Working Stress method

1.1 Objectives of design & detailing. State the different methods of design of concrete structures

1.2 Introduction to reinforced concrete, R.C. sections their behaviour, grades of concrete & steel. Permissible stresses, assumptions in WSM

1.3 Flexural design & analysis of single reinforced sections from first principles

1.4 Concept of under reinforced, over reinforced & balanced sections.

1.5 Advantages & disadvantages of WSM, reasons for its obsolescence.

Structural Design

Structural design is a methodical investigation to get the economical specification of a structure or a structural element to carry the predicted load safely. With the application of structural design we can obtain required size, grade, reinforcement etc of structural members to withstand the internal forces calculated from the structural analysis.

* Structural design is a part of structural engineering which is a sub-division of civil Engg. Structural engineers are trained to design the structural elements accurately & supervise during construction to ensure the proper implementation of the structural design.

Objectives of design & detailing

The objectives of structural design is to design the structure for stability, strength & serviceability. It must also be economical & aesthetic.

* The design of a structure must satisfy 3 basic requirements

(1) Stability to prevent overturning, sliding or buckling of the structure or parts of it, under the action of loads.

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(2) Strength to resist safely the stresses induced by the loads in the various structural members.

(3) Serviceability to ensure the satisfactory performance under service load conditions. which implies providing adequate stiffness & reinforcements to contain deflections, crack-widths & vibrations within acceptable limits & also providing impermeability & durability.

* There are 2 other considerations that a sensible designer ought to bear in mind, viz., economy & aesthetics. One can always design a massive structure, which has more than adequate stability, strength & ~~durability~~ serviceability, but the ensuing cost of the structure may be exorbitant & the end product far from aesthetic.

Different methods of design of concrete structures

There are 4 philosophies for the design of reinforced concrete ~~prestressed as well as steel~~ covered structures.

→ Working Stress design
→ Ultimate load design
→ Limit state design
→ Performance based design.

Introduction to reinforced Concrete

* Concrete being extremely weak in tension, has the tendency to crack when tensile stresses are induced in any part of the structure. To arrest these cracks, reinforcement is provided in the tension zone.

* It may, however, be noted that reinforcement does not avoid cracks but it simply arrests further widening of cracks. So by providing tensile reinforcement at a proper place, the microcracks do develop but generally they are so small in width that these are not visible to the naked eye. Such a concrete which has reinforcement in tension zone, is known as "Reinforced Cement Concrete". Concrete develops in

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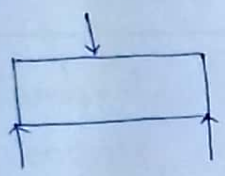
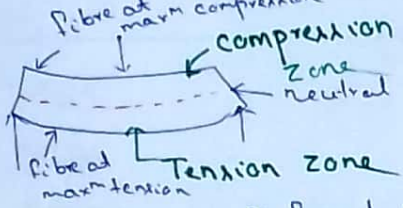
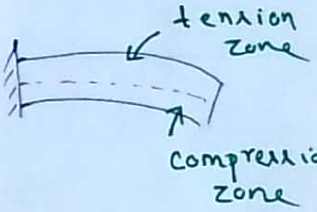
fact, a very good bond with steel.

* The ratio betⁿ the amounts of fine & coarse aggregates necessarily depends on the grading & other characteristics of the materials in order that the volume of sand is sufficient to fill the voids in the coarse aggregate to produce a dense concrete. Water used for mixing & curing should be clean & free from injurious amounts of oils, acids, alkalies, salts, sugar, organic materials or other substances that may be deleterious to concrete or steel. Potable water is generally considered satisfactory for mixing concrete. Mixing or curing of concrete with sea water is not recommended because of the presence of the harmful salts in sea water. The presence of tannic acid or iron compounds in water is objectionable.





Deformed shape of beam



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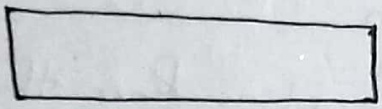
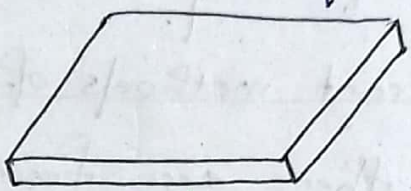
Different R.C. Sections

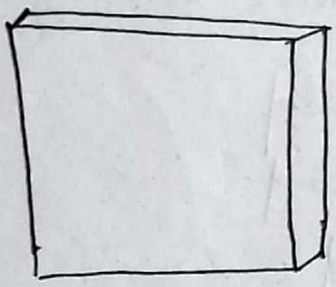
A reinforced Concrete flexural member may be a beam, slab, wall or component of a foundation.



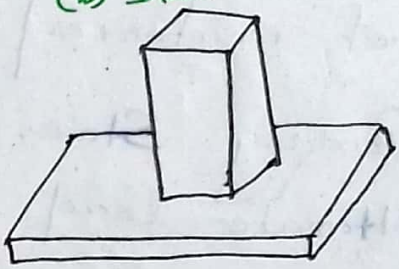
(a) Beam



(b) Slab

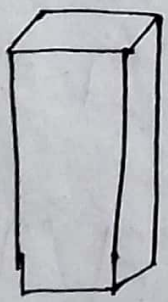


(c) wall

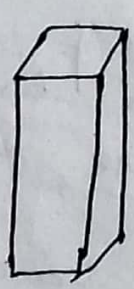


(d) Foundation.

A reinforced concrete compression member can be a column, which can be made rectangular, square or circular.



(a) Rectangular column



(b) Square column



(c) Circular Column.

Grade of concrete and Steel

Concrete is graded according to its compressive strength. The various grades

Grades of Concrete:

Concrete is graded according to its compressive strength. The various grades of concrete as stipulated in IS:456-2000 as shown in table below.

Group	Designation	Characteristic compressive strength of 150mm cube at 28 days (N/mm^2)
Ordinary Concrete	M10	10
	M15	15
	M20	20
Standard Concrete	M25	25
	M30	30
	M35	35
	M40	40
	M45	45
	M50	50
	M55	55
High Strength Concrete	M60	60
	M65	65
	M70	70
	M75	75
	M80	80

* In this classification of a concrete mix, letter M refers to the mix & the number to the specified characteristic compressive strength of 150mm cubes at 28 days expressed in N/mm^2 or MPa.

* The characteristic strength is defined as the strength of material below which not more than 5% of the test results are expected to fall.

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* Concretes of compressive strength lower than 20 MPa may be used for lean in ~~reinforced~~ concrete ~~works~~ bases & simple foundations for masonry walls. Grades lower than M20 should not be used in reinforced concrete works, lower than M40 should not be used in pre-tensioned prestressed concrete, and lower than M30 should not be used in post-tensioned prestressed concrete.

* For high strength concrete, the design parameters such as stress strain curve & stress block parameters given in IS 456 2000 code may have to be obtained from specialized literature.

* The mix proportions shall be selected to ensure the workability of the fresh concrete & when concrete is hardened, it shall have the required strength, durability & surface finish.

* The determination of the proportions of cement, aggregates & water to attain the required strengths shall be made as follows.

(a) by designing the concrete mix; such concrete shall be called 'Design mix concrete.' or

(b) by adopting nominal concrete mix; such concrete shall be called 'Nominal mix concrete'.

* Design mix concrete is preferred to nominal mix. If design mix concrete cannot be used for any reason on the work for grades of M20 or lower, nominal mixes may be used with the permission of engineer-in-charge, which however, is likely to involve a higher cement content.

(1) Design mix Concrete

The proportion of ingredients of concrete to obtain the desired mix can be found out by laboratory method of mix design. Design mix concrete is treated in IS:10262 & may be preferred.

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b) Nominal mix Concrete.

When the concrete is produced by taking standard arbitrary proportions of concrete ingredients, it is known as nominal mix concrete. Some nominal mix proportions are: -

M5 - 1:5:10
M7.5 - 1:4:8
M10 - 1:3:6
M15 - 1:2:4
M20 - 1:1.5:3
M25 - 1:1:2
M30 - 1:0.75:1.5
M35 - 1:0.5:1
M40 - 1:0.25:0.5

There is no guarantee that such mixes will give the desired strength & will be economical. Hence nominal mix may be used for grades of M20 or lower, if design mix cannot be used for some reason.

Grades of Steel

Steel as reinforcement:

In reinforced concrete, concrete being weak in tension, steel bars are used to carry the tension. Steel bars are also used to carry compression in beams and columns. For any material to be used as reinforcement for concrete, it should possess the following properties.

1) It should possess high tensile strength
2) It should be able to develop a good bond with concrete.
3) It should possess a high modulus of elasticity.
4) It should have the same temp. coefficient of expansion & contract^{n} as concrete to avoid the development of thermal stresses.
5) It should be easily available.

Types of reinforcement

Different types of reinforcement used are

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1> mild steel
2> Medium tensile steel
3> hot rolled deformed bars
4> high yield strength deformed (HYSD) steel
5> hard drawn steel wire fabric.

→ HYSD bars are available as CTD (cold twisted Deformed) bars & TMT (Thermo mechanically treated) bars.

Mild Steel bars (Fe250)

Mild steel bars have been widely used as reinforcement in construct^{n} industry. The yield strength of mild steel is 250N/mm^2. Among all kinds of steel, mild steel is the most ductile. A typical stress-strain curve for mild steel is shown in fig.

* In elastic range, the strain is directly proportional to stress. If the load is increased, the stress increases & proportionally the strain increases. At one particular point, the stress remains const with significant increase in strain. This is known as yielding of steel & the stress is known as yield stress which can be measured by drop of beam method on universal testing m/c using standard tensile test specimen.

* With a further increase of load, the stress increases slowly upto ultimate stress while strain increases rapidly. This is known as strain hardening. Beyond the ultimate load, the deformations are due to the necking & at a reduced load upto failure.

* Mild steel, when used as reinforcement, can yield on over loading. During yielding, extensive cracking in concrete takes place before it fails. This can give sufficient warning before failure. Now-a-days, because of the extensive use of high yield strength deformed bars, the use of mild steel is limited; however, it is sometimes used for secondary reinforcement.

* mild steel can be plain or deformed to increase the capacity in bond. The deformed bar is one which is provided with lugs, ribs or deformations on the surface of the bar to minimize the slippage of the bar in concrete.

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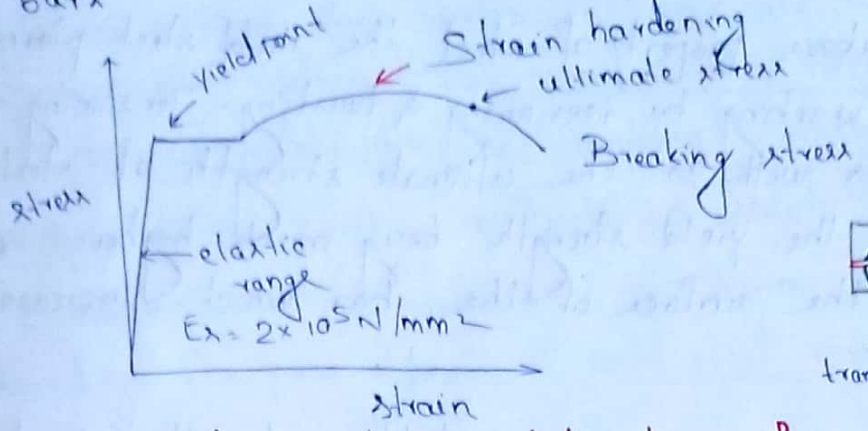
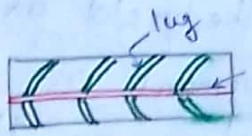
4 High Yield Strength Deformed bars:

HYSD bars

longitudinal rib
transverse rib





Rebar deformat^n
transverse (lug) & parallel (rib)

Stress - strain relationships for mild steel.

(-1) ~~4~~ High Yield Strength Deformed bars.

HYSD bars are available in two varieties.

(a) Cold Twisted Deformed [CTD] bars

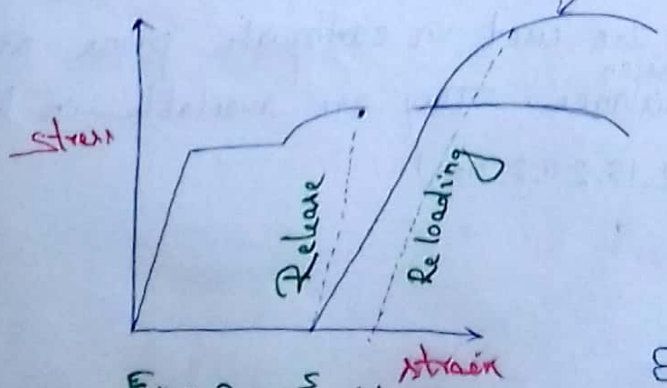
(b) Thermo Mechanically Treated (TMT) bars

(a) CTD bars

upto the late 1960s, only mild steel was used as reinforcement in India. There was a shortage of steel which made it costly. This created a demand to reduce the quantity of steel used.

* The quantity of steel can only be reduced by using the reinforcement of quality marked by a higher strength. Based on experiments & prevailing practice in foreign countries, the production of high strength ribbed bars was started.

* It was noticed that if a mild steel bar is strained beyond yield point by tensioning & twisting & then unloaded, the yield plateau is eliminated & a new stress-strain curve is obtained as shown in fig.



Effect of cold working of mild steel bars.

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* As the high yield strength does not possess the definite efield point, 0.2% proof stress is used as a yield stress.

* Based on the above property of steel, the mild steel plain bars are subjected to cold working by tensioning & twisting. Tensioning raises both the yield as well as the ultimate strength of steel, the improvement in the yield strength being much higher. Twisting produces deformations on the surface of the bar which increases its bond with concrete.

* In India, the CTD bars are available under the trade names of TOR steel, tistrong, Tiscon, GIRP etc. These bars are available in 3 grades, Fe 415, Fe 500, Fe 550. Where Fe denotes Ferrous material & the number, the yield strength of steel in N/mm^2.

Thermo-mechanically treated (TMT) bars

The TMT bars are manufactured by subjecting the hot rolled MS bars to a sophisticated & controlled cooling process which converts the outer surface of the bar into hardened structure.

* The subsequent cooling allow the hot inner core to soften the outer surface through thermal exchange resulting into tempered martensite in the peripherical zone & a fine grains ferrite-pearlite at the central zone.

* This process increases the yield strength of the bar without losing its yielding property. Thus, these bars possess definite yield point. In addition, the bond strength is also increased due to such treatment.

* TMT bars have high strength combined with high elongation. This increases the ductility & weldability of the bars. Due to higher ductility & toughness, these are used in earthquake prone areas. Also these bars have better corrosion resistance. They are available in limited variety of diameters (8, 10, 12, 16, 20, 25 mm)

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Permissible stresses (B.2, Table 21 & 22)

In working stress method, the stresses in materials are not exceeded beyond their permissible values. The permissible stresses are found by using suitable factors of safety to the material strengths; eg, for concrete in compression in bending, a factor of safety equal to 3 is considered on characteristic of concrete & a factor of safety equal to 1.8 is considered on the yield strength of mild steel reinforcement in tension due to bending.

