete.

PULL OUT TEST OF CONCRETE:

The pullout test produces a well defined in the concrete and measure a static strength property of concrete. The equipment is simple to assemble and operate. The compressive strength can be considered as proportional to the ultimate pullout force. The reliability of the test is reported as good.

NON-DESTRUCTIVE TESTING OF CONCRETE (NDT ON CONCRETE):-

Non destructive test is a method of testing existing concrete structures to assess the strength and durability of concrete structure. In the non destructive method of testing, without loading the specimen to failure (i.e. without destructing the concrete) we can measure strength of concrete. Now days this method has become a part of quality control process. This method of testing also helps us to investigate crack depth, micro cracks and deterioration of concrete.

Non destructive testing of concrete is a very simple method of testing but it requires skilled and experienced persons having some special knowledge to interpret and analyze test results. DIFFERENT METHODS OF NON-DESTRUCTIVE TESTING OF CONCRETE:-

Various non-destructive methods of testing concrete have been developed to analyze properties of hardened concrete, which are given below.

1. SURFACE HARDNESS TEST

These are of indentation type, include the Williams testing pistol and impact hammers, and are used only for estimation of concrete strength.

2. REBOUND HAMMER TEST

The rebound hammer test measures the elastic rebound of concrete and is primarily used for estimation of concrete strength and for comparative investigation.

3. PENETRATION AND PULLOUT TECHNIQUES

These include the use of the simbi hammer, spit pins, the Windsor probe, and the pullout test. These measure the penetration and pullout resistance of concrete and are used for strength estimation, but they can also be used for comparative studies.

NDT test on concrete

4. DYNAMIC OR VIBRATION TESTS

These include resonant frequency and mechanical sonic and ultrasonic pulse velocity methods. These are used to evaluate durability and uniformity of concrete and to estimate its strength and elastic properties.

5. COMBINED METHODS

The combined methods involving ultrasonic pulse velocity and rebound hammer have been used to estimate strength of concrete.

6. RADIOACTIVE AND NUCLEAR METHODS

These include the X-ray and Gamma ray penetration tests for measurement of density and thickness of concrete. Also, the neutron scattering and neutron activation methods are used for moisture and cement content determination.

7. MAGNETIC AND ELECTRICAL METHODS

The magnetic methods are primarily concerned with determining cover of reinforcement in concrete, whereas the electrical methods, including microwave absorption techniques, have been used to measure moisture content and thickness of concrete.

8. ACOUSTIC EMISSION TECHNIQUES

These have been used to study the initiation and growth of cracks in concrete.

ELASTICITY, CREEP, SHRINKAGE

Modulus of Elasticity of Concrete can be defined as the slope of the line drawn from stress of zero to a compressive stress of $0.45f'_c$. As concrete is a heterogeneous material. The strength of concrete is dependent on the relative proportion and modulus of elasticity of the aggregate.

Dynamic modulus is the ratio of stress to strain under vibratory conditions (calculated from data obtained from either free or forced vibration tests, in shear, compression, or elongation). It is a property of visco elastic materials.

Dynamic Modulus:-

The value of modulus of elasticity E_c determined by actual loading of concrete is known as static modulus of elasticity. This method of testing is known as destructive method as the specimen is stressed or loaded till its failure. The static modulus of elasticity does not represent the true elastic behaviour of concrete due to the phenomenon of creep. At higher stresses the modulus of elasticity is affected more seriously.

Thus a non-destructive method of testing known as dynamic method is adopted for determining the modulus of elasticity. In this case no stress is applied on the specimen. The modulus of elasticity is determined by subjecting the specimen to longitudinal vibration at their natural frequency that is why this is known as dynamic modulus.

Poisson's ratio:-

Poisson's ratio is the ratio of lateral strain to longitudinal strain in a material subjected to loading. Poisson's ratio varies between 0.1 for high strength concrete and 0.2 for weak mixes. It is normally taken as 0.15 for strength design and 0.2 for serviceability criteria.

Creep:-

Creep in concrete is defined as the deformation of structure under sustained load. Basically, long term pressure or stress on concrete can make it change shape. This deformation usually occurs in the direction the force is being applied. Like a concrete column getting more compressed, or a beam bending. Creep does not necessarily cause concrete to fail or break apart. When a load is applied to concrete, it experiences an instantaneous elastic strain which develops into creep strain if the load is sustained.

Creep is factored in when concrete structures are designed.

Factors Affecting Creep:-

These are the factors which affects creep of concrete.

1. Aggregate
2. Mix Proportions
3. Age of concrete

The magnitude of creep strain is one to three times the value of the instantaneous elastic strain, it is

proportional to cement-paste content and, thus, inversely proportional to aggregate volumetric content. The magnitude of creep is dependent upon the magnitude of the applied stress, the age and strength of the concrete, properties of aggregates and cementitious materials, amount of cement paste, size and shape of concrete specimen, volume to surface ratio, amount of steel reinforcement, curing conditions, and environmental conditions.

1. Influence of Aggregate:

Aggregate undergoes very little creep. It is really the paste which is responsible for the creep. However, the aggregate influences the creep of concrete through a restraining effect on the magnitude of creep. The paste which is creeping under load is restrained by aggregate which do not creep. The stronger the aggregate the more is the restraining effect and hence the less is the magnitude of creep. An increase from 65 to 75 % of volumetric content of the aggregate will decrease the creep by 10 %.

The modulus of elasticity of aggregate is one of the important factors influencing creep. It can be easily imagined that the higher the modulus of elasticity the less is the creep. Light weight aggregate shows substantially higher creep than normal weight aggregate.

2. Influence of Mix Proportions:

The amount of paste content and its quality is one of the most important factors influencing creep. A poorer paste structure undergoes higher creep. Therefore, it can be said that creep increases with increase in water/cement ratio. In other words, it can also be said that creep is inversely proportional to the strength of concrete. Broadly speaking, all other factors which are affecting the water/cement ratio are also affecting the creep.

3. Influence of Age:

Age at which a concrete member is loaded will have a predominant effect on the magnitude of creep. This can be easily understood from the fact that the quality of gel improves with time. Such gel creeps less, whereas a young gel under load being not so stronger creeps more. What is said above is not a very accurate statement because of the fact that the moisture content of the concrete being different at different age also influences the magnitude of creep.

Unlike brittle fracture, creep deformation does not occur suddenly upon the application of stress. Instead, strain accumulates as a result of long-term stress. Therefore, creep is a "time-dependent" deformation. It works on the principle of Hooke's law (stress is directly proportional to strain).

Effects of Creep on Concrete and Reinforced Concrete:-

• In reinforced concrete beams, creep increases the deflection with time and may be a critical consideration in design.

• In eccentrically loaded columns, creep increases the deflection and can load to buckling.

• In case of statically indeterminate structures and column and beam junctions creep may relieve the stress concentration induced by shrinkage, temperatures changes or movement of support. Creep property of concrete will be useful in all concrete structures to reduce the internal stresses due to non-uniform load or restrained shrinkage.

• In mass concrete structures such as dams, on account of differential temperature conditions at the interior and surface, creep is harmful and by itself may be a cause of cracking in the interior of dams. Therefore, all precautions and steps must be taken to see that increase in temperature does not take place in the interior of mass concrete structure.

• Loss of pre stress due to creep of concrete in pre stressed concrete structure.

• Because of rapid construction techniques, concrete members will experience loads that can be as large as the design loads at very early age; these can cause deflections due to cracking and early age low elastic modulus. So, creep has a significant effect on both the structural integrity and the economic impact that it will produce if predicted wrong.

Shrinkage:-

The volumetric changes of concrete structures due to the loss of moisture by evaporation is known as concrete shrinkage or shrinkage of concrete. It is a time-dependent deformation which reduces the volume of concrete without the impact of external forces.

Types of Shrinkage in Concrete:

To understand this aspect more closely, shrinkage can be classified in the following way:

(a) Plastic Shrinkage in concrete
(b) Drying Shrinkage in concrete
(c) Autogeneous Shrinkage in concrete
(d) Carbonation Shrinkage in concrete

The Types of shrinkage are explained as below:

a. Plastic Shrinkage:

Plastic shrinkage is contraction in volume due to water movement from the concrete while still in the plastic state, or before it sets. This movement of water can be during the hydration process or from the environmental conditions leading to evaporation of water that resides on the surface on the wet concrete. So, the more the concrete bleeds, the greater the plastic shrinkage should be. Plastic shrinkage is proportional to cement content and, therefore, inversely proportional to the w/c ratio.

Plastic shrinkage in concrete can be reduced mainly by preventing the rapid loss of water from surface. This can be done by covering the surface with polyethylene sheeting immediately on finishing operation; by fog spray that keeps the surface moist; or by working at night. Use of small quantity of aluminium powder is also suggested to offset the effect of plastic shrinkage. Similarly, expansive cement or shrinkage compensating cement also can be used for controlling the shrinkage during the setting of concrete.

b. Drying Shrinkage:

Just as the hydration of cement is an everlasting process, the drying shrinkage is also an everlasting process when concrete is subjected to drying conditions. The drying shrinkage of concrete is analogous to the mechanism of drying of timber specimen. The loss of free water contained in hardened concrete, does not result in any appreciable dimension change. It is the loss of water held in gel pores that causes the change in the volume. Under drying conditions, the gel water is lost progressively over a long time, as long as the concrete is kept in drying conditions. Cement paste shrinks more than mortar and mortar shrinks more than concrete. Concrete made with smaller size aggregate shrinks more than concrete made with bigger size aggregate. The magnitude of drying shrinkage is also a function of the fineness of gel. The finer the gel the more is the shrinkage.

c. Autogeneous Shrinkage:

Autogeneous shrinkage, also known as "basic shrinkage," is the shrinkage due to chemical reactions between cement with water, known as hydration, and do not include environmental effects such as temperature and moisture changes. Its magnitude is usually ignored in concretes with w/c more than 0.40.

In a conservative system i.e. where no moisture movement to or from the paste is permitted, when temperature is constant some shrinkage may occur. The shrinkage of such a conservative system is known as autogeneous shrinkage. Autogeneous shrinkage is of minor importance and is not applicable in practice to many situations except that of mass of concrete in the interior of a concrete dam.

d. Carbonation Shrinkage:

Carbon dioxide present in the atmosphere reacts in the presence of water with hydrated cement. Calcium hydroxide [$Ca(OH)_2$] gets converted to calcium carbonate and also some other cement compounds are decomposed. Such a complete decomposition of calcium compound in hydrated cement is chemically possible even at the low pressure of carbon dioxide in normal atmosphere. Carbonation penetrates beyond the exposed surface of concrete very slowly. The rate of penetration of carbon

dioxide depends also on the moisture content of the concrete and the relative humidity of the ambient medium. Carbonation is accompanied by an increase in weight of the concrete and by shrinkage. Carbonation shrinkage is probably caused by the dissolution of crystals of calcium hydroxide and deposition of calcium carbonate in its place. As the new product is less in volume than the product replaced, shrinkage takes place. Carbonation of concrete also results in increased strength and reduced permeability, possibly because water released by carbonation promotes the process of hydration and also calcium carbonate reduces the voids within the cement paste. As the magnitude of carbonation shrinkage is very small when compared to long term drying shrinkage, this aspect is not of much significance

Factors affecting durability of concrete

Durability of Concrete is influenced by the factors shown in the following figure

Sulphate attack

• Sulphate attack denotes an increase in the volume of cement paste in concrete or mortar due to chemical action between the products of hydration of cement and solution containing sulphate, and also sodium, magnesium and Chlorides.

• In hardened concrete, calcium aluminate hydrate (CAH) can react with sulphate salt from outside, product of reaction is calcium sulphoaluminate, which can cause an increase in volume up to 227%

• Rate of sulphate attack increases with a saturated sulphate solution.

• A saturate solution of magnesium sulphate can cause serious damage to concrete with high w/c ratio.

Chloride Attack.

• Chlorides in concrete increases risk of corrosion of steel (Electrochemical reaction)

• Higher Chloride content or exposure to warm moist conditions increase the risk of corrosion.

• To minimize the chances of corrosion, the levels of chlorides in concrete should be limited

• Total amount of chloride content (as Cl) in concrete at the time of placing is provided by common specifications and standards.

Efflorescence:

Efflorescence in concrete is a whitish coloured powdered deposition of salts on the concrete surface that is formed due to evaporation of water from the concrete. It is caused when water soluble salts are present in the concrete material, which comes on to the surface while evaporation of water from the concre

CHAPTER-6

MIX DESIGN

The various factors affecting the choice of concrete mix design are:

1. Compressive strength of concrete

• Concrete compressive strength considered as the most important concrete property. It influences many other describable properties of the hardened concrete.

• The mean compressive strength (fcm) required at a specific age, usually 28 days, determines the nominal water-cement ratio of the mix.

• ISO 456-200, British Standard, and Eurocode utilize the term mean compressive strength which is slightly greater than characteristic compressive strength. However, ACI Code do not use such term.

• Other factors which influences the concrete compressive strength at given time and cured at a specified temperature is compaction degree.

• Concrete compressive strength is inversely proportional to the water-cement ratio.

Fig.1:Means compressive strength vs characteristic compressive strength

Fig.2: compressive strength of concrete

2. Workability of concrete

• Concrete workability for satisfactory placement and compaction depends on the size and shape of the section to be concreted, the amount and spacing of reinforcement, and concrete transportation; placement; and compaction technique.

• Additionally, use high workability concrete for the narrow and complicated section with numerous corners or inaccessible parts. This will ensure the achievement of full compaction with a reasonable amount of effort.

• Slump test values used to evaluate concrete workability.

• Slump test values for various reinforced concrete sections ranges from 25 mm to 175 mm.

3. Durability of concrete

• The ability of concrete to withstand harmful environment conditions termed as concrete durability.

• High strength concrete is generally more durable than low strength concrete.

• In the situations when the high strength is not necessary but the conditions of exposure are such that high durability is vital, the durability requirement will determine the utilized water-cement ratio.

• Concrete durability decreases with the increase of w/c ratio.

4. Maximum nominal size of aggregate

• Reinforcement spacing controls maximum aggregate size.

• Aggregate size is inversely proportional to cement requirement for water-cement ratio. This is because workability is directly proportional to size of aggregate

• However, the compressive strength tends to increase with the decrease in size of aggregate. Smaller aggregate size offers greater surface area for bonding with mortar mix that gives higher strength.

• IS 456:2000 and IS 1343:1980 recommends that the nominal size of the aggregate should be as large as possible.

• Finally, in accordance with ACI code, maximum aggregate size shall not exceed minimum reinforcement spacing, bar diameter, or 25mm.

Fig.3: Maximum aggregate size

5. Grading and type of aggregate

• Aggregate grading influences the mix proportions for a specified workability and water-cement ratio.

• The relative proportions between coarse and fine aggregate in concrete mix influence concrete strength.

• Well graded fine and coarse aggregate produce a dense concrete because of the achievement of ultimate packing density.

• If available aggregate, which obtained from natural source, does not confirm to the specified grading, the proportioning of two or more aggregate become essential.

• Additionally, for specific workability and water to cement ratio, type of aggregate affects aggregate to cement ratio.

• Lastly, An important feature of a satisfactory aggregate is the uniformity of the grading that achieved by mixing different size fractions.

Fig.7: aggregate grading types

6. Quality Control at site

• The degree of control could be evaluated by the variations in test results.

• The variation in strength results from the variations in the properties of the mix ingredients, in addition to lack of control of accuracy in batching, mixing, placing, curing and testing.

• Finally, the lower the difference between the mean and minimum strengths of the mix lower will be the cement-content required. The factor controlling this difference is termed as quality control.

ACCEPTANCE CRITERIA FOR DESIGN MIX CONCRETE

The concrete shall be deemed to comply with the strength requirements if:

a) Every sample has a test strength not less than the characteristic value; or

b) The strength of one or more samples though less than the characteristic value, is in each case not less than the greater of :

1) The characteristic strength minus 1.35 times the standard deviation; and

2) 0.80 times the characteristic strength ; and the average strength of all the samples is not less than the characteristic strength plus

times the standard deviation.

II. The concrete shall be deemed not to comply with the strength requirements if:

a) The strength of any sample is less than the greater of :

1) the characteristic strength mix is 1.35 times the standard deviation; and

2) 0.80 times the characteristic strength ; or

b) The average strength of all samples is less than the characteristic strength

plus

times the standard deviation.

III Concrete which does not meet the strength requirements as specified in I, but has a strength greater than that required by II may, at the discretion of the designer, be

accepted as being structurally adequate without further testing.

IV. Concrete of each grade shall be assessed separately.

V Concrete shall be assessed daily for compliance.

VI. Concrete is liable to be rejected if it is porous or honey-combed; its placing has been interrupted

without providing a proper construction joint; the reinforcement has been
displaced beyond the tolerances specified; or construction tolerances have not been met. However, the hardened concrete may be accepted after carrying out suitable remedial
measures to the satisfaction.

VIII. Where the value of the average strength of the tests (preferably 30 tests or 15 tests) is less than
shall be rejected.

CONCRETE MIX PROPERTIONING:-

Determination of the proportion of the concrete ingredients such as cement, fine aggregate, coarse aggregate, water and admixtures is called concrete mix. • A proper mix design only can gives the specified properties such as workability, strength, permeability and durability with economy.

Methods of Proportioning Concrete
- Arbitrary Method of Proportioning Concrete.
- Fineness Modulus Method of Proportioning Concrete.
- Minimum Void Method.
- Maximum Density Method:
- Water – Cement Ratio Method of Proportioning Concrete.

ARBITRARY METHOD OF PROPORTIONING CONCRETE:-

The general expression for the propertions of cement, sand and coarse aggregare in 1:n:2n by volume.

- 1:1:2 and 1:1.2::2.4 for very high strength
- 1:1.5:3 and 1:2:4 for normal works
- 1:3:6 and 1:4:8 for foundations and mass concrete works.

Recommended mixes of concrete:

The concrete as per IS 456:2000, the grades of concrete lower than M20 are not to be used in RCC work.

M10 1 : 3 : 6
M15 1 : 2 : 4
M20 1 : 1.5 : 3
M25 1 : 1 : 2

Fineness Modulus Method of Proportioning Concrete:-

The term fineness modulus is used to indicate an index number which is roughly proportional to the average size of the particle in the entire quantity of aggregates.

The fineness modulus is obtained by adding the percentage of weight of the material retained on the following sieve and divided by 100.

The coarser the aggregates, the higher the fineness modulus.

Sieve is adopted for:

All aggregates : 80 mm, 40 mm, 20 mm, 10 mm, and Nos. 480, 240, 120, 60, 30 and 15.

Coarse aggregates : mm, 40 mm, 20 mm, 10 mm, and No. 480.

Fine aggregates : Nos. 480, 240, 120, 60, 30 and 15.

Proportion of the fine aggregate to the combined aggregate by weight

Where, P = desired fineness modulus for a concrete mix of fine and coarse aggregates.

= fineness modulus of fine aggregate

= fineness modulus of coarse aggregate.

Minimum Void Method:-

It does not give satisfactory result.

The quantity of sand used should be such that it completely fills the voids of coarse aggregate. Similarly, the quantity of cement used shown such that it fills the voids of sand, so that a dense mix the minimum voids is obtained.

In actual practice, the quantity of fine aggregate used in the mix is about 10% more than the voids in the coarse aggregate and the quantity of cement is kept as about 15% more than the voids in the fine aggregate.

Maximum Density Method:

This method is not very Popular.

It is determined by the formula as

Where, D = maximum size of aggregate (i.e. coarse aggregate)

P = percentage of material finer than diameter d (by weight)

d = maximum size of fine aggregate.

A box is filled with varying proportions of fine and coarse aggregates. The proportion which gives heaviest weight is then adopted.

Water – Cement Ratio Method of Proportioning Concrete:-

According to the water – cement ratio law given by Abram as a result of many experiments, the strength of well compacted concrete with good workability is dependent only on the ratio.

• The lower water content produces stiff paste having greater binding property and hence the lowering the water-cement ratio within certain limits results in the increased strength.

• Similarly, the higher water content increases the workability, but lower the strength of concrete.

• The optimum water-cement ratio for the concrete of required compressive strength is decided from graphs and expressions developed from various experiments.

• Amount of water less than the optimum water decreases the strength and about 10% less may be insufficient to ensure complete setting of cement. An increase of 10% above the optimum may decrease the strength approximately by 15% while an increase in 50% may decrease the strength to one-half.

• According to Abram's Law water-cement law, lesser the water-cement ratio in a workable mix greater will be the strength.

• If water cement ratio is less than 0.4 to 0.5, complete hydration will not be secured.

Some practical values of water cement ratio for structure reinforced concrete

• 0.45 for 1 : 1 : 2 concrete

• 0.5 for 1 : 1.5 : 3 concrete

• 0.5 to 0.6 for 1 : 2 : 4 concrete.

Concrete vibrated by efficient mechanical vibrators require less water cement ratio, and hence have more strength.

Thumb Rules for deciding the quantity of water in concrete:

(i) Weight of water = 28% of the weight of cement + 4% of the weight of total

aggregate
(ii) (ii) Weight of water = 30% of the weight of cement + 5% of the weight of total aggregate.

BIS METHOD OF MIX DESIGN:-

The process of selecting suitable ingredients of concrete and determining their relative amounts with the objective of producing a concrete of the required, strength, durability, and workability as economically as possible, is termed the concrete mix design.

Procedure for concrete mix design requires following step by step process:

1. Calculation of target strength of concrete
2. Selection of water-cement ratio
3. Determination of aggregate air content
4. Selection of water content for concrete
5. Selection of cement content for concrete
6. Calculation of aggregate ratio
7. Calculation of aggregate content for concrete
8. Trial mixes for testing concrete mix design strength

Step 1: Calculation of Target Strength of Concrete:-

Target strength is denoted by f_t which is obtained by characteristic compressive strength of concrete at 28 days (f_{ck}) and value of standard deviation (s)

$f_t = f_{ck} + 1.65 s$

Standard deviation can be taken from below table

Grade of concrete Standard deviation (N/mm$_2$)

M10 3.5
M15 3.5
M20 4.0
M25 4.0
M30 5.0
M35 5.0
M40 5.0
M45 5.0
M50 5.0

Step 2: Selection of Water-Cement Ratio:-

Ratio of the weight of water to weight of cement in the concrete mix is water-cement ratio. It is the important consideration in concrete mix design to make the concrete workable. Water cement ratio is selected from the below curve for 28 days characteristic compressive strength of concrete.

Similarly, we can determine the water-cement ration from the 7-day concrete strength, the curves are divided on the basis of strength from water cement ratio is decided. Which is observed from the below graph.

Step 3: Determination of Aggregate Air content:-

Nominal maximum size of aggregate Air content (% of volume of concrete)

Air content in the concrete mix is determined by the nominal maximum size of aggregate used. Below table will give the entrapped air content in percentage of volume of concrete.

Step 4: Selection of Water Content for Concrete:-
Select the water content which is useful to get required workability with the help of nominal maximum size of aggregate as given in below table. The table given below is used when only angular shaped aggregates are used in concrete as well as the slump should be 25 to 50mm.

Nominal maximum size of aggregate Maximum water content
10mm 208
20mm 186
40mm 165

If the shape of aggregate or slump value is differing from above, then some adjustments are required as follows.

CHAPTER-7
Concrete Production, Placement, & Curing Process

Concrete Production and placing Process includes following stages that are given below.

1. Batching of concrete
2. Mixing of concrete
3. Transportation of concrete
4. Placing of concrete
5. Compaction of concrete
6. Curing of concrete

1. Batching of concrete:

Batching is the process of measuring of material required for concrete mix by weight or volume.

Batching is done by two approaches.

- Volume batching
- Weight batching

Batching of Concrete

Volume Batching **Weight batching**

i. Volume batching:

Volume batching is done by a typical gauge box that is known as "farmas" in the field. The volume of farma is 0.035m3 which is similar to the volume of 1 bag of cement.

Farma is made of timber, steel or plastic materials.

ii. Weight batching:

Weight batching is done manually or semi-automatically, or fully automatically. Mostly it is used in Ready mix concrete plants.

This type of batching is mostly used for large construction.

2. Mixing of concrete:

Mixing is necessary to make homogeneous concrete. To obtain a good quality of concrete, it is necessary to do proper mixing of concrete ingredients. The mixing of concrete depends on the types of mixing.

Generally mixing is done by three ways.

- Hand mixing
- Machine mixing
- Ready-mix concrete

i. Hand Mixing:

In this way mixing of concrete done by the hands of workers. This type of mixing is used for small construction work due to high time consumption. Also the quality of the concrete we get very less.

ii. Machine Mixing:

In this way, the mixing of concrete done by various types of machinery. It is used for medium to large construction work for example 3 to 4 story building construction.

Three types of machineries used for mixing of concrete.

- **Tilting type**
- **Non-tilting type**
- **Reversing type**

iii. Ready mix concrete:

Now a day's ready mix concrete become most popular for large construction because it gives high-quality concrete. Also, it gives the flexibility to do concreting work at the site where we can't able to produce concrete like the valley, mountains region, etc.

In valley regions, concrete ingredients like cement, aggregate, sand, water, and other materials are not easy to available. So we need to transport material from other locations which increase the cost of construction.

In this type of mixing, concrete mixing is done in a ready mix concrete plant that is situated on-site or another site. In the case of the RMC plant situated in another place, the concrete can be transported by the transit mixer to the construction site.

It is economical for large construction.

Ready mix concrete plant Consist the following parts:

- Batcher (to measure material)
- Conveyor system (to move ingredients of concrete to the concrete mixer)
- Silos (to store cement)
- Concrete Mixer (To mix concrete)
- Screw Conveyor (to convey cement from silos to the concrete mixer)
- Etc,

3. Transportation of concrete:

Transportation of Concrete

There are various methods used to transport concrete. Few of its are given below.

Mortar pan:
A mortar pan is used for small construction work in which concrete or mortar can be transported by laborers using a mortar pan.

Wheel Borrow:
The wheel borrow method of concrete transportation is suitable for long-distance concrete transportation on the site. It is majorly used for the construction of roads, slabs, etc.

Bucket & Ropeway:
Bucket & ropeway method of concrete transportation is used to transport concrete in valley areas.

Transit mixer:
A transit mixer is suitable for long-distance concrete transportation. It is used to transport a large volume of concrete to the construction site which is situated a long distance away from the mixing plant.

Skip & Hoist:
It is used in concreting of skyscraper building.

Pumps & pipelines:
Now a day's this type of concrete transportation is most popular for concreting work in tall structures. In this method, concrete is transported through the pipes by using pumps. In this method, concrete conveys from a central discharge point to formwork.

Belt conveyors:
It is used to convey concrete horizontally or inclined on the construction site.

4. Placing of concrete:

Placing of concrete is the process of deposition of concrete in its required position.

Placing of concrete under water is done by various methods.

- Tremie method
- Bagged method
- Bottom dump method, etc.

5. Compaction of concrete:

Compaction of concrete is a process in which concrete is compacted by various methods to remove entrapped air from the concrete and increase the strength of concrete.

By doing compaction of concrete, concrete becomes dense, impermeable due to that the durability of concrete increases.

Compaction of Concrete

Compaction of concrete done by two ways.

- Hand compaction
- Mechanical compaction

i. Hand compaction:

Hand compaction of concrete is done by various techniques.

- Roding
- Ramming
- Tamping

ii. Mechanical compaction:

Mechanical compaction is done by various type of vibrating machineries which are given below.

- Needle vibrator
- Form vibrator
- Table vibrator
- Surface vibrator

6. Curing of concrete:

Curing of concrete is defined as a process to maintain temperature and moisture of concrete during the hydration process of cement due to that hydration of cement completed and concrete gains maximum strength.

CHAPTER-8

Factors Affecting Quality of Concrete

Concrete strength is affected by many factors, such as quality of raw materials, water/cement ratio, coarse/fine aggregate ratio, age of concrete, compaction of concrete, temperature, relative humidity and curing of concrete.

Quality of Raw Materials

Cement: Provided the cement conforms with the appropriate standard and it has been stored correctly (i.e. in dry conditions), it should be suitable for use in concrete. **Aggregates:** Quality of aggregates, its size, shape, texture, strength etc determines the strength of concrete. The presence of salts (chlorides and sulphates), silt and clay also reduces the strength of concrete. **Water:** frequently the quality of the water is covered by a clause stating "..the water should be fit for drinking..". This criterion though is not absolute and reference should be made to respective codes for testing of water construction purpose.

Water / Cement Ratio

The relation between water cement ratio and strength of concrete is shown in the plot as shown below:
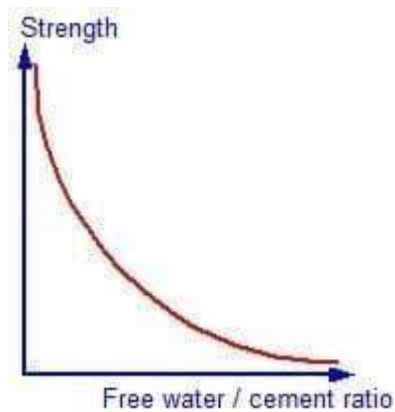
The higher the water/cement ratio, the greater the initial spacing between the cement grains and the greater the volume of residual voids not filled by hydration products. There is one thing missing on the graph. For a given cement content, the workability of the concrete is reduced if the water/cement ratio is reduced. A lower water cement ratio means less water, or more cement and lower workability. However if the workability becomes too low the concrete becomes difficult to compact and the strength reduces. For a given set of materials and environment conditions, the strength at any age depends only on the water-cement ratio, providing full compaction can be achieved.

Coarse / fine aggregate ratio

Following points should be noted for coarse/fine aggregate ratio:

- If the proportion of fines is increased in relation to the coarse aggregate, the overall aggregate surface area will increase.

- If the surface area of the aggregate has increased, the water demand will also increase.

- Assuming the water demand has increased, the water cement ratio will increase.

- Since the water cement ratio has increased, the compressive strength will decrease.

Aggregate / Cement Ratio

Following points must be noted for aggregate cement ratio:

- If the volume remains the same and the proportion of cement in relation to that of sand is increased the surface area of the solid will increase.
- If the surface area of the solids has increased, the water demand will stay the same for the constant workability.
- Assuming an increase in cement content for no increase in water demand, the water cement ratio will decrease.
- If the water cement ratio reduces, the strength of the concrete will increase.

The influence of cement content on workability and strength is an important one to remember and can be summarized as follows:

1. For a given workability an increase in the proportion of cement in a mix has little effect on the water demand and results in a reduction in the water/cement ratio.

2. The reduction in water/cement ratio leads to an increase in strength of concrete.

3. Therefore, for a given workability an increase in the cement content results in an increase in strength of concrete.

Age of concrete

The degree of hydration is synonymous with the age of concrete provided the concrete has not been allowed to dry out or the temperature is too low. In theory, provided the concrete is not allowed to dry out, then it wil always be increasing albeit at an ever reducing rate. For convenience and for most practical applications, it is generally accepted that the majority of the strength has been achieved by 28 days.

Compaction of concrete

Any entrapped air resulting from inadequate compaction of the plastic concrete will lead to a reduction in strength. If there was 10% trapped air in the concrete, the strength will fall down in the range of 30 to 40%.

Temperature

The rate of hydration reaction is temperature dependent. If the temperature increases the reaction also increases. This means that the concrete kept at higher temperature will gain strength more quickly than a similar concrete kept at a lower temperature. However, the final strength of the concrete kept at the higher temperature will be lower. This is because the physical form of the hardened cement paste is less well structured and more porous when hydration proceeds at faster rate. This is an important point to remember because temperature has a similar but more pronounced detrimental effect on permeability of the concrete.

Mixing

Once the ingredients are batched by volume or weight, they are introduced in a mixer and subjected to mixing. Initially, the dry ingredients are mixed for a small duration of time followed by the addition of water.

Once water is added, thorough mixing is carried out to ensure that the mass is homogenous, uniform in color, and consistent. Mixing may be done by hand or machine; however, the latter is preferred.

- For **small-scale concrete** works (e.g., repairing works), **hand mixing** can be done using a shovel. Fine and coarse aggregates are spread in alternate layers and cement is placed over them. Water is added and the ingredients are mixed till the attainment of uniform color. Manual mixing requires more cement than mechanical mixing to attain the same strength.

Fig.: Hand mixing of concrete

- For **medium and large-scale works**, machine mixing proves efficient in terms of delivering high-quality wet concrete. The machine mixer can either be a batch mixer or a continuous mixer.

3) Batch Mixer:

A batch mixer prepares concrete batch by batch. It can either be a tilting-drum type (T) mixer, a non-tilting drum (NT) type mixer, or a reversing (R) type mixer. Mixers with tilting drums are most commonly used.

a) Tilting drum (T) type mixers

They are either bowl-shaped or double conical frustum type. The material is added into the mixer by hand or is skip-fed. This type of mixer can easily be tilted in the downward direction for unloading mixed concrete.

The pros of using this mixer include the production of a concrete mix having relatively low workability, containing large aggregate sizes. However, a part of the cement mortar that clings or adheres to the mixer drum is often left out and not discharged which makes the use of this mixer a little disadvantageous.

Fig.: Tilting drum (T) type mixer

b) Non-tilting drum (NT) type mixers

Itconsist of cylindrical drums that are incapable of tilting. However, the drum is open from two sides and revolves about the horizontal axis. The dry mix is entered into the mixer at one end and the wet concrete is discharged from the other end. This type of mixer causes segregation owing to the slow discharge of concrete through it.

Fig.: Non-tilting drum (NT) type mixer

c) Reversing drum (R) type mixers

They are used for important concrete works. They are also termed forced action type mixers. The mixer consists of blades of two types functioning for different causes. One set of blades is used for mixing the concrete whilst the rotation of the drum, while the other set discharges the wet concrete as the drum is reversed.

Fig.: Reversing drum (R) type mixer

d) Pan type mixers

They are immovable and manufacture concrete either in the precast factory or at the central mixing plant. Their use is particularly worthwhile for stiff, homogeneous, and cohesive mixes. The pan or drum consists of two halves that are capable of rotating about the horizontal axis.

The circumferential speed at which both the halves rotate is different. This propels the concrete towards the center of the drum, producing some stirring up or turbulence that ultimately yields a homogenous mix of heterogenous concrete.