* From table 21, it can be observed that for a given grade of concrete $\sigma_{cc} < \sigma_{cbc}$, ie a greater factor of safety is adopted for direct stress than for a bending stress. When a c/s is subjected to bending stress, the stress induced in it is variable, being maxm at extreme fibre & zero at neutral axis.

* When the maxm stress ~~indu~~ exceeds, the permissible value, the extreme fibre will not fail actually but will transfer the additional force to the inner fibre which has a lower stress.

* When the sectn is subjected to a direct stress, all points of the sectn have uniform stress having no scope for such a transfer of the force. It is for this reason that larger factor of safety is adopted for direct stress than bending stress.

* In case of steel reinforcement of small dia, the stress will be uniform for direct stress as well as for bending stress. Therefore in steel bars, the permissible stresses in bending & direct stresses are the same for lower dia bars upto 20 mm dia. For dia more than 20 mm, permissible tensile stress in steel is usually reduced.

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Assumptions in WSM (B - Clause 1.3) (page-80)

The analysis & design of a reinforced concrete member are based on the following assumptions.

(i) Concrete is assumed to be homogeneous.

(ii) At any c/s plane sect^{n}s before bending remain plane after bending.

(iii) The stress-strain relationship for steel & concrete is a straight line, under working loads.

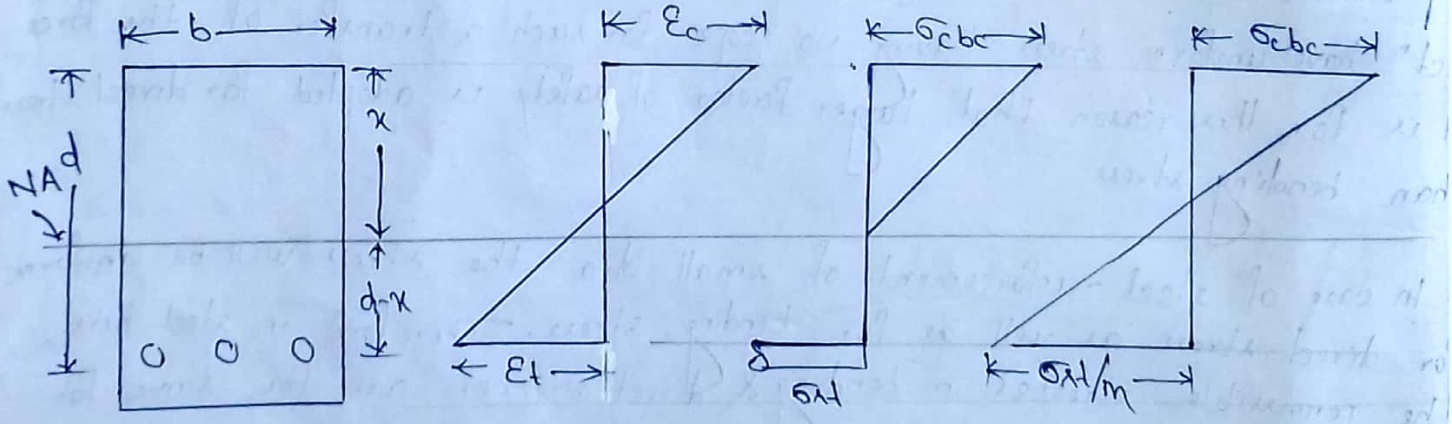
(4) Concrete area on the tension side is assumed to be ineffective.

(5) All tensile stresses are taken up by reinforcement & none by concrete except when specialy permitted

(6) The steel area is assumed to be concentrated at the centroid of the steel.

(7) The modular ratio has the value $\frac{280}{3\sigma_{cbc}}$ where σ_{cbc} is permissible compressive stress due to bending in concrete in N/mm^2.

* From the above assumptions, the equivalent stress diagram is developed as shown in fig.



Section of a beam — Strain diagram — Stress diagram — Equivalent stress diagram

$\sigma_{cbc} = \sigma_{st}/m$ prove

$\frac{\text{Stress in steel}}{\text{strain in steel}} = E_t$ ... (1)

$\frac{\text{stress in surrounding concrete}}{\text{strain " "}} = E_c$ ... (2)

$\therefore \frac{\text{Eqn(1)}}{\text{Eqn(2)}} \Rightarrow \frac{\sigma_{st}}{\sigma_{cbc}} = \frac{E_t}{E_c}$

$\Rightarrow \frac{\sigma_{st}}{\sigma_{cbc}} = m$

$\Rightarrow \sigma_{cbc} = \sigma_{st}/m$

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modular ratio

(page 106 of HJ Saha)

~~This is the ratio betn elastic mo~~

It is defined as the ratio of modulus of elasticity of steel (E_s) to that of concrete (E_c) & is denoted by m.

$$m = \frac{E_s}{E_c}$$

It is stated that the modulus of elasticity of all grades of steel 200kN 2×10^5 N/mm². The short term modulus of elasticity of concrete is given by $E_c = 5000\sqrt{f_{ck}}$. Also the long term modulus (Instantaneous load) of elasticity of concrete was shown to be $E_{ce} = \frac{E_c}{1+\theta}$ where θ is the creep coefficient (Creep & Shrinkage clause) given in 6.2.5 (page 16 of IS456:2000).

* ~~It shou~~ For the design work, long term modulus of elasticity of concrete shall be taken into account for calculation of modular ratio.

→ IS:456 stipulates that for the design with working stress method, the value of modular ratio m shall be taken as $m = \frac{280}{3\sigma_{cbc}}$

where σ_{cbc} = permissible compressive stress due to bending in concrete in N/mm²

* The modular ratio varies with the grade of concrete. No rounding off of the values of 'm' is permitted by Indian Standard.

Ex

Modular ratio values for different grades of concrete are as under

Grade of concrete	Modular ratio
M15	$m = \frac{280}{3 \times 5} = 18.66$
M20	$m = \frac{280}{3 \times 7} = 13.33$

etc

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Analysis of singly reinforced sections

Let b = breadth/width of the rectangular beam

d = effective depth of the beam
(which is defined as the distance from the extreme compression fibre to the centroid of tensile reinforcement)

x = depth of neutral axis which is defined as the distance from the extreme compression fibre.

A_{st} = c/s area of steel in tension

σ_{cbc} = permissible compressive stress in concrete due to bending

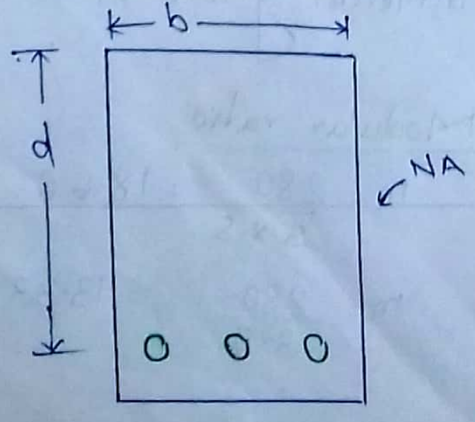
σ_{st} = permissible tensile stress in steel

m = modular ratio

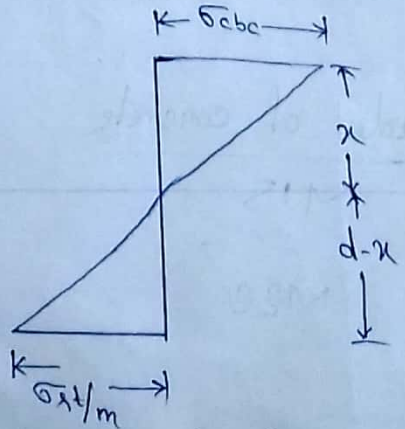
Neutral Axis: (N.A.)

Neutral axis is the axis at which the stresses are zero in the sect^n. The areas above & below it are subjected to compressive & tensile stresses respectively in the case of simply supported beams with vertical loads acting downwards while reverse in the case when the beam is a cantilever. Locating the position of neutral axis is of great importance to the structural Engineer. This helps in knowing the amount of strains both in concrete & steel.

* There are 2 methods for determining the neutral axis of a beam depending upon the data given.



Sect^n of a beam



Equivalent stress diagram

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Method-I (

Given data

(i) Dimensions of the section (b & d)

(ii) Permissible stresses in concrete & steel (σ_{cbc} & σ_{st})

(iii) modular ratio (m)

Referring to fig, from similar triangles in the equivalent concrete stress diagram,

$$\frac{\sigma_{cbc}}{\sigma_{cbc}/m} = \frac{x}{d-x}$$

$$\Rightarrow \sigma_{cbc}(d-x) = x \times \sigma_{st}/m$$

$$\Rightarrow \sigma_{cbc} \times d = x \cdot \frac{\sigma_{st}}{m} + x\,\sigma_{cbc}$$

$$\Rightarrow \sigma_{cbc} \times d = x\left(\sigma_{cbc} + \sigma_{st}/m\right)$$

$$\Rightarrow x = \frac{\sigma_{cbc}}{\left(\sigma_{cbc} + \sigma_{st}/m\right)} \times d$$

$$\Rightarrow x = \left(\frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}\right) \times d \quad \ldots\ldots (1)$$

$$\Rightarrow x = k \times d$$

where k = neutral axis const $= \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$

To find the area of steel A_{st}, equate total compressive force (C) to total tensile force (T)

Total compressive force (C) = area × average compressive stress

$$= b \cdot x \times \left(\frac{\sigma_{cbc} + 0}{2}\right) = \frac{bx\,\sigma_{cbc}}{2}$$

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Total tensile force (T) = area × tensile stress

$= A_{st} \sigma_{st}$

$\therefore \quad b x \cdot \frac{\sigma_{cbc}}{2} = A_{st} \sigma_{st} \quad \ldots\ldots (2)$

After calculating the neutral axis 'x' from eqn (1), the area of steel can be calculated from eqn 2.

The area of tensile steel (A_{st}) is expressed as a % (P_t) of the effective section.

$$P_t = \frac{A_{st} \times 100}{bd}$$

Problem-1

Determine the effective depths for the beam sectn of fig

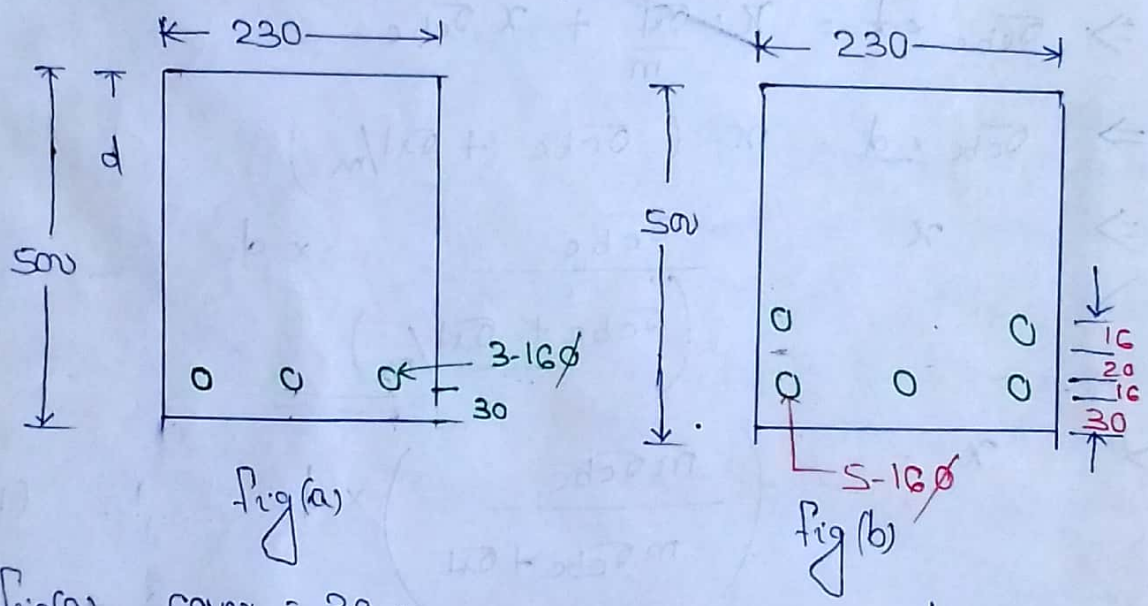

fig (a)

fig (b)

Soln

For fig(a) cover = 30 mm

clause 23

Overall depth (D) = 500mm

effective depth (d) = 500 − 30 (cover) − 8 (upto centroid of reinforcement)

= 462 mm

For fig(b)

In this fig if all the reinforcement bars are provided in a single layer, the concreting will be difficult. Therefore two layer shall be 20mm or more. The centroid of reinforcement is assumed at centre of 2 layers.

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$d = 500 - 30\,(\text{cover}) - 16\,(\text{first layer}) - 10\,(\text{centre of 2 layers})$

$= 500 - 56 = 444\ mm.$

Problem: 2

An RC beam 200mm wide has an effective depth of 350mm. The permissible stresses in concrete & steel are $5\ N/mm^2$ & $140\ N/mm^2$ respectively. Find the depth of neutral axis, area of steel & % of steel (m) = 18.66.

Given

width of beam (b) = 200mm

effective depth of beam (d) = 350mm

Permissible compressive stress in concrete due to bending (σ_{cbc}) $= 5\ N/mm^2$

permissible tensile stress in steel (σ_{st}) $= 140\ N/mm^2$

modular ratio (m) = 18.66

method-I

From similar triangles in the equivalent concrete stress diagram

depth of neutral axis $x = \left(\frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}\right) d$

$= \left(\frac{18.66 \times 5}{18.66 \times 5 + 140}\right) \times 350$

$= 139.97\ mm$

To find the area of steel

Total compressive force = Total tensile force

$\Rightarrow b \times \frac{x\,\sigma_{cbc}}{2} = A_{st}\,\sigma_{st}$

$\Rightarrow 200 \times 139.97 \times \frac{5}{2} = A_{st} \times 140$

$\Rightarrow A_{st} = 499.89\ mm^2$

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Percentage of steel

$$P_t = \frac{A_{st}}{bd} \times 100$$

$$= \frac{499.89}{200 \times 350} \times 100$$

$$= 0.714$$

Method - II

Given data

(i) Dimensions of section (b & d)

(ii) Area of tensile steel (A_{st})

(iii) modular ratio (m)

Referring to fig, we see that the neutral axis is situated at the centre of gravity of a sect^n. The moments of area on either side of it are equal.

∴ Moment of area on compression side = Moment of area on tension side

Moment of area on ~~tension~~ compression side

= Area of compression side × distance of c.g. of compression area from N.A.

$$= b \times x \times x/2$$

Moment of area on tension side

= equivalent area of concrete × distance of c.g. of tensile steel from N.A

$$= m A_{st} (d - x)$$

Therefore $b \cdot x \cdot x/2 = m A_{st} (d - x)$

Note that $m A_{st}$ = Equivalent [redacted] of concrete.