Fig.: Pan-type mixer

Batch mixers are available in various capacities. For general works, (10/7) th or (1/4) th cubic yard capacity mixers are used. 10/7 means that 10 cubic feet of dry ingredients will produce 7 cubic feet of wet concrete. The mixing time in no case is to be less than 90 seconds post addition of water for 1 cement bag capacity mixer.

The ready mixed concrete is batched in a central batching plant and delivered to the job site usually in trucks having mixers mounted on them. The concrete may be subjected to mixing during transportation or after arrival at the site. The concrete prepared using batch mixing can be kept workable or plastic for 1.5 hours by slow revolutions of the mixer.

However, it is preferred to add some water and mix again after the arrival at the site. This is termed as **retempering** of concrete and it aims at mobilizing the workability of concrete by adding some water into the prepared mix.

e) Continuous Mixer:

As the name suggests, these mixers are capable of providing a continuous discharge of wet concrete till the operation is deliberately put to a pause. The materials are loaded into the drum of the mixer that operates by the slow and continuous spinning of the blades mounted inside.

These mixers are used for large-scale projects that require an uninterrupted outflow of concrete during construction operations as in the case of a dam or bridge.

Fig.: Continuous mixer

Control on Mixing Time

In order to prevent cement from adhering to the blades or bottom of the mixer, a small quantity of water (about 25%) is usually added to the drum. The dry ingredients are then introduced into the mixer in sequential order, adding half the coarse and fine aggregates first, followed by the addition of the entire cement. After this, the remaining half of coarse and fine aggregates is added followed by the addition of remaining water.

The mixing time is initiated once the entire pre-determined water quantity is fed into the mixer. The mixer speed is generally kept between 25 to 30 rpm and the mixing time also hinges on the mixer capacity.

Transportation

The concrete is carried or transported to the site where it is to be placed or deposited. It should be ensured that during transportation neither does the concrete harden before time nor does it segregate or bleed. The transportation of concrete can be accomplished either manually or mechanically.

a) Manual transportation

It can be done using barrows or buckets and is employed for small concrete activities when the place of deposition is close to the place of mixing.

b) Mechanical transportation

It is preferred when the place of deposition is distant from the place of mixing. It is done using belt conveyors, dumpers, pumps, and lifts. In every case, the partial setting of concrete is to be prevented by consuming minimum time in transiting the concrete.

Dumpers, lorries, or trucks can transport concrete up to 5km and prove economical. During the transit process, the concrete should be covered with tarpaulins to prevent the evaporation of water.

Placing of Concrete

Placing of concrete refers to its deposition at the requisite site in the required shape. While placing the concrete, large lifts and free-falls must be avoided.

This possibly leads to segregation of concrete. When the concreting operating is to be halted for a certain duration of time, it is imperative to make grooves or construction joints in the finished work before its setting and hardening

Curing

When the concrete initially hardens and sets in place, moisture escape from the concrete body can create voids and ultimately result in huge strength loss. This is because concrete keeps on gaining strength for several weeks once it hardens initially.

Therefore, concrete needs to retain water that otherwise may evaporate leaving pores inside the body. To help concrete gain strength, curing is to be done and the method adopted depends upon the site conditions. The following curing techniques can generally be adopted:

- By jute bags

- By ponding

- By the sprinkling of water

- By immersing in water

- By steam

- By construction chemicals (chemical curing)

Fig.: Curing by the sprinkling of water

Fig.: Ponding of concrete

 Fig.: Steam curing of concrete

REQUIREMENTS FOR DURABILITY OF CONCRETE

1. One of the major properties of concrete affecting its durability is its permeability.

The more permeable a concrete is the less durable it will be.

Because when concrete is permeable it helps to the ingress of deleterious agent that affects both concrete and embedded reinforcement.

2. The factors which influence the durability of concrete are

- environment,
- cover to embedded steel,
- type and quality of construction materials,
- cement content and
- water-cement ratio,
- workmanship during compaction
- curing, and
- shape and size of the member.

CHAPTER-9

SPECIAL CONCRETE

Introduction

• Special types of concrete are those with out-of-the-ordinary properties or those produced by unusual techniques. Concrete is by definition a composite material consisting essentially of a binding medium and aggregate particles, and it can take manyforms.

• These concretes do have advantages as well asdisadvantages.

Types of special concrete

1. High Volume Fly AshConcrete.

2. Silica fumesconcrete.

3. GGBS, Slag based concrete.

4. Ternary blend concrete.

5. Light weight concrete.

6. Polymerconcrete.

7. Self-Compacting Concrete.

8. Coloured Concrete.

9. Fibre-reinforced Concrete. 10.Pervious Concrete. 11.Water-proofConcrete.

12.Temperature Controlled Concrete.

Silica fume concrete

• Very fine non-crystalline silica produced in electric arc furnaces as aby-product.

• Highly reactive pozzolana used to improve mortar andconcrete.

• Silica fume in concrete produces two types of effectviz.

• Physicaleffect

• Chemicaleffect

• The transition zone is a thin layer between the bulk hydrated cement paste and the aggregate particles in concrete. This zone is the weakest component in concrete, and it isalso the most permeable area. Silica fume plays a significant role in the transition zone through both its physical and chemicaleffects.

Physical Effect

• The presenceof any type of very small particles willimproveconcrete properties. Thiseffectistermedeither—particlepacking‖or—microfilling‖.

• Physical mechanisms do play a significant role, particularly at earlyages.

Chemical Effect

• Silica fume is simply a very effective pozzolanic material.

• Pozzolanic means a siliceous or siliceous and aluminous material, which in itself possess little or no cementious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementious properties.

High performance concrete

High-performance concrete (HPC) is concrete that has been designed to be more durable and, if necessary, stronger than conventional concrete. HPC mixtures are composed of essentially the same materials as conventional concrete mixtures, but the proportions are designed, or engineered, to provide the strength and durability needed for the structural and environmental requirements of the project. High-strength concrete is defined as having a specified compressive strength of 8000 psi (55 MPa) or greater. The value of 8000 psi (55 MPa) was selected because it represented a strength level at which special care is required for production and testing of the concrete and at which special structural design requirements may be needed.

Shortcrete

Shotcrete is a method of applying concrete projected at high velocity primarily on to a vertical or overhead surface. The impact created by the application consolidates the concrete. Although the hardened properties of shotcrete are similar to those of conventional cast-in-place concrete, the nature of the placement process results in an excellent bond with most substrates, and rapid or instant capabilities, particularly on complex forms or shapes. The shotcrete process requires less formwork and can be more economical than conventionally placed concrete. Shotcrete is applied using a wet- or dry-mix process. The wet-mix shotcrete process mixes all ingredients, including water, before introduction into the delivery hose. The dry-mix shotcrete process adds water to the mix at the nozzle. Shotcrete is used in new construction and repairs and is suitable for curved and thin elements

Guniting

Guniting is a procedure in which repairing concrete work can be done that has been harmed due to inferior work or other reasons and providing an impervious layer is also used.

For the application of slope stabilization and certain rehabilitation purposes, the guniting process is used in construction such as in the construction of retaining walls, swimming pool construction, tunnel construction, concrete repair works, and in fluid tank construction.

Ready mix concrete

concrete that is manufactured in a factory or within a batching plant based on the standard required specifications. The prepared concrete mix is then taken to the work site within transit mixers mounted over a truck.

This type of concrete guarantee higher durability and sustainability. As the work is carried out by an expert supplier, the mixture formed is precise and of higher quality. Special concrete mixtures too can be made efficiently by this concrete manufacturing method.

CHAPTER-10

Different types of Deterioration in Concrete Structures

Corrosion of Embedded Metals in concrete

When steel corrodes, the resulting rust occupies a greater volume than the steel. This expansion creates tensile stresses in the concrete, which can eventually cause cracking, delamination, and spalling. Steel corrodes because it is not a naturally occurring material. Rather, iron ore is smelted and refined to produce steel. The production steps that transform iron ore into steel add energy to the metal. The corrosion of structural steel is an electrochemical process that requires the simultaneous presence of moisture and oxygen. Essentially, the iron in the steel is oxidised to produce rust, which occupies a greater volume than the steel.

Freeze thaw deterioration in concrete

Freeze-thaw damage is a potentially serious deterioration process that occurs in concrete structures in cold climates. Premature damage to concrete slabs during freezing and thawing cycles represents a major challenge to pavement durability and resilience. When water freezes, it expands about 9%. As the water in moist concrete freezes, it produces pressure in the capillaries and pores of the concrete. If the pressure exceeds the tensile strength of the concrete, the cavity will dilate and rupture. The accumulative effect of successive freeze-thaw cycles and disruption of paste and aggregate can eventually cause significant expansion and cracking, scaling, and crumbling of the concrete Freeze/thaw damage occurs in concrete when the water molecules in concrete freeze and expand beyond the volume constraints of the concrete.

Chemical attack in concrete

Chemical attack is one of the more common causes of deterioration of concrete in industry today. Animal fats, natural and artificial oils, acids, alkalis, and various industrial salts are all damaging to concrete. Chemical attack occurs due to pollution products and following discharge activity on the insulator surface. Examination of field-aged insulators has found formation of uniform thin pollution layers on the surface. A chemical attack involves dissolution of substances or chemical reactions between substances and components of the concrete. Reaction products might cause problems, due to dissolution or expansion.

Alkali-Aggregate Reactivity in concrete

Alkali-aggregate reactions can be either alkali-carbonate reactions (ACRs) or alkali-silica reactions (ASRs). In ACR, the reaction is between the alkalies (sodium and potassium) and certain carbonate rocks, particularly calcitic dolomite and dolomitic limestones, present in some aggregates. It is a deleterious swelling reaction that occurs over time in concrete between the highly alkaline cement paste and the reactive amorphous (i.e., non-crystalline) silica found in many common aggregates, given sufficient moisture. This deleterious chemical reaction causes the expansion of the altered aggregate by the formation of a soluble and viscous gel of sodium silicate. This hygroscopic gel swells and increases in volume when absorbing water: it exerts an expansive pressure inside the siliceous aggregate, causing spalling and loss of strength of the concrete, finally leading to its failure.

Abrasion Erosion in concrete

The abrasion damage caused by the grinding action of silt, sand, and rock. The grinding action will remove the concrete surface and expose the aggregates in some cases. Hydraulic concrete structures frequently experience long-term abrasive erosion by water-borne sand, resulting in surface damage, eventually limiting their service life. Abrasion damage occurs when the surface of concrete is unable to resist wear caused by rubbing and friction. As the outer paste of concrete wears, the fine and coarse aggregate are exposed and abrasion and impact will cause additional degradation that is related to aggregate-to-paste bond strength and hardness of the aggregate

Plastic Shrinkage in concrete

Loss of water from fresh concrete, which leads to plastic shrinkage, can occur in a couple of ways. The predominant mode is, however, through evaporation from an exposed surface. Concrete can also lose water through suction by the subbase or, depending on the type of material used in its manufacture, the formwork. Such loss of water can aggravate the effects of surface evaporation. It is generally accepted that the loss of water from the paste fraction of concrete due to external factors generates negative capillary pressures that cause the volume of the paste to contract, hence the shrinkage. The rate of water evaporation is usually aggravated by a combination of high wind speed, low relative humidity, and high ambient and concrete temperatures.

Drying Shrinkage in concrete

The loss of moisture from concrete after it hardens, and hence drying shrinkage, is inevitable unless the concrete is completely submerged in water or is in an environment with 100 percent relative humidity. Thus, drying shrinkage is a phenomenon that routinely occurs and merits careful consideration in the design and construction of concrete structures. The actual mechanisms by which drying shrinkage occurs are complex, but it is generally agreed upon that they involve the loss of adsorbed water from the hydrated cement paste. When concrete is initially exposed to a drying condition – one in which there is a difference between the relative humidity of the environment and that of the concrete – it first loses free water. In the larger capillary pores this results in little or no shrinkage. In the finer water-filled capillary pores (2.5 to 50 nm size) due to loss of moisture, curved menisci are formed, and the surface tension of water pulls the walls of the pores

The following measures are taken to prevent deterioration of concrete:

1. From the consideration of permeability, the **water-cement ratio** is usually limited to **0.45 to 0.55**.

2. The **cement** content should be such that it ensures sufficient alkalinity to prevent corrosion of reinforcement. For concreting under marine environment, minimum cement content of **350 kg/m** or more is to be used

3. The water-cement ratio and the cement content must provide enough paste to overfill the voids in **compacted concrete**.

4. Use of Portland slag cement or Portland pozzolana cement is advantageous for **concreting in sea water**.

5. Use of Portland cement having **C3A** content less than **5%** is suitable for concreting under sulphate environment.

6. The super-sulphated cement provides acceptable durability against the **acidic environment**.

7. Addition of hydraulic additives is also helpful to prevent the deterioration of concrete.

8. It is possible to attain a marked improvement in the quality of concrete by encouraging natural or artificial carbonation of the surface layer.

9. Deterioration of concrete can also be prevented by treating the concrete with solutions of suitable salts or even acids in minor concentration.

10. The durability of concrete can also be increased by impregnating the pores with a suitable polyme

Corrosion of Steel Reinforcement: Causes, Effects and Remedies

Concrete, in itself, has poor tensile strength. To increase the tensile strength of concrete, steel reinforcement is used. Steel bars are embedded within the concrete mass. These steel bars carry most of the tensile load applied to the concrete.

The concrete renders the steel bars passive due to its highly alkaline nature, thus preventing them from corrosion. Still, due to various other reasons, the steel bars may get corroded over a long period of time. And due to the corrosion of the steel bars, various weaknesses arise in the concrete structure, which may eventually collapse if not taken proper care of within suitable time.

Corrosion of steel reinforcement bars is basically an electrochemical reaction. Small anodes and cathodes are created and a flow of ions between these two electrodes lead to the corrosion of the steel bars

The main factors responsible for corrosion of reinforcement bars are:

1. **Loss of alkalinity due to carbonation** – When the steel surface is left unprotected in the atmosphere, rust begins to form on the steel surface and gradually flakes off.
2. **Loss of alkalinity due to chlorides** – Chloride ions tend to de-passivate the steel surface by destroying the alkalinity of the concrete.
3. **Cracks in concrete** – Cracks may expose the steel bars to the atmosphere and hence increase carbonation.
4. **Moisture pathways** – Regular wetting of the concrete may lead to water reaching the steel reinforcement bars by diffusion through the pore structure of the concrete or cracks present in the concrete. Rusting of the steel bars follow thereafter.

5. **Insufficient Cover:** Insufficient dimension of concrete cover.

Prevention of concrete corrosion

Corrosion of steel reinforcement bars may be prevented or at least delayed by practising good measures. Also, damaged steel bars can be repaired and the concrete structure can be restored properly. Some steps are given below:

1. **Providing Sufficient Concrete Cover:** A good amount of concrete cover should be provided over the steel reinforcement bars. This ensures proper maintenance of the alkaline nature within the concrete and the passivity of the steel bars. The steel bars should be precisely placed in position
2. **Use of Good Quality Concrete:** High quality concrete must be used. It helps to maintain proper alkaline nature. For the concrete, a water/cement ratio of 0.4 or less is to be maintained. Excessive water may damage the steel bars
3. **Proper Compaction fo Concrete:** Concrete must be completely compacted such that there are no air voids or pockets present inside
4. **Use of FBE coated Bars:** Fusion Bonded Epoxy Coating (FBEC) may be applied on the steel bars to prevent them from corrosion. Epoxy powder is spread electrostatically on to the steel bars. The powder melts and flows over the bars upon heating, forming a protective coating. They are thermoset polymer coatings because application of heat will not melt the coating. Apart from rebar it also has wide application in pipeline construction
5. **Use of Cement Based Polymers:** Cement based polymers can be used in the concrete to enhance its protection against corrosion capabilities. The cement based polymers act as a binder in the concrete. They also increase the durability, tensile strength and vibration damping of the concrete
6. **RCPT test to assess degree of Corrosion:** The Rapid Chloride Permeability Test (RCPT) may be performed to assess the degree of corrosion. The quantity of electrical current that passes through a sample 50 mm thick and 100 mm in diameter in 6 hours is measured. Based on this a qualitative rating is made of the permeability of the concrete
7. **Use of Migratory Corrosion Inhibitors:** Migratory corrosion inhibitors may be used in the concrete mix or may be applied on the hardened surface of the concrete. These inhibitors diffuse through the concrete cover and reach the steel bars to protect them against corrosion. Calcium nitrite based inhibitors are quite common

CHAPTER-11

Repair technology for concrete structure

Types of Concrete Defects - Causes, Prevention

Various types of defects which can be observed in hardened concrete surface and their prevention methods are explained below:

1. Cracking

Cracks are formed in concrete due to many reasons but when these cracks are very deep, it is unsafe to use that concrete structure. Various reasons for cracking are improper mix design, insufficient curing, omission of

expansion and contraction joints, use of high slump concrete mix, unsuitable sub-grade etc. To prevent cracking, use low water – cement ratio and maximize the coarse aggregate in concrete mix, admixtures containing calcium chloride must be avoided. Surface should be prevented against rapid evaporation of moisture content. Loads must be applied on the concrete surface only after gaining its maximum strength.

Cracking

Fig 1: Cracking

2. Crazing

Crazing also called as pattern cracking or map cracking, is the formation of closely spaced shallow cracks in an uneven manner. Crazing occurs due to rapid hardening of top surface of concrete due to high temperatures or if the mix contains excess water content or due to insufficient curing. Pattern cracking can be avoided by proper curing, by dampening the sub-grade to resist absorption of water from concrete, by providing protection to the surface from rapid temperature changes.

Crazing

Fig 2: Crazing or Pattern Cracking

3. Blistering

Blistering is the formation of hollow bumps of different sizes on concrete surface due to entrapped air under the finished concrete surface. It may cause due to excessive vibration of concrete mix or presence of excess entrapped air in mix or due to improper finishing. Excessive evaporation of water on the top surface of concrete will also cause blistering. It can be prevented by using good proportion of ingredients in concrete mix, by covering the top surface which reduces evaporation and using appropriate techniques for placing and finishing.

Concrete Blisters

Fig 3: Concrete Blisters

4. Delamination

Delamination is also similar to blistering. In this case also, top surface of concrete gets separated from underlying concrete. Hardening of top layer of concrete before the hardening of underlying concrete will lead to delamination. It is because the water and air bleeding from underlying concrete are struck between these two surfaces, hence space will be formed. Like blistering, delamination can also be prevented by using proper finishing techniques. It is better to start the finishing after bleeding process has run its course.

Delamination

Fig 4: Delamination

5. Dusting

Dusting, also called as chalking is the formation of fine and loose powdered concrete on the hardened concrete by disintegration. This happens due to the presence of excess amount of water in concrete. It causes bleeding of water from concrete, with this fine particles like cement or sand will rise to the top and consequent wear causes dust at the top surface. To avoid dusting, use low slump concrete mix to obtain hard concrete surface with good wear resistance. Use water reducing admixtures to obtain adequate slump. It is also recommended to use better finishing techniques and finishing should be started after removing the bleed water from concrete surface.

Dusting

Fig 5: Dusting

6. Curling

When a concrete slab is distorted into curved shape by upward or downward movement of edges or corners, it is called curling. It occurs mainly due to the differences in moisture content or temperature between slab surface (top) and slab base (bottom). Curling of concrete slab may be upward curling or downward curling. When the top surface is dried and cooled before bottom surface, it begins to shrink and upward curling takes place. When bottom surface is dried and cooled due to high temperature and high moisture content, it will shrink before top surface and downward curling occurs. To prevent curling, use low shrink concrete mix, provide control joints, provide heavy reinforcement at edges or provide edges with great thickness.

Curling
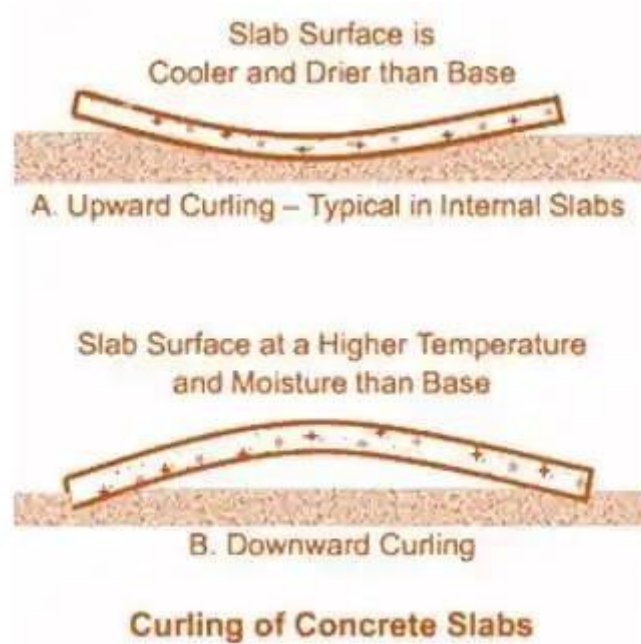
Fig 6: Curling of Concrete Slab

7. Efflorescence

Efflorescence is the formation of deposits of salts on the concrete surface. Formed salts generally white in color. It is due to the presence of soluble salts in the water which is used in making concrete mix. When concrete is hardening, these soluble salts gets lifted to the top surface by hydro static pressure and after complete drying salt deposits are formed on the surface. It can be prevented by using clean and pure water for mixing, using chemically ineffective aggregates etc. And make sure that cement should not contain alkalis more than 1% of its weight.

Efflorescence

Fig 7: Efflorescence

Cracks repair by injection technology

Injection materials

The selection of the appropriate injection material is the first key factor for a successful cracking

repair. The main materials requirements are the strong adhesion to the concrete, low viscosity,

flexibility and mechanical resistance suitable to structural or non-structural repairing, capacity of

deformation after hardening, volumetric shrinkage control and chemical stability of the

components mix that compose the product of injection .

Crack width is a crucial characteristic for injection material choice. Generally, lower width cracks

require an injection material with lower viscosity so they can enter in the void easily and with lower

injection pressure.

Cracks activity also has a huge influence in the material choice. In the case of passive/dead

cracks with small dimension, the best solution is the injection of rigid materials. In active cracks it

is necessary to apply a flexible material, with capacity of deformation after hardening [16].

The water presence will also affect the use of materials due to the chemical reactions (expansive)

of certain materials with the water molecules and to the materials adhesion (or the lack of it) with

the concrete in case of moist cracks.

Resins

Resins are the most used materials in injection systems. Epoxy resins have high compressive

and tensile strength comparing with concrete and, as a rigid material, they are used in structural

repairs ensuring the efficient transfer of strengths and recovering the structure rigidity conditions

, due to the strong adhesion between epoxy resins and concrete. The main injection epoxy

products properties are the hardening without shrinkage, the low viscosity, the applicability at low

temperature and the fact that they guarantee a barrier against water infiltration and corrosive

elements entering . On the negative side, regular epoxy resins are too sensitive to

water/moisture presence and water affects also the adhesion between epoxy and concrete, can

reduce their strength, and also the bad performance of these products at high temperature.

Polyurethane resins are flexible and guarantee a strong adhesion with the concrete in wet or

dry cracks. Within each polyurethane resins products, one can distinguish (i) polyurethane foams

and (ii) polyurethane gel. These two types of products have different characteristics and,

consequently, different functions, but, in general, these solutions are used in conjunction,

complement each other and make the best use possible of the properties of each one [15].

Polyurethane foams expand in contact with water, being used in areas with water flow to staunch

temporary the water entering. Its expansive speedily reaction in contact with water form a flexible

and elastic foam .

For a permanent waterproofing, the cracks with polyurethane foam could

be reinjected with not expansive resins like (i) polyurethane gel resins, that are flexible,

with a high chemical resistance, recommendable to fill cracks permanently with variable opening

over time, adapting to movements, with minimum width of approximately 0.2 mm or (ii) acrylic

gel resins, a very elastic material with an extremely reduced viscosity (similar to water), having

ideal properties to penetrate structures voids, they can be injected in cracks thinner than 0.05

mm. Acrylic resins application is common in the use of injection curtains technics as a repair

solution to other waterproofing systems or as preventive waterproofing in retaining walls in

underground structures.

Microcement grouts

Microcement grouts are not flexible and, consequently, they do not adjust to structures

movements. These materials can be used in injection works to repair structural cracks (wider than

0.5 mm), but are usually used in structural joints treatment. Therefore, microcements are not

products with a high demand for common cracks injections, because their penetrability and

mechanical characteristics are lower than those of epoxy resin products.

Injection equipment

The appropriate selection of injection equipment is the second key factor for the injection systems

success . The pumps and packers depend on the injections materials, the injection pressure

and the concrete quality in the cracks surround.

Injection pumps

There are two injection pumps technologies: (i) Single-component and (ii) bi-component. Singlecomponents
pumps can be manual, pneumatic (compressed air) or electrical. In this type of

pumps, the product composed by two components (resin + hardener) is mixed beforehand, being

deposited into a container. The product work time begins after components are mixed, and that it

is necessary to implement the complete volume mix inside the indicated pot life in the Product Technical Data Sheet. Bi-component pumps are usually pneumatic (delivery of compressed air) and have two storage containers, each one being filled with a part of product (usually part "A", mix of pre-dosed liquid resin and hardener and part "B" with a mix of an accelerator and water). The two components product mix is only made in the injection moment, in the pump head, when components are pumped, and products pot life begins counting when two parts are mixed.

3.2.2 Injection packers

Two types of injection packers can also be distinguished: (i) surface packers and (ii) mechanical packers. Surface packers (Fig. 1) are plastic or metal filling valves that are installed above the cracks, at structures surface. These packers are commonly used epoxy resins injections when a structural reinforcement is necessary [7] and in cases where it is not recommended to drill the concrete [15]. There are also simple cylindrical packers, similar to surface packers but without the circular injector basis and that are placed manually inside the crack.

References: book **Concrete Technology (M.L Gambhir)**

M.S Setty (Concrete technology)

Lecture notes on

STRUCTURE DESIGN-II

(TH-2,5THSEM)

Diploma Engineering

Prepared by-

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Structural Steel Fasteners & Connections: Bolted Connection.

→ Different elements or members of steel structures are required to be joined to one another either at their ends or at some intermediate length in order to facilitate the transmission of member forces which is known as <u>connection</u>.

→ The devices required for preparing a connection are called connectors or fasteners. There are mainly four types of fasteners commonly used.

 (a) Bolts (b) welds (c) Rivets (d) pins.

→ As a connection or a joint is the weakest part of a structure, which are to be properly designed.

<u>Bolted Connections.</u>

A bolt is a metal pin with a head at one end & a shank threaded at other end to receive a nut.

Bolt head — Nominal diameter. — Nut — D — shank — Thread
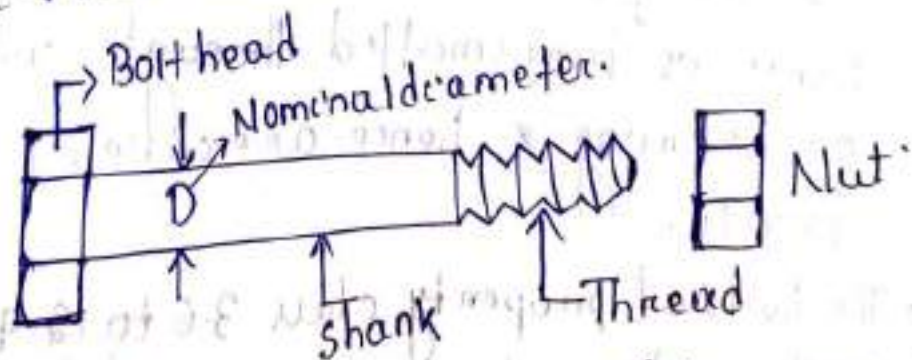
Bolts are used for the purpose of joining together pieces of metals having holes through which there are inserted & the nuts are

tightened at the threaded ends.

Types of bolts.

(i) unfinished bolts or black bolts.

(ii) finished bolts or turned bolt

(iii) High strength friction grip bolt (HSFG).

(i) unfinished or black bolts:

→ They are also known as ordinary or common bolts.

→ These bolts are made from low carbon mild steel round rods with square or hexagonal head & the shank is left unfinished or rough.

→ These are used for light structures subjected to static loads as well as for secondary members such as purlins, roof trusses etc, but not recommended for structures subjected to vibration & fatigue.

→ As the bolt is unfinished, it may not establish perfect contact with structural member resulting in loose joints.

→ In the joints made with such bolts, the force is transmitted through interlocking on bearing & hence are called bearing type joints.

* The bolts of property class 3.6 to 12.9 are available, out of which most commonly used black bolt is property class 4.6.

leaning of property class 4.6.:-

$$\frac{1}{100} \times \text{nominal ultimate strength} = 4$$
of bolt (f_u).

$$\frac{\text{yield stress of bolt } (f_{yb})}{\text{ultimate strength } (f_{ub}).} = 0.6.$$
of bolt

(ii) finished & turned bolts :-

→ These bolts are formed from mild steel hexagonal rods & are made by turning to circular shape.

→ As the connection is more tight, in this case it ensures better bearing contact between the bolts & holes: These are used where accurate alignment of components are necessary such as machine parts, structures subjected to dynamic loading etc.

(iii) High strength friction grip bolts:-

→ These bolts are made from high strength steel rods like black bolts, but the surface of the shank of these bolts is kept unfinished & these bolts are tightened until very high tensile stresses are developed using calibrated wrenches so that the connected plants are clamped tightly together between bolts & nut heads.

→ This permits the loads to be transferred primarily by friction & not by shear.

→ This results into no sleppage in the Joints the joints made with such bolts are known as slepcritical joint.

→ These are suitable for members subjected to dynamic load also.

Bolt holes :- table 19 (page 75)

Nominal size of fastener (d)	bolt hole (mm)
12-14	+1
16-22	+2
24	+2
>24	+3

Advantages of bolted connection :-

(i) use of simple tools & less skilled labour & working area.

(ii) speedy & noiseless connection.

(iii) Economical due to reduced labour & equipment cost.

(iv) Minimum strength reduction of joint due to less number of holes or bolts.

(v) Easy alternation or dismantling of connection.

Disadvantages of bolted connection:-

(i) high cost of material.

(ii) Reduced tensile strength due to area reduction

(iii) Susceptibility to loosening of bolts under vibration & dynamic loads.

Classification of bolts based on load transfer Mechanism

→ divided into two groups.
(i) bearing type or slip type connection.
(ii) friction grup type on slip critical connection.

(i) bearing type of bolt :

→ load trans fer takes place by shearing &
bearing of member.

ex finished bolt, unfinished bolt

(ii) friction grup type bolt :

→ load trans fer takes place by friction between
the members.

ex High strength friction grup (HSFG) bolt.

Advantages of HSFG bolts over bearing type bolt :

(i) Rigidity of joint due to no slip condition.

(ii) No shearing or bearing stresses on member
as the load trans fer mechanism is mainly
by friction.

(iii) large clamping forces provide high static
strength of joints.

Disadvantages of HSFG bolts over bearing type bolts.

(i) Material cost of HSFG bolts is greater.-
than that of ordinary bolts.

(ii) special workmanship is required in installing
& tightening of these bolts.

→ Additional labour cost is required for surface preparation of members to be joined.

Types of bolted connections / Joints :-
(a) Lap joint   (b) butt joint.

(a) Lap joint

→ The two members to be connected over lap one another.

→ Thus constitutes the simplest type of joint requiring no extra cover plates.

→ Since the centre of gravity of members joined in a lap joint are not collinear, the load on the lap joint has eccentricity, which may cause undesirable bending action. In order to minimize the effect of bending in lap joint, we have to increase no. of bolts.

(single bolted Lap joint)

(zig zag bolting)
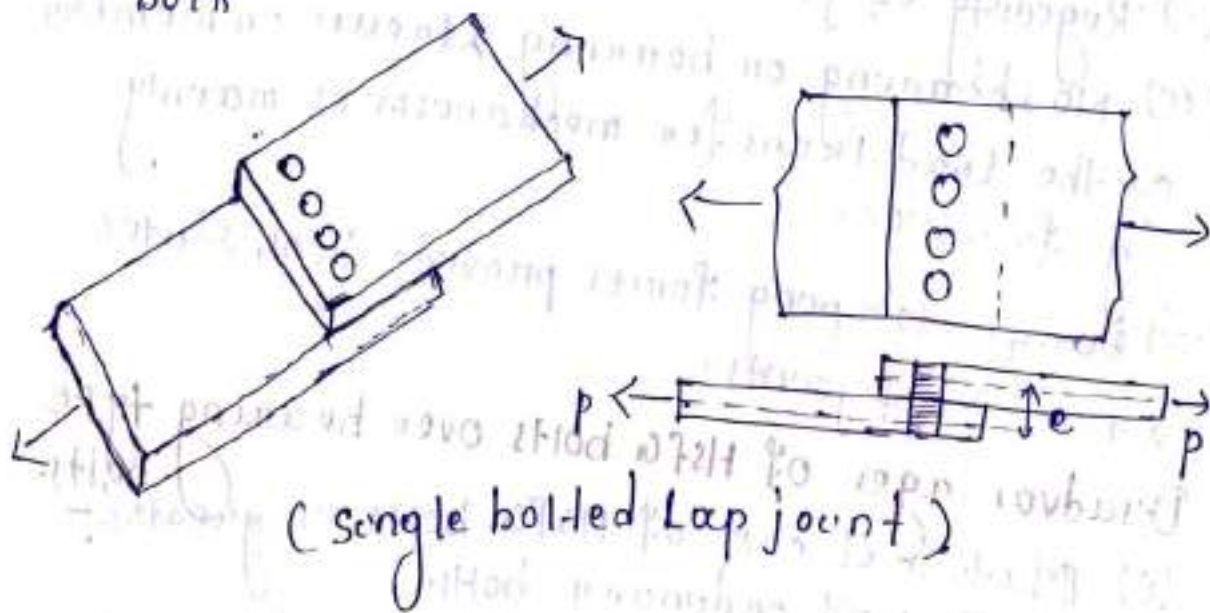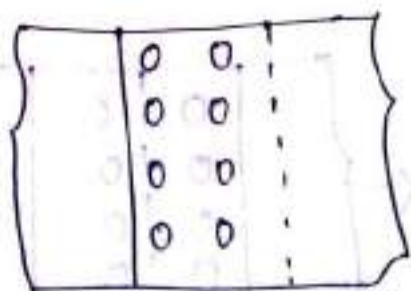
(double bolted lap joint)
(chain bolting)

(b) Butt joint :-

→ In this type of joint, the two members to be connected are placed end to end i.e. butt against each other & the connection is made by providing additional plates either on one side or on both sides. These additional plates are called cover plates where the members to be joined are called main plate.

→ bending is eliminated as there is no eccentricity of forces & the bolts are subjected to shear in two planes, known as bolts in double shear.
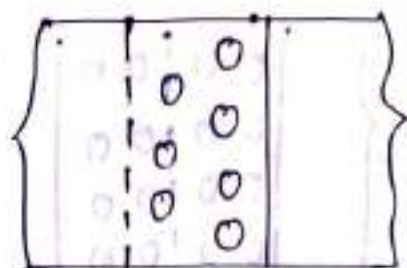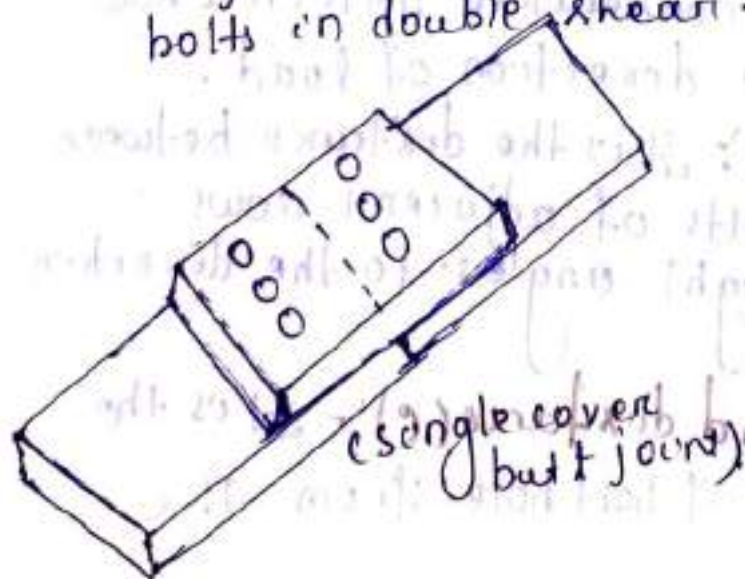
(single cover butt joint)
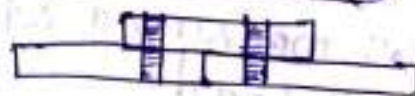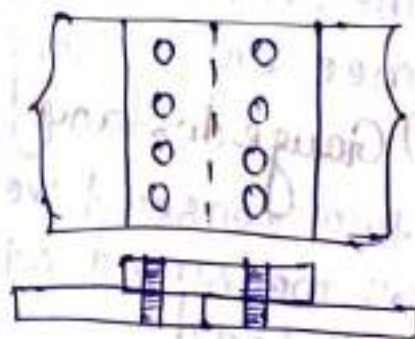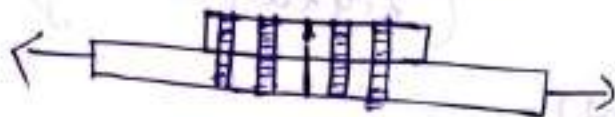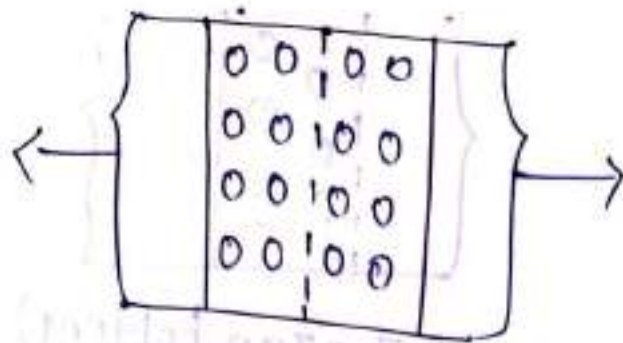
(single cover double
  bolted butt.
    joint).
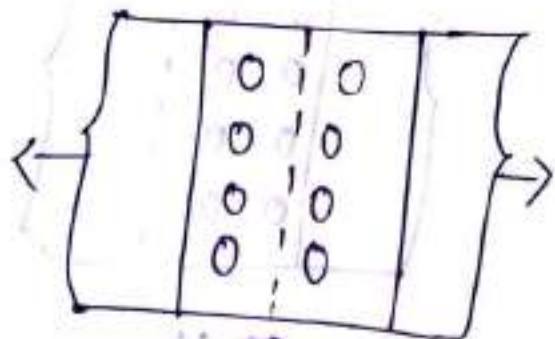
(double cover single
  bolted butt joint).
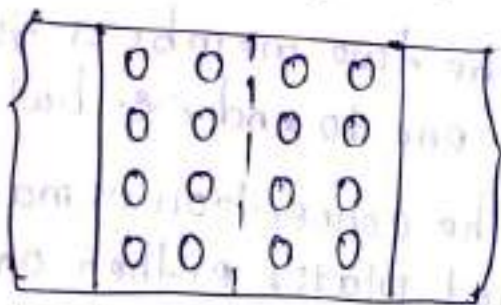
(double cover double
  bolted butt joint)

Terms used in bolted Connectuons :-

(1) pitch of bolt (p):- It is the distance between
the centres of two consecutive bolts in a row
measured along the direction of load.

(2) Gauge distance (g):- It is the distance between
two consecutive bolts of adjacent rows &
is measured at right angles. to the direction
of load.

(3) Edge distance/end distance (e):- It is the
distance of centre of bolt hole from the

adjacent edge or end of plate.

4. Staggered pitch :- (Ps) It is the centre to centre distance of staggered bolts measured along the direction of load.

(chain bolting).

(Staggered bolted connection)

$P_s \cdot P_s$
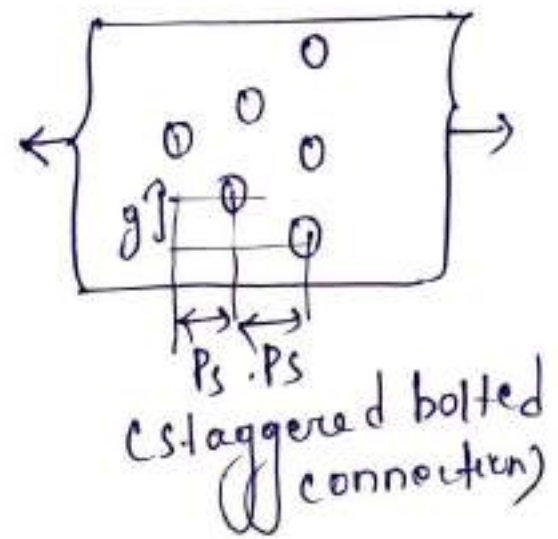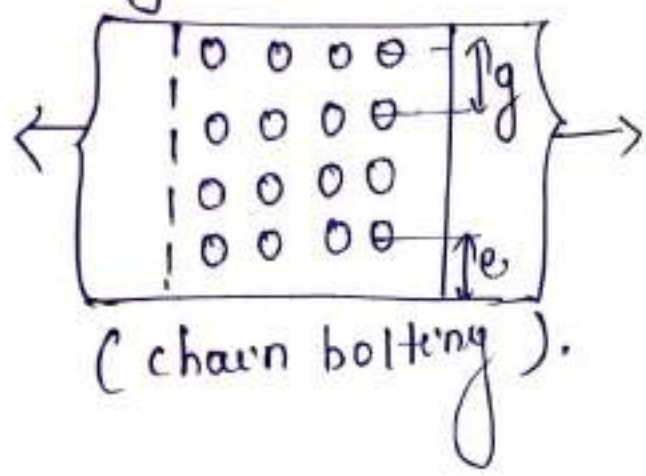
ssumptions in the analysis of bearing bolt.

following assumptions are made in the design of bearing type of bolted connection:-

1) The stress distribution on the plates between the bolt holes is uniform.

2) The friction between the plates is negligible.

3) The shearing stress is uniformly distributed over the cross section of bolt.

4) The bolts in a group share the direct load equality.

5) Bending stresses developed in bolts is neglected.

Coadal provisions for bolted joints:-

(i) Minimum pitch:- $2.5d$, where d = nominal diameter of bolt. (cl:10.2.2)

(ii) Maximum pitch (cl:10.2.3)

(a) $16t$ or $200mm$ whichever is less (for tensile member)

(b) $12t$ or $200mm$, whichever is less, (for compression member)

t = thickness of thinner member.

(iii) Maximum gauge distance:- $\left.\begin{array}{c}100+4t \\ \text{or} \\ 200\ mm\end{array}\right\}$ whichever is less. (cl:10.2.3.3)

(iv) Minimum edge distance:- (cl:10.2.4).

(a) $1.7do$ (for sheared or hand flame cut edges)

(b) $1.5do$ (rolled, machine flame cut, planed edge).

(v) Maximum edge distance:- (cl:10.2.4.3)

(a) $12t\epsilon$ $\left.\begin{array}{c} \\ \end{array}\right\}$ whichever is less.

(b) $40+4t$

where $\epsilon = \sqrt{\dfrac{250}{f_y}}$

Design strength of joint (bearing bolt)

Strength of bolt ————————→ Strength of plate (Rupture strength)

↓ ↓ Minimum will be the strength of joint.

bearing strength of bolt   **Shearing strength of bolt**

Minimum will be the strength of bolt.
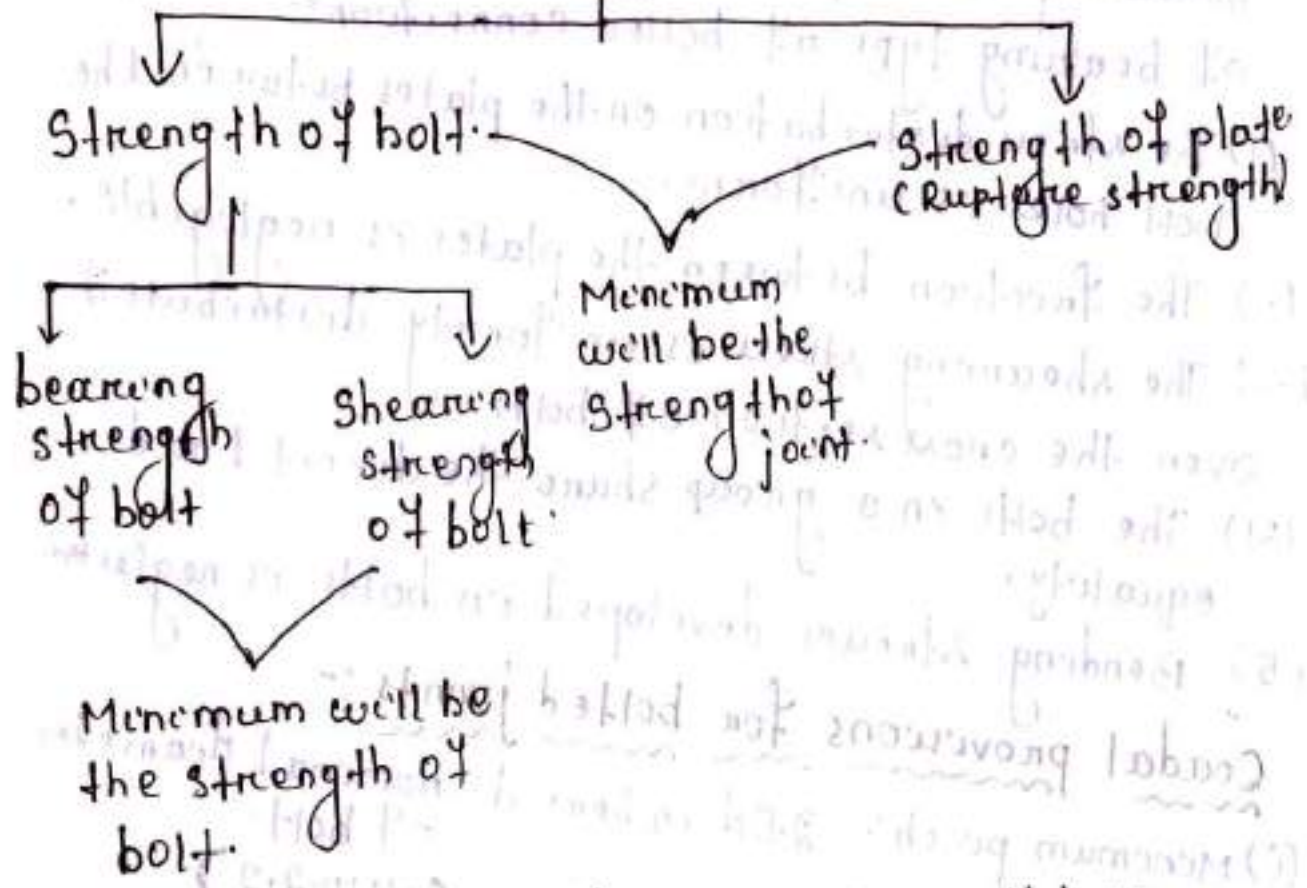
Design strength of bearing type of bolts on a joint :-

(A) shear strength of bolt   ⎱ Minimum

(B) Bearing strength of bolt   ⎰ will be taken.

(A) shear strength of bolt :- (cl : 10.3.3)

The design shear strength of the bolt (V_{dsb}) :-

$$V_{dsb} = \frac{V_{nsb}}{V_{mb}}$$

V_{mb} = partial safety factor of material of bolt (table No0-05)

V_{nsb} = nominal shear capacity of bolt.

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} \left(n_n A_{nb} + n_s A_{sb} \right)$$

f_{ub} = ultimate tensile strength of bolt

A_{sb} = nominal shank area of bolt = $\frac{\pi}{4} d^2$

A_{nb} = net area of bolt at threads = $0.78 \times \frac{\pi}{4} d^2$

n_n = number of shear planes with threads intercepting the shear planes.

($n_n = 1$ (for single shear)

($n_n = 1$) (for double shear).

n_s = number of shear planes without threads intercepting the shear plane.

($n_s = 0$ (for single shear)

($n_s = 1$ (for double shear))

Reduction factors for shear capacity of bolts: (cl: 10.3.3.1)

(i) Reduction factor for long joint:-

$$\beta_{lj} = 1.075 - \frac{l_j}{200 d} \quad (0.75 \leq \beta_{lj} \leq 1.0). \quad (\text{when } l_j > 15d)$$

(ii) Reduction factor for large grip length:- (β_{lg})

$$\beta_{lg} = \frac{8d}{3d + l_g} \quad (l_g > 5d) \qquad (cl: 10.3.3.2)$$

(iii) Reduction factor for packing plate (β_{pk})

$$\beta_{pk} = 1 - 0.0125 \, t_{pk} \quad (t_{pk} > 6mm) \quad (cl. 10.3.3.3)$$

t_{pk} = thickness of thicker packing in mm.

Fig.

$$\leftarrow y \rightarrow$$

if $y > 15d$, $lg > 5d$ & $tpk > 6$:-

then the modiffeed nomenal shear capacity

of bolt :-

$$Vnsb \neq \frac{fub}{\sqrt{3}} (nn\, Anb + ns\, Asb)\; Brj\; Brg\; Bpk.$$

(B) Bearing strength of bolt :- (cl. 10.3.4).

The dewgn strength of bolt, Vdpb

$$Vdpb = \frac{Vnpb}{\gamma mb.}$$

Vnpb = nomenal bearing strength of bolt

$$2.5\, kb\, d\, t\, fub.$$

$kb =$

$\dfrac{e}{3do}$

$\dfrac{P}{3do} - 0.25$

$\dfrac{fub}{fu}$

1

whichever is less.

e = end distance

P = putch distance

fub = ultumate strength of bolt

do = diameter of bolt hole.

d = nomenal diameter of bolt.

t = summation of thickness of coverplate / thickness of main plate less.

fu = ultimate strength of plate.
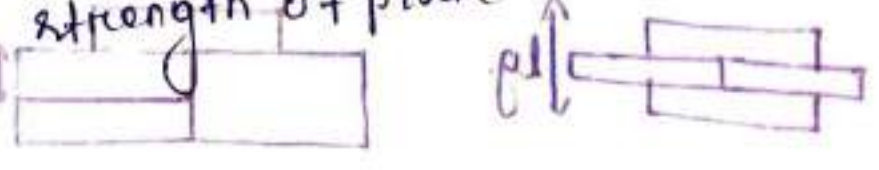
Rupture strength of plate :- (Cl: 6.3)

The design tensile strength of a plate in the joint is the strength of thinner member against rupture which is given by :-

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

γ_{m1} = partial safety factor for failure at ultimate stress = 1.25

f_u = ultimate stress of material

A_n = net effective area of plate at critical section

$$= \left(b - n d_o + \sum \frac{P_{si}^2}{4 g_i}\right) t \quad \text{(for staggered bolted connection)}$$

$$= (b - n d_o) t \quad \text{(for chain bolted connection)}.$$

Efficiency of a joint :-

$$\eta \ (efficiency) = \frac{Strength\ of\ joint}{Strength\ of\ solid\ plate} \times 100$$

Strength of solid plate :-

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

f_y = yield stress of material

A_g = gross area of cross section

γ_{mo} = partial safety factor for failure by yielding

= 1.1 (from table)

Q-7 find the maximum force that can be transmitted through a double bolted chain lap joint consisting of 6 bolts in 2 column. Given that M16 bolts of grade 4.6 & plates of fe 410 are used. Also find the efficiency of joint. (Given e=30, p=50).

Given data: Double bolted lap joint

Thickness of plate $t_1 = 10mm$, $t_2 = 12mm$.

Total no. of bolts = 6

Dia of bolt $d = 16mm$.

hole dia $d_0 = 16 + 2 = 18mm$.

end distance $(e) = 30mm$.

pitch $(p) = 50mm$.

Grade of bolt 4.6

$f_{ub} = 400$ MPa, $f_{yb} = 240$ MPa

$f_y = 250$ MPa.

ultimate strength of plate $f_u = 410$ MPa.

Required: efficiency of joint.

Solution:-

(A) Strength of bolt :-

(a) shear strength of a bolt :-

nominal shear strength $V_{nsb} = \dfrac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$

$f_{ub} = 400$, $A_{nb} = 0.78 \times \dfrac{\pi}{4} \times (16)^2$

$A_{sb} = \dfrac{\pi}{4} \times (16)^2$

for single shear (lap joint):-

$n_n = 1$, $n_s = 0$.
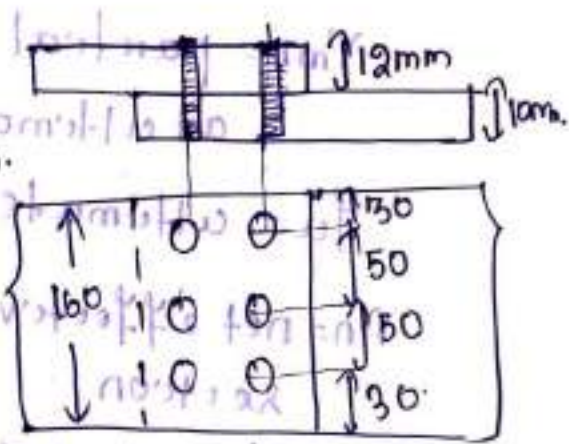
$$V_{nsb} = \frac{400}{\sqrt{3}} \left(1 \times 0.78 \times \frac{\pi}{4} \times 16^2 + 0 \times A_{sb}\right).$$

$$= 36.217 \, kN$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{36.216}{1.25} = 28.974 \, kN.$$

Design shear strength of a bolt $:- 28.974 \, kN$

Design shear strength of 6 no. of bolt $:-$

$$28.974 \times 6 = \boxed{173.846 \, kN.}$$

(b) Bearing strength of bolt $:-$

The bearing strength of bolt against the thinner plate will be critical.

Nominal strength / bolt $\cdot$ (V_{npb}) $:-$

$$V_{npb} = 2.5 \, k_b \, d \, t \, f_u$$

$d = 16 \, mm$

$f_u = 410$

$t =$ thickness of thinner plate $= 10 \, mm.$

$$k_b = \begin{cases} \dfrac{e}{3d_0} = \dfrac{30}{3 \times 18} = 0.56 \\[2mm] \dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 18} - 0.25 = 0.676 \\[2mm] \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.9756 \\[2mm] 1 \end{cases} \quad \begin{array}{l} \text{whichever} \\ \text{is less} \end{array}$$

$$k_b = 0.56.$$

$$V_{npb} = 2.5 \times 0.56 \times 16 \times 10 \times 410$$
$$= 91.840 \text{ kN per bolt}.$$

Design bearing strength per bolt

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{91.840}{1.25}$$

$$= 73.47 \text{ kN}.$$

Design bearing strength of 6 no. of bolt.

$$73.47 \times 6 = \boxed{440.832 \text{ kN}.}$$

Design strength of joint bolt in joint.

minimum of $\begin{cases} V_{dsb} = 173.846 \\ V_{dpb} = 440.832 \end{cases}$

$$= \boxed{173.846}$$

(B) Rupture strength of plate (T_{dn}).

$$T_{dn} = \frac{0.9 \, f_u A_n}{\gamma_{ml}} \qquad \begin{aligned} f_u &= 410, \\ \gamma_{ml} &= 1.25 \end{aligned}$$

$$A_n = (b - n \, do) \, t$$
$$= (160 - 3 \times 18) \times 10$$
$$= 1060 \text{ mm}^2$$

$$T_{dn} = \frac{0.9 \times 410 \times 1060}{1.25} = \boxed{312.912 \text{ kN}}$$

Strength of joint :- minimum of $\begin{cases} \text{strength of bolt} = 173.846 \text{ kN} \\ \text{rupture strength of plate} = 312.912 \text{ kN} \end{cases}$

$$= \boxed{173.846 \text{ kN}.}$$

Strength of solid plate (Tdg):- $\dfrac{Ag \cdot fy}{\gamma mo}$

$fy = 250 \, MPa$, $\gamma mo = 1.1$

$Ag = b \times t = 160 \times 10$
$\qquad = 1600 \, mm^2$

$Tdg = \dfrac{1600 \times 250}{1.1} = \boxed{363.636 \, kN}$

∴ efficiency of joint $\eta = \dfrac{\text{strength of joint}}{\text{strength of solid plate}} \times 100$

$\qquad = \dfrac{173.846}{363.636} \times 100$

$\qquad \boxed{\eta = 47.8 \%}$

Q-02 A single bolted double cover butt joint is used to connect two plates each 10mm thick. The bolts used were 20mm dia of grade 4.6 & cover plates were of 8mm thick. If 4 bolts were provided in a bolt line a pitch of 60mm with edge/end distance 40mm, Calculate the strength & efficiency of the joint.

8mm = t1
10mm = t2
8mm = t3

40
60
60
60
40
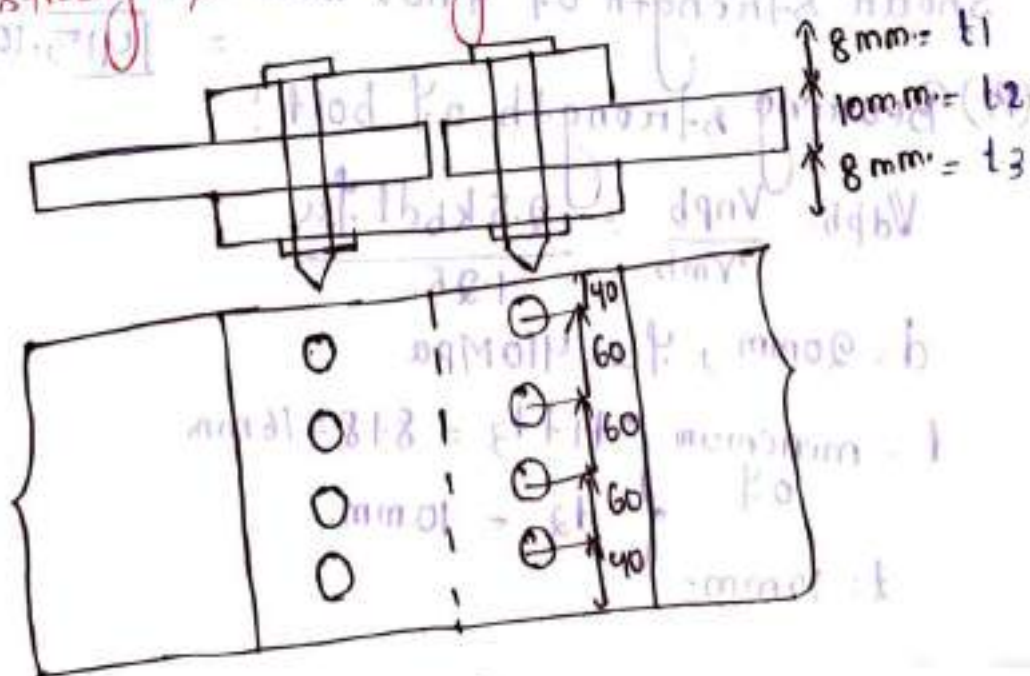
Given data:

$f_u = 410$ MPa, $f_{ub} = 400$ MPa ← for bolt grade 4.6

$f_y = 250$ MPa, $f_{yb} = 240$ MPa

$d = 20$ mm, $d_0 = 22$ mm.

$P = 60$ mm, $e = 40$ mm.

Solution:

(A) Strength of a bolt in joint:-

(a) Shear strength of bolt.

$$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times (20)^2 = 314 \, mm^2$$

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times (20)^2 = 245 \, mm^2$$

For butt joint $n_n = 1$, $n_s = 1$.

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3} \, \gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

$$= \frac{400}{\sqrt{3} \times 1.25} (1 \times 245 + 1 \times 314)$$

$$= 103.276 \, kN.$$

Shear strength of one bolt $= 103.276 \, kN.$

Shear strength of 4 no's bolt $= 103.276 \times 4$

$$= \boxed{413.105 \, kN.}$$

(B) Bearing strength of bolt.

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 \, k_b \, d \, t \, f_u}{1.25}$$

$d = 20$ mm, $f_u = 410$ MPa

$t =$ minimum of $\begin{cases} t_1 + t_3 = 8 + 8 = 16 \, mm \\ t_2 = 10 \, mm \end{cases}$

$t = 10$ mm.
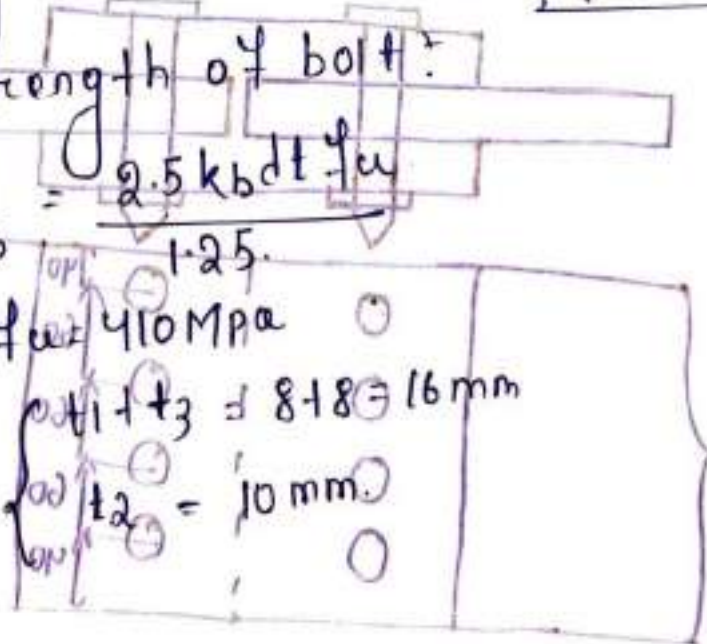
$$kb = \begin{cases} \dfrac{e}{3d0} = \dfrac{40}{3 \times 22} = 0.606 \\[3mm] \dfrac{P}{3d0} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.659 \\[3mm] \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976 \end{cases}$$ whichever is minimum

$$kb = 0.606$$

$$Vdpb = \dfrac{2.5 \times 0.606 \times 20 \times 10 \times 410}{1.25}$$

$$= 99.384 \, kN.$$

bearing strength of one bolt = 99.384 kN.

bearing strength of 4 no's bolt = 99.384×4

$$= \boxed{397.54 \, kN}$$

(A) Design strength of bolt in joint :-

for minimum of $\begin{cases} Vdsb = 413.105 \\ Vdpb = 397.54 \end{cases}$

$$= \boxed{397.54 \, kN}$$

(B) Rupture strength of plate :-

$$Tdn = \dfrac{0.9 An f_u}{\gamma_{m1}} \qquad f_u = 410, \; \gamma_{m1} = 1.25$$

$$An = (b - nd0) \, t$$
$$= (260 - 4 \times 22) \times 10 = 1720 \, mm^2$$

$$Tdn = \dfrac{0.9 \times 1720 \times 410}{1.25} = \boxed{507.74 \, kN.}$$

Design strength of joint :-

minimum of $\begin{cases} \text{Design strength of} \\ (397.54 \text{kN}) \text{ bolt} \\ \text{Rupture strength} \\ \text{of plate} (174.036\text{kN}) \\ (507.74 \text{kN}) \end{cases}$

$= 397.54 \text{ kN}.$

Strength of solid plate :-

$$T_{dg} = \frac{Ag \cdot f_y}{\gamma_{mo}}$$

$Ag = b \times t = 260 \times 10$
$= 2600 \text{ mm}^2$

$f_y = 250 \text{ MPa}$

$\gamma_{mo} = 1.1$

$$T_{dg} = \frac{2600 \times 250}{1.1} = 590.91 \text{ kN}$$

efficiency of joint $\eta = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$

$$= \frac{397.54}{590.91} \times 100$$

$$\boxed{\eta = 67.28\%.}$$

Design procedure for bearing bolted joint :-

For the design of a lap or butt joint, when the thickness of plates & force to be transmitted is known, the following are steps for design :

(1) The size of bolt is determined from Unwin's formula as $d = 6\sqrt{t}$

$t =$ thickness of plate in mm.

The diameter of bolts so computed is rounded off to available size of bolts.

(2) The strength of bolts in shear & bearing are computed assuming suitable value of pitch, edge distance. The minimum of the above is taken as bolt value & the number of bolts required is obtained by dividing the applied force by bolt value.

(3) The bolts are suitably arranged to produce a convenient & efficient joint.

(4) The joint is checked for rupture strength of plate with the assumed arrangement of bolts which should be more than the applied load.

Q-03 Two steel plates of 10mm & 12mm thick are to be joined by a lap joint so as to transmit a load of 120kN using 20mm dia bearing bolts of property class 4.6 & plates of grade fe410. Find the number & arrangement of bolts, if each of (i) plates are (ii) 100mm wide (iii) 200mm wide.

Given data : dia of bolt $(d) = 20mm$.
$$d_0 = 22mm$$
$$P_u = 120kN.$$
$$f_{ub} = 400 MPa.$$
$$f_u = 410 MPa.$$

(i) For 100 mm wide plate

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times (20)^2$$
$$= 245 mm^2$$

$$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times (20)^2$$
$$= 314 mm^2$$

$$\gamma_{mb} = 1.25$$

$$V_{dsb} = \frac{1}{\gamma_{mb}} \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

$$= \frac{1}{1.25} \times \frac{400}{\sqrt{3}} (1 \times 245 + 0 \times 314)$$

$$= 45.26 \, kN.$$

Design strength of a bolt in bearing:-

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}}$$

$f_u = 410 \, Mpa$, $\gamma_{mb} = 1.25$

$d = 20 \, mm.$

$t = 10 \, mm.$

$$k_b = \begin{cases} \frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.606 \\ \frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.66 \\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976 \\ 1 \end{cases} \quad \text{whichever is less}$$

$$k_b = 0.606.$$

$$V_{dpb} = 2.5 \times 0.606 \times 20 \times 10 \times \frac{410}{1.25}$$

$$= 99.38 \, kN.$$

Design strength of a bolt = minimum of $\begin{cases} V_{dsb} = 45.26 \\ V_{dpb} = 99.38 \end{cases}$

$$= 45.26.$$

Number of bolts required to transmit a load of 120 kN:-

$$n = \frac{120}{45.26} = 2.65 \approx 3 \, nos.$$

(i) The bolts are to be arranged along the length in a row because width is not sufficient to accomodate them in a row along the width.
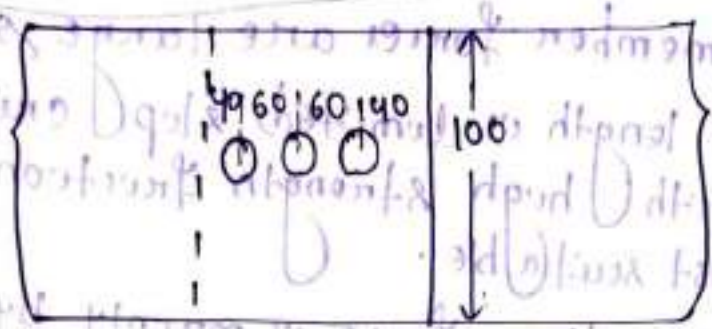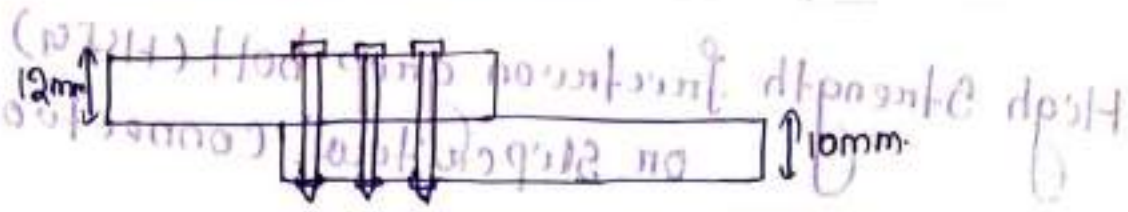
Check for rupture strength of plate :-

$$An = (b - ndo)t$$
$$= (160 - 1 \times 22)10 = 780 \, mm^2$$
$$Tdn = \frac{0.9 \, An \, fu}{\gamma_{m1}} = \frac{0.9 \times 780 \times 410}{1.25} = 230.260 \, kN > 120 \, kN \quad (ok)$$

(ii) When each plate is 200 mm wide :-
To reduce the length of joint, bolts may be arranged along the width in a row :-

check for rupture strength of plate :-
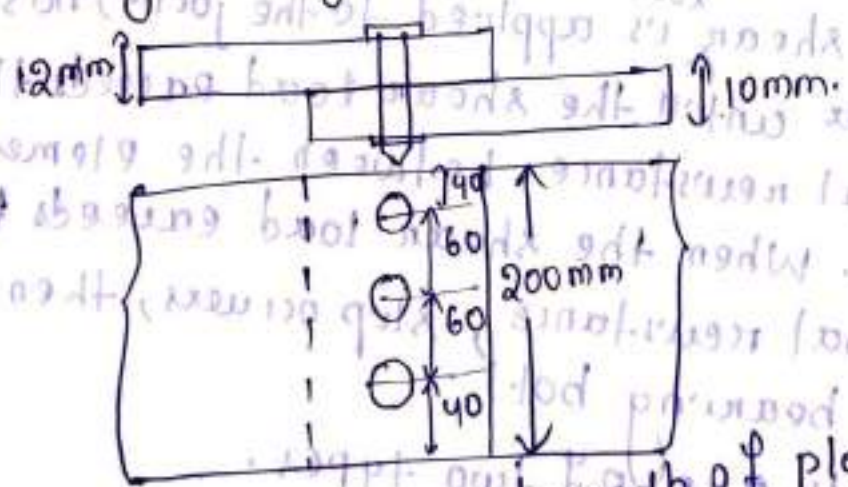
$$An = (b - ndo).t$$
$$= (200 - 3 \times 22) \times 10 = 1340 \, mm^2$$
$$Tdn = \frac{0.9 \, An \, fu}{\gamma_{m1}} = \frac{0.9 \times 1340 \times 410}{1.25}$$
$$= 395.57 \, kN > 120 \, kN \quad (ok).$$

High Strength friction grip bolt (HSFG)
            or supercritical connection.