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Problem-3

Find the position of the neutral axis of a reinforced concrete beam 150mm wide & 400mm deep (effective). Area of tensile steel is 804 mm^2. m = 18.66

Given data

width of beam (b) = 150mm
effective depth of beam = 400mm
Area of tensile steel (Ast) = 804 mm^2
modular ratio (m) = 18.66

Taking moments of area of compression & tension sides about N.A.

$$b \cdot x \cdot x/2 = m\, Ast\, (d-x)$$

$$\Rightarrow 150x^2/2 = 18.66 \times 804\,(400-x)$$

$$\Rightarrow 75x^2 = 6001056 - 15002.64x$$

$$\Rightarrow 75x^2 + 15002.64x - 6001056 = 0$$

$$\Rightarrow x = 200.0mm$$

Concept of balanced, Under-reinforced, Over-reinforced sections

Depending on modes of failure of a beam, three types of design are possible, viz, balanced, underreinforced & over-reinforced design. These are now briefly discussed.

(1) Balanced ~~design~~ sectn:

Reinforced concrete sectns are designed on the assumptn that the maxm stresses in concrete & steel reach their permissible stresses at the same time. Both the materials are stressed to their fullest allowable stress at one & the same time. As such the section is called 'balanced' or 'critical' or 'economic'.

* The neutral axis of this sectn is called critical neutral axis & may be denoted by x_c. Critical n.A is calculated as follows

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$$\frac{\sigma_{cbc}}{\sigma_{st}/m} = \frac{x_c}{d-x_c}$$

$$x_c = \left(\frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}}\right)d$$

The failure in this case is called tension failure & the design is referred to as under-reinforced. Note that steel is a ductile material & yields before failure. Thus the under reinforced sect^n suffers large deflections and concrete in tension zone cracks extensively before failure. The failure in this case is a ductile failure.

Under-reinforced Sect^n

If less steel than that required for balanced section is used, the sect^n is under-reinforced. In such a condition, the steel is stressed to its max^m permissible stress. Under such conditions, the beam will fail initially due to overstress in the steel.

* The neutral axis is calculated by taking moments of area of compression & tension sides about neutral axis. Thus

$$b \cdot x \cdot x/2 = m A_{st}(d-x)$$

* This actual N.A is above the critical N.A ($x < x_c$) as shown in fig. The neutral axis moves upwards because the concrete area is greather than the area of steel, the latter being less than what it should be for perfect balance.

Over-reinforced sect^n

If more steel than the steel required for balanced sect^n is used, the sect^n is over-reinforced. The steel is not fully stressed to its max^m permissible tensile stress, while the compressive stress in concrete at extreme fibre reaches its max^m permissible value. In this case, the beam will fail initially due to overstress in the concrete.

* N.A is calculated by taking moments of area of compression & tension side about the N.A.

$$b \cdot x \cdot x/2 = m A_{st}(d-x)$$

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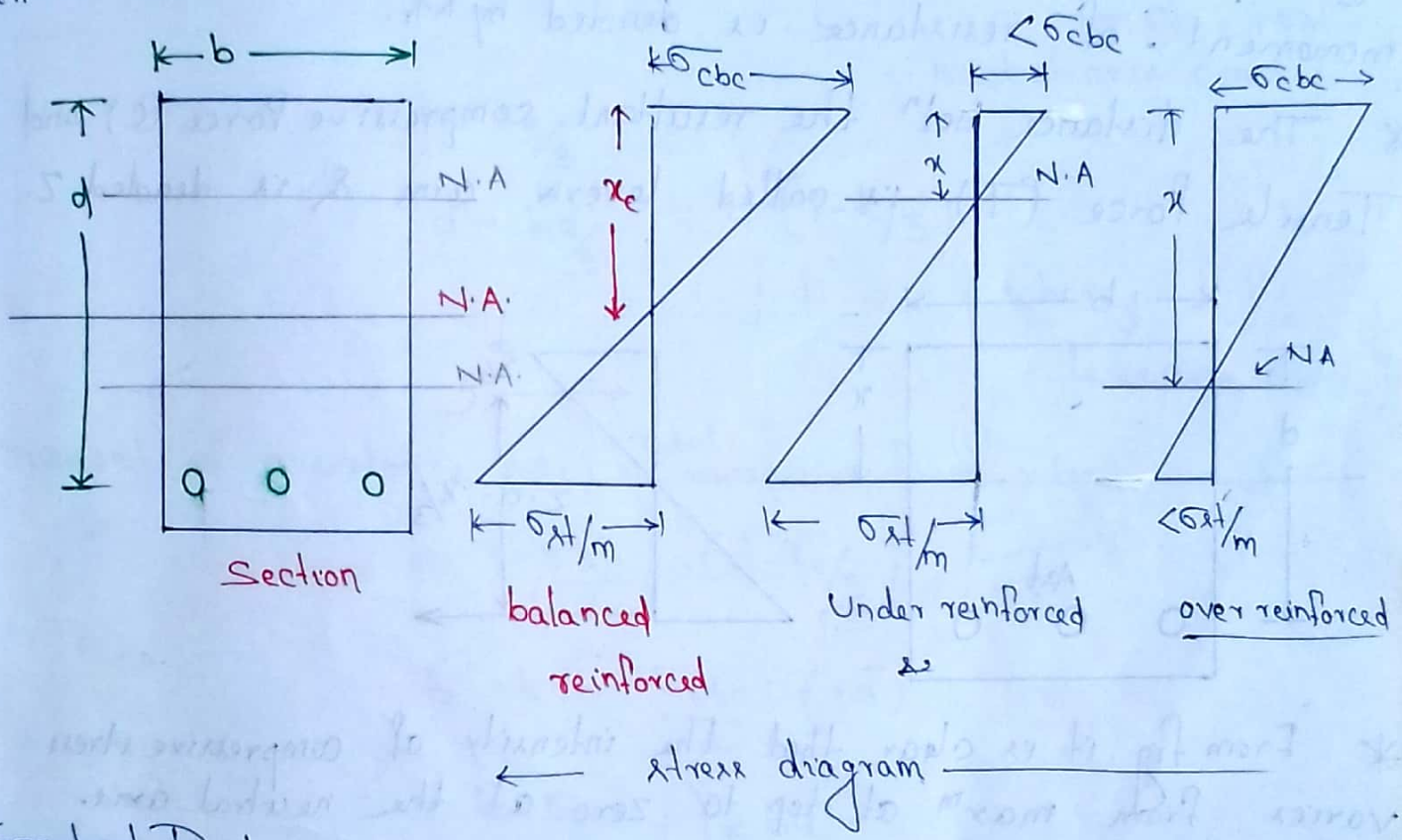
* This actual N.A. is below the critical N.A. The N.A. moves ($x > x_c$) downwards because the area of steel is more than that of balanced sect^n.

* The failure in this case is called compression failure and the design is referred to as over reinforced. Note that concrete is a brittle material & therefore the failure will be brittle & sudden.



← stress diagram ——————

Important Points

A beam, if under-reinforced, gives notice before failure as the steel yields & concrete in tension zone shows cracks. If it is over-reinforced concrete fails first.

* Prior to failure, concrete does not gives notice as it does not yield. Thus, over-reinforced beam may collapse without giving a notice when overloaded. Therefore normally balanced or under-reinforced design is preferred. The mode of failure in this case will be ductile.

* In fact, we should design the structure as ductile as possible so that in case of failure due to overloads, it gives notice before failure and prevention can be taken. IS 456 stipulates all provision to see that the designd structure is ductile as per the importance of the structure.

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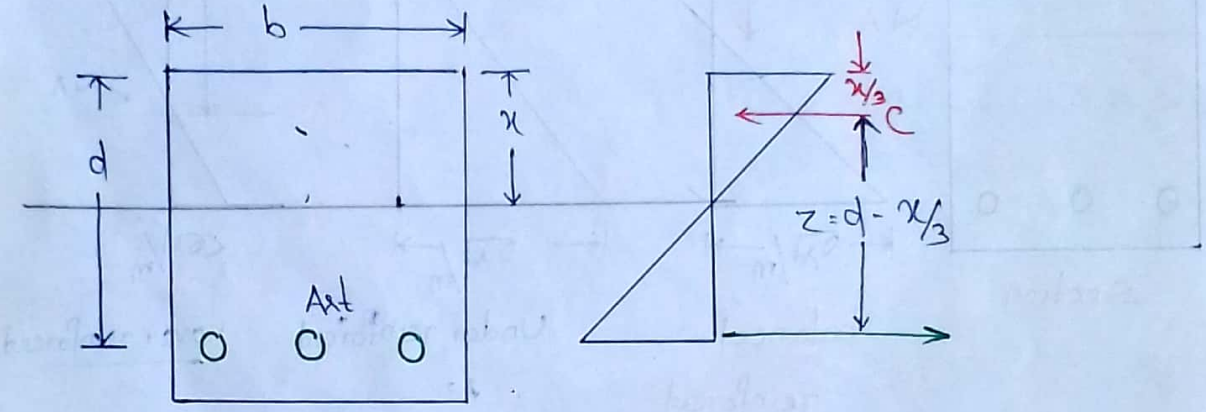
Moment of resistance.

The moment of resistance of the concrete ~~beam~~ section is the moment of couple formed by the total tensile force (T) in the steel acting at the c.g. of reinforcement & the total compressive force (C) in the concrete acting at the c.g. of the compressive stress diagram as shown in fig. The momoment of resistance is denoted by M_r.

* The distance betn the resultant compressive force (C) and Tensile force (T) is called lever arm & is denoted z.



* From fig, it is clear that the intensity of compressive stress varies from maxm at top to zero at the neutral axis. There force c.g. of the compressive force is at a distance $x/3$ from the top edge sect^n

$$\therefore z = d - x/3$$

Moment of resistance

M_r = resultant / Total compressive force (C) × lever arm (Z)

$$= b \cdot x \cdot \frac{\sigma_{cbc}}{2} \left(d - x/3\right)$$

or

M_r = resultant / Total tensile force (T) × lever arm (Z)

$$= A_{st} \cdot \sigma_{st} \cdot \left(d - x/3\right)$$

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~~moment~~

For balanced sect^n

Critical / balanced / economic depth of of neutral axis

$$x_c = \left(\frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}}\right)d$$

$$= kd$$

[where $k = \frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}}$ = neutral axis const]

lever arm $z = d - x_c/3$

$= d - kd/3 = d(1-k/3)$

$= j \cdot d$

[where $j = 1 - k/3$ = lever arm const]

moment of resistance (M_r) = Total compressive force (C) × lever arm (z)

$= b \cdot x_c \cdot \frac{\sigma_{bc}}{2} \cdot (d - x_c/3)$ (1)

$= b \cdot kd \cdot \frac{\sigma_{cbc}}{2} \cdot (jd)$

$= \left(\frac{k \cdot \sigma_{cbc} \cdot j}{2}\right) \times bd^2$

$= Q\, bd^2$

[where $Q = \frac{k\,\sigma_{cbc} \cdot j}{2}$ = moment of resistance const]

or moment of resistance (M_r)

= Total tensile force × lever arm

$= A_{st}\,\sigma_{st} \dfrac{(d - x_c/3)}{(jd)}$

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for under reinforced sect^n

M_r = Total tensile force (T) × lever arm

$= A_{st} \sigma_{st} (d - x/3)$

for over-reinforced sect^n

M_r = Total compressive force (C) × lever arm

$= b \cdot x \cdot \frac{\sigma_{cbc}}{2} (d - x/3)$

Problem: 4

For a rectangular beam of size 250mm wide × 520 mm effective depth, find out the balanced depth of neutral axis, balanced lever arm, balanced moment of resistance & balanced steel area. The materials are M20 grade concrete & HYSD reinforcement of grade Fe415. Also find out the design const.

Soln

given data

bridth of the beam (b) = 250mm

effective depth " (d) = 520 mm

for, M20 grade concrete,

$\sigma_{cbc} = 7 N/mm^2$ (Table 21, IS:456-2007)

for HYSD reinforcement of grade Fe415

$\sigma_{st} = 230 N/mm^2$ (Assuming bar dia $\leq$ 20mm)

(i) balanced depth of neutral axis (x_c) = kd

where k = neutral axis const $= \left(\frac{m \sigma_{cbc}}{m \sigma_{cbc} + \sigma_{st}} \right) = \frac{\frac{280}{3\sigma_{cbc}} \times \sigma_{cbc}}{\frac{280}{3\sigma_{cbc}} \times \sigma_{cbc} + \sigma_{st}}$

$= \left(\frac{\frac{280}{3\times 7} \times 7}{\frac{280}{3\times 7} \times 7 + 230} \right)$

$= 0.29$

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$\therefore x_c = 0.29 \times 520 = 150.8\ mm$

(ii) balanced lever arm $(z) = d - \frac{x_c}{3}$

$= d - \frac{kd}{3}$

$= d(1 - k/3)$

$= jd$

lever arm const $(j) = 1 - k/3 = 1 - \frac{0.29}{3} = 0.90$

$\therefore z = 0.90 \times 520 = 468mm$

(iii) balanced moment of resistance $M_r = Q bd^2$

where Q = moment of resistance const

$= \frac{k \sigma_{cbc} \times j}{2}$

$= \frac{0.29 \times 7 \times 0.90}{2}$

$= 0.91$

$M_r = 0.91 \times 250 \times 520^2 = 61.52\ KNm$

(iv) balanced steel area

moment of resistance for balanced sect^n (M_r)

$M_r = A_{st} \sigma_{st} (d - x_c/3)$

$\Rightarrow 61.52 \times 10^6 = A_{st} \sigma_{st} \times 230 \times 468$

$\Rightarrow A_{st} = \frac{61.52 \times 10^6}{230 \times 468}$

$= 571.53\ mm^2$

Percentage of steel $(P_t) = \frac{100 A_{st}}{bd} = \frac{100 \times 571.53}{250 \times 520}$

$\frac{A_{st}}{bd} \times 100$

$= 0.44$

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Types of Problem

The following types of problems can be encountered in the design of reinforced concrete beam

(1) To find the moment of resistance of a given sect^n & load carrying capacity of a beam
(2) To find the actual stress in steel & concrete
(3) To find the area of tensile reinforcement.
(4) To Design ~~of singly reinforce~~ the section

To find the moment of resistance of a given sect^n

Generally the following data are given

b = breadth of a beam

d = depth of a beam

A_{st} = area of tensile steel

σ_{cbc} = permissible stress in concrete

σ_{st} = " " in steel

m = modular ratio

To solve the problem, the following procedure may be used

(a) Find x_c by the formula

$$\therefore x_c = \left(\frac{m \sigma_{cbc}}{m \sigma_{cbc} + \sigma_{st}} \right) \times d$$

(b) Find x by the formula

$$b \cdot x \cdot x/2 = m A_{st} (d - x)$$

(c) Find whether the section is under reinforced or over reinforced

If $x < x_c$, then the sect^n is under reinforced.