→ When the member forces are large, where the connection length is limited, slip critical. connection with high strength friction grip bolts are most suitable.

→ Resistance to shear force is mainly by friction & shear or bearing are not the criteria for load transmission as in case of bearing type bolts.

→ The nut is tightened to develop a clamping force on the plates which is indicated as the tensile force in the bolt; this tension is about 90% of proof load.

→ When a shear load is applied to the joint, no slip well occur until the shear load exceeds the frictional resistance between the elements jointed. When the shear load exceeds the frictional resistance, slip occurs, then it acts as bearing bolt.

→ HSFG bolts are of two types.

(i) parallel shank type.
(ii) Waisted shank type.

(i) parallel shank type:-

(a) → parallel shank type HSFG bolts are designed for no slip at serviceability loads. Hence they slip at higher loads & slip into bearing at ultimate load. therefore such bolts should be checked for their bearing strength at ultimate load.

(ii) Inlaisted shank type:-

→ Inlaisted shank HSFG bolts are designed for no slip even at ultimate load & hence there is no need to check for their bearing strength.

Shear Capacity of HSFG bolts :- $(c1:10.4.)$
is 800

The design slip resistance = $\dfrac{\text{nominal shear capacity of bolt}}{\gamma_{mf}}$

$$\boxed{V_{dsf} = \dfrac{\mu_f n_e K_h F_o}{\gamma_{mf}}}$$

μ_f = coefficient of friction (slip factor)
(table 20).

n_e = number of effective interfaces offering fructional resistance to slip.

for Lap joint, $n_e = 1$
for butt joint, $n_e = 2$.

$K_h = 1$ for fasteners in clearance holes

$= 0.85$ for fasteners in oversized & short slotted holes & for Long slotted holes Loaded perpendicular to the slot

$= 0.70$ for fasteners in Long slotted holes Loaded parallel to the slot

T_o = minimum bolt tension (proof load) at installation = $A_{nb} f_o$

A_{nb} = Net area of bolt at heads = $0.78 \times \frac{\pi}{4} d^2$.

f_o = proof stress = $0.70 f_{ub}$

$\gamma_{mf} = 1.1$ (if slip resistance is designed at service load)

$= 1.25$; (if slip resistance is designed at ultimate load)

Reduction factor for shear capacity of HSFG bolt

The provision for long joints in 10.3.3.1 shall apply to friction grip connections also.

Ⓠ Two plates of 12mm thick are joined by double cover butt joint with 20mm HSFG bolts of property class 8.8 & cover plates of 8mm thick. Assuming that the fasteners are in clearance holes & slip factor as 0.30, determine the shear capacity of a bolt if (i) slip resistance is designed at service load (ii) slip resistance is designed at ultimate load

For HSFG bolt of grade 8.8 :-
$$f_{ub} = 800 \, N/mm^2$$

For fasteners in clearance holes $k_h = 1$
slip factor $(\mu_f) = 3$ (Given)

Nominal shear capacity of bolt $(V_{nsf}) \Rightarrow$
$$V_{nsf} = \mu_f \, n_e \, k_h \, f_0$$

For butt joint $n_e = 2$,
$$f_0 = 0.7 \, f_{ub} \, A_{nb}$$

$$= 0.7 \times 800 \times 245.04 \qquad \left(A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2 \right)$$

$$= 137.22 \, kN \qquad = \left(0.78 \times \frac{\pi}{4} \times 20^2 \right)$$

$$V_{nsf} = \mu_f \, n_e \, k_h \, f_0 \qquad\qquad\qquad = 245.04$$

$$= 0.3 \times 2 \times 1 \times 137.22$$

$$= \cancel{823.32} \; 82.332 \, kN$$

ⅰ) If slip resistance is designated at service load :-
Design shear capacity of bolt $(V_{dsf}) = \dfrac{V_{nsf}}{\gamma_{mf}}$

$$V_{dsf} = \frac{\cancel{823.32} \; 82.332}{1.1} = \cancel{748.50 \, kN} \; 74.85 \, kN.$$

ⅱ) If slip resistance is designated at ultimate
load $\gamma_{mf} = 1.25$

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{\cancel{823.32}}{1.25} \; \frac{82.332}{1.25}$$

$$= 65.86 \, kN$$

Q An ISA 110×110×8 mm carries a factored tensile force of 60kN. It is connected to a 12 mm thick gusset plate. Design a high strength bolted joint when (a) no slip is permitted (b) slip is permitted. Steel is of grade Fe418. Assume bolts in clearance holes & slip factor as 0.3.

Solution:- Dia of bolt, $d = 6\sqrt{t}$

$$= 6\sqrt{8}$$
$$= 16.97 \text{ mm}$$
$$\approx 16 \text{ mm}$$

For HSFG bolt of property class 8.8:-

$f_{ub} = 800 \text{ MPa}$, $A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2$

$$= 0.78 \times \frac{\pi}{4} \times (16)^2$$
$$= 156.83 \text{ mm}^2$$

For Fe410 grade of steel; $f_u = 410 \text{ MPa}$

a) when slip is not permitted (slip critical connection):-

proof load $f_0 = A_{nb} \times 0.7 f_{ub}$
$$= 156.83 \times 0.7 \times 800$$
$$= 87.824 \text{ kN}$$

design shear capacity of bolt,

$$V_{sf} = \frac{\mu_f \, n_e \, k_n \, f_0}{\gamma_{mf}}$$

For lap joint ne = 1
For bolts in clearance hole kn = 1

$$V_{sf} = \frac{0.3 \times 1 \times 1 \times 87.824}{1.25}$$
$$= 21.10 \text{ kN}$$

(γ_{mf} = 1.25 for slip resistance at ultimate load)

No. of bolts required =

$$\frac{external\ load}{shear\ capacity\ of\ a\ bolt} = \frac{60}{21.16}$$

$$= 2.84 \approx 3\ nos.$$

Hence provide 3 nos of 16mm dia 8.8 grade HSFG bolts for making the connection.

(b)
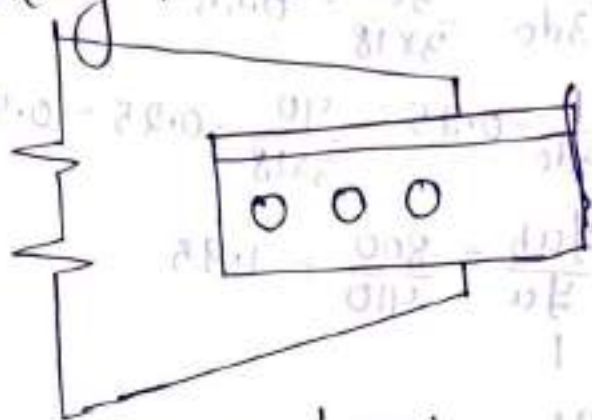
(b) Bearing type Connection :-

For bearing type connection we will design for bearing strength & shearing strength.

For Lap joint $n_n = 1, n_s = 0,$

(i) Strength of the bolt in single shear :-

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left\{\frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\right\}$$

$$= \frac{1}{1.25}\left\{\frac{800}{\sqrt{3}}(1\times157+0)\right\}$$

$$\boxed{V_{dsb} = 58.01\ kN}$$

(ii) Strength of bolt in bearing :-

$$V_{dpb} = \frac{2.5\ k_b d\ t\ f_u}{\gamma_{mb}.}$$

$\gamma_{mb} = 1.25, \quad d_0 = 16+2 = 18\text{mm}$

Assume $e = 1.5 d_0 = 1.5 \times 18$

$\qquad = 27\text{mm}$

$\qquad \approx 30\text{mm}.$

$P = 2.5d = 2.5 \times 16$

$\qquad = 40\text{mm}.$

$K_b = \begin{cases} \dfrac{e}{3d_0} = \dfrac{30}{3 \times 18} = 0.55 \\[3mm] \dfrac{P}{3d_0} - 0.25 = \dfrac{40}{3 \times 18} - 0.25 = \boxed{0.491} \\[3mm] \dfrac{f_{ub}}{f_u} = \dfrac{800}{410} = 1.95 \\[3mm] 1 \end{cases}$ least of

$K_b = 0.491$

$V_{dpb} = 2.5 \, k_b \, d \, t \, \dfrac{f_u}{\gamma_{mb}}$

$\qquad = 2.5 \times 0.491 \times 16 \times 08 \times \dfrac{410}{1.25}$

$\boxed{V_{dpb} = \quad 51.54 \text{ kN.}}$

Strength of bolt = minimum of $\begin{cases} V_{dsb} = 58.01\text{kN} \\ V_{dpb} = 51.54\text{kN} \end{cases}$

$\qquad = 51.54 \text{ kN.}$

No. of bolts required $= \dfrac{60}{51.54} = 1.16 \approx 2$ NOS.

Hence provide 2 nos of 16mm dia, HSFG bolts.
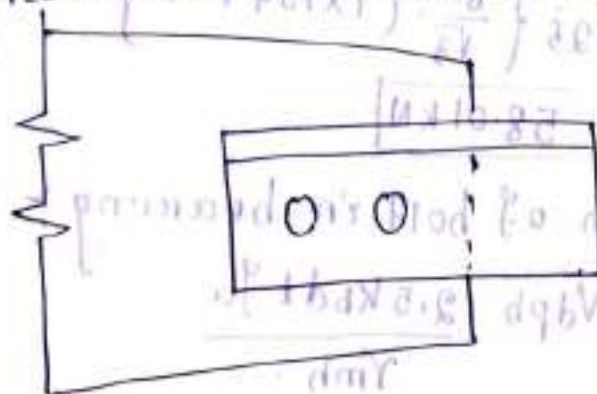
Welded Connection.

Welding is a method of connecting two pieces of metal by heating to a plastic (or) fluid state.
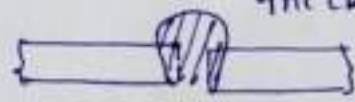
→ The elements to be connected are brought closer & the metal is melted by means of electric arc along with weld rod which adds metal to the joint. The bond is established between the two elements after cooling.

Types of welded joint :-

(i) butt weld   (iii) slot weld & plug weld

(ii) fellet weld

(1) Butt weld :- This is also known as groove weld.

→ Butt welds are provided when the members to be joined are placed end to end or aligned in the same plane. → (used for plates upto 8 mm thickness).

→ Square butt weld on one side.   [▨]

→ Square butt weld on both side ; [⧖]

→ Single V butt joint :-   [▽]

→ Double V-butt joint :-   [⧖]

→ Single U butt joint :-   [⨆]

(2) Fillet weld :- Fillet welds are provided when two members to be joined are in different planes ;

[◣]

Figure
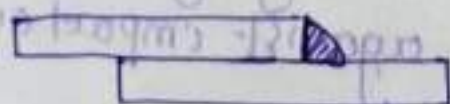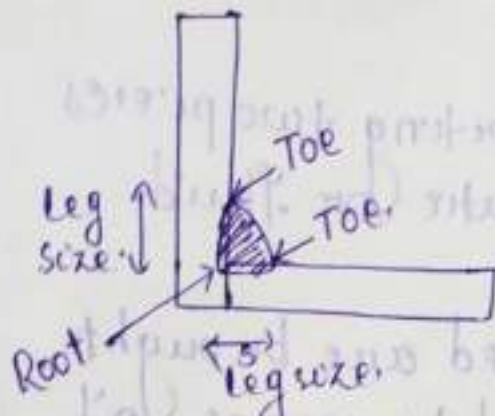
(iii) **Slot & plug welds :-** Slot & plug welds are used to supplement the fillet welds, when the required length of fillet weld can not be provided.

-> In slot weld, slots are made on one of the plates, which is kept with another plate & then fillet welding is made along the periphery of hole.

-> In case of plug weld, small holes are made in one plate & is kept over another plate to be connected & then entire hole is filled with fillet material.

Advantages of welded connection.

(i) welding is more adaptable than bolting or reveting, as even circular sections can be easily connected by welding.

(ii) 100% efficiency can be achieved in contrast to bolted connection.   deduction

(iii) Since there is no deduction for holes; the gross section is effective in carrying loads.

(iv) Better resistant against impact or vibrative loads.

(v) Results in light structure due to absence of connecting plates, gusset plates etc.

(vi) Noise pollution is nearly eliminated.

(vii) presents good asthetic appearance.

(viii) Connections are water & air tight.

Disadvantages of welded connection :-

(i) Skilled labour & electricity is neccesary for welding.

(ii) Welded joints are more brittle.

(iii) Difficult to detect defects like internal air pockets.

(iv) Welded joints are over rigid & proper welding in field condition is difficult task.

IS code provisions for welding :-

(1) Butt weld. :-

(i) Reinforcement :- It is the extra material deposit above the surface level of parent metal to be connected. (It increases efficiency of joint).

→ but in case of repeated or vibrating loads stress concentration develops in the reinforcement leading to early failure.

→ Maxm reinforcement = 3mm

→ Reinforce ment

te
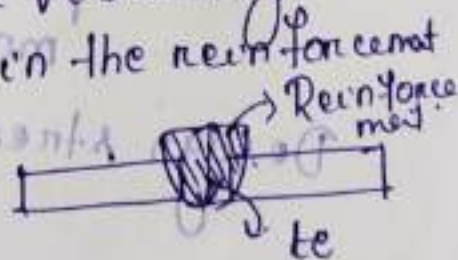
(ii) Size of weld | $\frac{Tw \times Lw}{...}$ | =

size of weld = effective throat thickness (te)

for complete penetration :
size of weld = effective throat
thickness = thickness of thinner member (t_{min}).

For incomplete penetration :-

Size of weld (s) = effective throat = $\frac{5}{8} \times tmin$.
thickness
(te)

(iii) effective length (Leff)

= length of full size weld;

(v) effective area = effective × effective
of weld   throat   length of
thickness (te)   butt weld
(Leff).

* Minimum Length of weld = 4 × size of weld.

Design strength of butt weld :-

The stress in weld shall not exceed those
permitted stress in the parent metal

→ It can resist external load by tension or
compression & shear action

Design stress of butt / groove weld in tension
or (compression) $fdw = \dfrac{fy}{\gamma mw}$

fy = yield stress of material
γmw = partial safety factor for weld
material (1.25). (from table 5)

Design strength of butt weld
= Design stress × Area of
weld   weld.

(Aw = te × Le)

= $\boxed{\dfrac{fyw \times Aw.}{\gamma mw}}$

Design strength of weld in shear :-

= $\boxed{\dfrac{fyw}{\gamma mw \times \sqrt{3}} \times Aw}$

4

Reduction of design stress for Long joints:-
when the length of joint (L_j) > 150 t_t. (t_t = throat thickness)
the design capacity of weld (f_{wd}) shall be reduced by a factor:-

$$\beta_{tw} = 1.2 - \frac{0.2 L_j}{150 \, t_t} \leq 1$$

Q:-1 Two steel plates of 16mm & 12mm thick are to be joined by butt welding. If effective length of weld is 180mm, determine the strength of joint for following cases (i) single V groove weld joint (ii) Double V groove weld joint.

(i) Single V butt weld joint:-

As for incomplete penetration:

$t_1 = 16$mm $t_2 = 12$mm
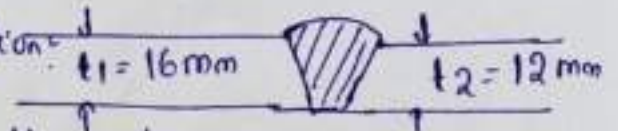
Size of weld = effective throat thickness = $\frac{5}{8}$ t min

$$= \frac{5}{8} \times 12$$
$$= 7.5 \text{mm.}$$

effective Length of weld (L_e) = 180mm.

Design strength of welded joint =
$$\frac{f_{yw}}{\gamma_{mw}} \times (L_e \times t_e)$$

$$= \frac{250}{1.25} \times 180 \times 7.5$$

$$= 270 \text{kN}$$

(ii) Double V butt weld joint:-
For complete penetration:

Size of weld = effective throat thickness = t min = 12mm.
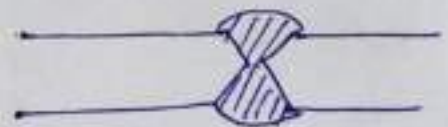
Design strength of weld :-

$$= \frac{fy}{\gamma_{mw}} (\text{lext e})$$

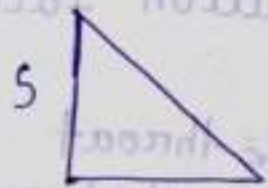$$= \frac{250}{1.25} (180 \times 12)$$

$$= 432 \text{ kN}$$

Fillet Weld

IS code provisions for welding :-

(i) **Size of weld :-**

Generally the size of weld is taken as the minimum weld Leg size.

Minimum size of weld :- (table No 21)

Minimum size of weld depends upon thickness of thicker member.

Thickness of thicker part (mm)	Minimum size of weld (mm).
1-10	3
11-20	5
21-32	6
33-50	8 of first run, 10 for minimum size of weld.

(ii) **Maximum size of weld :-**

for square edge of plate

for rounded toe.

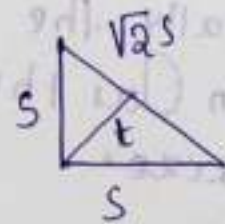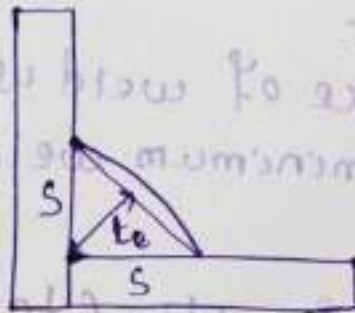$$S_{min} = t - 1.5$$

(cl : 10.5.8.1 / 10.5.8.2)
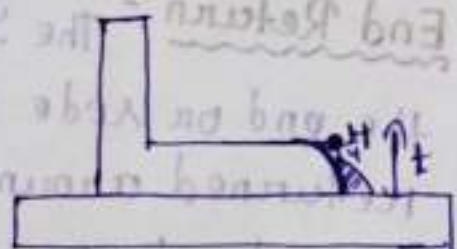
$$S_{min} = \frac{3}{4} t$$

Throat Thickness:- The throat of a fillet is the length of perpendicular from the right angle corner to hypotenuse. The effective throat thickness is calculated as :-

> Throat thickness $= k \times$ fillet size of fillet weld.
> (te)

(Cl No: 10.5.3.2)

The value of k depends upon the angle between fusion faces.

$t \dot{=}$ throat thickness

$$t = \frac{\sqrt{2}s}{2} = \frac{s}{\sqrt{2}}$$
$$= 0.7s.$$

K value (table No-22).

Angle between fusion faces.	60-90°	91-100°	101-106°	107-113°	114-120°
constant (k).	0·7	0·65	0·60	0·55	0·50.

* In most cases, a right angled fillet is used, for which $k = 0.7$.

* Minimum value of throat thickness (te):- 3 mm
  Maximum value of throat thickness :- 0.7t
  (temax)

  $t =$ thickness of thinner plate

End Return:- The fillet weld terminating at the end or side of a member should be returned around the corner. (Cl: 10.5.1.1)

> end return $= 2 \times$ size of weld cs.

7

Assumptions in the analysis of welded joint:

a) Welds connecting various parts are homogenous isotropic & elastic.

b) The parts connected by the welds being rigid, their deformation is usually neglected.

c) Only stresses due to external loads are considered & effects of residual stresses, stress concentration & shape of welds are neglected.
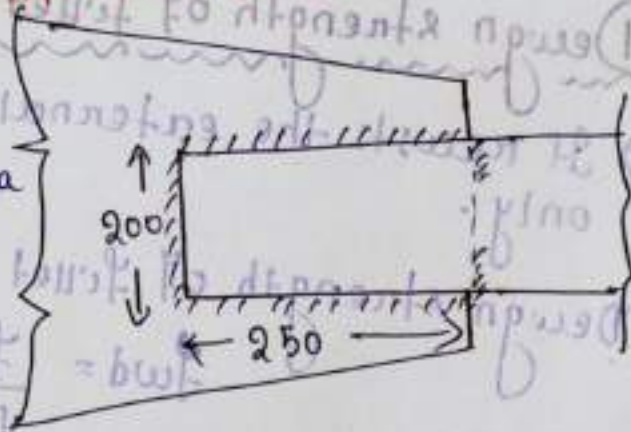
Q A steel plate 200mm × 12mm is welded to a 10mm thick gusset plate such that the overlap of member is 250mm. If fillet weld of size 6mm is used for the connection, determine the design strength of the joint. Given that shop welding is to be done on three sides & grade of steel is Fe 410.

Given data:
$f_u = 410 MPa$, $f_y = 250 MPa$

For shop welding,
$\gamma_{mw} = 1.25$ (Table No. 05)

$l_w = 2 \times 250 + 200 = 700 mm$ (Assuming.

$l_e = l_w - 2 \times 2 \times s$

$= 700 - 4 \times 6$

$\Rightarrow 700 + 24 = 676 mm$

Effective throat thickness (te) = $k \times s$ (for 90° angle $k = 0.7$)

$= 0.7 \times 6 = 4.2$

Design strength of weld:-

$$T_{dw} = L_w \cdot t_e \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 876 \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25}$$

$$= 537.661 \, kN$$

Design strength of plate:-

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}}$$

$$A_g = 200 \times 12$$
$$= 2400$$
$$f_y = 250 \, MPa$$
$$\gamma_{mo} = 1.1$$

$$= \frac{2400 \times 250}{1.1}$$

$$T_{dg} = 545.45 \, kN$$

Design strength of joint $\doteq$ minimum of $\begin{cases} T_{dw} = 537.661 \\ T_{dg} = 545.45 \end{cases}$

$$= 537.661 \, kN.$$

Design procedure for fillet weld:-

(1) The size of weld is selected based on the thickness of members to be joined.

(2) Depending on the angle between fusion faces, effective throat thickness is calculated.

(3) If force to be transmitted is not given, gross design strength should be taken as the rupture strength of plate to develop maximum force.

(4) strength of weld per mm length is calculated.

(5) effective length of weld required is calculated by dividing the factored load on design strength
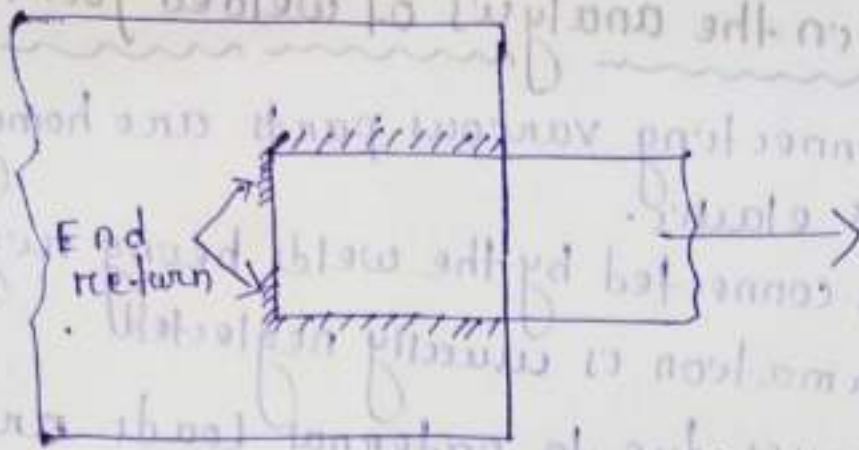
Effective Length :- (cl;10:5:4)

effective length (Le) = overall length — end return
of weld (2 × S)

→ effective length of weld is designed to transmit load.

→ Minimum effective length = 4 × size of weld (S).

Lap Length — Minimum lap = 4 × tmin or 40mm } whichever is more.
Length

Design strength of fillet weld :- (cl; 10.5.7).

→ It resists the external load by shear action only.

Design strength of fillet weld :-

$$fwd = \frac{fu}{\sqrt{3} \times \gamma mw} \times \text{Area of weld}$$

$$\boxed{fwd = \frac{fu}{\sqrt{3} \times \gamma mw} \times (Le \times te)}$$

fu = ultimate stress of weld

γmw = partial safety factor (table 5).

Le = [L - 2×S], te = k × S.

by the strength of weld per mm length.

(6) Total length of weld is suitably arranged.

(7) If the length of weld in the direction of load exceeds 150tt, the design capacity of weld is reduced by reduction factor for long joints.

(8) Check for minimum lap joint.

(9) End returns of length equal to twice the size of weld at each end of fillet weld are provided.

Q. Design a suitable fillet weld to connect a tie bar 60mm × 8mm to a 12mm thick gusset plate so as to develop maximum force if

(i) Shop welding is done on two sides.

(ii) Field welding is done on three sides.

Assuming the grade of steel for tie bar as fe 410

$$f_u = 410 MPa, \; f_y = 250 MPa, \; \gamma_{mo} = 1.1$$

For 12mm thick gusset plate,

Minimum size of weld = 5 mm

For 8mm thick tie bar,

Maximum size of weld = 8-1.5

= 6.5mm.

Hence, let us provide a weld size of (S) = 6 mm.

effective throat thickness (te) = 0.7 × S

= 0.7 × 6

= 4.2 mm.

To develop maximum force, the design strength of weld should be equal to strength of the plate strength of plate in yielding

$$T_{dg} = \frac{A_g\, f_y}{\gamma_{mo}} = \frac{(b \times t)\, f_y}{\gamma_{mo}} = \frac{60 \times 8 \times 250}{1.1}$$

$$= 109.09 \text{ kN}.$$

(c) For shop welding on two sides, $\gamma_{mw} = 1.25$

(table-5).

Strength of weld per mm length :-

$$f_{dw} = L_w t_e \frac{f_u}{\sqrt{3}\, \gamma_{mw}} = L_w \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25}$$

$$= 0.795 \text{ kN per } L_w$$

For maximum efficiency :-

$$T_{dg} = f_{dw}$$

$$\Rightarrow 109.09 = 0.795 \, L_w$$

$$\Rightarrow L_w = 137.22 \text{ mm}$$

$$\approx 140 \text{ mm}.$$

Length of weld on each side $= \frac{140}{2} = 70 \text{ mm}.$

$L_w = 140 \text{ mm} > 4 \times 5 \quad (\text{Ok})$
$\qquad = 4 \times 6$
$\qquad = 24 \text{ mm}$

minimum Lap length

$$\text{whichever is more} \begin{cases} = 4 \times t_{min} \\ = 4 \times 8 \\ = 32 \text{ mm} \\ \text{or} \\ 40 \text{ mm} \end{cases}$$

$= 40 \text{ mm.} \qquad > 70 > 40 \text{mm} \quad (\text{ok}).$
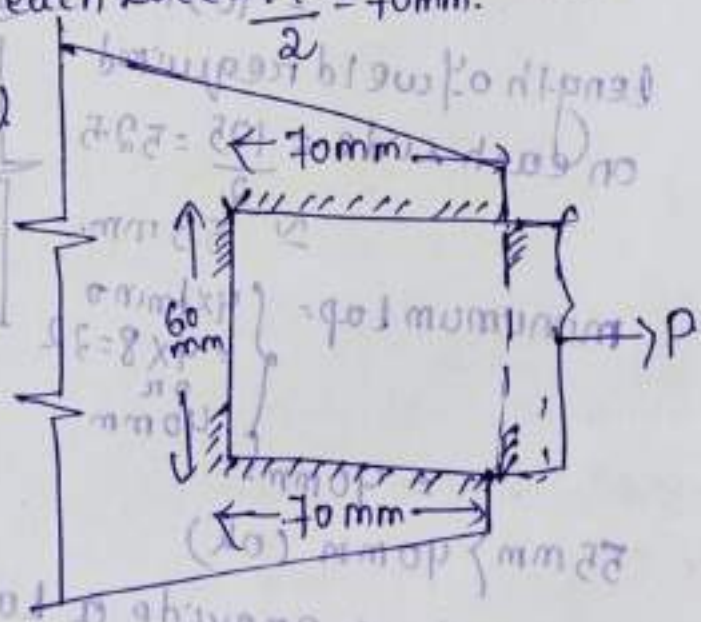
Length on each side $= 70 + 2 \times 2 \times 6$

$= 94\,mm$.

Total length of weld $= 94 \times 2 = 188\,mm$.

(ii) For fillet welding on three sides :-

$\gamma_{mw} = 1.5$ (table-5).

strength of weld per mm length:

$$lw \cdot te \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$lw \times 4.2 \times \frac{410}{\sqrt{3} \times 1.5} = 0.663\,lw$$

For maximum efficieny :-

$T_{dg} = 0.663\,lw \implies 109.09 = 0.663\,lw$

$\implies lw = 164.54\,mm$

$\implies lw \approx 165\,mm$. $\left\{ \begin{array}{l} 4 \times 5 = 4 \times 6 \\ = 24\,mm \\ (ok). \end{array} \right.$
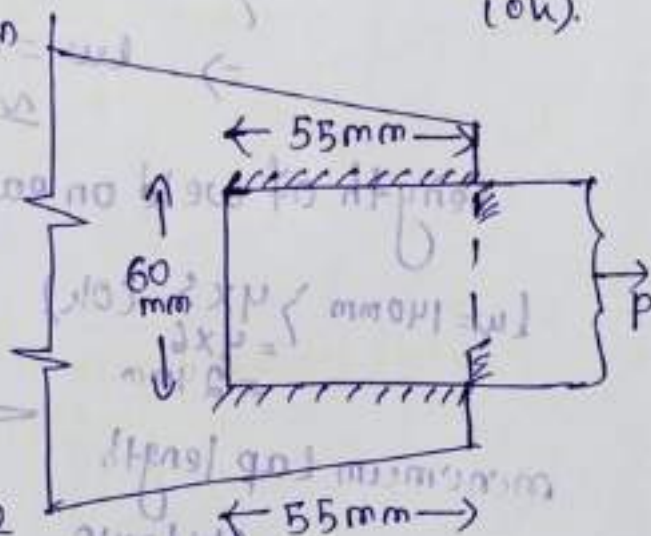
length of weld required on both sides $\Rightarrow 165 - 60$

$= 105\,mm$.

length of weld required on each side $= \frac{105}{2} = 52.5$

$\approx 53\,mm$.

$9 < $ minimum Lap $= \left\{ \begin{array}{l} 4 \times tmin = \\ = 4 \times 8 = 32 \\ \text{or} \\ 40\,mm \end{array} \right.$
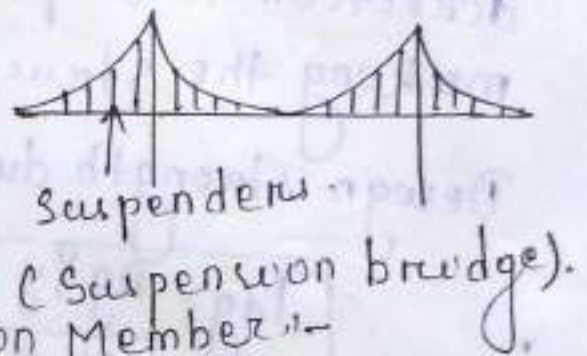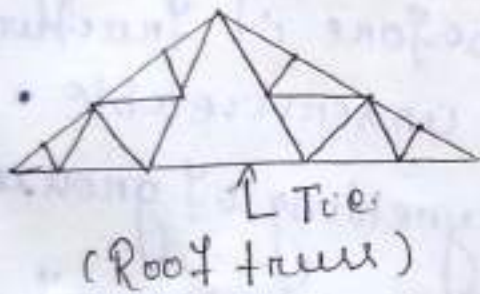
$= 40\,mm$

$55\,mm > 40\,mm$ (ok)

Hence let us provide a Lap of 55mm & 6mm size fillet weld.

welds on three sides with end return:-

Total length of weld $= 60 + 2 \times 55 + 24$ $2 \times 2 \times 6$

$= 194\,mm$ $= 24\,mm$.

Tension Member

Tension members are linear members pre-dominatly subjected to pulling (direct axial tensile load) which tend to stretch or elongate the member.

Ex Rope, tie of roof truss, suspenders of suspension bridge.

Tie
(Roof truss)

Suspenders
(Suspension bridge).