If $x > x_c$ " " " over "

If $x = x_c$ " " " balanced sect^n

(d) Find M_r

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(a) If the sectn is under-reinforced

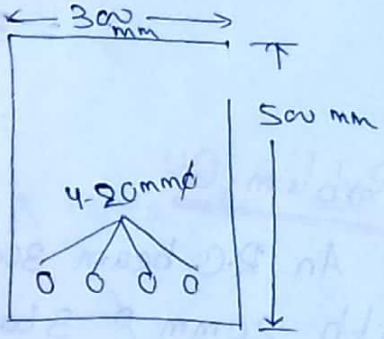
$$M_r = T \times Z$$
$$= A_{st}\, \sigma_{st} \left(d - x/3\right)$$

If the sectn is over-reinforced

$$M_r = C \times Z$$
$$= b \cdot x \cdot \frac{\sigma_{cbc}}{2} \left(d - x/3\right)$$

Problem-5

A singly reinforced rectangular beam is 300 mm wide & 500 mm deep to the centre of tensile steel. It is reinforced with 4 bars of 20mm dia on tension side. If the allowable stresses in concrete & steel are $5 N/mm^2$ & $140 N/mm^2$ respectively. Find the moment of resistance of the section. $m = 18.66$



Given data

$b = 300 mm$

$d = 500 mm$

$A_{st} = 4 \times \pi/4 \times 20^2 = 1256.63 mm^2$

$\sigma_{cbc} = 5 N/mm^2$

$\sigma_{st} = 140 N/mm^2$

$m = 18.66$

To find the critical depth of N.A

$$x_c = \left(\frac{m \sigma_{cbc}}{m \sigma_{cbc} + \sigma_{st}}\right) d$$

$$= \left(\frac{18.66 \times 5}{18.66 \times 5 + 140}\right) \times 500$$

$$= 199.95 mm$$

To find the actual depth of neutral axis

$$\frac{b x^2}{2} = m A_{st} (d - x)$$

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$$\Rightarrow \frac{300x^2}{2} = 18.66 \times 1256.63 (500 - x)$$

$$\Rightarrow 150x^2 = 23448.71 (500 - x)$$

$$\Rightarrow 150x^2 = 11724357.9 - 23448.71x$$

$$\Rightarrow 150x^2 + 23448.71x - 11724357.9 = 0$$

$$\Rightarrow x = 212.13 mm$$

So $x > x_c$, the beam is overreinforced & hence concrete will reach its maxm permissible value first

$$M_r = C \times Z$$

$$= b x \frac{\sigma_{cbc}}{2} (d - x/3)$$

$$= 300 \times 212.3 \times 5/2 \left(500 - \frac{212.B}{3}\right)$$

$$= 68.29 kN.m$$

Problem-6

An R.C. beam 300mm wide & 700mm deep overall, is reinforced with 20mm ϕ 3 bars. The centre of bars are 50mm from the underside of the beam. The maxm stresses are not to exceed 7 N/mm² for concrete and 190 N/mm² for steel. Find the moment of resistance & the safety distributed load, the beam can carry. The span is 6m & weight of concrete is 25 kN/m³.

Given data

$b = 300 mm$

$D = 700 mm$

$d = 700 - 50 = 650 mm$

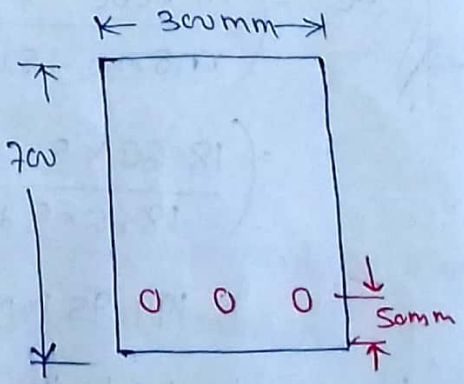
$\sigma_{cbc} = 7 N/mm^2$

$\sigma_{st} = 190 N/mm^2$

$m = 13.33$

span $(l) = 6m$

$A_{st} = 3 \times \pi/4 \times 20^2 = 942.47 mm^2$



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To find the critical depth of neutral axis

$$x_c = \left(\frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}\right) d$$

$$= \left(\frac{13.33 \times 7}{13.33 \times 7 + 190}\right) \times 650$$

$$= 214.08 \text{ mm}$$

To find the actual depth of neutral axis

$$\frac{bx^2}{2} = m A_{st} (d - x)$$

$$\Rightarrow \frac{300 x^2}{2} = 13.33 \times 942.47 \ (650 - x)$$

$$\Rightarrow 150x^2 = 12563.12 \ (650 - x)$$

$$\Rightarrow 150x^2 + 12563.12 x - 8166031.31 = 0$$

$$\Rightarrow x = 195.17 \text{ mm}$$

Here $x < x_c$. So the beam is under-reinforced & hence the steel will reach its maxm permissible value first.

$$M_r = T \times Z$$

$$= A_{st} \sigma_{st} (d - x/3)$$

$$= 942.47 \times 190 \times \left(650 - \frac{195.17}{3}\right)$$

$$= 104.74 \text{ kN-m}$$

Let w kN be the load on the beam per metre

$$\text{B.M.} = \frac{wl^2}{8} = \frac{w \times 6^2}{8} = 4.5w \text{ kN-m}$$

Equating M_r to B.M. we get

$$4.5w = 104.74$$

$$\Rightarrow w = 23.27 \text{ kN/m}$$

Net weight / live load the beam can carry will be

$$23.27 - 5.25 = 18.02 \text{ kN/m}$$

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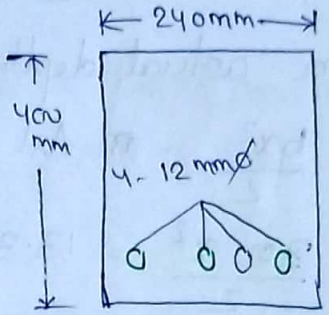
Problem-7

A reinforced concrete cantilever beam 2 m long of 240 x 400 mm is designed to support a concentrated load of 3000 N at the free end in addition to a UDL. The beam is reinforced with 4 bars 12 mm dia on the tension side. Determine the permissible UDL the beam can be support. Take $\bar{\sigma}_{cbc} = 5 N/mm^2$ & $\bar{\sigma}_{st} = 140 N/mm^2$.

Sol:



Given data

$b = 240mm$

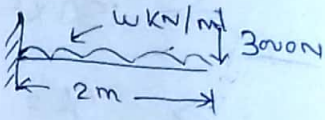
$D = 400mm$

$\bar{\sigma}_{cbc} = 5 N/mm^2$

$\bar{\sigma}_{st} = 140 N/mm^2$

$d = 400 - 25 - 12/2 = 369mm$

$A_{st} = 4 \times \pi/4 \times 12^2 = 452.38 mm^2$



modular ratio $(m) = \frac{280}{3\bar{\sigma}_{cbc}} = \frac{280}{3 \times 5} = 18.66$

To find the critical depth of neutral axis (x_c)

$$x_c = \left(\frac{m\bar{\sigma}_{cbc}}{m\bar{\sigma}_{cbc} + \bar{\sigma}_{st}}\right) d$$

$$= \left(\frac{18.66 \times 5}{18.66 \times 5 + 140}\right) \times 369$$

$$= 147.56 mm$$

To find the actual depth of neutral axis

$$\frac{bx^2}{2} = m A_{st} (d - x)$$

$$\Rightarrow \frac{240 x^2}{2} = 18.66 \times 452.38 (369 - x)$$

$$\Rightarrow 120 x^2 = 8441.41 (369 - x)$$

$$\Rightarrow 120 x^2 + 8441.41 x - 3114880.58 = 0 \Rightarrow x = 129.73 mm$$

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Here $x_{lim} > x$, so the beam is under-reinforced sectn & steel reaches its maxm permissible value first.

$$M_r = T \times Z$$

$$= A_{st} \sigma_{st} (d - x/3)$$

$$= 452.38 \times 140 \times \left(369 - \frac{129.73}{3}\right)$$

$$= 20.63 \text{ KN-m}$$

Bending moment of the sectn

$$B.M. = \frac{wl^2}{2} \times wt$$

$$= \frac{w \times 2^2}{2} + 3 \times 2$$

Equating B.M to M_r we get

$$\frac{w \times 2^2}{2} + 3 \times 2 = 20.63$$

$$\Rightarrow 2w = 20.63 - 6$$

$$\Rightarrow w = 7.315 \text{ KN-m}$$

T

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To find the actual stresses in concrete & steel

Generally the following data are given

b = breadth of a rectn

d = effective depth of a rectn

Ast = area of steel

m = modular ratio

M = bending moment

To solve a problem, the following procedure may be used.

(a) Find x by the formula

$$\frac{bx^2}{2} = m\,A_{st}(d-x)$$

(b) Find z from

$$z = d - x/3$$

(c) Equate M_r to M

$$M_r = T \times z$$

$$\Rightarrow BM = A_{st}\,\sigma_{st}\left(d - x/3\right)$$

The stress σ_{st} can be worked since M, Ast & z are known

(d) Find the stress in concrete from

$$\frac{\sigma_{cbc}}{\sigma_{st}/m} = \frac{x}{d-x}$$

Problem.8

A rectn of RC beam 300×700 mm is reinforced with 4 no's of 25mm ϕ bars placed 30mm from the bottom of the beam. The beam is subjected to a bending moment of 130 kNm. Find the stresses set up concrete & steel. If m = 18.66

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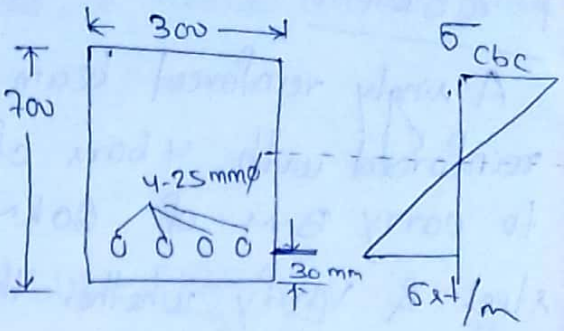
Sol^n



$b = 300\,mm$

$d = 700 - 30 = 670\,mm$

$A_{st} = 4 \times \pi/4 \times 25^2 = 1963.49\ mm^2$

$B.M = 130\,kN.m = 130 \times 10^6\ N.mm$

$m = 18.66$

To find the actual depth of neutral axis

$$\frac{bx^2}{2} = m A_{st} (d-x)$$

$$\Rightarrow \frac{300x^2}{2} = 18.66 \times 1963.49 (670-x)$$

$$\Rightarrow 150x^2 = 36638.72 (670-x)$$

$$\Rightarrow 150x^2 + 36638.72x - 24547944.68 = 0$$

$$\Rightarrow x = 300.44\ mm$$

To find $z = d - x/3 = 670 - \frac{300.44}{3} = 569.85\ mm$

To find M_r,

$$M_r = T \times Z$$
$$= A_{st}\, \sigma_{st}\, (d - x/3)$$
$$= 1963.49 \times \sigma_{st} \times 569.85$$
$$= 1118901.32\, \sigma_{st}$$

Equating M_r to B.M we get

$$130 \times 10^6 = 1118901.32\, \sigma_{st}$$

$$\Rightarrow \sigma_{st} = 116.18\ N/mm^2$$

To find σ_{cbc} from the following eq^n

$$\frac{\sigma_{cbc}}{\sigma_{st}/m} = \frac{x}{d-x} \Rightarrow \frac{\sigma_{cbc}}{116.18/18.66} = \frac{300.44}{670 - 300.44}$$

$$\Rightarrow \sigma_{cbc} = 5.06\ N/mm^2$$

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Problem-9

A singly reinforced beam is 300mm wide & 550mm deep is reinforced with 4 bars of 20mm dia in tension side. It is designed to carry B.M of 60kNm. Calculate the actual stress in concrete & steel & verify whether the section safe or not. Use concrete M15 & mild steel grade.

Soln

$b = 300mm$

$D = 550mm$

$d = 550 - 25 - \frac{20}{2} = 515mm$

$A_{st} = 4 \times \frac{\pi}{4} \times 20^2 = 1256.63\ mm^2$

$B.M. = 60kNm$

$\sigma_{cbc} = 5N/mm^2$

$\sigma_{st} = 140\ N/mm^2$

$$\text{modular ratio} = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times 5} = 18.66$$

To find the critical depth of neutral axis

$$x_c = \left(\frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}\right) d$$

$$= \left(\frac{18.66 \times 5}{18.66 \times 5 + 140}\right) \times 515$$

$$= 205.95\ mm$$

To find the actual depth of neutral axis

$$\frac{bx^2}{2} = m A_{st} (d - x)$$

$$\Rightarrow \frac{300x^2}{2} = 18.66 \times 1256.63 (515 - x)$$

$$\Rightarrow 150x^2 = 23448.71 (515 - x)$$

$$\Rightarrow 150x^2 + 23448.71x - 12076088.64 = 0 \Rightarrow x = 216.14\ mm$$

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Here $x > x_c$, so the beam is over reinforced & hence concrete reach its maxm permissible value first.

$$M_r = b x \frac{\sigma_{cbc}'}{2} \left(d - x/3\right)$$

$$\Rightarrow 60 \times 10^6 = 300 \times 216.14 \times \frac{\sigma_{cbc}'}{2} \left(515 - \frac{216.3}{3}\right)$$

$$\Rightarrow \sigma_{cbc}' = 4.17 \text{ N/mm}^2 < 5 \text{ N/mm}^2$$

so the sect^n is safe.

To find σ_{st}

$$\frac{\sigma_{cbc}'}{\sigma_{st}'/m} = \frac{x}{d-x}$$

$$\Rightarrow \frac{4.17}{\sigma_{st}'/18.66} = \frac{216.14}{515 - 216.14}$$

$$\Rightarrow \frac{\sigma_{st}'}{18.66} = 5.76$$

$$\Rightarrow \sigma_{st}' = 107.59 \text{ N/mm}^2$$

Hence the sect^n is safe.

Problem: 10

A beam 350 × 650 mm overall is provided with tensile reinforcement consisting of 4 no's of 22 mm ϕ bars. Calculate the stresses developed in steel & concrete, if the beam carries a uniformly distributed load of 20kN/m (inclusive of self weight). Take lef = 7m & m = 13.33 & clear cover = 25mm.

Soln

Given data $b = 350$ mm

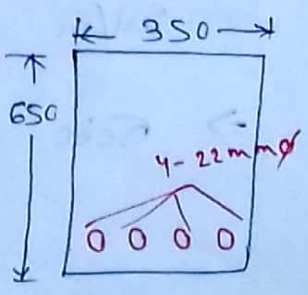
$D = 650$ mm

$d = 650 - 25 - 11 = 614$ mm

$m = 13.33$

lef = 7m

$A_{st} = 4 \times \pi/4 \times 22^2 = 1520.53 \text{ mm}^2$



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To find the actual depth of neutral axis

$$\frac{bx^2}{2} = m A_{st} (d-x)$$

$$\Rightarrow \frac{350 x^2}{2} = 13.33 \times 1520.53 (614-x)$$

$$\Rightarrow 175x^2 = 20268.66(614-x)$$

$$\Rightarrow 175x^2 + 20268.66x - 12444957.24 = 0$$

$$\Rightarrow x = 214.98 mm$$

To find lever arm

$$Z = d - x/3 = 614 - \frac{214.98}{3} = 542.34 mm$$

To find moment of resistance

$$M_r = A_{st} \sigma_{st} (d - x/3)$$

$$= 1520.53 \times \sigma_{st} \times 542.34$$

bending moment (M) $= \frac{wl^2}{8} = \frac{20 \times 7^2}{8} = 122.5 kN.m$

Equating M_r to B.M. we get

$$122.5 \times 10^6 = 1520.53 \times \sigma_{st} \times 542.34$$

$$\Rightarrow \sigma_{st} = 148.595 N/mm^2$$

To find σ_{cbc}

$$\frac{\sigma_{cbc}}{\sigma_{st}/m} = \frac{x}{d-x}$$

$$\Rightarrow \sigma_{cbc} = \frac{x}{d-x} \times \sigma_{st}/m$$

$$= \frac{214.98}{614 - 214.98} \times \frac{148.55}{13.33}$$

$$= 6.00 N/mm^2$$

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To find the area of tensile reinforcement

Generally, the following data are given

b = breadth of a beam

d = depth of a beam

σ_{cbc} = permissible stress in concrete

σ_{st} = " " in steel

m = modular ratio

B.M = bending moment

To solve the problem, the following procedure may be followed

1> Determine k, j, Q for the given stress

2> Find the critical moment of resistance $M_r = Q b d^2$ from the dimensions of the beam

3> Compare the bending moment to the critical moment of resistance

a> If $BM < M_r$ the sec^n is under reinforced

$$M_r = A_{st}\, \sigma_{st}\, (d - x/3)$$

To find A_{st} in term of x take moment of areas about NA

$$\frac{bx^2}{2} = m A_{st} (d - x)$$

$$\Rightarrow A_{st} = \frac{bx^2}{2m(d-x)}$$

so $$M = \frac{b x^2 \sigma_{st} (d - x/3)}{2m(d-x)} = B.M \text{ to be resisted}$$

solve for x & then A_{st} can be calculated.

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(b) If $BM > M_r$ the sect^n as over reinforced sect^n

$$M_r = \frac{b x \sigma_{cbc}}{2}\left(d - x/3\right) = B.M \text{ to be resisted}$$

Determine x. Then Ast can be obtained by taking moments of areas about N.A axis.

$$Ast = \frac{bx^2}{2m(d-x)}$$

Problem: 11

A singly reinforced rectangular beam 300mm wide & 600 mm deep has an effective span of 6m. Find necessary tension reinforcement to enable the beam to carry a live load of 8kN/m in addition to its own weight. Use M15 grade of concrete & steel of grade Fe250. modular ratio = 18.66 & density of concrete = 25kN/m^3

Soln

Given data $b = 300mm$

$D = 600mm$

$d = 600 - 35 = 565mm$

$\sigma_{cbc} = 5 N/mm^2$

$\sigma_{st} = 140 N/mm^2$

live load of the beam = 8kN/m

dead load / self weight of the beam = $0.3 \times 0.6 \times 25 = 4.5 kN/m$

So the total load on the beam (w)

$= 8 + 4.5 = 12.5 kN/m$

The maxm bending moment is at centre of the beam

$$B.M. = \frac{wl^2}{8} = \frac{12.5 \times 6^2}{8} = 56.25 kN\text{-}m$$

neutral axis const (k) $= \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$

$$= \frac{18.66 \times 5}{18.66 \times 5 + 140} = 0.39$$

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lever arm const (j)

$$= 1 - k/3$$

$$= 1 - \frac{0.39}{3} = 0.87$$

moment of resistance const (Q)

$$= \frac{k\, \sigma_{cbc}\, j}{2}$$

$$= \frac{0.39 \times 5 \times 0.87}{2}$$

$$= 0.85 N/mm^2$$

Critical moment of resistance

$$M_r = Q b d^2$$

$$= 0.85 \times 300 \times 565^2$$

$$= 81.4 KN\text{-}m$$

Here $M_r > B.M.$ Hence the sect^n is under reinforced & steel will fail first.

$$M_r = A_{st}\, \sigma_{st} (d - x/3)$$

To find A_{st} in terms of x take moment of areas about N.A

$$A_{st} = \frac{b x^2}{2m(d-x)}$$

$$\therefore M_r = \frac{b x^2}{2m(d-x)}\, \sigma_{st} (d - x/3)$$

$$\Rightarrow 56.25 \times 10^6 = \frac{300\, x^2}{2 \times 18.66 (565 - x)} \times 140 \times (565 - x/3)$$

$$\Rightarrow 49982.14 = \frac{x^2 (565 - x/3)}{565 - x}$$

by solving trial & error method we get

$$x = 192.75 mm$$

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So $A_{st} = \frac{bx^2}{2m(d-x)}$

$= \frac{300 \times 192.7^2}{2 \times 18.66 \times (565 - 192.7)}$

$= 801.77 mm^2$

Problem-12

A singly reinforced beam 250x 400mm deep to the centre of steel as a span of 5m. It carries a total UDL of 12500 N/m including of its weight. The stress in concrete & steel shall not exceed $5 N/mm^2$ & $230 N/mm^2$

Soln

modular ratio (m) $= \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 5} = 18.66$

The total load the beam can taken $= 12500 N/m = 12.5 kN/m$

The max^m bending moment is at centre of beam

$BM = \frac{wl^2}{8} = \frac{12.5 \times 5^2}{8} = 39.06 kN.m$

neutral axis const (k)

$= \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$

$= \frac{18.66 \times 5}{18.66 \times 5 + 230}$

$= 0.29$

Lever arm const (j) $= 1 - k/3$

$= 1 - \frac{0.29}{3}$

$= 0.90$

moment of resistance const (Q) $= \frac{k\sigma_{cbc} j}{2}$

$= \frac{0.29 \times 5 \times 0.90}{2} = 0.65 N/mm^2$

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Critical moment of resistance

$$M_r = Q b d^2$$

$$= 0.65 \times 250 \times 400^2$$

$$= 26 \text{ kNm}$$

Here BM > M_r. Hence the sect^n is over reinforced & concrete reaches its max^m permissible value first

$$M = b n \frac{\sigma_{cbc}}{2} \left(d - x/3\right)$$

$$\Rightarrow 39.06 \times 10^6 = 250 \times x \times 5/2 \left(400 - x/3\right)$$

$$\Rightarrow 62496 = 400x - x^2/3$$

$$\Rightarrow x^2 - 1200x + 187488 = 0$$

$$\Rightarrow x = 184.85 \text{ mm}$$

Then Ast can be calculated by taking moments about NA

$$A_{st} = \frac{bx^2}{2m(d-x)}$$

$$= \frac{250 \times 184.85^2}{2 \times 18.66 \times (400 - 184.65)}$$

$$= 1060.60 \text{ mm}^2$$

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Design of singly reinforced beam

Generally the following data are given

σ_{cbc} = permissible stress in concrete

σ_{st} = " " " steel

m = modular ratio

BM = bending moment

To solve the problem the following procedure may be followed

1) Determine the k, j, Q for the given stress

2) Find the breadth of the beam by the following relation ship

$$\text{breadth of beam}(b) = \frac{\text{effective span}}{30} + 80\text{mm}$$

3) Find the depth of beam

$$d = 2b \text{ or } 3b$$

4) Find the critical moment of resistance

$$M_r = Qbd^2$$

5) Find the sect^n is under reinforced or over reinforced

6) Then the area of steel can be calculated by the before procedure.

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Problem: 13

A singly reinforced RCC beam has a effective span of 5m. It carries a total UDL of 12500 N/m including its self weight. The stresses in concrete & steel shall not exceed 5 N/mm² & 230 N/mm². design the beam.

Sol'n:

Given data

effective span (l) = 5m

$w = 12500\ N/m = 12.5\ KN/m$

$\sigma_{cbc} = 5\ N/mm^2$

$\sigma_{st} = 230\ N/mm^2$

modular ratio (m) $= \frac{280}{3\sigma_{cbc}} = \frac{280}{3 \times 5} = 18.66$

For the beam maxm B.M at centre

$B.M. = \frac{wl^2}{8} = \frac{12.5 \times 5^2}{8} = 39.06\ KN.m$

neutral axis const (k) $= \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$

$= \frac{18.66 \times 5}{18.66 \times 5 + 230} = 0.29$

Lever arm const (j) $= 1 - k/3 = 1 - \frac{0.29}{3} = 0.90$

moment of resistance const (Q) $= \frac{K \cdot \sigma_{cbc} j}{2}$

$= \frac{0.29 \times 5 \times 0.90}{2}$

$= 0.65$

breadth of the beam b $= \frac{\text{effective span}}{30} + 80\ mm$

$= \left(\frac{5000}{30} + 80\right)\ mm = 246.66\ mm \simeq 250\ mm$

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So effective depth of the beam (d)

$$d = 2b$$
$$= 2 \times 250 = 500 \text{ mm}$$

The critical moment of resistance

$$M_r = Q b d^2$$
$$= 0.65 \times 250 \times 500^2$$
$$= 40.62 \text{ KN.m}$$

Here $M_r > BM$. So the sect^n is underreinforced & hence steel reaches its max^m permissible value first

$$M = A_{st} \sigma_{st} (d - x/3)$$

To find A_{st} in terms of 'x', take moment of areas about N.A.

$$\frac{bx^2}{2} = m A_{st} (d - x)$$

$$\Rightarrow A_{st} = \frac{bx^2}{2m(d-x)}$$

So $$M = \frac{b n^2 . \sigma_{st}}{2m(d-x)} (d - x/3)$$

$$\Rightarrow 39.06 \times 10^6 = \frac{250 \times x^2 \times 230 (500 - x/3)}{2 \times 18.66 \times (500 - x)}$$

$$\Rightarrow 25351.63 = \frac{x^2}{500 - x} (500 - x/3)$$

by solving trial & error method we get

$$x = 141.7 \text{ mm}$$

$$A_{st} = \frac{bx^2}{2m(d-x)} = \frac{250 \times 141.7^2}{2 \times 18.66 \times (500 - 141.7)}$$
$$= 375.39 \text{ mm}^2$$

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Advantages & Disadvantages of WSM (Working Stress Method)

This is the traditional method of design, used not only for reinforced concrete but also for structural steel & timber. This method of designed was evolved around the year 1900. This method was accepted by many tradition national code. India's code IS:456-1953 was based on this method. It was revised in 1957, 1964, 1978, 2000 & reaffirmed in 2005 & tenth reprint was in 2007. This method is based on linear elastic theory.

* This method ensures adequate safety by suitably restricting the stresses in the materials induced by expected working loads on the structure.

* In this method it is assumed that the concrete & steel are elastic. At the worst combinations of working loads, the stresses in the materials are not exceeded beyond permissible values. The permissible stresses are found out by using a suitable factor of safety to the material strength, e.g. for concrete in compression due to bending, a factor of safety equals to 3.0 is considered on 28 days characteristics strength & factor of safety equals to 1.8 is considered on the yield strength for mild steel reinforcement in tension due to bending.

* Working stress method does not considered the mode of failure of the structure that whether it is ductile or brittle. Also the reserve strength of the material beyond yield point is not considered in this method of design.

* Reinforced concrete is a composite material. The WSM assumes the strain compatibility, whereby the strain reinforcing steel is assumed to be equal to that in the adjoining concrete to which it is bonded. Consequently the stress in concrete by a const. factor, called the modular ratio, defined as the ratio of modulus of elasticity of steel to that of concrete. The WSM therefore also known as modular ratio method.

modular ratio $m = E_s/E_c = 280/3\sigma_{cbc}$

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Advantages of WSM

* It is simple, both in concept as well as in application
* It is reasonably reliable
* It is essential to have a knowledge of WSM, since it forms a part of LSM. IS:456-2000 has incorporated LSM & WSM. But the concept of WSM is retained for checking serviceability states of deflection & cracking.
* The design usually results in large sections of structural members as compared to LSM. Due to this, structural design by WSM gives better serviceability performance, i.e. less deflection, less crack width etc.

Dis-advantages of WSM

→ It gives larger sectn of structural members, thus it is uneconomical
→ It assumes that, stress & strain relationship is const for concrete, which is not true.
→ WSM does not consider the mode of failure of the structure (brittle or ductile)
→ Because of creep & non linear stress-strain relationship, concrete does not have definite modulus of elasticity.
→ The modular ratio design results in larger percentage of compression steel than that given by the limit state design, thus it's lead to uneconomic design.
→ WSM fails to discriminate betn different types of loads that act simultaneously but have different uncertainities

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Philosophy of Limit State Method (LSM)

The object of reinforced concrete design is to achieve a structure that will result in a safe & economical solution.

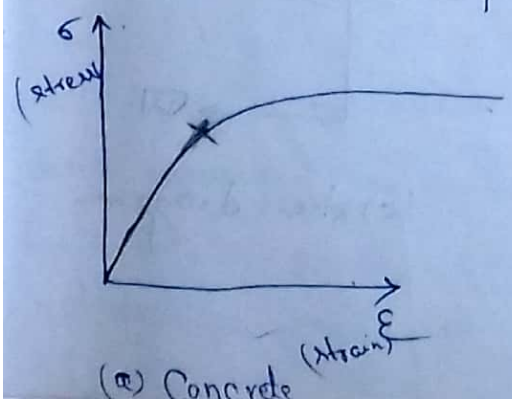
* There are four philosophies for the design of reinforced concrete, prestressed concrete as well as steel structure.

(1) Working stress ~~method~~ Design
(2) Ultimate load design
(3) Limit state design
(4) ~~Permance~~ for performance based design

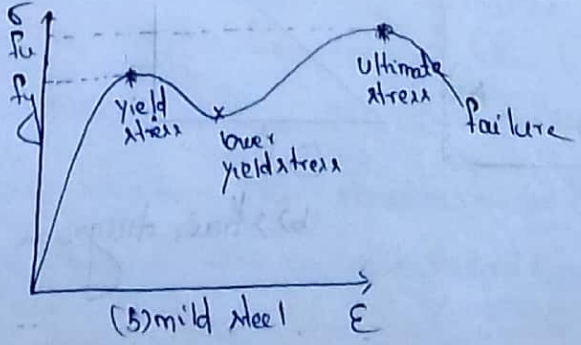
Working Stress method / modular ratio method / Elastic stress method

* The working stress method was the prevalent in the use from the early 1900's until the early 1960's.