Common Shapes of Tension Member :-

(angle section)   (double angle section)   (channel section)

(I section)   (circular section)   (steel flat)   (built up section)
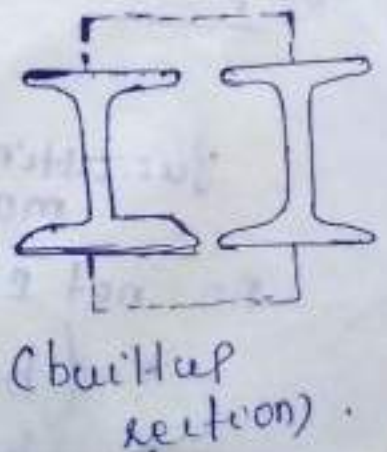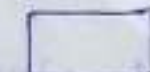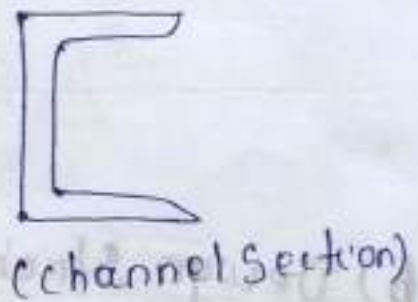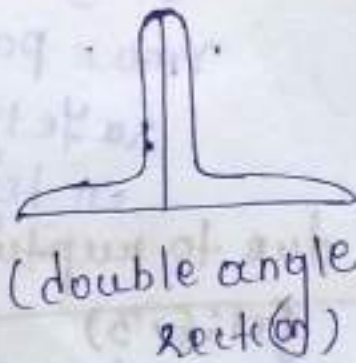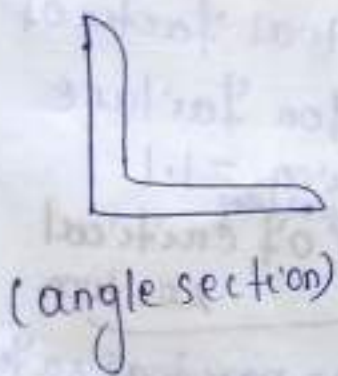
Types of failures of tension Members :-

(a) yielding of gross section

(b) Rupture of net critical section.

(c) block shear failure.

(a) Yielding of gross section :- (cl : 6.2, page No 32)

yielding of gross section occurs when considerable deformation of member in longitudinal direction takes place before it fractures, making the structure unserviceable.

Design strength due to yielding of gross section

$$T_{dg} = \dfrac{A_g f_y}{\gamma_{mo}}$$

f_y = yield stress of material

A_g = gross area of section.

γ_{mo} = partial factor of safety for failure in tension by yielding = 1.1

b) Design Strength due to rupture of critical section.

(cl : 6.3)

(i) plates

$$T_{dn} = \dfrac{0.9 A_n f_u}{\gamma_{m1}}$$

where γ_{m1} = partial safety factor for failure at ultimate stress = 1.25

f_u = ultimate stress of material

A_n = net effective area of member :-

$$= \left(b - n d_0 + \sum_{i=1}^{n} \dfrac{p_{si}^2}{4 g_i}\right) t \quad (\text{for staggered bolted connection})$$

$$= (b - n d_0) t \quad (\text{for chain bolted connection}).$$

b = width of plate

do = diameter of bolt hole.

g = gauge distance between bolt holes.

Ps = staggered pitch distance between the line of bolt holes.

t = thickness of plate.

n = number of bolt holes on the critical section.

(b) Threaded rods:-

$$Tdn = \frac{0.9 \, An \, fu}{\gamma_{mL}} \quad (cl: 6.3.2)$$

An = net root area at the threaded section

$$= 0.78 \times \frac{\pi}{4} \times d^2.$$

(c) Single angle :- (cl: 6.3.3).

Rupture strength $(Tdn) = \dfrac{0.9 \, Anc \, fu}{\gamma_{mL}} + \dfrac{\beta A_{go} \, fy}{\gamma_{mo}}$.
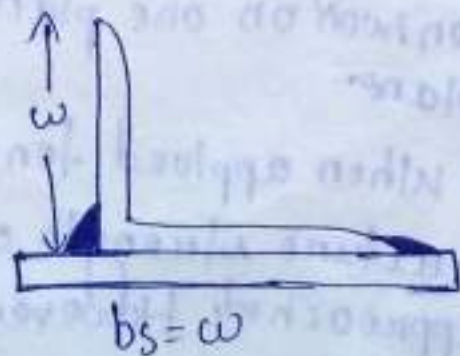
Anc = Net area of connected leg.

Ago = Gross area of outstanding leg.

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{fy}{fu}\right)\left(\frac{bs}{Lc}\right) \leq \frac{fu \, \gamma_{mo}}{fy \, \gamma_{mL}} \not> 0.7.$$

$$\not> 0.7$$

w = outstand leg width

bs = shear lag width.

bs = w + wi - t

outstand leg width ← w →

← wi →

connected leg width

w

bs = w
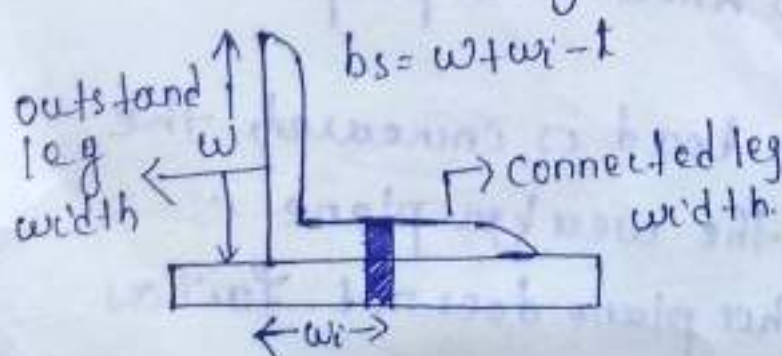
t = thickness of the leg.

L_c = length of end connection, that is the distance between the outermost bolts in the end joint measured along the load direction.

or when the segments of welds in the end connections are of different length in the direction of load, the length of longest segment will be taken as L_c.
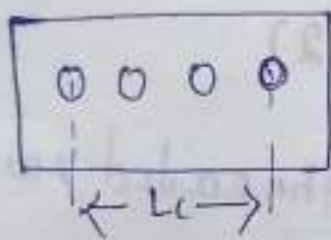
* The rupture strength T_{dn} of double angles, channels, T sections & other rolled steel sections may be calculated by the same equation as for single angle, but width bs taken from the farther edge of the outstanding edge of outstanding leg to nearest bolt in connected leg.

Design strength due to block shear :-

This type of failure occurs along a path involving tension on one plane & shear on a perpendicular plane.

→ When applied tensile load is increased, the fracture strength of the weaker plane is approached. However, this plane does not fail as

it is restrained by stronger planes the load can still be increased until the fracture strength of stronger plane is reached. By this time the weaker plane would have yielded. Thus the total strength equals fracture strength of stronger plane plus yield strength of weaker plane (cl: 6.4).

(a) Bolted Connection:-

The block shear strength T_{db} of bolted connection may be taken as smaller of the following:-

(i) Yielding of shear + fracture of tensile section

$$T_{db1} = \frac{Avg \cdot fy}{\gamma_{mo}} + 0.9 \frac{Atn \cdot fu}{\gamma_{mL}}$$

(ii) Yielding of tensile + fracture of shear section

$$= \frac{Atg \cdot fy}{\gamma_{mo}} + \frac{0.9 \, Atn \cdot fu}{\sqrt{3} \, \gamma_{mL}}$$

Atg = Shear o gross area in shear tensile section

Avg = gross area in shear section

Atn = net area in tensile section

Avn = net area in shear section.

(b) Welded connection:- the block shear strength, T_{db} for welded connection shall be calculated by taking an appropriate section is member around the end weld. However for welded

angle tension members, hence it is the gross area only that is involved. A_{tn} & A_{vn} is to be replaced by A_{tg} & A_{vg} respectively in above equation:
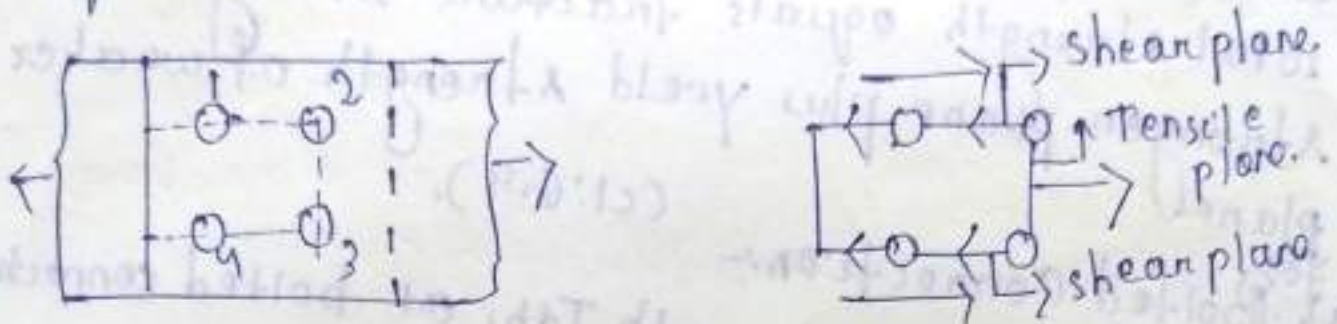
(Block shear failure for plates).
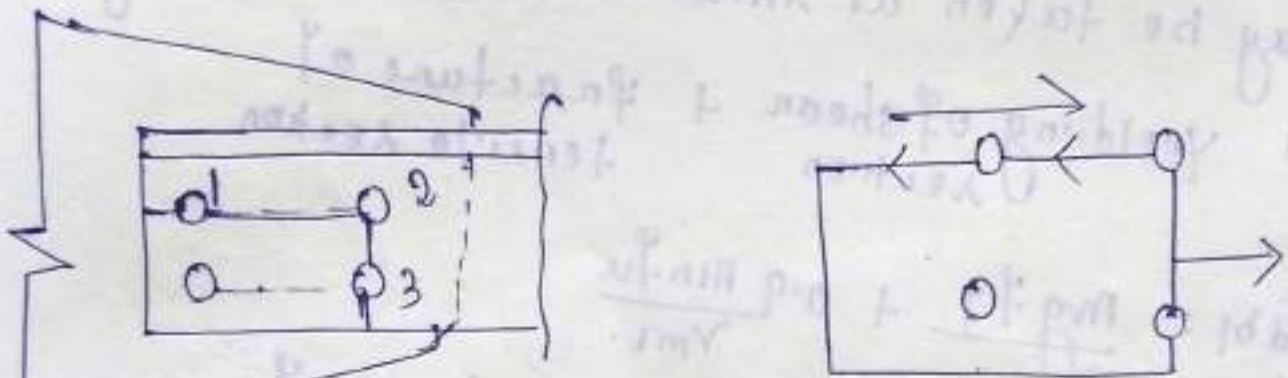
(Block shear failure for angles).

shear plane

Tension plane
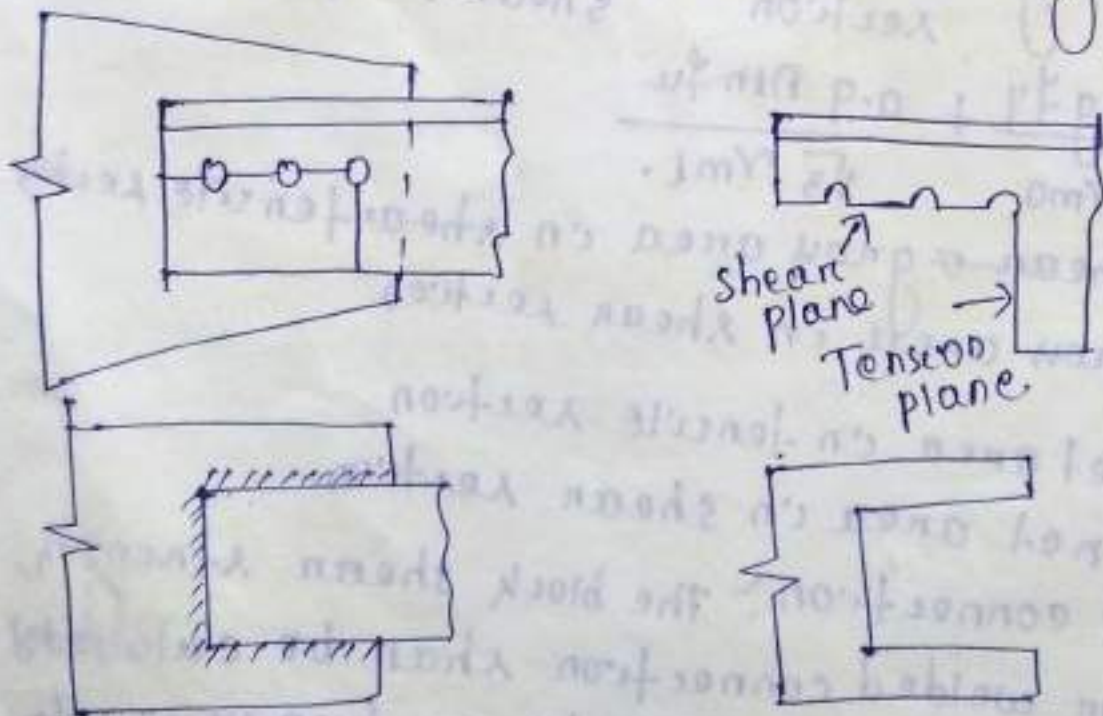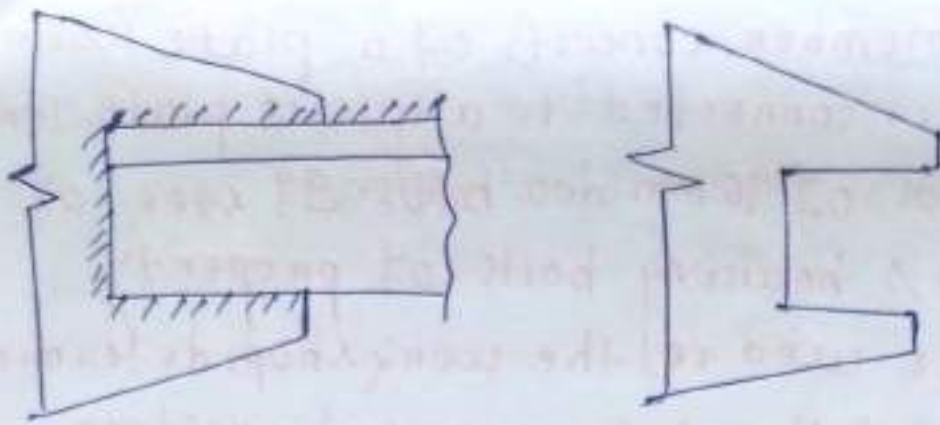
Slenderness Ratio (λ).

The slenderness ratio of a tension member is the ratio of effective length (KL) to its least radius of gyration r.

where K = coefficient depending on the end condition. (table No 11)

→ Theoretically, there should be no limitation on the slenderness ratio of tension member since stability or buckling is of little concern. But they may be subjected to stress reversals during erection, wind or earthquake load etc. From these point of views, the design specifications usually limit the slenderness ratio of tension member.

→ Value of λ (table No-3) IS 800).

Q A tension member consists of a plate 100mm × 8mm which is connected to a gusset plate 10mm thick by 2 nos of 16 mm dia bolts. If steel of grade Fe410 & bearing bolts of property class 4.6 are used in the workshop, determine the strength of the plate against yielding, rupture & block shear.

Solution :- For Fe410 grade of steel :-

$f_u = 410\,MPa$, $f_y = 250\,MPa$, $\gamma_{mo} = 1.1$, $\gamma_{m1} = 1.25$

For 16mm dia shop bolts of property class 4.6:

$d = 16mm$, $d_0 = 16 + 2$
$\qquad = 18\,mm$, $\gamma_{mb} = 1.25$

Strength of the plate :-

(i) Strength of plate against yielding of gross cross-section :-

$T_{dg} = \dfrac{A_g \, f_y}{\gamma_{mo}}$

$A_g = 100 \times 8 = 800\,mm^2$

$T_{dg} = \dfrac{800 \times 250}{1.1}$

$\qquad = 181.82\,kN$
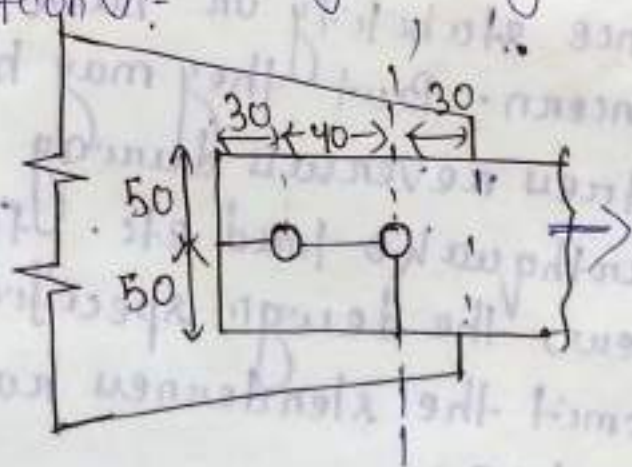
(ii) strength of plate against rupture of critical section
In this case section I-I is critical.

Net area $A_n = (b - n\,d_0)\,t$
$\qquad = (100 - 1 \times 18) \times 8 = 656\,mm^2$

$T_{dn} = \dfrac{0.9 A_n f_u}{\gamma_{m1}} = \dfrac{0.9 \times 410 \times 656}{1.25} = 193.65\,kN$

Block shear strength = minimum of T_{db1} & T_{db2}

$$T_{db} = 149.54 \, kN.$$
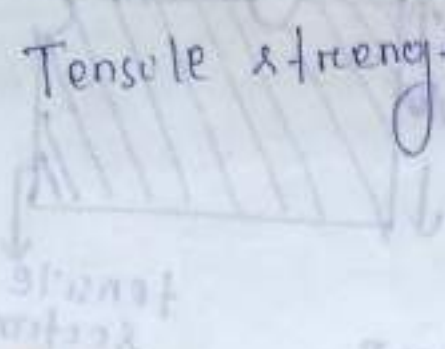
Tensile strength of plate = minimum of $\begin{cases} T_{dg} = 181.82 \\ T_{dn} = 193.65 \\ T_{db} = 149.54 \end{cases}$

$$= 149.54 \, kN.$$

Q A single unequal angle ISA $100 \times 75 \times 8 \, mm$ is connected by longer leg to a 12 mm gusset plate at the ends with 4 nos of 20mm dia yield bearing bolts of property class 4.6 (in a line) to transfer tension as shown in figure. Determine the design tensile strength of the angle. Take Fe 410 grade of steel).

For Fe 410 grade of steel:

$f_u = 410 \, MPa$

$f_y = 250 \, MPa$.

For 20mm dia. yield bolts of property class 4.6:

$d = 20mm, d_0 = 22mm, \gamma_{mb} = 1.25.$
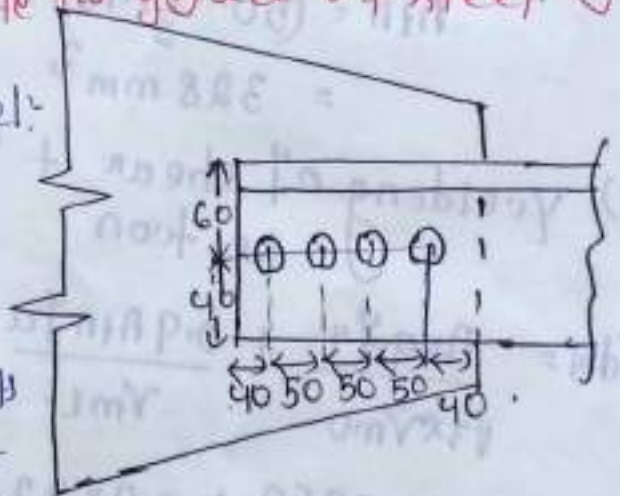
$A_{nb} = 0.78 \times \frac{\pi}{4}$

Strength of angle section:

(c) Strength of angle against yielding of gross section

For ISA $100 \times 75 \times 8$)

$A_g = 1336 \, mm^2$ (from steel table).

(ii) Strength of the plate against block shear:-

gross area at shear section:-

$$A_{vg} = 70 \times 8$$
$$= 560 mm^2$$

net area at shear section:-

$$A_{tn} - A_{vn} = \{70 - (1.5 \times 18)\} \times 8$$
$$= 344 mm^2$$

gross area at tensile section:-

$$A_{tg} = \cancel{100 \times 8} \ 50 \times 8$$
$$= 400 mm^2$$

net area at tensile section:-

$$A_{tn} = (50 - \frac{1}{2} \times 18) \times 8$$
$$= 328 mm^2$$
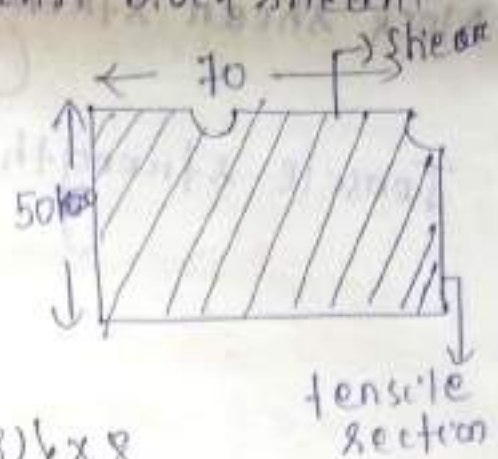
(i) yielding of shear + Rupture of tensile section

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3} \times \gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{mL}}$$

$$= \frac{560 \times 250}{1.1 \times \sqrt{3}} + \frac{0.9 \times 328 \times 410}{1.25}$$

$$= 170.31 \ kN.$$

(ii) Rupture of shear + yielding of tensile section

$$T_{db2} = \frac{0.9 \times A_{vn} \times f_u}{\gamma_{mL} \times \sqrt{3}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times \cancel{328} \ 344 \times 410}{1.25 \times \sqrt{3}} + \frac{400 \times 250}{1.1} = 149.54 \ kN.$$

$$T_{dg} = \frac{Ag\,f_y}{\gamma_{mo}} = \frac{1336 \times 250}{1.1} = 303.64 \text{ kN}$$

(v) Strength of the angle against rupture of critical section:

Net area of connected leg;

$$Anc = \left(100 - 22 - \frac{8}{2}\right) \times 8$$
$$= 592 \text{ mm}^2$$

Gross area of outstanding leg,

$$Ago = \left(75 - \frac{8}{2}\right) \times 8$$
$$= 568 \text{ mm}^2$$

w = outstand leg width = 75mm.

$w_1 = w - g$.
$\quad = 100 - 40 = 60 \text{ mm}$.

Shear lag width (bs) =

$$w + w_1 - t$$
$$= \cancel{100}\,75 + 60 - 8 = 127 \text{ mm}.$$

Length of connection (Lc) = 3×50
$$= 150 \text{ mm}.$$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{bs}{Lc}\right) \leq \frac{f_u\,\gamma_{mo}}{f_y\,\gamma_{m1}} \geqslant 0.7$$

$$= 1.4 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{127}{150}\right) \leq \frac{410 \times 1.1}{250 \times 1.25}$$
$$\qquad\qquad = \frac{250 \times 1.1}{250 \times 1.25}$$

$$= 1.032 \leq 1.4432$$
$$\qquad \geqslant 0.7 \quad (ok).$$

hence $\beta = 1.032$.

12mm
←75mm→
8mm
100 mm
60mm
40mm
ISA 100 × 75 × 8 mm
Gusset plate of 12 mm thick.
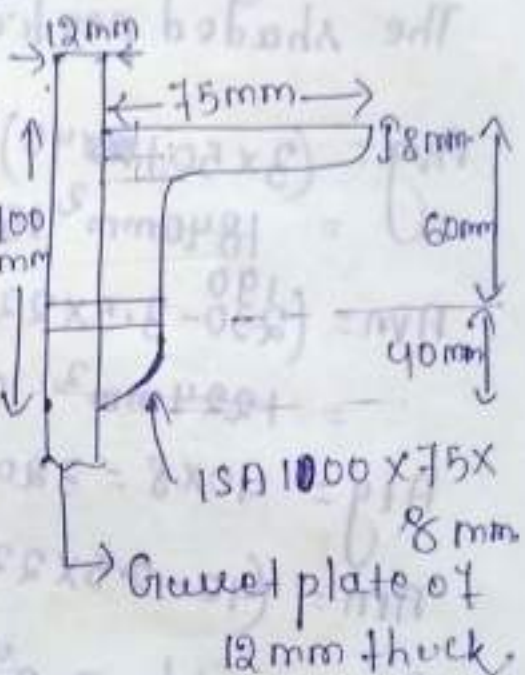
$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{mo}} \quad (cl: 6.3.3),$$

$$= \frac{0.9 \times 592 \times 410}{1.25} + \frac{1.032 \times 368 \times 250}{1.1}$$

$$= 301.980 \, kN.$$

(vi) Strength of angle section against block shear:-

The shaded portion may shear off.] →Shear section
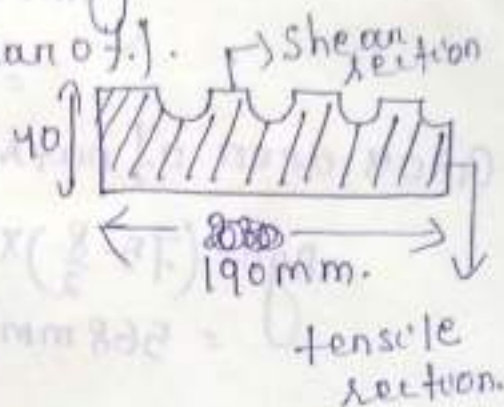
$A_{vg} = (3 \times 50 + 40) 8$

$= 1840 \, mm^2 \quad 1520 \, mm^2$

$A_{vn} = \overset{190}{(230} - 3.5 \times 22) \times 8$

$= 1224 \, mm^2. \quad 904 \, mm^2$

tensile section

$A_{tg} = 40 \times 8 = 320 \, mm^2$

$A_{tn} = (40 - 0.5 \times 22) \times 8 = 232 \, mm^2.$

(i) yielding of shear + rupture of tensile section section

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3} \, \gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \frac{1520 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 232 \times 410}{1.25}$$

$$= 267.934 \, kN$$

$$T_{db2} = \left(\begin{array}{c} \text{yielding of tensile} \\ \text{section} \end{array} \right) + \left(\begin{array}{c} \text{rupture of shear} \\ \text{section} \end{array} \right)$$

$$= \frac{A_{tg} f_y}{\gamma_{mo}} + \frac{0.9 \times A_{vn} f_u}{\sqrt{3} \, \gamma_{m1}}$$

$$= \frac{320 \times 250}{1.1} + \frac{0.9 \times 904 \times 410}{\sqrt{3} \times 1.25} = 226.799 \, kN.$$

block shear strength = minimum $\begin{cases} Tdb_1 = 267.934\ kN \\ Tdb_2 = 226.799\ kN \end{cases}$

$$Tdb = 226.799\ kN.$$

strength of angle = minimum of $\begin{cases} (C)\ Tdg = 303.64\ kN \\ (C^{ii})\ Tdn = 307.98\ kN \\ (C^{iii})\ Tdb = 226.79\ kN \end{cases}$

$$= 226.79\ kN.$$

Q A steel plate 250mm × 6mm is connected to a 10mm thick gusset plate as a tension member by fillet welding. Determine the block shear strength of member. (Take fe410 grade steel)

for fe410 grade steel

$$f_u = 410\ MPa, \quad f_y = 250\ MPa.$$

$$\gamma_{mo} = 1.1, \quad \gamma_{ml} = 1.25$$

tension plane
→ shear plane
150
250
→ P
→ shear plane
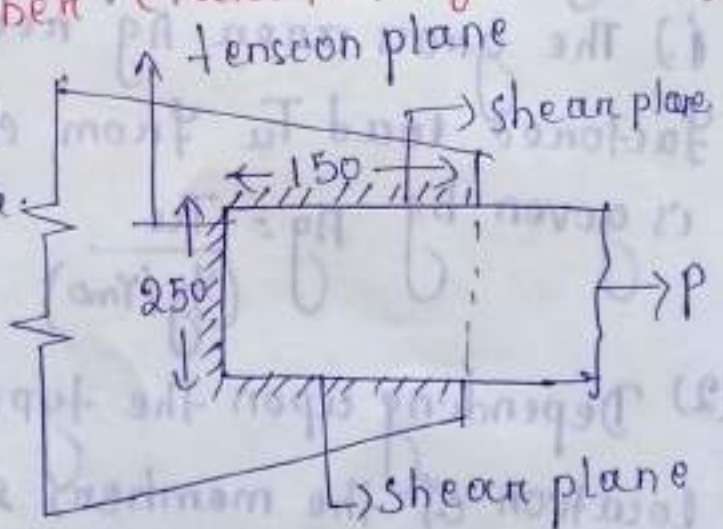
$$Avg = Avn = 150 \times 6 \times 2$$
$$= 1800\ mm^2$$

$$Atg = Atn = 250 \times 6 = 1500\ mm^2$$

block shear strength:

$$(C)\ Tdb_1 = \frac{Avg \cdot f_y}{\sqrt{3}\ \gamma_{mo}} + \frac{0.9 \times Atn \times f_u}{\gamma_{ml}}$$

$$= \frac{1800 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 1800 \times 410}{1.25}$$

$$= 678.98\ kN.$$

$$T_{db2} = \frac{A_{tg} f_y}{\gamma_{mo}} + \frac{0.9 A_{vn} f_u}{\gamma_{ml} \times \sqrt{3}}$$

$$= \frac{1500 \times 250}{1.1} + \frac{0.9 \times 1800 \times 410}{1.25 \times \sqrt{3}}$$

$$= 647.689 \text{ kN}.$$

block shear strength T_{db} = minimum of $\begin{cases} T_{db1} = 678.95 \text{ kN} \\ T_{db2} = 647.68 \text{ kN} \end{cases}$

$$T_{db} = 647.689 \text{ kN}.$$

Design of tension members subjected to axial load :-

Design steps :

1) The gross area 'A_g' required to carry the factored load T_u from consideration of yielding is given by $A_g = \dfrac{T_u}{(f_y / \gamma_{mo})}$

2) Depending upon the type of structure & the location of the member, suitable shape & area of section is selected from steel table.

3) The connection is designed by calculating the number of bolts on the length of weld required, which is suitably arranged as per requirement.

$$\text{no. of bolts} = \frac{\text{external load}}{\text{strength of a bolt}}$$

$\hookrightarrow$ minimum of bearing strength & shearing capacity of bolt.

(4) The design strength Td of trual section is calculated considering minimum of Tdg, Tdn, & Tdb which should be more than the factored load.

(5.) If Td < P Td < P, then the section is suitably revised.

(6.) The effective slenderness ratio of member is checked which should satisfy I.S. specification.

Q Design a single angle section for a tie of a roof truss to carry a factored tensile force of 300 kN. The member is subjected to possible reversal of stress due to action of wind. The effective length of member is 2.5m. Given that bearing bolts of property class 4.6 & steel of grade Fe410 are used.

For steel grade Fe410:-

$f_u = 410 MPa$, $f_y = 250 MPa$.

$\gamma_{mo} = 1.1$, $\gamma_{ml} = 1.25$

(i) Calculation of sectional area required:-

$$A_g = \frac{T_u \gamma_{mo}}{f_y} = \frac{300 \times 10^3 \times 1.1}{250} = 1320 mm^2$$

Let us adopt an angle section whose area > 1320 mm²

from steel table

so choose an angle section 90x60x10mm having gross area 1401mm² connected by long leg.

(i) Design of Connection:-

From unwin's formula, $d = 6\sqrt{t}$
$$= 6\sqrt{10} = 18.97$$
$$\approx 20mm.$$

Let us adopt 20mm bearing type bolts
$d_0 = 20 + 2 = 22mm$, $\gamma_{mb} = 1.25$.

$A_{nb} = 0.78 \times \frac{\pi}{4} \times (20)^2 = 245.04 mm^2$

$A_{sb} = \frac{\pi}{4} \times (20)^2 = 314.16 mm^2$

$f_{ub} = 400 Mpa$.

For lap joint $n_n = 1$, $n_s = 0$.

(a) Design strength of bolt in single shear:-
$$\hspace{6cm} (Cl. 6.2).$$

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

$$= \frac{400}{\sqrt{3} \times 1.25} (1 \times 245.04)$$

$$= 45.27 \, kN$$

(b) assume $e = 1.5 \times 22 = 33mm$, $P = 2.5 \times 20$
$$\hspace{8cm} = 50mm.$$
$$e \approx 40mm.$$

$k_b = $ minimum of $\begin{cases} \dfrac{e}{3d_0} = \dfrac{40}{3 \times 22} = 0.606 \\[2mm] \dfrac{P}{3d_0} - 0.25 = \dfrac{50}{3 \times 22} - 0.25 = 0.51 \\[2mm] \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.975 \\[2mm] 1 \end{cases}$

$k_b = 0.51$

Design bearing strength of bolt $(Vdpb)$:-

$$= \frac{2.5\,kbd\,t\,fu}{\gamma mb}$$

$$= \frac{2.5 \times 0.51 \times 20 \times 10 \times 410}{1.25}$$

$$= 83.640\ kN.$$

Strength of one bolt =

minimum of $\begin{cases} (v)\ Vdsb = 45.27\ kN. \\ (vi)\ Vdpb = 83.64\ kN \end{cases}$

$$= 45.27\ kN.$$

number of bolts required $= \dfrac{Tu}{\text{strength of one bolt}}$

$$= \frac{300}{45.27} = 6.63$$
$$\approx 7\ no.$$

Hence let us provide 7 nos of
20mm dia bolts in a single row.

check for long joint
$(cl:10.3.3.1)$

$lj = 6 \times 50 = 300mm = 15 \times d$

$300mm = 15 \times 20$
$\qquad = 300\ mm$

$(\ lj = 15 \times d\ )\ (ok)$
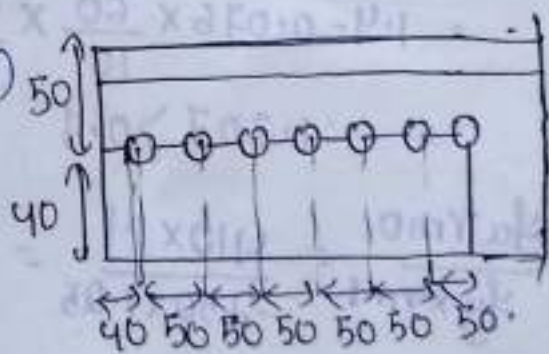
(3) check for strength of plate:-

(i) strength against yeilding of gross section

$Tdg = \dfrac{Ag\,fy}{\gamma m0} = \dfrac{1401 \times 250}{1.1}$

$\qquad Ag = 1401$
$\qquad$ from steel table
$\qquad fy = 250\ Mpa.$

$$= 318.409\ kN. > 300kN\ (ok).$$

(c) Strength against rupture of section:

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \beta A_{go} f_y}{\gamma_{m0}}$$

(Cl: 6.3.3).