* The working-stress method of design was developed first which is based on the behaviour of the structure at working load. The stress distribution in concrete & steel at working load is assumed to be linear. Hence the design is made by assuming linear stress-strain relationship ensuring that the stresses in concrete & steel do not exceed their permissible values at service load, which is taken as the fixed proportion of the ~~ultimate~~ yield strength of materials. It is obtained by dividing the yield strength of materials by the factor of safety. The factor of safety for ~~steel~~ concrete in compression in bending & in direct compression are 3.0 & 4.0 respectively. The factor of safety for steel in tension & compression is 1.8. ↓ 1.78 - 1.8



(a) Concrete



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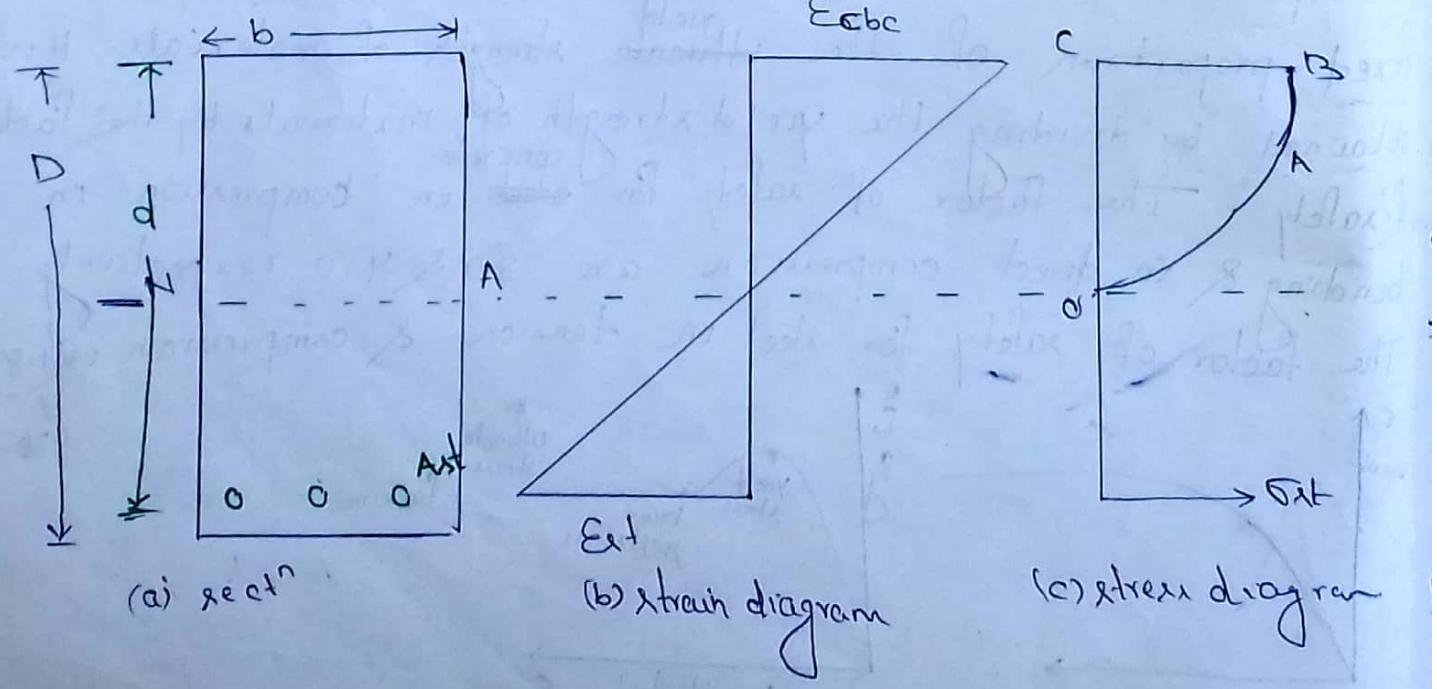
Ultimate load design / Load factor method

In the ultimate load method, the working loads are increased by suitable factors to obtain ultimate loads. These factors are called load factors. The structure is then designed to resist the desired ultimate loads. This method takes into account the non linear stress-strain behaviour of concrete.

* The term safety factor has been used in the working stress method to denote the ratio bet^n the yield stress and the permissible stress. It had little meaning as far as the ratio bet^n collapse load & working load was concerned. The term load factor has been traditionally used to denote the ratio bet^n the collapse or ultimate load to the working load.

* The knowledge of load factor is more important than the knowledge of factor of safety. In the former case, the designer is able to predict the excess load which a given structure can carry beyond the working load without collapse.

* When, stress-strain diagram in this method is idealised into individual well defined graphs, then the stress diagram becomes parabolic, which is straight line in elastic theory or the working stress method.



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Limit state method

* It takes into account the inelastic behaviour of the materials, which resulted into economical design. At the same time, it resulted in slender section, large deformations & larger crack widths in concrete. This created the necessity of refining the method so that in addition to strength, the structure ~~shall~~ would be serviceable also.

Limit State Method

Limit state method of design is a further improvement of ultimate load design. We know that the ultimate load theory is based on the assum that a structure reaches a collapse condition forming a mechanism pt when a certain load is applied. The ratio of the ultimate load to the ~~factored~~ working load is called the load factor. This method appeared to be ~~more~~ satisfactory and acceptable & was recommended by the codes of practice of many countries.

* After more research & study it has now been felt that this method of design has to be modified, so that a structure can be designed to function satisfactorily at service condition.

* The ultimate load method appears to emphasize on the adequate strength of a member. Now it is realized that the adequate strength could not be considered as the only criterion for the satisfactory performance of the member at service conditions. In most of the cases, a member designed by ultimate load theory is too slender to be adopted & if adopted may exhibit new problems like excessive deflection, excessive crack width etc at service conditions.

* In the limit state method, a structure is designed to withstand all loads likely to act on it in the duration of its life span & also to satisfy the serviceability requirements before failure can occur. The design provides a condition that the structure will not become unfit for use for which it is meant or in otherwords the structure will not reach a limit state.

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* All limit states which are relevant are considered in the design to ensure an adequate degree of safety & serviceability. The structure in general shall be designed on the basis of the most critical limit state & shall also be checked for other limit states.

* The ~~limit st~~ most important limit states to be considered are
  (a) formation of a mechanism leading to collapse &
  (b) excessive deflection & development of excessive cracks.

* The limit state corresponding to the formation of a mechanism depends on the ultimate strength. But the limit state corresponding to excessive deflection & excessive crack width is determined by elastic theory. Thus the limit state method combines the principles of ultimate load theory & elastic theory.

Limit State - The acceptable limit for the safety & serviceability requirements before failure occur.

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Types of limit states

The following limit states ~~refers to the strength of the structure~~ are considered in the design of structures.

(a) **Limit state of collapse / failure** → [Flexural (bending), compression, shear, Torsion] — Based on imaginary behaviour of concrete].

This limit state refers to the strength of the structure. A structure or its part should be strong enough to resist the applied ~~stress~~ design loads. This is called limit state of collapse.

* The limit state of collapse includes the design for axial forces, flexture, shear, torsion, buckling etc.. The strength of each section must be more than the applied stresses on that section due to all expected combination of loads.

(b) **Limit state of serviceability** — Based on actual behaviour of concrete (deflection, cracking, vibration, fire resistance etc)

The structure or its part thereof shall be serviceable during its expected life span. The serviceability corresponds to the deflection & cracking of the structure

Deflection:

Excessive deflections that can reduce the efficiency of the structure must be avoided.

Cracking:

Concrete structures have innumerable cracks, however if the crack widths are larger, the appearance of the structure will be affected. Also ingress of water & gas from the atmosphere can rusting of reinforcements.. For normal structures, the surface crack width of 0.3 mm is acceptable.

* It may happen that the structure is strong & safe to resist applied design loads but is not serviceable. Increasing the section dimensions usually solves the problem.

(c) **Limit state of durability:** Rain, floods, fire, earthquakes, severe weather etc are forces of nature which may affect the durability of a structure. Limits in the form of specifications are provided to cover the durability of the structures.

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Partial safety factors :

There are the factors which when applied to loads & material give the design values. The partial safety factors take into account the possible overloads, the kind of limit state & inaccurate assessment of the effects of loading. There are set out in table 18 of IS456 accordingly the partial safety factors for various loads shall be as follows.

(A) Partial safety factors (γ_f) for loads

Sr No.	Description	Partial safety factors	
		Collapse	Serviceability (short term)
1.	Gravity loads		
	DL	1.5	1.0
	IL (IL)	1.5	1.0
2.	DL+WL (or EL) combination		
	(i) for normal case		
	DL	1.5	1.0
	WL(EL)	1.5	1.0
	(ii) for checking stability against overturning/stress reversal		
	DL	0.9	1.0
	WL(or EL)	1.5	1.0
3.	DL+LL+WL(or EL) combination		
	DL	1.2	1.0
	LL	1.2	0.8
	(WL)EL	1.2	0.8

* For long term effect only permanent loads like dead load & a few live loads likely to be permanent should be considered

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(B) Partial safety factors (γ_m) for materials

Material	Partial safety factor
For concrete	1.5
" steel	1.15

Characteristic strength of materials

The characteristic strength of a material is that value of the strength of a material below which not more than 5 percent of the test results are expected to fall.

* The characteristic strength (f_{ck}) for various grades of concrete are given below

Grade of concrete	M15	M20	M25	M30	M35	M40
characteristic strength (28 days compressive strength f_{ck} in N/mm^2)	15	20	25	30	35	40

* The characteristic strength for steel (f_y) shall be assumed as min^m yield stress or 0.2% proof stress.

Grade of steel	Fe250	Fe415	Fe500
characteristic strength (f_y) N/mm^2	250	415	500

Characteristic load (F)

The term 'characteristic load' means that value of load which has a 95% probability of not being exceeded during the life of structure & is denoted by (F)

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Design values

The design values are obtained when partial safety factors are applied to characteristic loads ~~to cha~~ and strengths. These are obtained as below.

(i) Materials:

The design strength of the materials, f_d is given by

$$f_d = f/\gamma_m$$

where f = characteristic strength of the material

γ_m = partial safety factor appropriate to the material & limit state being considered

* While assessing the strength of a structural member for the limit state of collapse, the value of partial safety factor γ_m should be taken as 1.5 for concrete & 1.15 for steel.

(ii) Loads: The design load F_d is given by

$$F_d = F\gamma_f$$

F = characteristic load

γ_f = partial safety factor appropriate to the nature of loading & the limit state being considered. (Table 18 of IS 456 2000)

Loading on the structure as per IS 875

1) Characteristic dead loads are due to weight of the structure itself. These shall be found out using the unit weights of materials as given in IS:875-1987 (part I)

(ii) Characteristic live loads and wind loads in the absence of statistical data, shall be taken from IS:875-1987 (part II & III)

(iii) same → wind loads (IS 875 1987 part IV)

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(v) characteristic seismic loads in the absence of statistical data shall be taken from IS: 1893 -2002 (part-I)

~~Stru~~ IS specification regarding spacing of reinforcement in slab.

The bars shall be placed in such a way that they allow the concrete to enter when placed or a vibrator can be immersed.

maxm spacing of main bars - cl-26.3.3b)

The spacing of the main bar shall not exceed the following

i) 3 times the effective depth of the slab.

ii) 300mm, which ever is smaller

maxm spacing of distribution bars. The spacing of distribution bars shall not exceed the following

i) Five times the effective depth of the slab

ii) ~~300~~ 450 mm, which ever is smaller

minm spacing of bars: The spacing of the bars shall not be less than 75 mm for satisfactory concreting.

Cover to reinforcement

A concrete cover shall have to provided to the reinforcement for the following reasons.

(1) to protect the reinforcement from weather & fire, so that it does not corrode or melt &

(2) to ensure the grip of concrete over reinforcement so that they act as one & resist the loads.

cover (concrete cover)/ clear cover/nominal cover is defined as the minm distance betn the surface of concrete to the outside of the reinforcement.

Thus overalldepth (D) = effective depth (d) + bar radius + cover

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Cover to reinforcement:

slab For mild exposure, specified clear cover is 20mm; This can be reduced by 5mm when the reinforcement of 12 mm dia or less is u In most slabs, the bar dia is not more than 12 mm. we may take clear cover of 15mm for $\phi \leq 12$ mm & clear cover of 20mm when $\phi > 12$ mm for mild exposure.

beam (Table 16) of 1456

The cover to all reinforcement including stirrups is 20mm for mild exposure. Assuming max^m diameter of stirrups for normal beams be 10mm, the clear cover to main reinforcemen for beams may be taken as 20+10 = 30mm.

column

The longitudinal reinforcing bar in a column nominal cover shall in no case not be less than 40mm or less than the diameter of such bar.

* In the case of columns of min^m dimension, of 200mmor under, whose reinforcing bar do not exceed 12mm, a nominal cover of 25mm may be used

footing

For footing min^m cover shall be 50mm.

Min^m reinforcement

2) Beams

(a) Tension reinforcement: (26.5.1)

min^m reinforcement: The min^m area of tension reinforcemen shall be not less than that given by the following

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where A_s = min^m area of tension reinforcement

b = breadth of beam

d = effective depth

f_y = characteristic strength of reinforcement in N/mm^2

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b) max^m reinforcement:

The max^m area of tension reinforcement shall not exceed 0.04 bD.

Compression reinforcement:

The max^m area of compression reinforcement shall not exceed 0.04 bD.

min^m reinforcement in slab: (26.5.2.1)

The reinforcement in either direction in slab shall not be less than the following

0.15% bD for mild steel bar

0.12% bD for HYSD bars.

Column

The c/s area of longitudinal reinforcement, shall be not less than 0.8% nor more than 6% of the gross sectional area of column.

Effective span (22.2)

Simply supported beam or slab:

The effective span of a member that is not built integrally with it supports shall be taken as clear span plus the effective depth of slab or beam or c/c of supports which ever is less.

b) Cantilever - beam/slab The effective length of a cantilever shall be taken as its length to the face of support plus half the effective depth except where it forms the end of a continuous beam where the length to the centre of support shall be taken

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Analysis & Design of Singly & Doubly Reinforced Sections (LSM)

Limit state of collapse: Flexture:

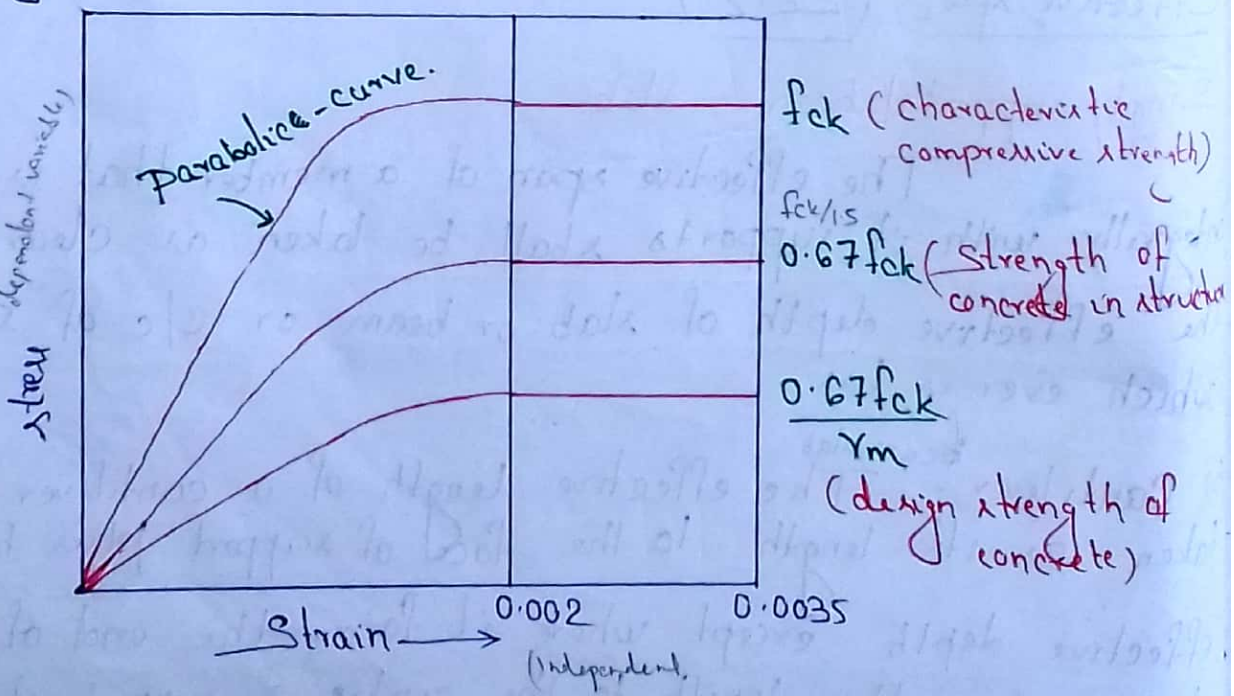
Assumptions (clause 38.1)

Design for the limit state of collapse in flexture shall be based on the assumptions given below

(a) plane sections normal to the axis remain plane after bending

(b) The max^m strain in concrete at the outermost compression fibre is taken as 0.0035 in bending.