Net area of connected leg (A_{nc}) =
$$\left(90 - 22 - \frac{10}{2}\right) \times 10$$
$$= 630 \, mm^2$$

Area of outstand leg (A_{go}) =
$$\left(60 - \frac{10}{2}\right) \times 10$$
$$= 550 \, mm^2$$

w = outstand leg width
$$= 60 \, mm.$$

shear lag width (b_s) = $w + w_1 - t$
$$= 60 + 50 - 10 = 100 \, mm.$$

Length of connection (L_c) = $6 \times 50 = 300 \, mm.$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$$= 1.4 - 0.076 \times \frac{60}{10} \times \frac{250}{410} \times \frac{100}{300}$$

$$\boxed{= 1.307 > 0.7}$$

$$\frac{f_u \gamma_{m0}}{f_y \times \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44.$$

$\beta = 1.307 > 0.7$
$\quad < 1.44 \; (ok).$

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 630 \times 410}{1.25} + \frac{1.307 \times 550 \times 250}{1.1}$$

$$= 349.35 \, kN > 300 \, kN \; (ok).$$

(iii) Strength of plate in block shear :-

(1:6:4)

$Avg = 340 \times 10$
$\qquad = 3400 \, mm^2$

$Avn = (340 - 6.5 \times 22) \times 10$
$\qquad = 1970 \, mm^2$

$Atg = 40 \times 10 = 400 \, mm^2$

$Atn = (40 - 0.5 \times 22) \times 10 = 290 \, mm^2$

$6 \times 50 + 40$
$= 340 mm$
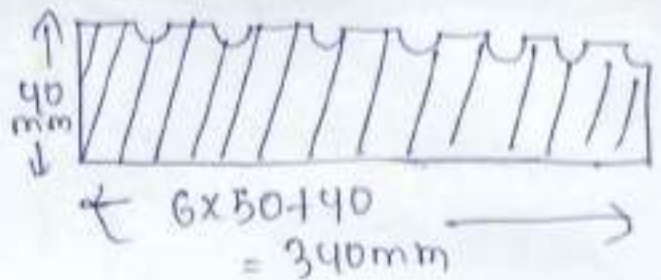
$Tdb1 = \dfrac{Avg \, fy}{\sqrt{3} \times \gamma mo} + \dfrac{0.9 \, Atn \, fu}{\gamma mL}$

$\qquad = \dfrac{3400 \times 250}{\sqrt{3} \times 1.1} + \dfrac{0.9 \times 290 \times 410}{1.25} = 531.742 \, kN$

$Tdb2 = \dfrac{Atg \, fy}{\gamma mo} + \dfrac{0.9 \, Avn \, fu}{\sqrt{3} \, \gamma mL}$

$\qquad = \dfrac{400 \times 250}{1.1} + \dfrac{0.9 \times 1970 \times 410}{\sqrt{3} \times 1.25} = 426.67 \, kN$

block shear strength $(Tdb) =$

minimum of $\begin{cases} Tdb1 = 531.74 \\ Tdb2 = 426.67 \end{cases}$

$\qquad = 426.67 \, kN > 300 \, kN$

(ok)

(4) check for slenderness ratio:

for a member subjected to possible reversal of stress due to action of wind $\lambda = 350$

(from table
3)

minimum radius gyration $r = 12.7 \, mm$ (from steel
table)

maximum slenderness ratio $\lambda = \dfrac{kL}{r} = \dfrac{2500}{12.7} = 196.8 < 350$

(ok)

Compression Members

→ Pure compression members are structural elements subjected to axial compressive force only.

→ Axial compressive force means force applied along the centroid of longitudinal axis of the cross-section. A column is defined as a structural member whose longitudinal dimension is comparatively more than its lateral dimension & subjected to compressive force in a direction parallel to its longitudinal axis.

Common Shapes of compression members:-

(1) Single angle section

(2) Double angles back to back

(3) T sections.

(4) Single channels.

(5) Circular hollow sections.

(6) Square and rectangular hollow section

(7) I section

(8) Built up section.

Buckling of Columns.

→ Buckling is defined as the sudden bending of the compression members under compression.

→ Due to buckling, deformation developed in a column occurs in a direction or plane normal to the direction of loading.

→ Buckling resistance depends on magnitude of the applied load, stiffness of member & length of member.

← buckled shape
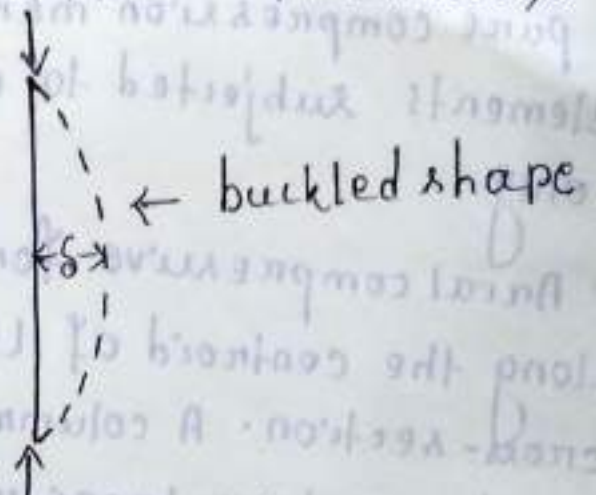
↑ (buckling of column).

Effective length of Column :-

Effective Length (le) of a column is defined in terms of equivalent length of column hinged at both ends for various end condition.

Effective length (le) = KL

L = actual Length of column.

k = constant depends upon end support condition

value of k → table No-11 of IS 800 : 2007

* Where the boundary conditions in the plane of buckling can be assened, the effective length 'kL' can be calculated on the basis of Table No-11 of IS 800 : 2007.

* In case of bolted or welded trusses, the effective length kL shall be taken as 0.7 to 1 times the actual length.

Appropriate radius of gyration :-

The radius of gyration may be different about different transverse axes (YY, ZZ, UU, VV etc).

→ However the radius of gyration of compression member about the axis of buckling is known as appropriate radius of gyration.

Slenderness Ratio

It is defined as the ratio of effective length to the corresponding radius of gyration of section.

$$\lambda = \frac{l_e}{r_{min}} = \frac{kL}{r_{min}}$$

$$\boxed{P < P_E}$$

→ Maximum value of slenderness ratio :-
    from table No 3 of IS 800: 2007.

* Also the compression members are required to satisfy the limiting width to thickness ratio depending upon class of section as per table-2 of IS 800: 2007.

Buckling class of cross sections (cl: 7.1.2.2)

(Table No-10 of IS 800: 2007)

→ The minimum load, that causes collapse of column section, is to be determined by checking safety of column about all axes.

→ However columns have tendency to buckle about Z, Y, V or U axis.

→ Depending upon geometric dimensions of individual structural elements & their corresponding limits, buckling classes are of

Four types :-

(i) buckling class a   (iii) buckling class c

(ii) buckling class b   (iv) buckling class d :-

Design compressive stress & strength of compression member :-

The design compressive strength P_d of a member is given by :-

$$\boxed{P_d = A_e \, f_{cd}}$$

A_e = effective sectional area (from steel table)

f_{cd} = design compressive stress

(by interpolation from table 9-a,b,c,d)

$$\boxed{P < P_d}$$

→ P = axial compressive Load (load applied).

→ To calculate value of f_{cd} from Table $q(a), q(b), q(c), q(d)$, we require λ, f_y & buckling class.

Q Calculated factored axial load on the column section ISHB 400 @ 806.38 N/m. The height of column is 3m & it is pin-ended. use steel of Fe 410 grade.

(1) for steel grade Fe 410,

f_u = 410 Mpa, f_y = 250 Mpa.

γ_{mo} = 1.1, E = 2×10^5 Mpa.

(2) for ISHB 400 @ 806.38 N/m

section properties (from steel table) :-

h = 400 mm, b_f = 250 mm, t_f = 12.7 mm, t_w = 10.6 mm.

A = 104.66 cm² = 104.66 r 10466 mm², r_{xx} = 166.1 mm

r_{yy} = 51.6 mm

(3) buckling class (cl: 7.1.2.2) (Table 10).

$$\frac{h}{b_f} = \frac{400}{250} = 1.6 > 1.20$$

$t_y = 12.7\,mm \le 40\,mm.$

for buckling about x-x axes
$$= \text{buckling class } \underline{a}$$

for buckling bout y-y axis
$$= \text{buckling class } \underline{b}$$
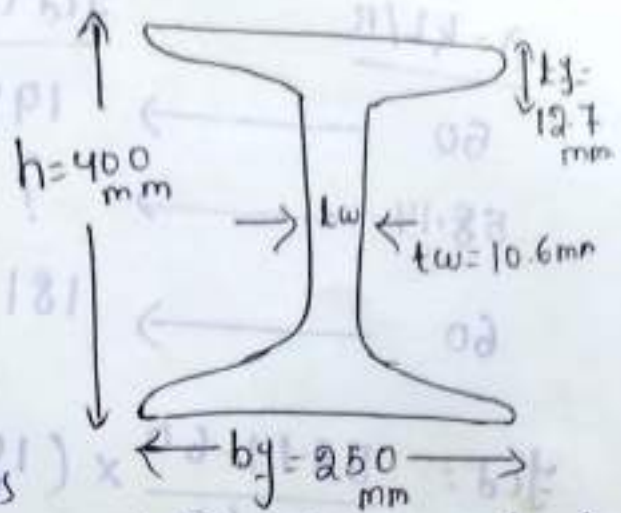
(Rolled I section)
(ISHB 400 @ 806.38 N/m)

(4) effective sectional area & effective length of column.

$A_e = 10466\,mm^2$

$L_e = kL$

for columns pinned at both ends value of $k = 1$ (from table No-11)

L = length of column given = 3 m.

$L_e = 1 \times 3 = 3\,m = 3000\,mm$

(5) slenderness ratio (λ):-

$$\lambda_{YY} = \frac{kL}{r_{yy}} = \frac{3 \times 1000\,mm}{51.6\,mm} = 58.14$$

$$\lambda_{xx} = \frac{L_e}{r_{zz}} = \frac{kL}{r_{zz}} = \frac{3000}{166.1} = 18.06$$

(6) Design compressive strength: f_{cd}

(c) About buckling axis YY & buckling class b

$\lambda_{yy} = 58.14, f_y = 250\,Mpa.$

We can get value of f_{cd} by interpolation

From table No 9(b) of IS 800:

$\lambda = kL/r$	f_{cd} (for f_y 250 Mpa)
50	194
58.14	?
60	181

$$f_{cd} = \frac{58.14-60}{50-60} \times (194-181) + 181$$

$$= 183.42 \, Mpa.$$

Factored axial load on design compressive strength of column

$(P_d) = A_e \, f_{cd}$

$= 10466 \times 183.42$

$= 1919 \, kN.$

(ii) About buckling axis zz/s buckling class a:

$\lambda_{zz} = 18.06$, $f_y = 250 \, Mpa$

by interpolating from table No 9(a):-

$\lambda = kL/r$	f_{cd} (for $f_y = 250 \, Mpa$)
10	227
18.06	?
20	226

$$f_{cd} = \frac{18.06-20}{10-20} \times (227-226) + 226$$

$$= 226.194 \, Mpa.$$

Factored axial load $P_d = A_e \, f_{cd}$

$= 10466 \times 226.194$

$= 2367.35 \, kN.$

∴ Design factored axial load = minimum of two = $1919 \, kN.$ (ANS)

Q A compound column consists of ISLB @ 400
@ 558 N/m with one cover plate 300 mm × 20 mm
on each flange. The actual length of column is
4.2 m and is fixed at one end and hinged at
other end, using the grade of steel fe 410,
determine the working load that the column
can carry.

(1) for fe410 grade steel
$f_u = 410 \, MPa, \quad f_y = 250 \, MPa$
$\gamma_{mo} = 1.1,$

(2) for ISLB 400 @ 558 N/m
Section properties (from steel table):-

$h = 400 \, mm \quad I_{zz} = 19306.2 \times 10^4 \quad 20 \, mm$
$b_f = 165 \, mm \qquad mm^4$
$t_f = 12.5 \, mm \quad I_{yy} = 716.4 \times 10^4 \, mm^4.$
$t_w = 8 \, mm \quad, \quad r_{zz} = 163.3 \, mm \, , \, r_{yy} = 31.5 \, mm.$
$A = 7243 \, mm^2$

(3) buckling class:-
As it is a built up section, so it belongs to
buckling class 'c' about any axis. (table-10)

(4) effective area & effective length:-
$A_e = (Area \, of \, I \, section) + 2 \times (area \, of \, one \, cover \, plate)$
$= 7243 + 2 \times (300 \times 20)$
$= 19243 \, mm^2$

(5) Moment of Inertia of whole section
about y-y & z-z axis
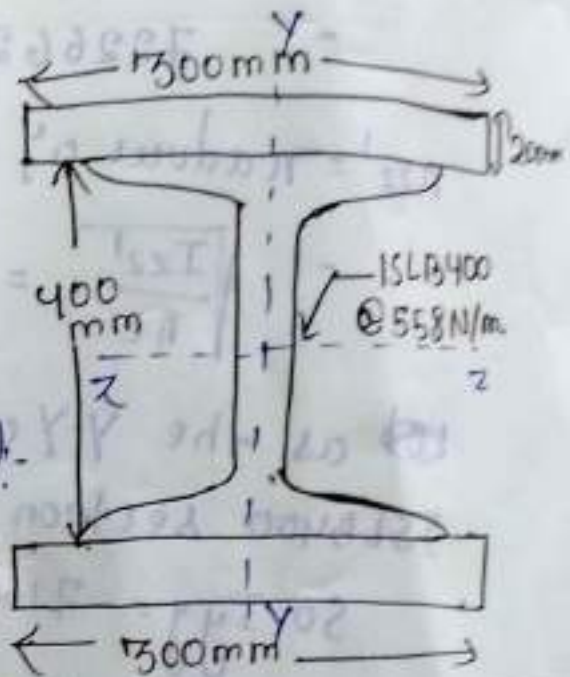
Let I_{zz}^1 = moment of Inertia of whole section about zz^1 axis

I_{zz}^1 = (Moment of Inertia of ISLB400 section about zz axis i.e. I_{zz}) + 2× (moment of Inertia of one cover plate about z-z axis).

= 19306.2×10^4 mm⁴ + 2× $\left(\dfrac{300 \times (20)^3}{12} + 300 \times 20 \times (210)^2 \right)$

= 722662×10^3 mm⁴

r_{zz}^1 = radius of gyration of whole section

= $\sqrt{\dfrac{I_{zz}^1}{A}} = \sqrt{\dfrac{722662 \times 10^3}{19243}}$ = 193.78 mm.

as the Y-Y axis of both cover plates & ISLB400 section is same, so Iyy will not be modified.

so I_{yy} = 716.4×10^4 mm⁴

r_{yy} = 31.5 mm

(6) effective length of column & slenderness ratio-

for one end fixed & other end hinged :
effective length $L_e = kL = 0.8 \times 4.2 \times 1000$

= 3360 mm
(table No-11)

slenderness ratio:

$\lambda_{zz} = \sqrt{} = \dfrac{KL}{r_{zz}^1} = \dfrac{3360}{193.78} = 17.34$

$\lambda_{yy} = \dfrac{KL}{r_{yy}} = \dfrac{3360}{31.5} = 106.67$

(7) Design compressive strength (P_d):

$(P_d)_{z-z}$ axis:

We can get value of f_{cd} by interpolating from table 9(c):

λzz	f_{cd} (for $f_y = 250$ MPa)
10	227
17.34	?
20	224

$$f_{cd} = \frac{17.34 - 20}{10 - 20} \times (227 - 224) + 224$$

$$= 224.798 \text{ Mpa}.$$

Factored axial load $(P_d)_{zz} = A_e f_{cd}$

$$= 19243 \times 224.798$$

$$= 4325.787 \text{ kN}$$

$(P_d)_{y-y}$ axis:

λyy	f_{cd} (for $f_y = 250$ MPa)
100	107
106.67	?
110	94.6

$$f_{cd} = \frac{106.67 - 110}{100 - 110} \times (107 - 94.6) + 94.6$$

$$= 98.7292 \text{ Mpa}$$

Factored axial load $(P_d)_{yy} = A_e f_{cd}$

$$= 19243 \times 98.73$$

$$= 1899.86 \text{ kN}$$

∴ Factored axial load that can carry = minimum of $(P_d)_{zz}$ & $(P_d)_{yy}$.
(the column)

$P_d = 1899.86 \, kN$

working load capacity of column $= \dfrac{P_d}{1.5}$

$$= \dfrac{1899.86}{1.5}$$

$$= 1266.57 \, kN$$

Design of axially loaded compression member.

Design steps:-

(1) (i) Assume slenderness ratio & determine design compressive stress (fcd), considering grade of steel & assuming buckling class.

assume

we can consider slenderness ratio as :-

→ 70 to 90 for rolled steel beam

→ 110 to 130 for angle strut

→ 40 for member carrying large load.

(ii) Alternatively the design stress in compression (fcd), we can consider directly assume as :-

→ 130 Mpa to 140 Mpa for rolled steel I section

→ 80 to 100 Mpa for angle strut & channel.

→ 190 to 200 Mpa for heavy / built up section.

(2) calculate effective sectional area required.

$$(Ae)_{req} = \dfrac{\oplus \text{applied factored axial load}}{\text{compressive stress}}$$

$$(Ae)_{req} = \dfrac{Pd}{fcd.}$$

choose a trial section from steel table whose area should be greater than area required. Write down the section properties of that section from steel table & find r_{min} of the trial section.

(3) find effective length & maximum slenderness ratio $\lambda_{max} = \dfrac{le}{r_{min}}$ considering end conditions & type of connection.

(4) Then find out buckling class of the provided section

(5) Determine design compressive stress (f_{cd}) considering grade of steel & actual buckling class. Compute design compressive strength (P_d) of member.

$$P_d = A_e \, f_{cd}$$

(6) check $P < P_d$.; if $P_d \, \text{act} \, \text{De} \, P > P_d$; then redesign by choosing another section.

(7) The section may be checked for limiting thickness also.

Q. Design a column section (using channel section only) to carry a factored axial load of 400kN. The column is 4m long & is effectively held in position at both ends but restrained against rotation at one end only. Consider $f_y = 250 \, MPa$. Assume wind/earthquake actions.

(1) Let us assume permissible design compressive stress (f_{cd}) = 80 MPa (for channel section).

$$(A_e)_{reqd} = \frac{Pd}{f_{cd}} = \frac{400 \times 10^3 N}{80} = 5000\, mm^2$$

(2) choose a section from steel table whose area should be greater than Area required.

So Try ISMC 350 @ 413 N/m, having A = 5366 mm² (from steel table).

$h = 350\,mm$ $t_f = 13.5\,mm$

$b_f = 100\,mm$ $t_w = 8.1\,mm$

$r_{zz} = 136.6\,mm$, $r_{yy} = 28.3\,mm$.

$r_{min} = 28.3\,mm$.

root of radius $r_1 = 14\,mm$

(3) L = 4m.

For one end fixed & other end pinned :

k = 0.8 (table 11).

$l_e = kl = 0.8 \times 4 \times 1000$

$= 3200\,mm$.

$$\lambda_{max} = \frac{kL}{r_{min}} = \frac{3200}{28.3} = 113.07 < 250 \quad \text{(for wind/earthquake condition)}$$
(from table 3).

(4) The buckling class is 'C' for channel section.

We can get value of f_{cd} for ISMC 350 section by interpolating from table 9(C):

λ	f_{cd} (for $f_y = 250\,MPa$)
110	94.6
113.07	?
120	83.7

(4) $f_{cd} = \dfrac{113.07-120}{110-120} \times (94.6-83.7) + 83.7$

$= 91.254 \, Mpa$

(5) Design compressive strength (P_d):-

$P_d = A_e \, f_{cd}$

$= 5366 \times 91.254$

$= 489.67 \, kN > 400 \, kN(P) \quad (ok)$

hence safe.

(c) check for limiting thickness:-

$\varepsilon = \sqrt{\dfrac{250}{f_y}} = \sqrt{\dfrac{250}{250}} = 1$

$d_w = h - 2(t_f + r_1)$

$= 350 - 2(13.5 + 14) = 295 \, mm.$

$b = \dfrac{b_f}{2} \not= 100 \quad (for \, channel \atop section).$

$\dfrac{b}{t_f} = \dfrac{100}{13.5 \atop 8.1} = 7.41 \atop 12.34 \quad , \quad \dfrac{d}{t_w} = \dfrac{295}{8.10} = 36.42 < 42\varepsilon \atop = 42$

$< 15.7 \varepsilon$
$= 15.7$

⊛ $\dfrac{b}{t_f} < 15.7 \, , \, \dfrac{d}{t_w} < 42 \quad (ok)$

$(table \, 2)$

Design of Steel Beam:-

→ Flexural members or bending members are called beams.

→ A beam is a structural member subjected to transverse load i.e. load perpendicular to the longitudinal axis.

→ The load produces bending moment & shear forces.

Ex Joist, girder, purlin.

Nature of forces acting on beam:-

→ It is assumed that the beam is subjected to only transverse loading.

→ All the loads & reactions lie in the plane of symmetry. So it follows that such a beam will be primarily subjected accompanied by shear in the loading plane with no external torsion & axial force.

→ but the problem of torsion can not completed be avoided in a beam even if the beam shape is symmetrical & loads are in the plane of symmetry. The reason is the instability caused by compressive stresses, such instability is defined as lateral buckling when it is involving only local components of a beam it is called local buckling.

modes of failure:-

(i) bending failure:- due to crushing of compression flange or fracture of tension flange.

(ii) Shear failure:- when shear stress value exceeds the limiting value. Thus occurs due to buckling of web of beam near location of high shear forces. The beam can fail locally due to crushing or buckle of web near the reaction of concentrated loads.

(iii) deflection failure:- A floor having large deflection not only produces a feeling of insecurity but also damages the non structural elements attached to it. Hence deflection in beam should be restricted by specify deflection limitation imposed by code which is usually in terms of ~~deflection~~ ~~depth~~ $\frac{deflection}{span}$ to span ratio

Sections available for beam members-

→ Angles $\}$ weaker bending   built up sections → when

→ T-section   stress exceeding

→ channel $\}$ used for light load.   limiting value.

→ I section → most effective
                    as a beam member.
                    (economical, bending
                    strength is more as $m = \frac{M}{y} I$
                    I is more

Common cross sections & their classification:-

Slender cross section	Semi-compact cross section		Compact cross section	Plastic cross-section.

$n < ay$ $a = ay$ $n = ay$ $a = ay$ $a = ay$

$a < ay$ $a = ay$ $a = ay$ $a = ay$ $a = ay$

→ $M < my$ $M = my$ $my < m < mp$ $M = Mp$ $M = Mp$.

→ elements

My → yield moment

Mp → plastic moment
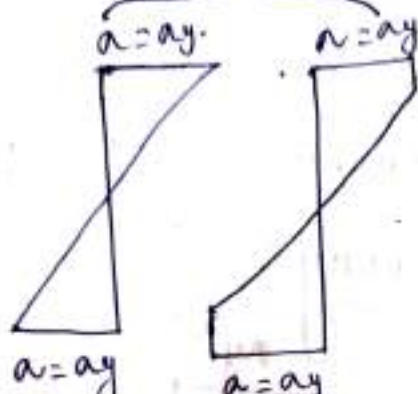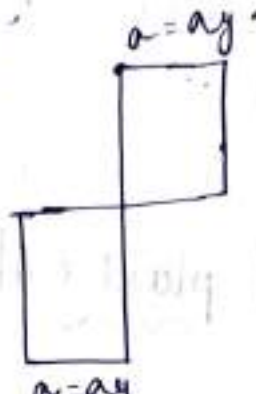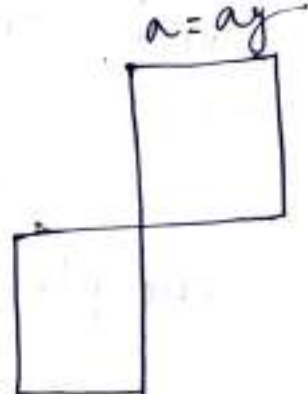
(1) plastic cross-section → plastic cross-sections are those which can develop plastic hinges & have the rotation capacity required for failure of structure by formation of plastic mechanism.

(2) compact section → compact cross-section are those which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism due to local buckling.

(3) semi compact cross-section → semi compact cross sections are those in which the stress in the extreme fibres in compression should be limited

to yield stress. These sections can not develop plastic moment of resistance due to local buckling.

Slender cross-section :- Slender cross-section are those in which the elements buckle locally even before reaching yield stress.

Concepts of plastic theory :-

considering equilibrium conditions.
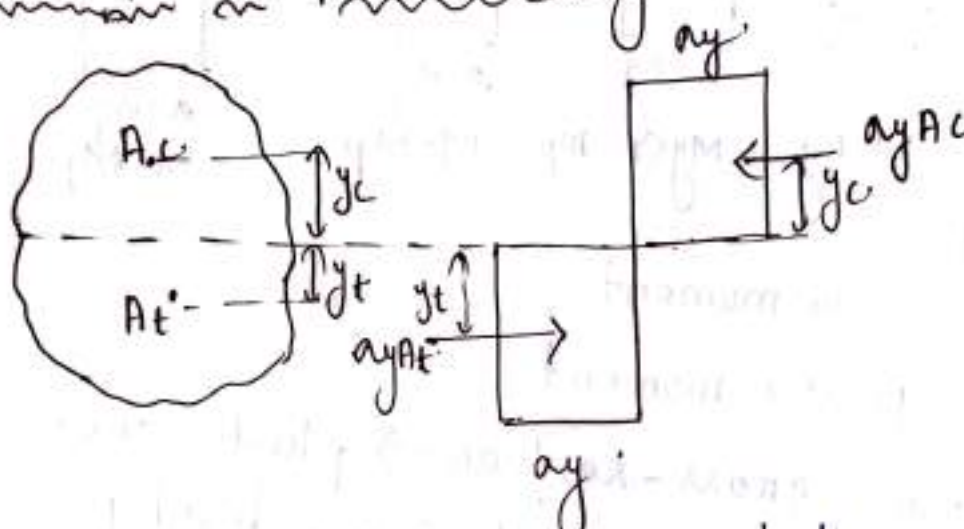
$$\Sigma H = 0$$

Total force in compression = Total force in tension.

$$\sigma_y A_c = \sigma_y A_t$$

$$\Rightarrow \boxed{A_c = A_t}$$

So the neutral axis that divides the cross-section into two equal haves is known as plastic neutral axis.

$$\boxed{A_t = A_c = \frac{A}{2}}$$

A_t = area in tension.
A_c = area in compression.
A = total area.

Here,
plastic moment capacity (M_p) -

$$(a_y A_c) y_c + (a_y A_t) y_t$$

$$\Rightarrow a_y (A_c y_c + A_t y_t)$$

$$\boxed{M_p = a_y z_p}$$

z_p = plastic section modulus.

$$= A_c y_c + A_t y_t.$$

shape factor: shape factor of a cross section is defined as the ratio between plastic moment to yield moment.

$$\boxed{\text{shape factor} = \frac{M_p}{M_y} = \frac{a_y z_p}{a_y z_e} = \frac{z_p}{z_e.}}$$

shape factor of a rectangular section:-
plastic moment (m_p).   yielding moment (M_y)
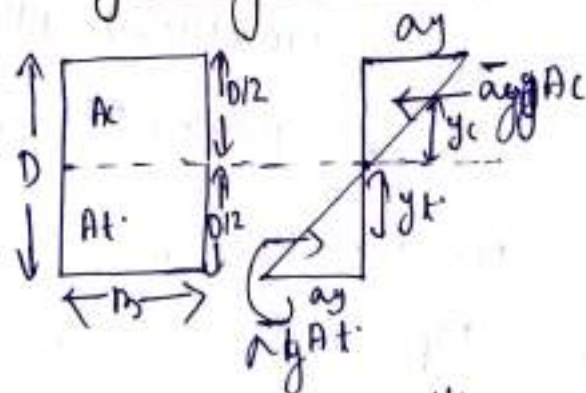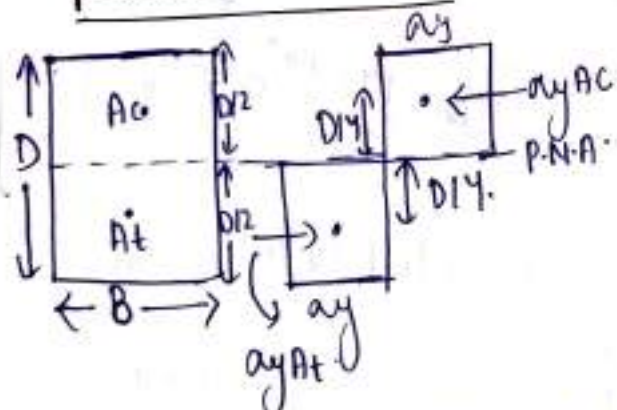
$M_p = a_y A_c \cdot \frac{D}{4} + a_y A_t \cdot \frac{D}{4}$

$= a_y B \cdot \frac{D}{2} \cdot \frac{D}{4} + a_y \cdot B \cdot \frac{D}{2} \cdot \frac{D}{4}$

$\boxed{M_p = a_y \frac{BD^2}{4}}$

Hence $z_p = \frac{BD^2}{4}$

Me or $M_y = a_y A_c \cdot y_c + a_y A_t \cdot y_t$

$= a_y B \cdot \frac{D}{2} \cdot \frac{2}{3} \cdot \frac{D}{2} + a_y B \cdot \frac{D}{2} \cdot \frac{2}{3} \frac{D}{2}$

$\boxed{M_y = a_y \cdot \frac{BD^2}{6}}$

Here $z_e = \frac{BD^2}{6}$

Shape factor $= \dfrac{M_P}{M_y} = \dfrac{a_y \cdot \frac{bd^2}{4}}{a_y \frac{bd^2}{6}} = 1.25$.

Q Determine the plastic moment capacity & plastic section modulus of a symmetrical I section having depth of section 350 mm, width of flange 140mm, thickness of flange 14.2 mm & thickness of web 8.1 mm (a) about x-x axis (b) about y-y axis.
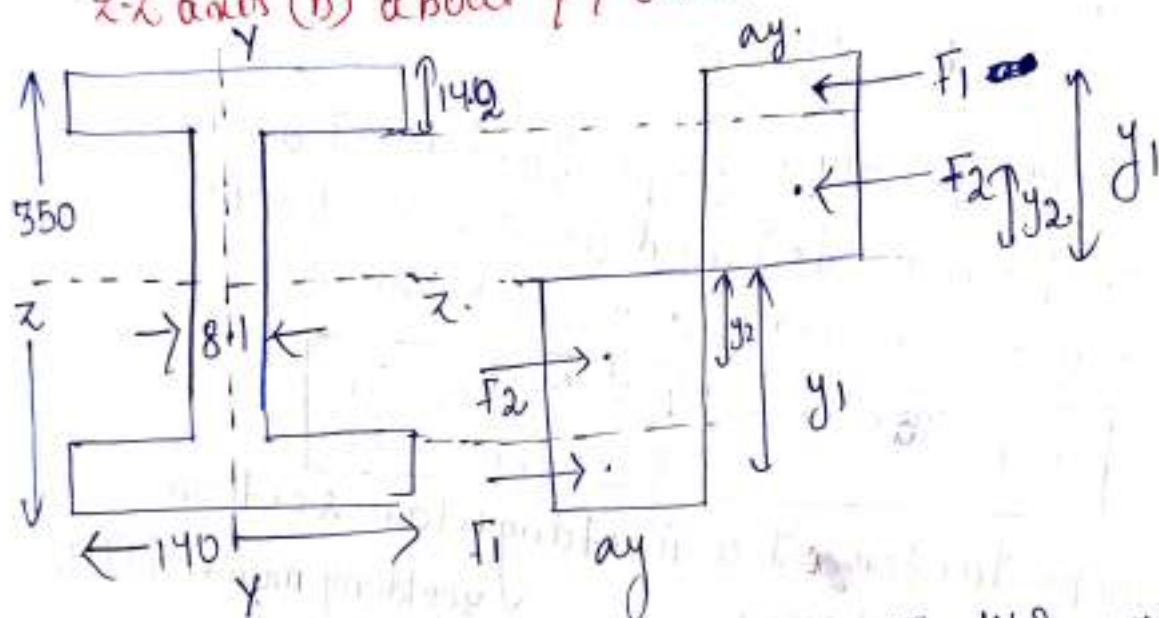
$F_1 = a_y (140 \times 14.2) = 1988 \, a_y$ $y_1 = 175 - \dfrac{14.2}{2} = 167.9$

$F_2 = a_y (160.8 \times 8.1) = 1302.48 \, a_y$ $y_2 = \dfrac{175-14.2}{2} = 80.4$

About xx axis :-

$M_p = (F_1 y_1 + F_2 y_2)_{\text{from compression region}} + (F_1 y_1 + F_2 y_2)_{\text{from tension region}}$

$= (1988 \, a_y \times 167.9 + 1302.48 \, a_y \times 80.4) +$

$(1988 \, a_y \times 167.9 + 1302.48 \, a_y \times 80.4).$

$= 877.009 \times 10^3 \times a_y$

$$Z_p = \frac{M_P}{\oint_{ay}} = \frac{877.009 \times 10^3 \times ay.}{ay} = 877.009 \times 10^3 \, mm^3$$

Q Determine the plastic section modulus of a 'T' section having flange width 12cm, flange thickness 1cm, depth of web 160mm & width of web 12mm.
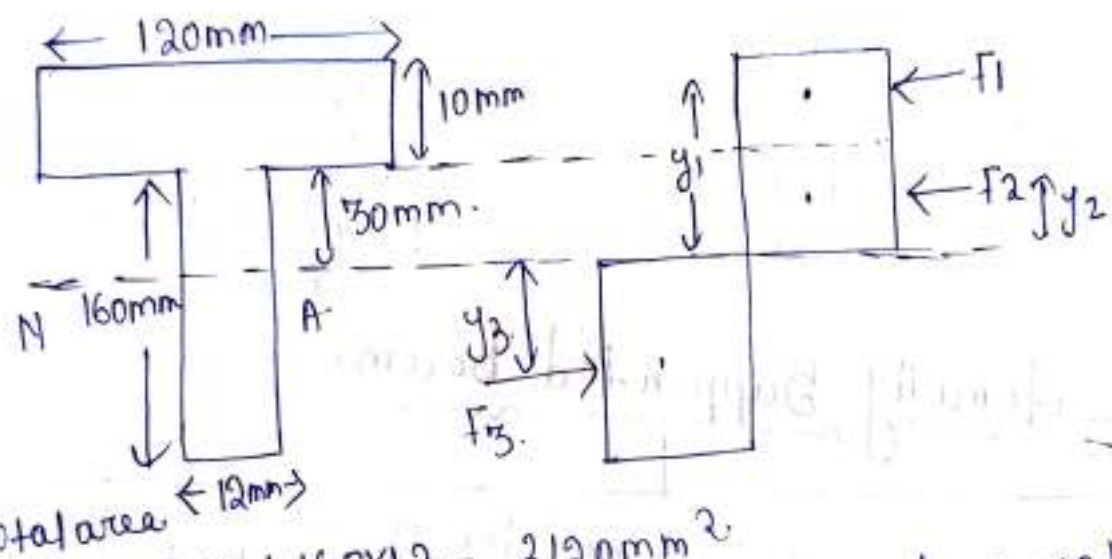
Total area $\leftarrow 12mm \rightarrow$

$$A = 120 \times 10 + 160 \times 12 = 3120 \, mm^2$$

Plastic N.A. devides the total section into two part of same area.

$$\frac{A}{2} = \frac{3120}{2} = 1560 \, mm^2$$

$$120 \times 10 + 12 \times y = 1560$$

$$\Rightarrow \boxed{\bar{y} = 30mm} \rightarrow \text{Location of plastic N.A.}$$
$$\text{from the top of flange web}.$$

$$M_P = F_1 y_1 + F_2 y_2 + F_3 y_3$$
$$= ay \times 120 \times 10 \times 35 + ay \times 12 \times 30 \times 15 + ay \times 12 \times 130 \times 65$$
$$= 148.800 \, ay$$
$$Z_p = \frac{M_P}{ay} = \frac{148.800 \, ay}{ay} = 148.800 \, mm^3$$

Beam (based on lateral supports to compression flange).

laterally supported beam.

↓

laterally unsupported beam.

↓

If compression flanges are laterally supported

↓

subjected to bending & shear stress.

If compression flange is not laterally supported.

↓

Lateral buckling occurs.

Laterally Supported beam.

low shear.
$V \leqslant 0.6 V_d$
(page no 53)

V = factored design shear force due to external load

V_d = design shear strength of cross section (page No. 59)

high shear.
$V \geqslant 0.6 V_d$
(page No- 70)

Design shear strength (V_d) (Go to page No 59)

From clause No 8.4:-

design shear strength $(V_d) = \dfrac{A_v f_{yw}}{\sqrt{3}\, \gamma_{mo}}$

f_{yw} = yield strength of web

γ_{mo} = partial safety factor against shear failure (refer cl. 5.41)

= 1.1

Av = shear area

For I / channel section:

	A_v (For major axis
Hot rolled section $\rightarrow$	$h t_w$ (bending".
welded $\rightarrow$	$d t_w$

Hot rolled or welded $\rightarrow$ $2 b_f t_f$ (Minor axis bending).

circular hollow tube of uniform thickness $\rightarrow$ $2A/\pi$

plates & solid bars $\rightarrow$ A

A = cross section area.
h = overall depth of section
t_f = thickness of flange
t_w = thickness of web.
d = clear depth of web between flanges.
b_f = width of flange

* For classification of section, go to page No 18 (table 2) :-

Here $b = \dfrac{b_f}{2}$, $d = h - 2(t_f + r)$.

r = root of radius (from steel table we can get this value)
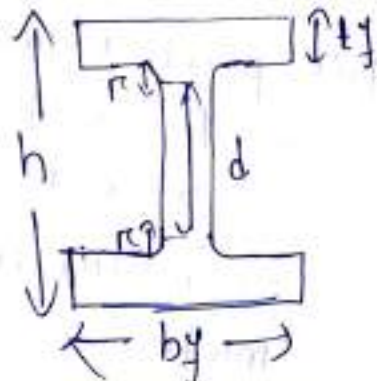
Design bending strength of laterally supported beam.

bending strength for low shear $(V \leq 0.6\,Vd)$. (Page 53)

$$\left[C\frac{d}{t_w} \leq 67\,\epsilon \right),\quad \epsilon = \sqrt{\frac{250}{f_y}} \right]$$

bending strength for high shear $(V > 0.6\,Vd)$. (page 70)

$M \leq Md$

M = factored bending moment due to applied load.

Md = design bending strength.

$Md = \beta_b\, z_p\, f_y / \gamma_{mo}$

$\leq \dfrac{1.2\, z_e f_y}{\gamma_{mo}}$ (for simply supported)

$\leq \dfrac{1.5\, z_e f_y}{\gamma_{mo}}$ (for cantilever beam)

where,

$\beta_b = 1$ for plastic/compact section

$= \dfrac{z_e}{z_p}$ for semi compact section.

z_p, z_e = plastic & elastic section moduli of cross section

f_y = yield stress of material.

$\gamma_{mo} = 1.1$

$M \leq Mdv$.

Mdv = design moment capacity under high shear.

$Mdv = Md - \beta(Md - Mfd)$ (for plastic & compact section) $\leq \dfrac{1.2\, z_e f_y}{\gamma_{mo}}$

where,

$\beta = \left(\dfrac{2V}{Vd} - 1\right)^2$

V = factored applied shear force.

Vd = design shear strength.

Mfd = plastic design strength of area of cross-section excluding shear area, considering partial safety factor, γ_{mo}

z_e = elastic section modulus of whole section.

(b) For semicompact section.

$Mdv = \dfrac{z_e f_y}{\gamma_{mo}}$

Steps for analysis problem.

(1) write down the section properties of choosen section. ~~coming 600 to load doing~~ given in question.

(2) clarify the section whether it is compact section, plastic section, semicompact section, or slender section.

(3) Assume the section is low shear case.
i.e. $V \leq 0.6 V_d$. for low shear case calculate the design bending strength (Md).

(4) calculate the load by taking $\Rightarrow M = M_d$.

(5) calculate the maximum shear force (V). (by taking the load that is calculated from bending strength).

(6) Then calculate design shear strength (Vd).
   ⊙ then check whether the section is low shear case or high shear case.
   ⊙ if it is low shear case, then our assumption is ok but if it is high shear case, then solve according to high shear case.

(7) calculate load carrying capacity from deflection criteria by taking
$$(\delta max) \, beam = \frac{span}{240.} \quad \text{(for simple supported beam)}$$

(from table - 06)
code IS 800.

Q A laterally supported beam ISMB 600 @ 1202.71 N/m is placed between two supports. Determine the safe uniformly distributed load per meter length which can be placed over the beam for an effective span of 12m. Take $f_y = 250 \, N/mm^2$. Neglect web buckling & web crippling.

(1) Section properties of ISMB 600 @ 1202.71 N/m (from steel table):

plastic section modulus $z_p = 3510630 \, mm^3$

depth of section $h = 600 \, mm$.

$bf = 210 \, mm$, $A = 15621 \, mm^2$, root of radius (rv) = 20 mm.

$tf = 20.8 \, mm$, $tw = 12 \, mm$.

depth of web $dw = h - 2(tf + r_1)$

$$= 600 - 2(20.8 + 20)$$
$$= 518.4 \, mm$$

$I_{zz} = 91813 \times 10^4 \, mm^4$.

elastic section modulus $(ze) = 3060.4 \times 10^3 \, mm^3$
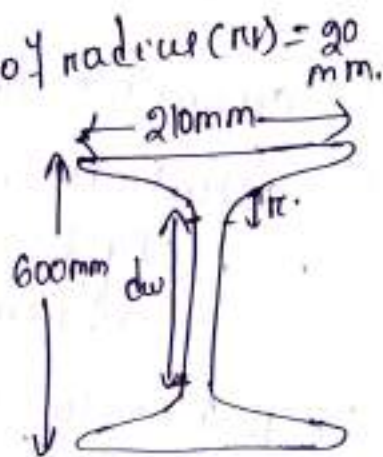
(2) Section classification:-

$$e = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1 \quad , \quad b = \frac{bf}{2} = \frac{210}{2} = 105 \, mm.$$

for rolled section-
    outstand of compression flange:-

$$\frac{b}{tf} = \frac{105}{20.8} = 5.05 < 9.4 \, \varepsilon.$$

So flange is plastic section.   (table 2)

for web with N.A. at mid depth:- $\frac{d}{tw} = \frac{518.4}{12}$

$$= 43.2 < 84 \, \varepsilon.$$
(table 2).

So web vs plastic section.

Hence overall classification of section is plastic.

(3) $\dfrac{d}{t_w} = \dfrac{518.4}{12} = 43.2 < 67\varepsilon$.

Let assume $\cancel{\text{os}}$ it is a low shear case

i.e. $V \leq 0.6 V_d$.

Design bending strength $M_d = \dfrac{\beta_b Z_p f_y}{\gamma_{mo}} \leq \dfrac{1.2 Z_e f_y}{\gamma_{mo}}$

$\beta_b = 1$ (∵ for plastic section).

$\qquad\qquad\qquad$ (for simply supported beam)

$M_d = \dfrac{1 \times 3510630 \times 250}{1.1}$

$\quad = 797.87 \times 10^6 \, N\,mm$

$\quad = 797.87 \, kNm.$

$\dfrac{1.2 Z_e f_y}{\gamma_{mo}} = \dfrac{1.2 \times 3060.4 \times 10^3 \times 250}{1.1}$

So $M_d \leq \dfrac{1.2 Z_e f_y}{\gamma_{mo}}$

$\qquad\qquad = 834.65 \times 10^6$

$\qquad\qquad\qquad N\,m\,m$

$\qquad\qquad = 834.65 \, kN\,m.$

(4) Load carrying capacity based on design bending strength.

$M = M_d.$ $\qquad$ (For simply supported beam maxm bending moment $= \dfrac{\omega l^2}{8}$)

$\dfrac{\omega \times 12^2}{8} = 797.87 \times 10^6$

$\Rightarrow \omega = 44.326 \, kN/m.$

Safe uniformly distributed load $= \dfrac{\omega}{1.5}$

$\qquad\qquad\qquad = \dfrac{44.326}{1.5}$

Safe u.d.l. that the beam $\omega' = 29.551 \, kN/m.$

can carry $= \omega' - $ self wt. of beam

$\qquad\qquad = 29.551 - 1.202 = 28.349 \, kN/m.$

(5) Factored shear force $V = \dfrac{\omega l}{2}$

$(\omega = 1.5\omega')$

$$V = \dfrac{44.326 \times 12}{2}$$

$$= 265.956 \, kN. \text{ (for simply supported beam)}$$

Design shear strength

$$(V_d) = \dfrac{A_v \, f_{yw}}{\sqrt{3} \, \gamma_{mo}}$$

{ For rolled section,
$A_v = htw$
$(A_v = 600 \times 12)$ }

$$V_d = \dfrac{600 \times 12 \times 250}{\sqrt{3} \times 1.1}$$

$$= 944.75 \, kN$$

$0.6 \, V_d = 0.6 \times 944.75$

$$= 566.85 \, kN.$$

$V < 0.6 \, V_d$ (low shear case) so, our assumption

is ok.

(6) Load carrying capacity from deflection criteria.

$(\delta_{max})_{beam}$ for udl $= \dfrac{5 \omega l^4}{384 EI}$

$$= \dfrac{5 \times \omega \times (12 \times 10^3)^4}{384 \times 2 \times 10^5 \times 91813 \times 10^4}$$

$$= 1.47 \, \omega$$

$\delta_{max} = \dfrac{span}{240} \Rightarrow 1.47\omega = \dfrac{12 \times 10^3}{240} \Rightarrow \omega = 34.01 \, kN/m$

working load $\omega' = \dfrac{34.01}{1.5} = 22.67 \, kN/m.$

safe working load placed on beam =

$\omega' - self \, wt = 22.67 - 1.202 = 21.47 \, kN/m$

Hence the safe working load that the beam
can carry is minimum of the two = 21.47 kN/m.

Design steps.

(1) (a) Determine the service load on the beam
& multiply with γ_f (1.5) to find the ultimate
load on factored load.

(b) Determine the effective span of the beam.

(c) Calculate the maximum bending moment
M & maximum shear force V.

(2) Trial Section & section classification:-

(a) Determine a trial plastic section using
the formula:-
$$Z_{p(reqd)} = \frac{M \gamma_{mo}}{f_y}$$

M = maximum bending moment determined in
step(1)(c).

γ_{mo} = partial factor of safety (1.1)

f_y = yield stress of material (250 MPa).

(b) From steel table, choose a trial section having
plastic section modulus more than that required.

(c) check the classification of section (see
table-2, IS 800).

(3) check for bending & shear strength:-
(a) Determine shear strength of section
(cl 8.4 of IS 800) & compare with maximum

shear force V determined in step $1(c)$ is check
(b) whether the $*$ if is low shear case on high shear case.

(b) find design bending strength $\oplus$ depending on $V < 0.6 Vd$ on $V > 0.6 Vd$ as cl $8.2.1.2$ on cl: $9.2.2$ of IS $800: 2007$.

Check $M($ maximum bending moment$) <$ Design bending strength. if not satisfied, repeat from step-$2(b)$ on $3(b)$.

(4) check for deflection $(cl: 5.6.1 \text{ & table-6})$. Calculate the maximum deflection on the beam considering effective span, loading & support condition. The maximum deflection shall be less than the permissible value given in table 6 of IS 800.

Q Design a simple supported beam of clear span 1.2m carrying a concentrated load of 260kN at mid span. Width of support is 200mm. Consider $f_y = 250 N/mm^2$.

(1) (a) assume self wt. of beam $= 1 kN/m$.
effective span $=$ centre to centre of supports.
$$= 1.2 + 2 \times \frac{0.2}{2}$$
$$= 1.4m.$$

Given point load $(W) = 260kN.$

~~self wt of beam~~ $= 1 kN/m \times 1.2 m$
$$= 1.2 kN.$$

$\leftarrow$ 1.2m $\rightarrow$ 0.2   0.2
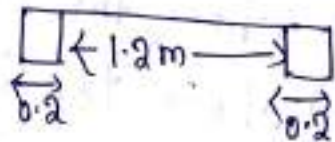
factored point load (Wu) = 260 × 1·5
                         = 390 kN.

factored 8014 wt (Wu) = 14 × 1·51 kN/m × 1·5
                      = 2·1 kN. 1·5 kN/m.

(b) calculation of maximum bending moment
(M) & maximum shear force (V)

$$Wu = \frac{2·1 kN}{(1·4 × 1·5)},\quad 1·5 kN/m.\quad Wu = 390 kN.\ (260×1·5)$$

$$\leftarrow 1·4 m \rightarrow$$
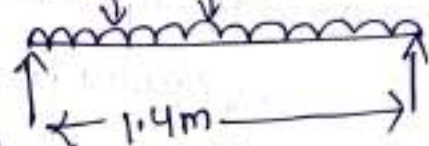
maximum bending moment at centre:-

$$(M) = \left(\frac{Wu\,l_{eff}^2}{8} + \frac{W\,l_{eff}}{4}\right)$$

$$M = \frac{1·5 × 1·4^2}{8} + \frac{390 × 1·4}{4}$$

$$= 136·87\ kNm.$$
$$= 136·87 × 10^6\ Nmm.$$

maximum shear force at support (V) =

$$\left(\frac{Wu}{2} + \frac{Wu\,l_{eff}}{2}\right)$$

$$= \left(\frac{390}{2} + \frac{1·5 × 1·4}{2}\right) = 196·05\ kN$$
$$= 196·05 × 10^3 N.$$

(2) Trial section & section classification:-

(a) $(Z_p)_{reqd} = \dfrac{M\gamma_{mo}}{f_y} = \dfrac{136·87 × 1·1 × 10^6}{250} = 602228\ mm^3$

(b) then choose a section from steel table
having $Z_p > 602228\ mm^3$.

So try ISMB 350 @ 514 N/m having $Z_p = 889600\ mm^3$

Section properties:

$h = 350$ mm. $t_y = 14.2$ mm $I_{zz} = 13630.3 \times 10^4 \, mm^4$ (a)

$b_y = 140$ mm $t_w = 8.1$ mm. $z_p = 889600 \, mm^3$

$A = 6670 \, mm^2$ $z_e = 778.9 \times 10^3 \, mm^3$ $r_1 = 14$

$\gamma = 0.514$ kN/m

(C) Section classification:-

$d = h - 2(t_y + r_1)$.

$= 350 - 2(14.2 + 14)$

$= 293.6$ mm.
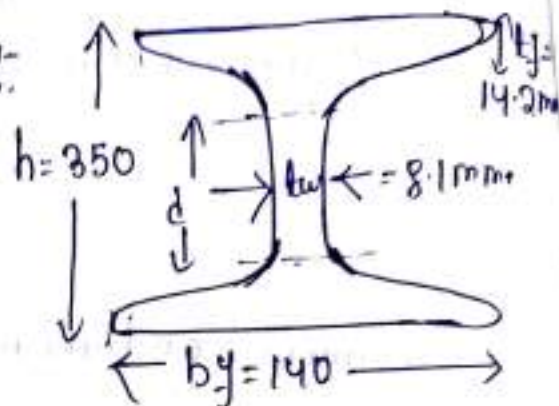
flange

$b = \dfrac{b_y}{2} = \dfrac{140}{2} = 70$ mm. , $\epsilon = \sqrt{\dfrac{250}{yy}} = \sqrt{\dfrac{250}{250}} = 1$

for outstand of compression flange for
rolled section, $\dfrac{b}{t_y} = \dfrac{70}{14.2} = 4.93 < 9.4\epsilon$

  (plastic section).

web of I-section with NA at mid depth:-

$\dfrac{d}{t_w} = \dfrac{293.6}{8.1} = 36.25 < 84\epsilon$

  (plastic section)

Hence the section is classified as plastic
section.

check for assumed self wt:-

wt. of section $= 0.514$ kN/m < 1 kN/m (assumed)

  (hence ok).

(3) check for bending & shear strength:-

Design shear strength of section:-

$$V_d = \frac{f_{yw} \times A_v}{\sqrt{3} \times \gamma_{mo}}$$

$$A_v = h \times t_w$$
$$= 350 \times 8.1$$

$$= \frac{250 \times 350 \times 8.1}{\sqrt{3} \times 1.1}$$

$$V_d = 371.997 \text{ kN}.$$

$$\Rightarrow 0.6 \, V_d = 0.6 \times 371.997$$
$$= 223.198 \text{ kN}.$$

③ $(V = 196.05 < V_d = 371.997)$

(hence safe).

∴ $V = 196.05 < (0.6 \, V_d) = 223.198$ (low shear case).

(b) For low shear case $(V < 0.6 \, V_d):-$

design bending strength $(M_d):-$

$$M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}} < 1.2 Z_e \frac{f_y}{\gamma_{mo}}$$ (For simply supported beam)

($\beta = 1$ (for plastic section))

$$M_d = \frac{1 \times 889.6 \times 10^3 \times 250}{1.1}$$

$$< \frac{1.2 \times 778.9 \times 10^3 \times 250}{1.1}$$

$$M_d = 202.18 \text{ kNm} < 212.43 \text{ kNm}. \text{ (ok)}$$

$$M = 136.87 < M_d = 202.18 \text{ (Hence the section is safe)}.$$

(4) check for deflection:-

Permissible deflection for a beam on building assuming elastic cladding:- $\frac{l_e}{240}$

$$= \frac{1400}{240} = 5.83$$

maximum deflection corresponding to load:

$$\delta = \frac{5wl^4}{384EI} + \frac{Wl^3}{48EI}$$

$$= \frac{5 \times 1.5 \times 10^3 \times (1400)^4}{384 \times 2 \times 10^5 \times 13630.3 \times 10^4} + \frac{260 \times 10^3 \times (1400)^3}{48 \times 2 \times 10^5 \times 13630.4 \times 10^4}$$

$$= 5.208 < \frac{Le}{240}. \quad \text{(so the section is safe).}$$

Web crippling.

→ When a member is subjected to concentrated load, then this load is resisted by compressive stress of member. So stress concentration occur at the junction of webs flange.

→ Crippling occur near the stress concentration zone (local buckling occurs at perticular area).
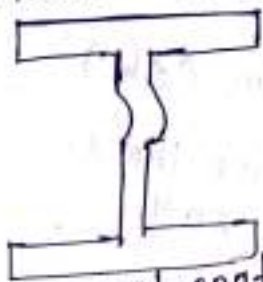
→ so bearing strength should be greater than the concentrated load.

→ so to avoid web crippling bearing stiffner is provided on thickness of web is provided.

Web buckling.

→ When vertical compressive stress exceed critical buckling stress for web acting as column.

→ not occur in hot rolled beam section.

→ occurs in case of plate girder.

assumed to exist if the frictional or other positive restraint of a floor connection to the compression flange of the member is capable of resisting a lateral force not less than 2.5 percent of the maximum force in the compression flange of the member. This may be considered to be uniformly distributed along the flange, provided gravity loads constitute the dominant loading on the member and the floor construction is capable of resisting this lateral force.

The design bending strength of a section which is not susceptible to web buckling under shear before yielding (where $d/t_w \leq 67\varepsilon$) shall be determined according to 8.2.1.2.

8.2.1.1 *Section with webs susceptible to shear buckling before yielding*

When the flanges are plastic, compact or semi-compact but the web is susceptible to shear buckling before yielding ($d/t_w \leq 67\varepsilon$), the design bending strength shall be calculated using one of the following methods:

a) The bending moment and axial force acting on the section may be assumed to be resisted by flanges only and the web is designed only to resist shear (*see* **8.4**).

b) The bending moment and axial force acting on the section may be assumed to be resisted by the whole section. In such a case, the web shall be designed for combined shear and normal stresses using simple elastic theory in case of semi-compact webs and simple plastic theory in the case of compact and plastic webs.

8.2.1.2 When the factored design shear force does not exceed $0.6 V_d$, where V_d is the design shear strength of the cross-section (*see* **8.4**), the design bending strength, M_d shall be taken as:

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than $1.2 Z_e f_y / \gamma_{m0}$ incase of simply supported and $1.5 Z_e f_y / \gamma_{m0}$ in cantilever beams;

where

β_b = 1.0 for plastic and compact sections;

β_b = Z_e / Z_p for semi-compact sections;

Z_p, Z_e = plastic and elastic section modulii of the cross-section, respectively;

f_y = yield stress of the material; and

γ_{m0} = partial safety factor (*see* **5.4.1**).

8.2.1.3 When the design shear force (factored), V exceeds $0.6V_d$, where V_d is the design shear strength of the cross-section (*see* **8.4**) the design bending strength, M_d shall be taken

$$M_d = M_{dv}$$

where

M_{dv} = design bending strength under high shear as defined in **9.2**.

8.2.1.4 *Holes in the tension zone*

a) The effect of holes in the tension flange, on the design bending strength need not be considered if

$$(A_{nf} / A_{gf}) \geq (f_y/f_u) (\gamma_{m1}/\gamma_{m0}) / 0.9$$

where

A_{nf} / A_{gf} = ratio of net to gross area of the flange in tension,

f_y/f_u = ratio of yield and ultimate stress of the material, and

γ_{m1}/γ_{m0} = ratio of partial safety factors against ultimate to yield stress (*see* **5.4.1**).

When the A_{nf}/A_{gf} does not satisfy the above requirement, the reduced effective flange area, A_{gf} satisfying the above equation may be taken as the effective flange area in tension, instead of A_{gf}

b) The effect of holes in the tension region of the web on the design flexural strength need not be considered, if the limit given in (a) above is satisfied for the complete tension zone of the cross-section, comprising the tension flange and tension region of the web.

c) Fastener holes in the compression zone of the cross-section need not be considered in design bending strength calculation, except for oversize and slotted holes or holes without any fastener.

8.2.1.5 *Shear lag effects*

The shear lag effects in flanges may be disregarded provided:

a) For outstand elements (supported along one edge), $b_o \leq L_o / 20$; and

b) For internal elements (supported along two edges), $b_i \leq L_o / 10$.

where

L_o = length between points of zero moment (inflection) in the span,

b_o = width of the flange with outstand, and

b_i = width of the flange as an internal element.

Where these limits are exceeded, the effective width of flange for design strength may be calculated using

forces required shall be taken as 2.5 percent of the maximum force in the compression flange plus 1.25 percent of this force for every member of the series other than the first, up to a maximum total of 7.5 percent.

8.3.5 Purlins adequately restrained by sheeting need not be normally checked for the restraining forces required by rafters, roof trusses or portal frames that carry predominately roof loads provided there is bracing of adequate stiffness in the plane of rafters or roof sheeting which is capable of acting as a stressed skin diaphragm.

8.3.6 In case of beams with double curvature bending, adequate direct lateral support to the compression flange in the hogging moment region may be provided as given above for simply supported beam. The effect of support to the tension (top) flange in the hogging moment region on lateral restraint to the compression flange may be considered as per specialist literature.

8.4 Shear

The factored design shear force, V, in a beam due to external actions shall satisfy

$$V \le V_d$$

where

V_d = design strength

= V_n / γ_{m0}

where

γ_{m0} = partial safety factor against shear failure (see **5.4.1**).

The nominal shear strength of a cross-section, V_n, may be governed by plastic shear resistance (see **8.4.1**) or strength of the web as governed by shear buckling (see **8.4.2**).

8.4.1 The nominal plastic shear resistance under pure shear is given by:

$$V_n = V_p$$

where

$$V_p = \frac{A_v f_{yw}}{\sqrt{3}}$$

A_v = shear area, and

f_{yw} = yield strength of the web.

8.4.1.1 The shear area may be calculated as given below:

I and channel sections:

Major Axis Bending:

Hot-Rolled $- h t_w$
Welded $- d t_w$

Minor Axis Bending:

Hot-Rolled or Welded $- 2 b t_f$

Rectangular hollow sections of uniform thickness:

Loaded parallel to depth (h) $- A h / (b + h)$
Loaded parallel to width (b) $- A b / (b + h)$

Circular hollow tubes of uniform thickness $- 2 A / \pi$

Plates and solid bars $- A$

where

A = cross-section area,

b = overall breadth of tubular section, breadth of I-section flanges,

d = clear depth of the web between flanges,

h = overall depth of the section,

t_f = thickness of the flange, and

t_w = thickness of the web.

NOTE — Fastener holes need not be accounted for in plastic design shear strength calculation provided that:

$$A_{vn} \ge (f_y / f_u) (\gamma_{m1} / \gamma_{m0}) A_v / 0.9$$

If A_{vn} does not satisfy the above condition, the effective shear area may be taken as that satisfying the above limit. Block shear failure criteria may be verified at the end connections. Section 9 may be referred to for design strength under combined high shear and bending.

8.4.2 *Resistance to Shear Buckling*

8.4.2.1 Resistance to shear buckling shall be verified as specified, when

$$\frac{d}{t_w} > 67\varepsilon \text{ for a web without stiffeners, and}$$

$$> 67\varepsilon \sqrt{\frac{K_v}{5.35}} \text{ for a web with stiffeners}$$

where

K_v = shear buckling coefficient (see **8.4.2.2**), and

$\varepsilon = \sqrt{250 / f_y}$

8.4.2.2 *Shear buckling design methods*

The nominal shear strength, V_n, of webs with or without intermediate stiffeners as governed by buckling may be evaluated using one of the following methods:

a) *Simple post-critical method* — The simple post critical method, based on the shear buckling strength can be used for webs of I-section girders, with or without intermediate transverse stiffener, provided that the web has transverse stiffeners at the supports. The nominal shear strength is given by:

$$V_n = V_{cr}$$

9.2.2 When the factored value of the applied shear force is high (exceeds the limit specified in **9.2.1**), the factored moment of the section should be less than the moment capacity of the section under higher shear force, M_{dv} calculated as given below:

a) *Plastic or Compact Section*

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \le 1.2 Z_e f_y / \gamma_{m0}$$

where

$$\beta = (2V/V_d - 1)^2$$

M_d = plastic design moment of the whole section disregarding high shear force effect (*see* **8.2.1.2**) considering web buckling effects (*see* **8.2.1.1**),

V = factored applied shear force as governed by web yielding or web buckling,

V_d = design shear strength as governed by web yielding or web buckling (*see* **8.4.1** or **8.4.2**),

M_{fd} = plastic design strength of the area of the cross-section excluding the shear area, considering partial safety factor γ_{m0}, and

Z_e = elastic section modulus of the whole section.

b) *Semi-compact Section*

$$M_{dv} = Z_e f_y / \gamma_{m0}$$

9.3 Combined Axial Force and Bending Moment

Under combined axial force and bending moment, section strength as governed by material failure and member strength as governed by buckling failure shall be checked in accordance with **9.3.1** and **9.3.2** respectively.

9.3.1 *Section Strength*

9.3.1.1 *Plastic and compact sections*

In the design of members subjected to combined axial force (tension or compression) and bending moment, the following should be satisfied:

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \le 1.0$$

Conservatively, the following equation may also be used under combined axial force and bending moment:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \le 1.0$$

where

M_y, M_z = factored applied moments about the minor and major axis of the cross-section, respectively;

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone (*see* **9.3.1.2**);

N = factored applied axial force (Tension, T or Compression, P);

N_d = design strength in tension, T_d as obtained from **6** or in compression due to yielding given by $N_d = A_g f_y / \gamma_{m0}$;

M_{dy}, M_{dz} = design strength under corresponding moment acting alone (*see* **8.2**);

A_g = gross area of the cross-section;

α_1, α_2 = constants as given in Table 17; and

γ_{m0} = partial factor of safety in yielding.

9.3.1.2 For plastic and compact sections without bolts holes, the following approximations may be used for evaluating M_{ndy} and M_{ndz}:

a) *Plates*

$$M_{nd} = M_d (1 - n^2)$$

b) *Welded I or H sections*

$$M_{ndy} = M_{dy} \left[1 - \left(\frac{n-a}{1-a}\right)^2\right] \le M_{dy} \text{ where } n \ge a$$

$$M_{ndz} = M_{dz} (1 - n) / (1 - 0.5a) \le M_{dz}$$

where

$$n = N/N_d \qquad \text{and } a = (A - 2 b t_f)/A \le 0.5$$

c) *For standard I or H sections*

for $n \le 0.2 \quad M_{ndy} = M_{dy}$

for $n > 0.2 \quad M_{ndy} = 1.56 M_{dy} (1 - n) (n + 0.6)$

$$M_{ndz} = 1.11 M_{dz} (1 - n) \le M_{dz}$$

d) *For rectangular hollow sections and welded box sections*

When the section is symmetric about both axes and without bolt holes

$$M_{ndy} = M_{dy} (1 - n) / (1 - 0.5a_t) \le M_{dy}$$

$$M_{ndz} = M_{dz} (1 - n) / (1 - 0.5a_w) \le M_{dz}$$

where

$a_w = (A - 2 b t_f)/A \le 0.5$

$a_t = (A - 2 h t_w)/A \le 0.5$

e) *Circular hollow tubes without bolt holes*

$$M_{nd} = 1.04 M_d (1 - n^{1.7}) \le M_d$$

Table 2 Limiting Width to Thickness Ratio

(*Clauses* 3.7.2 *and* 3.7.4) $b = \dfrac{b}{2}$

Compression Element			Ratio	Class of Section		
				Class 1 Plastic	Class 2 Compact	Class 3 Semi-compact
(1)			(2)	(3)	(4)	(5)
Outstanding element of compression flange	Rolled section		b/t_f	9.4ε	10.5ε	15.7ε
	Welded section		b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending		b/t_f	29.3ε	33.5ε	42ε
	Axial compression		b/t_f	Not applicable		
Web of an I, H or box section	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\dfrac{84\varepsilon}{1+r_1}$ but $\le 42\varepsilon$	$\dfrac{105.0\varepsilon}{1+r_1}$	$\dfrac{126.0\varepsilon}{1+2r_1}$ but $\le 42\varepsilon$
		If r_1 is positive :	d/t_w		$\dfrac{105.0\varepsilon}{1+1.5r_1}$ but $\le 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component			d/t	9.4ε	10.5ε	15.7ε
Stem of a T-section, rolled or cut from a rolled I- or H- section			D/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to: a) moment			D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
b) axial compression			D/t	Not applicable		$88\varepsilon^2$

NOTES

1 Elements which exceed semi-compact limits are to be taken as of slender cross-section.

2 $\varepsilon = (250/f_y)^{1/2}$.

3 Webs shall be checked for shear buckling in accordance with 8.4.2 when $d/t > 67\varepsilon$, where, b is the width of the element (may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), t is the thickness of element, d is the depth of the web, D is the outer diameter of the element (see Fig. 2, 3.7.3 and 3.7.4).

4 Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favourable classification.

5 The stress ratio r_1 and r_2 are defined as:

$$r_1 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of web alone}}$$

$$r_2 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of overll section}}$$

Tubular Steel Structure.

Tubular steel structures are used in truss members, scaffolding of building, stadium, exhibition halls, transmission towers.

codes Required: IS 1161.1998 , IS 806.1968.

Advantages:-

(i) These have small self weights. Also because of direct connections, gusset plates are eliminated further reducing dead loads.

(2) Torsional strength of these structures is more than any other rolled section.

(3) For the same load, the surface area of a tube is about 60 to 70% of that for other rolled sections. Because of less area economy is achieved in maintenance, painting & fire proofing.

(4) Due to smooth finished surface, dirt & moisture do not collect over the surface, reducing the possibility of corrosion.

(5) ~~Due to~~. The change in load with the floor levels can be accomodated by varying the tube thickness & the external tube dimension may be maintained.

(6) The internal hollow space of tubular columns may be used for carrying drain pipes, wires cable etc. Also these spaces may

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be filled with concrete to increase the load carrying capacity s to improve fire resistance.

Disadvantage:

1) They pose difficulty in connection among themselves or to any plate element due to their shape problems.

2) Bolting s riveting on those sections are not convenient.

3) There light weight some times become responsible for the structural instability.

4) Highly skilled man power s special welding techniques are required for their connection.

Designation of steel tubes:

Steel tubes are designated by their nominal bore size.

classification of steel tube. →

YS → Yield stress
t → tube.

22, 25, 32 ares on kg/1mm2.