(3) The ~~stress~~ relationship bet^n the compressive stress distribution in concrete & the strain in concrete may be assumed to be rectangle, trapezoid, parabola or any other shape which results in predict^n of strength in substantial agreement with the results of test. An acceptable stress-strain curve is given in fig. For design purpose 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.



fig(1) Stress-strain curve for concrete

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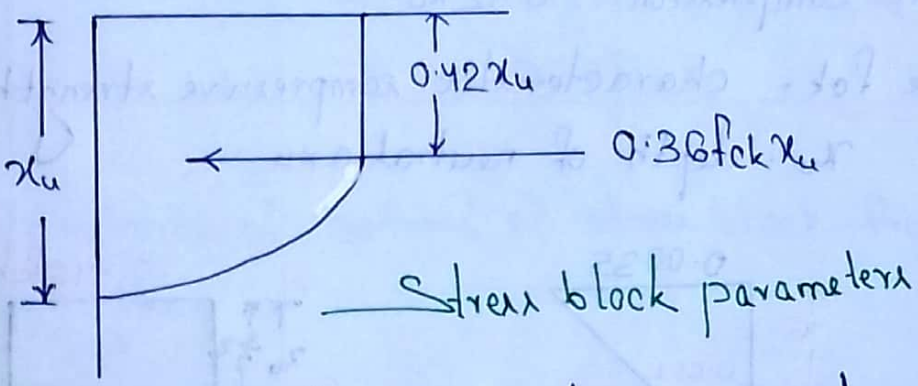
Note: For the stress-strain curve in fig (a) the design stress block parameters are as follows

Area of stress block = $0.36 f_{ck} x_u$

Depth of centre of compressive force from the extreme fibre in compression = $0.42 x_u$

where f_{ck} = characteristic compressive strength of concrete &
x_u = depth of neutral axis.



Stress block parameters

(e) The tensile strength of the concrete is ignored.

(f) The stresses in the reinforcement are derived from representative stress-strain curve for the type of steel used. Typical curves are given in (fig 23 of 456). For design purpose the ~~the~~ partial safety factor γ_m, equal to 1.15 shall be applied.

(f) The max^m strain in the tension reinforcement in the sect^n at failure shall not be less than.

$$\frac{f_y}{1.15 E_s} + 0.002$$

where f_y = characteristic strength of steel
E_s = modulus of elasticity of steel

Stress-strain relationship for concrete & steel

fig 21 & 23 of IS 456: 2000

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Stress block diagram & strain diagram for singly reinforced RC section

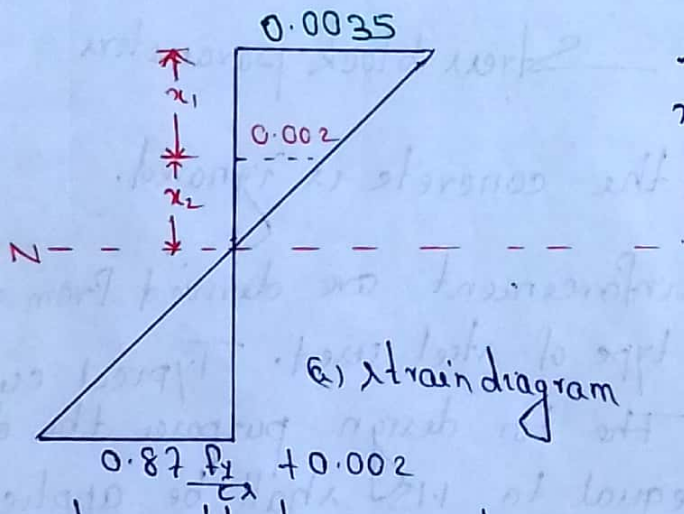
The strain diagram & design stress block are separately shown in fig. The design stress block parameters are as follows

Area of stress block = $0.36 f_{ck} x_u$

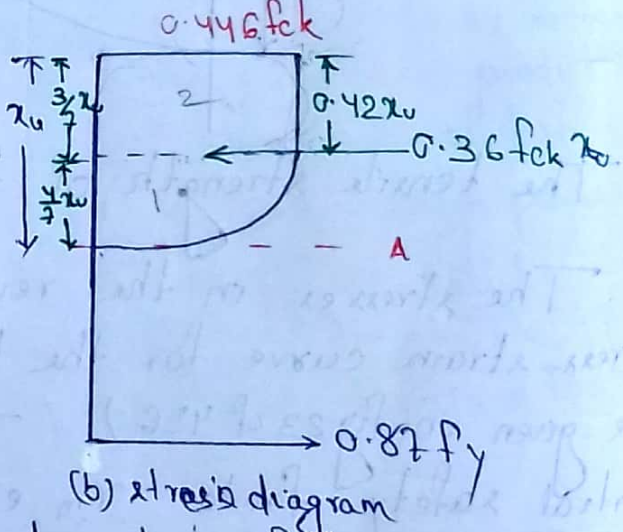
depth of centroid of compressive force from the extreme fibre in compression = $0.42 x_u$

where f_{ck} = characteristic compressive strength of concrete

x_u = depth of neutral axis



(a) strain diagram



(b) stress diagram

The stress block parameters are derived as follows

See stress block diagram & strain diagram for concrete

Let x_1 = height of rectangular part of the stress block

x_2 = " of parabolic " " "

As the strain in concrete is proportional to the distance from the neutral axis,

Now $\frac{x_2}{x_u} = \frac{0.002}{0.0035} = 4/7$

$\therefore x_2 = \frac{4}{7} x_u$

& $x_1 = x_u - 4/7\, x_u = 3/7\, x_u$

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Area of the parabolic part of the stress block

$$= \frac{2}{3} \times 0.446 f_{ck} \times \frac{4}{7} x_u$$

$$= 0.17 f_{ck} x_u$$

Area of the rectangular part of the stress block

$$= 0.446 f_{ck} \times \frac{3}{7} x_u = 0.19 f_{ck} x_u$$

∴ Total area of the stress block $= 0.17 f_{ck} x_u + 0.19 f_{ck} x_u$

$$= 0.36 f_{ck} x_u$$

Let $\bar{y}$ be be distance of centroid of stress block from extreme compression fibre, then

$$\bar{y} = \frac{0.17 f_{ck} x_u \times \left(\frac{3}{7} x_u + \frac{3}{8} \cdot \frac{1}{2} x_u \right) + 0.19 f_{ck} x_u \times x_u/2}{0.36 f_{ck} x_u}$$

$$= \frac{0.17 f_{ck} x_u \times \left(\frac{3}{7} x_u + \frac{3}{8} \times \frac{4}{7} x_u \right) + 0.19 f_{ck} x_u \times \left(\frac{1}{2} \times \frac{3}{7} x_u \right)}{0.36 f_{ck} x_u}$$

$$\bar{y} = 0.42 x_u$$

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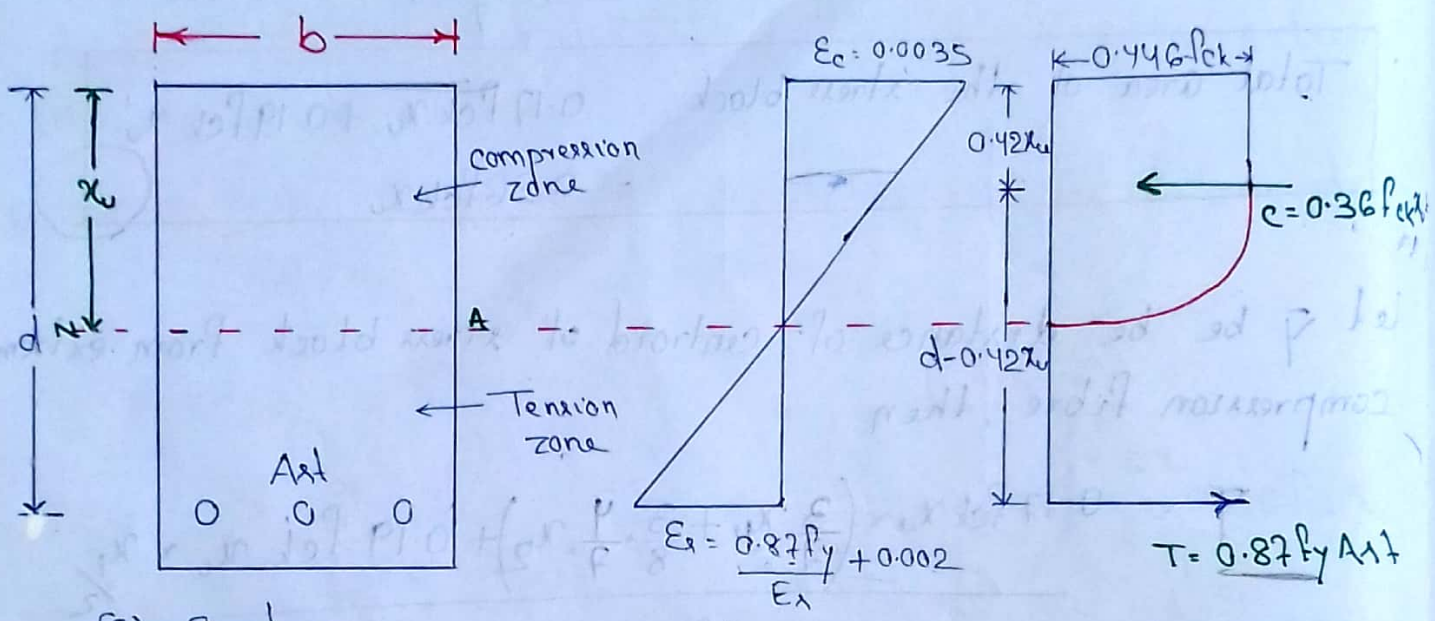
Singly Reinforced Rectangular Beams

A singly reinforced rectangular beam sections, strain diagram & stress diagram are shown in fig.

* ~~To find neu~~



(a) Section (b) Strain diagram (c) Stress diagram

Singly reinforced beam

* To find neutral axis

Total compression = total tension

$0.36 f_{ck} x_u b = 0.87 f_y A_{st}$

$$\Rightarrow x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} \quad \ldots (1)$$

Total compression
C = Area of stress block × b

Total tension / tensile force = Area of steel × stress

Max^m depth of neutral axis

The max^m depth of neutral axis is governed by the assumption (b) & (f) in members in flexture

Fig shows the strain & the stress diagram at limiting conditions

Let $x_{u,max}$ be the max^m depth of neutral axis

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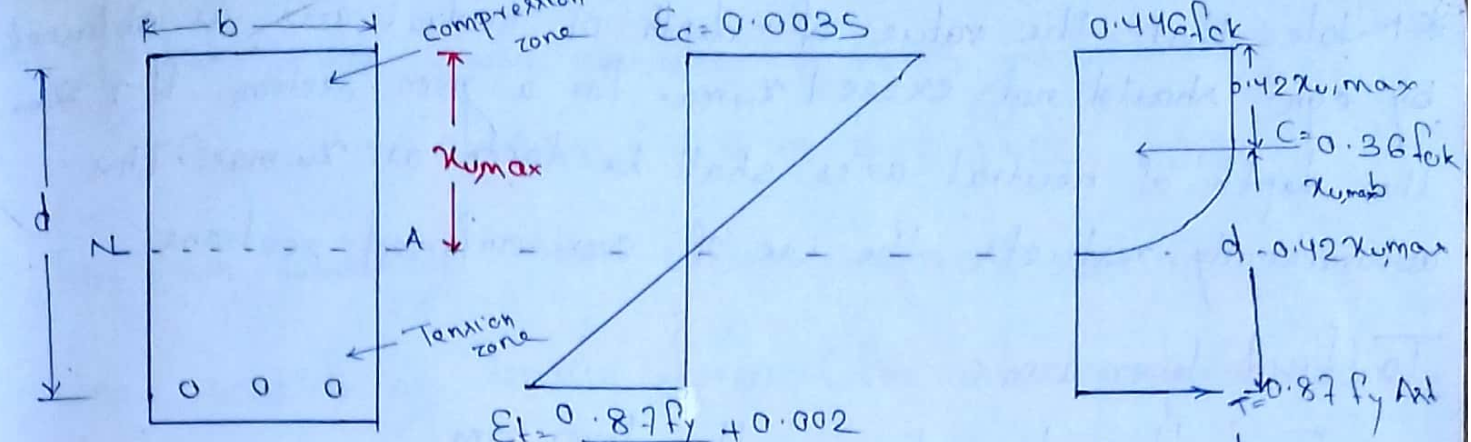

(a) section (b) strain diagram (c) stress diagram

by geometry of the strain diagram

$$\frac{x_{u,max}}{d - x_{u,max}} = \frac{0.0035}{\frac{0.87 f_y}{E_s} + 0.002}$$

$$\Rightarrow \frac{x_{u,max}}{d} = \frac{0.0035}{0.0035 + \frac{0.87 f_y}{E_s} + 0.002}$$

$$\Rightarrow \frac{x_{u,max}}{d} = \frac{0.0035}{0.0055 + \frac{0.87 f_y}{E_s}}$$

$$\Rightarrow \frac{x_{u,max}}{d} = \frac{0.0035 E_s}{0.0055 E_s + 0.87 f_y}$$

Taking $E_s = 2 \times 10^5$ N/mm^2, we get

$$\frac{x_{u,max}}{d} = \frac{700}{1100 + 0.87 f_y}$$

The following table shows the values of $\frac{x_{u,max}}{d}$ for various grades of steel

Value of $\frac{x_{u,max}}{d}$ for various grades of steel

f_y (N/mm^2)	$\frac{x_{u,max}}{d}$
250	0.53
415	0.48
500	0.46

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* Note that the value of depth of neutral axis (x_u) as obtained by eqn should not exceed $x_{u,max}$ for a given section. If $x_u > x_{u,max}$ the depth of neutral axis shall be taken as $x_{u,max}$. Thus automatically restricts the use of over-reinforced sections.