Based on wall thickness
→ light   medium   heavy.

Based on yield stress
→ YSt 22   YSt 25   YSt 32.
   (YSt 210)   YSt 240   YSt 310
   Mpa   s mpa   s mpa
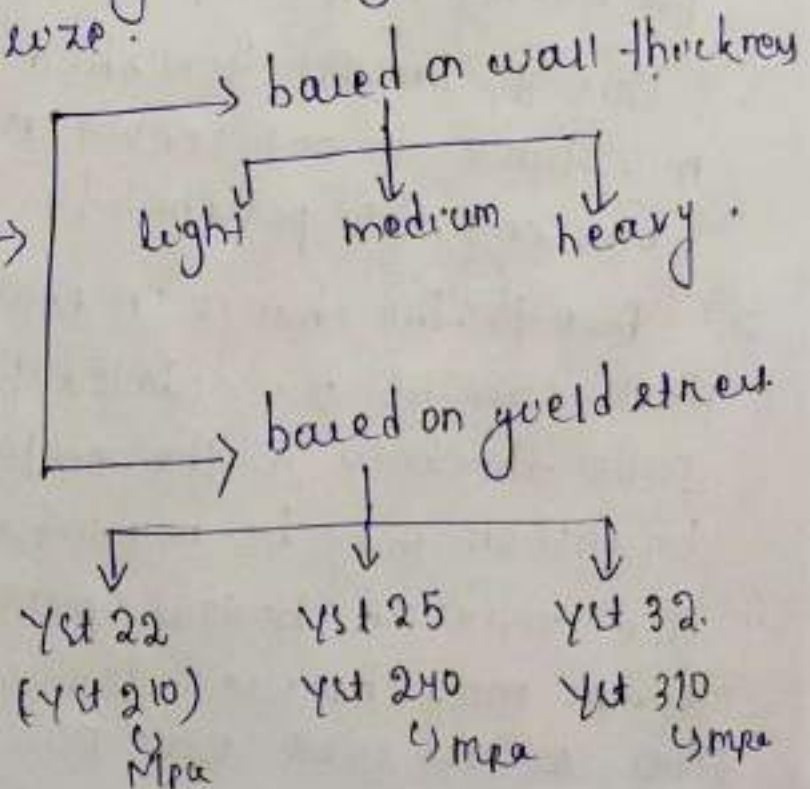
Note
(•)* $1 MPa = 1 N/mm^2 = 0.102 kgf/cm^2$

Tensile properties of steel tubes for
structural purposes :-

(table-2, clause -11.2.1., IS code. 1161)

Grade	tensile strength (Min)	Yield stress (Min).
Yst 210	330	210
Yst 240	410	240
Yst 310	450	310.

* The standard sizes of tubular sections,
their mass/weight, relevant geometrical
properties are given in table 1 as per
IS :1161: 1998.

Behaviour of tubular Sections :-

(1) Compression member :-
depends upon slenderness ratio.

slenderness ratio $\lambda = \dfrac{l_{eff}}{r}$

$l_{effective} \rightarrow$ depends of end conditions.
$= kL$ (table-7, cl.6.4) IS 800
$r =$ we will get from section properties
given is IS1161:1998.

maximum limit of slenderness ratio :

type of member. $\lambda = \frac{l}{r}$

(a) Carrying loads resulting
from dead loads & super-
imposed load.   180

(b) carrying loads resulting
from wind or seismic forces
only provided the deformation
of such members does not   250
adversely, affect the stress
in any part of the structure.

(c) Normally acting as a tie
in a roof truss but subject   350.
to possible reversal on
stress resulting from the
action of wind.

Axial stress in compression: The direct stress
in compression on the cross-sectional area
of axially loaded steel tubes shall not
exceed the value of F_c given in table-2)
of IS 806:1968.

Permissible axial stress in compression →
                                        (F_c).
          (table 2, clause 5.2)
              (IS 806)

the cross-sectional area of axially loaded steel tubes shall not exceed the values of F_c given in Table 2 in which l/r is equal to the effective length of the member divided by the radius of gyration.

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*Specification for steel tubes for structural purposes (*revised*) (*Second revision* in 1968).

†Specification for covered electrodes for metal arc welding of mild steel (*revised*) (**Third** revision in 1970).

‡Code of practice for structural safety of buildings: Loading standards (*revised*).

TABLE 2 PERMISSIBLE AXIAL STRESS IN COMPRESSION
(*Clause* 5.2)

l/r	F_c		
	GRADE YSt 22 kgf/cm²	GRADE YSt 25 kgf/cm²	GRADE YSt 32 kgf/cm²
(1)	(2)	(3)	(4)
0	1 250	1 500	1 900
10	1 217	1 448	1 821
20	1 175	1 400	1 760
30	1 131	1 352	1 679
40	1 088	1 303	1 610
50	1 046	1 255	1 539
60	1 002	1 207	1 468
70	970	1 156	1 375
80	929	1 088	1 283
90	876	1 003	1 128
100	814	910	989
110	745	813	869
120	674	721	758
130	603	638	665
140	540	565	584
150	480	503	517
160	432	443	450
170	381	392	396
180	339	348	353
190	304	311	316
200	271	278	280
210	243	249	250
220	219	225	227
230	198	204	203
240	180	185	187
250	162	167	167
300	106	106	106
350	71	71	72

NOTE 1 — Intermediate values may be obtained by linear interpolation.

NOTE 2 — The formula, from which these values have been derived, is given in Appendix A.

5.3 Bending Stresses — In tubes, the tensile bending stress and the compressive bending stress in the extreme fibres shall not exceed the values of F_b given in Table 3.

TABLE 3 PERMISSIBLE BENDING STRESS IN EXTREME FIBRES IN

6.4 Compression Members

6.4.1 *Effective Length of Compression Members* — Effective length (*l*) of a compression member for the purpose of determining allowable axial stresses shall be assumed in accordance with Table 7, where *L* is the actual length of the strut, measured between the centres of lateral supports. In the case of a compression member provided with a cap or base, the point of lateral support at the end shall be assumed to be in the plane of the top of the cap or bottom of the base.

TABLE 7 EFFECTIVE LENGTH OF COMPRESSION MEMBERS

TYPE	EFFECTIVE LENGTH
Effectively held in position and restrained in direction at both ends	0·67 *L*
Effectively held in position at both ends and restrained in direction at one end	0·85 *L*
Effectively held in position at both ends but not restrained in direction	*L*
Effectively held in position and restrained in direction at one end, and at the other end effectively restrained in direction but not held in position	*L*
Effectively held in position and restrained in direction at one end, and at the other end partially restrained in direction but not held in position	1·5 *L*
Effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end	2·0 *L*

Table 1 Sizes and Properties of Steel Tubes for Structural Purposes

(*Clauses* 3.1, 6.1, 6.1.1 *and* 6.1.2)

| Nominal Bore | Outside Diameter | Class | Thickness | Weight | Area of Cross Section | Internal Volume | Surface | | Moment of Inertia | Modulus of Section | Radius of Gyration | Square of Radius of Gyration |
| | | | | | | | External | Internal | | | | |
mm (1)	mm (2)	(3)	mm (4)	kg/m (5)	cm² (6)	cm³/m (7)	cm²/m (8)	cm²/m (9)	cm⁴ (10)	cm³ (11)	cm (12)	cm² (13)
15	21.3	Light	2.0	0.947	1.21	235		543	0.57	0.54	0.69	0.47
		Medium	2.6	1.21	1.53	203	669	506	0.69	0.64	0.66	0.44
		Heavy	3.2	1.44	1.82	174		468	0.75	0.70	0.55	0.42
20	26.9	Light	2.3	1.38	1.78	399		700	1.36	1.01	0.87	0.76
		Medium	2.6	1.56	1.98	370	845	681	1.48	1.10	0.86	0.74
		Heavy	3.2	1.87	2.38	330		644	1.70	1.26	0.84	0.71
25	33.7	Light	2.6	1.98	2.54	638		895	3.09	1.83	1.10	1.21
		Medium	3.2	2.41	3.06	585	1 059	857	3.61	2.14	1.08	1.17
		Heavy	4.0	2.93	3.73	518		807	4.19	2.48	1.05	1.11
32	42.4	Light	2.6	2.54	3.25	1 086		1 168	6.47	3.05	1.41	1.98
		Medium	3.2	3.10	3.94	1 017	1 332	1 130	7.62	3.59	1.39	1.93
		Heavy	4.0	3.79	4.82	929		1 080	8.99	4.24	1.36	1.86
40	48.3	Light	2.9	3.23	4.13	1 418		1 335	10.70	4.43	1.61	2.59
		Medium	3.2	3.56	4.53	1 378	1 517	1 316	11.59	4.80	1.59	2.54
		Heavy	4.0	4.37	5.56	1 275		1 265	13.77	5.70	1.57	2.47
50	60.3	Light	2.9	4.08	5.23	2 332		1 711	21.59	7.16	2.03	4.13
		Medium	3.6	5.03	6.41	2 213	1 667		25.88	8.58	2.00	4.02
		Heavy	4.5	6.19	7.88	2 066		1 611	30.90	10.2	1.98	3.92
65	76.1	Light	3.2	5.71	7.32	3 814		2 189	48.79	12.82	2.58	6.66
		Medium	3.6	6.42	8.20	3 727	2 391	2 163	54.02	14.20	2.57	6.60
		Heavy	4.5	7.93	10.1	3 534		2 107	65.12	17.1	2.54	6.43
80	88.9	Light	3.2	6.72	8.61	5 343		2 591	79.23	17.82	3.03	9.19
		Medium	4.0	8.36	10.7	5 138	2 793	2 540	96.36	21.68	3.00	9.00
		Heavy	4.8	9.90	12.7	4 930		2 490	112.52	25.31	2.98	8.88
90	101.6	Light	3.6	8.70	11.1	6 993		2 964	133.27	26.23	3.47	12.03
		Medium	4.0	9.63	12.3	6 877	3 192	2 939	146.32	28.80	3.45	11.91
		Heavy	4.8	11.5	14.6	6 644		2 889	171.44	33.75	3.43	11.76
100	114.3	Light	3.6	9.75	12.5	9 004		3 363	192.03	33.60	3.92	15.36
		Medium	4.5	12.2	15.5	8 704	3 591	3 306	234.3	41.0	3.89	15.10
		Heavy	5.4	14.5	18.5	8 409		3 250	274.5	48.0	3.85	14.86
110	127.0	Light	4.5	13.6	17.3	10 930		3 705	325.3	51.2	4.33	18.78
		Medium	4.8	14.5	18.4	10 819	3 990	3 686	344.58	54.27	4.32	18.69
		Heavy	5.4	16.2	20.0	10 599		3 649	382.0	60.2	4.30	18.32
125	139.7	Light	4.5	15.0	19.1	13 410		4 104	437.2	62.6	4.78	22.87
		Medium	4.8	15.9	20.3	13 287	4 389	4 085	463.44	66.35	4.77	22.76
		Heavy	5.4	17.9	22.8	13 063		4 047	514.5	73.7	4.75	22.38
135	152.4	Light	4.5	16.4	20.9	16 142		4 503	572.2	75.1	5.23	27.37
		Medium	4.8	17.5	22.2	16 008	4 788	4 484	606.92	79.65	5.22	27.25
		Heavy	5.4	19.6	25.0	15 740		4 446	674.3	88.3	5.20	27.03
150	165.1	Light	4.5	17.8	22.7	19 128		4 902	732.6	88.7	5.68	32.27
		Medium	4.8	18.9	24.2	18 981	5 187	4 883	777.32	94.16	5.67	32.14
		Heavy	5.4	21.3	27.1	18 690		4 845	864.7	105.0	5.65	31.92
150	168.3	Light	4.5	18.2	23.1	19 921		5 002	777.2	92.4	5.79	33.56
		Medium	4.8	19.4	24.7	19 771	5 287	4 983	824.78	98.01	5.78	33.42
		Heavy 1	5.4	21.7	27.6	19 473		4 946	917.7	109.0	5.76	33.21
		Heavy 2	6.3	25.2	32.0	19 030		4 889	1 053	125.0	5.73	32.85
175	193.7	Light	4.8	22.4	28.5	26 606		5 781	1 271.71	131.31	6.68	44.63
		Medium	5.4	25.1	32.0	26 260	6 085	5 743	1 417	146	6.66	44.36
		Heavy	5.9	27.3	34.8	25 974		5 712	1 535.2	158.65	6.64	44.11
200	219.1	Light	4.8	25.4	32.3	34 454		6 578	1 856.51	169.47	7.58	57.45
		Medium	5.6	29.5	37.5	33 930	6 883	6 528	2 141	195	7.55	57.02
		Heavy	5.9	31.0	39.5	33 734		6 509	2 247	205	7.54	56.80
225	244.5	Heavy	5.9	34.7	44.2	42 507	7 681	7 307	3 149	258	8.44	71.21
250	273.0	Heavy	5.9	38.9	49.5	53 557	8 578	8 202	4 412	323	9.45	89.30
300	323.9	Heavy	6.3	49.3	62.8	76 073	10 177	9 775	7 992	493	11.2	125.44
350	355.6	Heavy	8.0	68.6	87.3	90 533	11 173	10 663	13 111	737	12.3	151.29

Q-1 A tubular steel column of 4.8m length is hinged at both ends. It has nominal dia of 225mm φ of Yst 25 grade. Determine the safe load carrying capacity of column.

Solution:-

step-1 Given data & section properties:-

thickness = 5.9mm, weight = 34.7 kg/m.

Area of cross section = 44.2 cm²

radius of gyration (r) = 8.44 cm.

nominal dia = 225mm.   Yst = 25

outside dia = 244.5mm.

length of column @ (L) = 4.8m.

as both the ends are hinged, effective length

$(l_e) = kL$
$= 1 \times L = 4.8m.$
(table-7, IS 806).

Step (2)

calculation of slenderness ratio:-

$\lambda = \dfrac{l_{eff}}{r}$, $\lambda = \dfrac{4.8 \times 10^3}{8.44 \times 10} = 56.87 < 180$ (cl-6.4.2)
(IS 806)
(ok)

Step-(3) permissible stress in compression-
(for Yst = 25) Y_c (kg/cm²) (from table-2)
(IS 806)

$\lambda_1 = 50 \longrightarrow 1255$

$\lambda = 56.87 \longrightarrow ?$

$\lambda_2 = 60 \longrightarrow 1207$

by interpolating we will get value of permissible compressive stress.

$$\frac{56.87 - 60}{50 - 60} \times (1255 - 1207) + 1207$$

$\Rightarrow f_c = 1222.024 \, kg/cm^2$ $\left(1 kg\text{-}f = 9.81 \, N \approx 10 \, N\right)$

$= 1222.024 \, N/cm^2 =$
$= 12220.24 \, N/cm^2$ $1 cm^2 = 100 mm^2$

$\frac{1 cm^2}{100 mm^2}$

Safe load carrying capacity :-

$$A \times f_c$$

$P = 44.2 \times 12220.24$
$= 5401.35 \, N$
$= 540;134 \, N \approx 540.134 \, kN.$

Assignment :-

Q-2 A tubular column consists of IS 1161 grad yst 32 steel. The column is hanged in both the ends. The outside diameter of tube is 219.1mm. The weight of 1m length of tube is 310N. The length of column is 4.5m. Determine the safe load carrying capacity of column.

Minimum thickness of material:

For tubes painted with one prime coat of red oxides then periodically painted, the thickness should not be less than:-

(i) For construction exposed to weather → ~~3·5mm~~ 4mm

(ii) For construction not exposed to weather → 3·2 mm.

(iii) For members not readily accessible for maintenance = 5mm.

For tubes painted with one coat of zinc primer followed by two coats of paint, the thickness should not be less than:-

(i) For construction exposed to weather = 3·2 mm

(ii) For construction not exposed to weather = 2·6mm.

Permissible axial stress in tension (table-1) IS 806)

Grade	F_t (kgf/cm²)
Yst 22	1250
Yst 25	1500
Yst 32	1900

permissible stresses as per IS 806 (table-3, 4, 5)

| Grade	Permissible bending stress (F_b)(kgf/cm²)	Permissible maxm shear stress (F_s)	permissible maximum bearing stress (F_p)
Yst 22	1400	900	1700
Yst 25	1655	1100	1900
Yst 32	2050	1350	2500

Connection:- (cl-5.7.2, appendix B, IS 806).
Determination of the length of the curve of
intersection of a tube with another tube
or with a flat plate !-

The length of the curve of intersection
may be taken as :-

$$P = a + b + 3\sqrt{a^2 + b^2}$$

$$a = \frac{d}{2} \cos\theta$$

$$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$$
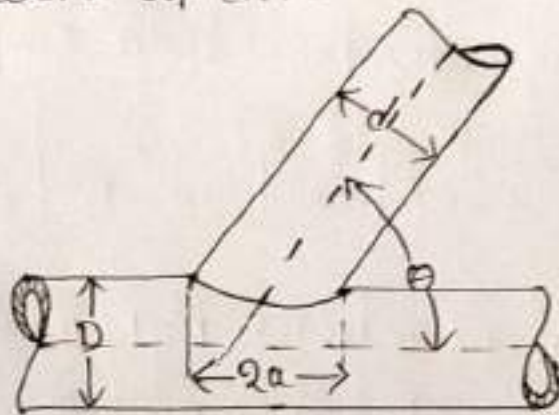
$\hookrightarrow$ For intersection with a tube.

$$= \frac{d}{2} \text{ For intersection with a flat plate.}$$

d = outside diameter of branch. tube

θ = angle between branch & main tube.

D = outside diameter of main tube.

Permissible stresses in welds:- (cl:- 5.7).

* For butt weld, the allowable tensile, compre-
ssive & shear stresses shall not exceed
the stresses respectively permissible in
Yst 25 tubes or in the parent metal, whichever
is less.

* In a fillet weld or in a fillet butt weld,
the permissible stress shall not exceed
the shear stress permissible in Yst 25 tubes
or in the parent metal, whichever is less.

Q A tension member carrying a force of 40kN meets the principal tie of tubular truss at an angle of 45°. If the force in principal tie is 100kN, Design the members & welded joint between the two tube. Use Yst 22 grade of steel. Assume these member are placed around inside location.

Design of Branch tube
(40kN member)
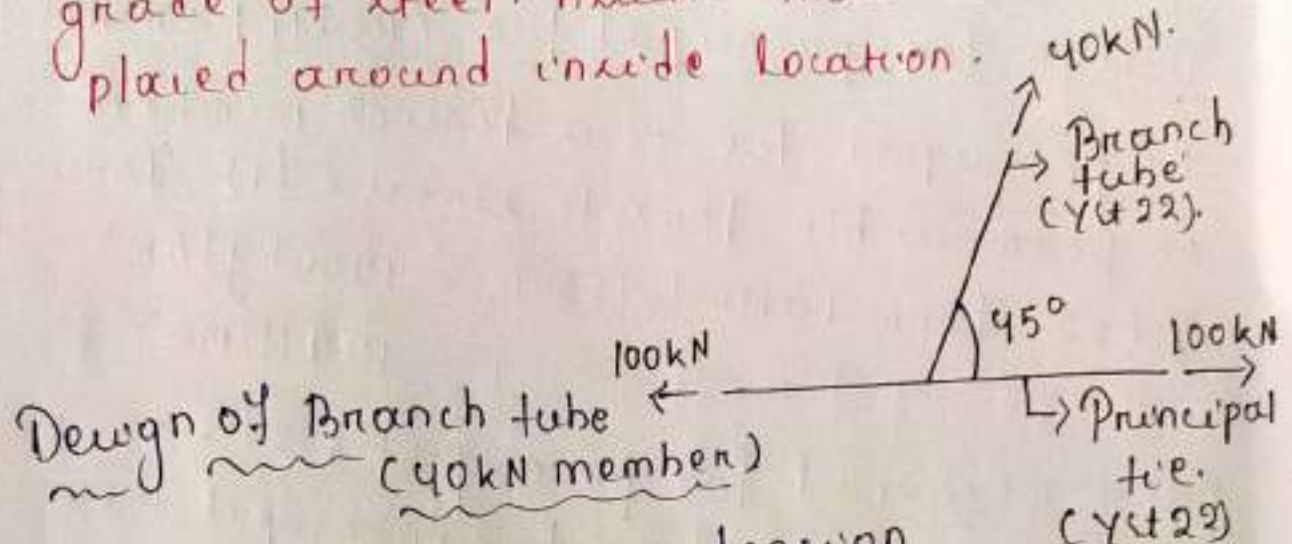
As the branch tube is a tension member, so permissible tensile stress for Yst 22 (ft) = 1250 kgf/cm² (table-1 IS806)
= 125 N/mm²

→ Load coming on the branch tube (P) = 40kN = 40×10³ N.

→ Area of the branch tube = $\frac{P}{f_t} = \frac{40×10^3}{125}$

= 320 mm²

Let choose cross-sectional area = 373 mm²

So let us provide steel tube of nominal bore size 25mm (class heavy) & outside diameter of 33.7mm & area of cross section 373mm².

check the minimum thickness from durability consideration →

we provide thickness of member = 4mm > 3.2mm.

(as this member is located inside, minimum thickness should be 3.2 mm as per cl 6.3.1).

Design for principal tie (100kN tie member)

As principal tie is a tension member so permissible tensile stress (ft) for Yst 22 from table-1, IS 806 ÷ 1250 kgf/cm²

$$= 125 \text{ N/mm}^2$$

Area required for principal tie :-

$$\frac{\text{load on principal tie}}{f_t}$$

$$= \frac{100 \times 10^3}{125} = 800 \text{mm}^2$$

Let provided area = 820mm² (choose from table-1, IS 1161).

So let us provide a steel tube of nominal bore of 65mm (medium class) & outside dia of 76.1 mm & provide thickness of 3.6 mm.

check for thickness.

provided thickness (3.6mm) > 3.2mm.

↳ minimum thickness required as per cl 6.3.1 for durability consideration.

so it is ok.

Design of connection :- (refer appendix B of
IS 806).

Length of connection =
$$p = a + b + 3\sqrt{a^2 + b^2}$$

$a = \dfrac{d}{2} \operatorname{cosec}\theta$

$= \dfrac{33.7}{2} \operatorname{cosec} 45$

$= 23.83$

$b = \dfrac{d}{3} \times \dfrac{3 - (d/D)^2}{2 - (d/D)^2}$

$\Rightarrow \dfrac{33.7}{3} \times \dfrac{3 - \left(\dfrac{33.7}{76.1}\right)^2}{2 - \left(\dfrac{33.7}{76.1}\right)^2} = 17.46$

d = outside dia
of branch
tube
$= 33.7$ mm.

D = outside dia of
principal tie
$= 76.1$ mm.

$P = a + b + 3\sqrt{a^2 + b^2}$
$= 23.83 + 17.46 + 3\sqrt{(23.83)^2 + (17.46)^2}$
$= 129.91$ mm.

let us assume fillet weld :-
permissible shear stress in weld :-

(i) permissible shear stress in
parent material (Yst 22) = 90 N/mm^2
(table - 4)

(ii) for Yst 25, permissible shear
stress = 110 N/mm^2

} minimum
will be
taken.

so permissible shear stress in weld :- 90 N/mm^2
Required area of weld = $\dfrac{40 \times 10^3}{90} = 444.44 \dfrac{}{\text{mm}^2}$

Area of weld = effective throat × length of
thickness   weld

$$444.44 = t_e \times 129.91$$

$$\Rightarrow t_e = 3.42 \text{ mm}.$$

effective throat thickness $(t_e) = 0.7 \times$ size of
weld

$$3.42 = 0.7 \times S$$

$$\Rightarrow S = \frac{3.42}{0.7} = 4.88 \text{ mm}.$$

So provide size of weld = 4.88 mm.

Q The principal rafter in round tubular truss
(compression member)
carries maximum force of 80kN. A tie member
meeting at a joint at right angle to carry
a force of 25 kN. If the panel length of
principal rafter is 2m. Design the member
as well as welded joint. Assume outside
location is Yst 25 grade of material.

Design of branch tie member (25kN)

As the branch tie member is
tension member, so
permissible tensile
stress for yst 25 from
table 1 (IS 806):-

$$f_t = 1500 \text{ kgf/cm}^2$$

$$= 150 \text{ N/mm}^2$$

tie (25kN)
(Yst 25)

← 2m →

(80kN) Principal
rafter
(Yst 25)
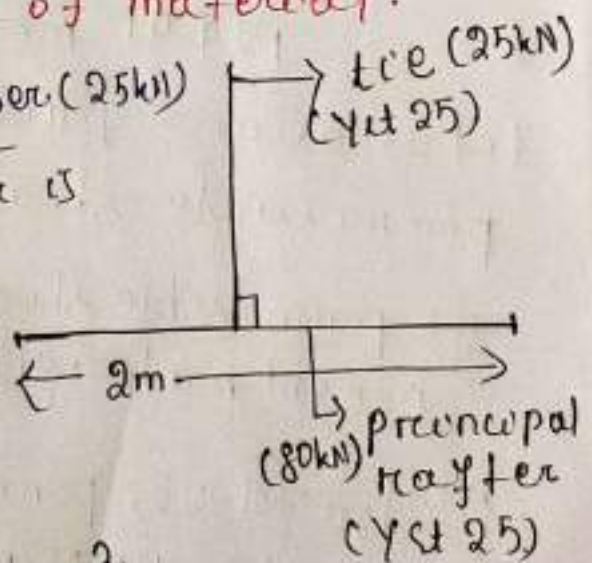
Area required for branch tee:-

$$= \frac{\text{Load on branch tee}}{f_t}$$

$$= \frac{25 \times 10^3}{150} = 166.67 \text{ mm}^2.$$

So provide a section of 373 mm² (class heavy) of nominal bore size 25mm ⌀ of outside diameter of 33.7 mm ⌀ thickness 4mm to satisfy durability requirements.

Ⓐ Design of principal rafter.

let assume slenderness ratio $\lambda = 100$

so for $\lambda = 100$, the safe compressive stress

$$(f_c) = 91 \text{ N/mm}^2$$

(from table-2)

Required area = $\dfrac{\text{Load on principal rafter}}{f_c}$

$$= \frac{80 \times 10^3}{901} = 879.12 \text{ mm}^2$$

letus provide a steel tube of nominal bore of 76.1mm of heavy class having cross section area 1010 mm², outside dia 76.1mm ⌀ the thickness 4.5mm which is more than 4mm. as per 3.6.3.1. (hence ok).

as nothing is given about end support condition, let assume effective length of principal rafter (leff) = 0.85L.

so, $L_e = 0.85 \times 2$
$= 1.7 \, m$.

radius of gyration of choosen section(r) $= 2.54 \, cm$.

slenderness ratio$(\lambda) = \dfrac{1.7 \times 100}{2.54}$

$= 66.92$

permissible stress according to $\lambda = 66.92$.
(from table-2
IS 806).

$\lambda = \dfrac{l}{r}$	$f_c \, (kg\mathcal{f}/cm^2)$ (for Yst 25)
60	~~1002~~ 1207
$\lambda = 66.92 \longrightarrow$	?
70	~~970~~ 1155

by interpolating,

$f_c = \dfrac{66.92 - 70}{60 - 70} \times (1207 - 1155) + 1155$

$= 1171.016 \; \cancel{N/mm^2} \; kg\mathcal{f}/cm^2 = 117.1016 \, N/mm^2$

Load $= f_c \times A \text{ provided}$

$= \cancel{1171.016 \times 1010}$

$= 117.1016 \times 1010$

$= 118273 \, N = 118.273 \, kN > 80 \, kN$
(OK).

Design of connection:- (appendex B of
IS 806).

Length of connection :-
$P = a + b + 3\sqrt{a^2 + b^2}$

$a = \dfrac{d}{2} \cos\theta$

$= \dfrac{33.7}{2} \cos 90°$

$= 16.85$

$d = 33.7 \, mm$
$D = 76.1 \, mm$.

$$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$$

$$= \frac{76.1}{3} \quad \frac{33.7}{3} \times \frac{3 - \left(\frac{33.7}{76.1}\right)^2}{2 - \left(\frac{33.7}{76.1}\right)^2} = 17.46$$

Length of weld $P = a + b + 3\sqrt{a^2 + b^2}$

$$P = 16.85 + 17.46 + 3\sqrt{(16.85)^2 + (17.46)^2}$$

$$= 107.104 \ mm$$

Let us assume fillet weld :

permissible shear stress in weld :-

(i) permissible shear stress in parent material (YSt 25) = 110 N/mm² (table-4)

(ii) for Yst 25, permissible shear stress = 110 N/mm² $\Big\}$ minimum will be taken.

So permissible shear stress in weld = 110 N/mm²

Required area of weld = $\dfrac{\text{load on branch tube}}{\text{permissible shear stress in weld}}$

$$= \frac{25 \times 10^3}{110} = 227.27 \ mm^2$$

Area of weld = effective throat thickness × Length of weld

$$227.27 = t_e \times 107.14$$

$$\Rightarrow t_e = 2.12 \ mm.$$

$t_e = 0.7 \times$ size of weld(s)

$$\Rightarrow 2.12 = 0.7 \times s \Rightarrow s = 3.028 \ mm.$$

So provide size of weld = 3.028mm

○ Tubular beam :-

limiting deflection of beam :-

The maximum deflection should not exceed 1/325 of the span for simply supported members. This requirements may be satisfied if the bending stress in compression or tension does not exceed $31500 \dfrac{D}{L}$ kgf/cm², where 'D' is the outer diameter of the tube in cm & 'L' is the effective length of beam in cm.

Q A medium steel tubular section of 200mm nominal diameter is simply supported as a flexural member over effective span of 4.5m. Determine the safe uniformly distributed super imposed load which can be placed over it? Assume Yst 25 grade of steel).

Section properties of 200mm nominal dia medium class steel tube :-
(from table)

thickness (t) = 5.6mm., γ = 29.5 kg/m.
                        = 289.395 N/m.

E = 2×10⁵ Mpa.

Area of cross section (A) = 37.5cm²
                        = 3750 mm²

modulus of section (Z) = 195 cm³
                        = 195000 mm³.

moment of Inertia (I) = 2141 cm⁴
                        = 2141×10⁴ mm⁴

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$$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$$

$$= \frac{\cancel{76.1}}{3} \frac{33.7}{3} \times \frac{3 - \left(\frac{33.7}{76.1}\right)^2}{2 - \left(\frac{33.7}{76.1}\right)^2} = 17.46$$

Length of weld $P = a + b + 3\sqrt{a^2 + b^2}$

$$P = 16.85 + 17.46 + 3\sqrt{(16.85)^2 + (17.46)^2}$$

$$= 107.104 \text{ mm}$$

Let us assume fillet weld :-

(i) permissible shear stress in weld :-
permissible shear stress in parent
material (Y4t 25) = 110 N/mm² (table-4)

(ii) For Y4t 25, permissible shear $\Big\}$ minimum
stress = 110 N/mm² $\quad$ will be
taken.

So permissible shear stress in weld = 110 N/mm²

Required area of weld = $\dfrac{\text{load on branch tube}}{\text{permissible shear stress in weld}}$

$$= \frac{25 \times 10^3}{110} = 227.27 \text{ mm}^2$$

Area of weld = effective throat thickness × length of weld

$$227.27 = t_e \times 107.14$$

$$\Rightarrow t_e = 2.12 \text{ mm}$$

$t_e = 0.7 \times \text{size of weld (s)}$
$\Rightarrow 2.12 = 0.7 \times S \Rightarrow S = 3.028 \text{ mm}$

(i) load carrying capacity based on bending stress:-

permissible bending stress for Y_{st} 25 (f_b) =
$$1655 \, kgf/cm^2$$
$$= 165.5 \, N/mm^2$$

allowable bending stress due to load applied (f_{ab}) $= \frac{M}{z}$.

For simply supported beam, ① bending moment due to uniformly distributed load $(M) = \frac{\omega \, l_{eff}^2}{8}$ (leff given = 4.5m)

$$= \frac{\omega (4.5)^2}{8} = 2.53 \omega N \cdot m.$$

section modulus (z) from section properties
$$19500 mm^3.$$

$$f_{ab} = f_b$$

$$\frac{M}{z} = f_b \implies \frac{2.53 \, \omega \times 10^3}{195000} = 165.5$$

$$\implies \omega = 12755.928 \, N/m.$$

$$= \boxed{12.756 \, kN/m.}$$

(2) based on shear stress:-

For Y_{st} 25, permissible maximum shear stress $(f_s) = 110 \, N/mm^2$

allowable shear stress due to applied load $(f_{as}) = 2 \times \dfrac{\text{maximum shear force} (V)}{\text{Area of section.}}$

maximum shear force (V) due to applied

load $= \frac{w \, leff}{2} = \frac{w \times 4.5}{2} = 2.25 w$ N $\binom{\text{let } w}{\text{in N/m}}$

Area of section from section properties:

$A = 3750 \, mm^2$

Now, $f_{as} = f_s$

$\Rightarrow 2 \times \frac{V}{A} = f_s \Rightarrow 2 \times \frac{2.25 w}{3750} = 110$

$\Rightarrow w = 91667 \cancel{10000}$ N/m

$= \boxed{91.667 \text{ kN/m.}}$

(B) Load carrying capacity in view of deflection:-

Maximum permissible deflection for simple supported beam $\delta_{max} =$

$\frac{1}{325} \times leff$

$= \frac{1}{325} \times 4.5 \times 10^3$

$= 13.85 \, mm.$

(w is in N/mm)

Deflection due to applied load:-

$\delta = \frac{5}{384} \cdot \frac{w(leff)^4}{EI}$

$= \frac{5}{384} \times \frac{w \times (4.5 \times 10^3)^4}{2 \times 10^5 \times 2141 \times 10^4 \times 10^3}$

$= \cancel{10258 w} \, 1246.93 w$

Now $\delta = \delta_{max}$

$\Rightarrow 1.25 w = 13.85 \Rightarrow w = 11.08 \, N/mm.$

$= \boxed{11.08 \text{ kN/m.}}$

load carrying capacity = minimum of above 3 loads calculated

$$w = 11.08 \text{ kN/m}.$$

self wt. of member (dead load) or $\gamma =$ 289.395 N/m.

$w =$ super imposed load (live load) + dead load. $= 0.289$ kN/m.

$\Rightarrow$ super imposed load = $w -$ dead load

$= 11.08 - 0.289$

$= 10.79 \text{ kN/m}.$