To find lever arm:

From the stress diagram, the lever arm

$$z = d - 0.42x_u \quad \ldots (2)$$

* To find moment of resistance:

(1) For a balanced section

MR: total compression × lever arm
= total tension × lever arm

Considering the compressive forces (over reinforced & balanced sect^n)

$$M_u = 0.36 f_{ck} b x_u (d - 0.42x_u)$$

$$= 0.36 f_{ck} \frac{x_u}{d}\left(1 - 0.42\frac{x_u}{d}\right) f_{ck} bd^2 \quad \ldots (3)$$

for a limiting value, substitute $x_{u,max}$ for x_u & $M_{u,lim}$ for M_u

Then $$M_{u,lim} = 0.36 \frac{x_{u,max}}{d}\left(1 - 0.42\frac{x_{u,max}}{d}\right) f_{ck} bd^2 \quad \ldots (4)$$

$$= Q_{lim} bd^2 \quad \ldots$$

where $$Q_{lim} = \frac{M_{u,lim}}{bd^2}$$

$$= 0.36 \frac{x_{u,max}}{d}\left(1 - 0.42\frac{x_{u,max}}{d}\right) f_{ck}$$

Q_{lim} = limiting moment of resistance factor for balanced rectangular sect^n

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for eg; for M20 grade concrete & Fe415 grade steel.

$$Q_{lim} = 0.36 \times 0.48 (1 - 0.42 \times 0.48) \times 20 = 2.76$$

(2) For an underreinforced section

Now considering tensile forces (For under-reinforced sections)

$$M_u = 0.87 f_y A_{st} (d - 0.42 x_u) \quad \ldots (5)$$

Substituting $x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b}$

$$M_u = 0.87 f_y A_{st} \left(d - 0.42 \times \frac{0.87 f_y A_{st}}{0.36 f_{ck} b}\right)$$

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{b d f_{ck}}\right) \quad \ldots (6)$$

Dividing both the sides by $0.87 f_y b d^2$

$$\frac{M_u}{0.87 f_y b d^2} = \frac{A_{st}}{bd} \left(1 - \frac{f_y A_{st}}{f_{ck} b d}\right)$$

Substituting $A_{st} = \frac{p_t b d}{100}$

$$\frac{M_u}{0.87 f_y b d^2} = \frac{p_t}{100} - \frac{f_y}{f_{ck}} \left(\frac{p_t}{100}\right)^2$$

$$\Rightarrow \frac{f_y}{f_{ck}} \left(\frac{p_t}{100}\right)^2 - \left(\frac{p_t}{100}\right) + \frac{M_u}{0.87 f_y b d^2} = 0$$

which on solving gives

$$\frac{p_t}{100} = \frac{1 - \sqrt{1 - \frac{4.6}{f_{ck}} \times \frac{M_u}{b d^2}}}{2 f_y / f_{ck}}$$

$$\therefore p_t = \frac{50}{f_y / f_{ck}} \left[1 - \sqrt{1 - \frac{4.6}{f_{ck}} \times \frac{M_u}{b d^2}}\right]$$

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The positive value of the root of eqn will give larger area of steel than $P_{t,lim}$ & hence it is not considered.

Limiting value of moment of resistance.

Properties of the beam section at limiting section.

At limiting condition $\frac{x_u}{d} = \frac{x_{u,max}}{d}$

The moment of resistance at limiting condition

$$M_{u,lim} = 0.36 \frac{x_{u,max}}{d}\left(1 - 0.42 \frac{x_{u,max}}{d}\right) bd^2 f_{ck}$$

$$= 0.36 \frac{x_{u,max}}{d}\left(1 - 0.42 \frac{x_{u,max}}{d}\right) f_{ck}\, bd^2$$

Corresponding to the various grades of steel the moments of resistance at limiting cond^n are given in the below

Moment of resistance at limiting cond^n

Grade of steel	$\frac{x_{u,max}}{d}$	Limiting moment of resistance
Fe250	0.53	$M_{u,lim} = 0.149 f_{ck} bd^2$
Fe415	0.48	$M_{u,lim} = 0.138 f_{ck} bd^2$
Fe500	0.46	$M_{u,lim} = 0.133 f_{ck} bd^2$

Here the const $\frac{M_{u,lim}}{f_{ck} bd^2} = 0.36 \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{u,max}}{d}\right)$ is known as limiting moment of resistance index

Limiting percentage of steel required for limiting singly-reinforced section

steel requirement at limiting cond^n

Equating the total compression to total tension at limiting cond^n

$$0.36 f_{ck}\, b\, x_{u,max} = 0.87 f_y A_{st,lim}$$

$$\Rightarrow A_{st,lim} = \frac{0.36 f_{ck}\, b\, x_{u,max}}{0.87 f_y}$$

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$$= \frac{0.36\, f_{ck}\, x_{u,max}}{0.87\, f_y\, d} \times bd$$

$$= 0.414 \frac{f_{ck}}{f_y} \cdot \frac{x_{u,max}}{d} bd$$

Limiting percentage of steel : $P_{t,lim} = \frac{A_{st,lim}}{bd} \times 100$

$$= 41.4 \frac{f_{ck}}{f_y} \cdot \frac{x_{u,max}}{d}$$

The limiting percentages of steel for various combinations of grades of concrete & steel are given in the table below.

Limiting percentage of steel $P_{t,lim} = 41.4 \frac{f_{ck}}{f_y} \times \frac{x_{u,max}}{d}$

f_{ck} N/mm²	f_y (N/mm²)		
	250	415	500
15	1.32	0.72	0.57
20	1.75	0.96	0.76
25	2.19	1.20	0.95
30	2.63	1.44	1.14

Types of Problems

The different types of problems are considered for singly reinforced rectangular beams are as follows

Type 1

To find out the depth of neutral axis & specifying the type of beam

For a given section, equate total tension & total compression & thus find out the depth of neutral axis using eqn

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i.e. $x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b}$

Also find out the limiting value of depth of neutral axis $x_{u,max}$ using limiting value of $\frac{x_{u,max}}{d}$

Then If $x_u < x_{u,max}$ the sect^n is under-reinforced

If $x_u = x_{u,max}$ the sect^n is balanced.

If $x_u > x_{u,max}$ the sect^n is over reinforced & adopt

$x_u = x_{u,max}$

Alternatively the reinforcement percentage P_t shall be compared with $P_{t,lim}$

Then If $P_t < P_{t,lim}$ the section is under-reinforced.

If $P_t = P_{t,lim}$ " " is balanced.

If $P_t > P_{t,lim}$ " " is over-reinforced

Type 2

To find out moment of resistance for a given section

1) find out the depth of neutral axis (x_u) & the type of the beam as discussed in type 1.

2) For over-reinforced & balanced sections, obtain moment of resistance using eqn (3)

i.e.

$$M_{u,lim} = 0.36 \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{u,max}}{d}\right) f_{ck}\, b d^2$$

3) For under reinforced sections, obtain moment of resistance by using eqn (3) (5) or (6)

i.e.

$$M_u = 0.36 \frac{x_u}{d}\left(1 - 0.42 \frac{x_u}{d}\right) f_{ck}\, b d^2$$

or

$$M_u = 0.87 f_y A_{st} (d - 0.42 x_u)$$

or

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{f_{ck} b d}\right)$$

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Type 3:

To design a singly reinforced rectangular section for a given width & applied factored moment.

The width is usually decided by the functional or architectural requirements. To decide the depth, first determine the balanced depth.

$$d_{bal} = \sqrt{\frac{M_{lim}}{Q_{lim} b}} = \sqrt{\frac{M_u}{0.36 \frac{x_{u,max}}{d}\left(1 - 0.42 \frac{x_{u,max}}{d}\right) f_{ck}}}$$

$$\left[\because M_{u,lim} = 0.36 \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{umax}}{d}\right) f_{ck}\, b d^2 \right]$$

* If the balanced area is found with this depth, difficulty will arise while providing the reinforcements. Selecting exact no of reinforcement with available dia. is hardly possible. We cannot provide lesser area.

* If we provide larger area, the section becomes over-reinforced. Therefore, the under-reinforced design should be followed by using larger depth than balanced one.

* The following procedure may be adopted as one of the alternatives for depths upto 1000 mm to arrive at a practical soln.

(1) Assume 5% larger effective depth for $d \leq 500$ mm (case 1) & 10% larger depth for $1000\text{mm} \geq d \geq 500\text{mm}$ (case 2).

(2) Assume one layer of 20mm dia bars for (case 1) & 2 layers of 20mm dia. bars for (case 2)

(3) If the clear cover to main reinforcement is 30 mm

effective cover = 30 + 10 = 40mm for case 1 (Assume one layer of 20 ϕ)

= 40 + 20 = 60mm for case 2 (Assume 2 layer of 20 ϕ)

(30 + 20 + 10)

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(4) The overall depth can be obtained as

$D_{overall} = d + 40mm$ for case 1

1 $= d + 60mm$ " " 2

(5) : The values thus obtained shall be rounded upto a nearest 25mm.

(6) Now $d = D_{overall} - 40mm$ for case 1

$= D_{overall} - 60mm$ " " 2

(7) Determine $\frac{M_u}{bd^2}$, P_t & A_{st}, using eqn as per the case of under-reinforced sect^n

$$P_t = 50\left[\frac{1-\sqrt{1-\frac{4.6}{f_{ck}}\times\frac{M_u}{bd^2}}}{f_y/f_{ck}}\right]$$ or $M_u = 0.87 f_y A_{st} d\left(1-\frac{A_{st} f_y}{bdf_{ck}}\right)$

then $P_t = \frac{A_{st}}{bd}\times 100$. find out A_{st} $\Rightarrow$ findout A_{st}

Also determine $A_{st,lim}$ by from the following eqn.

$$0.36 f_{ck} b x_{umax} = 0.87 f_y A_{st,lim}$$

8> Select the bar size & number such that $A_{st} > A_{st,required}$ & also $A_{st} < A_{st,lim}$

Type: 4

To find the steel area for a given factored moment

(1) For a given ultimate/factored moment & assumed width of section, find out d from eqn

$$d = \sqrt{\frac{M_u}{Q_{lim} b}}$$

Thus is a balanced section & balanced steel area may be found out using eqn

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$M_u = 0.87 f_y A_{st} (d - 0.42 x_u)$

or

(2) for a given factored moment, width & depth of section
M_u, b, d

obtain $M_{u,lim} = Q_{lim} \cdot bd^2$

If $M_u < M_{u,lim}$ ($x_u < x_{u,max}$): design as under-reinforced

If $M_u = M_{u,lim}$: design as ~~over-reinforced~~ balanced sectn

If $M_u > M_{u,lim}$: redesign the section either increasing the dimension of section or design as doubly reinforced beam.

For under-reinforced sectn, the steel area can be obtained by using eqn

$$M_u = 0.87 f_y A_{st} (d - 0.42 x_u)$$

or

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{b d f_{ck}}\right)$$

Problem: 1

A rectangular beam 230mm wide & 520 mm effective depth is reinforced with 4 no's 16mm dia bars. Find out the depth of neutral axis & specify the type of beam. The materials are are M20 grade concrete & HYSD reinforcement of grade Fe415. Also find out the depth of neutral axis if the reinforcement is increased to 4 no 20mm dia. bars.

Soln

breadth of the beam (b) = 230mm

effective depth of the beam (d) = 520mm.

characteristic comp. strength of concrete (f_{ck}) = 20 N/mm²

" strength of steel (f_y) = 415 N/mm²

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Case-I

$A_{st} = 4 \times \frac{\pi}{4} \times 16^2 = 804.25\ mm^2$

Total compressive force (C) $= 0.36 f_{ck} b x_u$

$= 0.36 \times 20 \times 230\, x_u$

$= 1656\, x_u$

Total tensile force (T) $= 0.87 f_y A_{st}$

$= 0.87 \times 415 \times 804.25$

$= 290374.46\ N$

Equating $1656\, x_u = 290374.46$

$\Rightarrow x_u = 175.35\ mm$

Limiting value of neutral axis

$x_{u,max} = 0.48 d = 0.48 \times 520 = 249.6\ mm$

Here $x_u < x_{umax}$

$\therefore$ Section is under-reinforced.

Case-2

Area of steel $(A_{st}) = 4 \times \pi/4 \times 20^2 = 1256.64\ mm^2$

Equating total compressive force (C) to total tensile force

$\Rightarrow 0.36 f_{ck} b x_u = 0.87 f_y A_{st}$

$\Rightarrow 0.36 \times 20 \times 230 \times x_u = 0.87 \times 415 \times 1256.64$

$\Rightarrow x_u = 273.98\ mm$

Here $x_u > x_{umax}$ i.e. over reinforced section

$\therefore x_u = x_{max} = 249.6\ mm$

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Problem: 2

A singly reinforced rectangular beam of width 230mm & 460mm effective depth is reinforced with 3 no. 20mm dia bars. Find out the factored moment of resistance of the section. The materials are M20 grade concrete & HYSD reinforcement of grade Fe 415. Also find out out the factored moment of resistance if it is reinforced with 4 no of 20mm dia bars.

Sol^n

breadth of the beam(b) = 230 mm

effective depth " " (d) = 460mm

Characteristic compressive strength of concrete (f_{ck}) = 20 N/mm^2

" strength of steel (f_y) = 415 N/mm^2

Case-1

Area of steel (A_{st}) = $\frac{3\pi}{4} \times 20^2$ = 942.47 mm^2

To find the depth of neutral axis (x_u)

Equating total tensile & compressive forces

$$0.36 f_{ck} b x_u = 0.87 f_y A_{st}$$

$$\Rightarrow 0.36 \times 20 \times 230 \times x_u = 0.87 \times 415 \times 942.47$$

$$\Rightarrow x_u = 205.48 \text{ mm}$$

Limiting value of neutral axis ($x_{u,max}$) = 0.48×460

= 220.8 mm

Here $x_u < x_{u,max}$

acceptable value of

$\therefore$ The section is under-reinforced & x_u = 205.48 mm

Moment of resistance = $0.87 f_y A_{st} (d - 0.42 x_u)$

$= 0.87 \times 415 \times 942.47 \times (460 - 0.42 \times 205.48)$

= 127.16 kN-m

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Alternatively $M_u = 0.36 f_{ck} b x_u (d - 0.42 x_u)$

$= 0.36 \times 20 \times 230 \times 205.48 (460 - 0.42 \times 205.48)$

$= 127.16 \text{ kN.m}$

Case 2

Area of steel $(A_{st}) = \pi/4 \times 20^2 = 1256.64 \text{ mm}^2$

To find the depth of neutral axis (x_u)

Equating total compressive forces to total tensile force

$\Rightarrow 0.36 f_{ck} b x_u = 0.87 f_y A_{st}$

$\Rightarrow 0.36 \times 20 \times 230 \times x_u = 0.87 \times 415 \times 1256.64$

$\Rightarrow x_u = 273.98 \text{ mm}$

Limiting value of neutral axis $(x_{u,max}) = 0.48 \times 460$

$= 220.8 \text{ mm}$

Here $x_u > x_{u,max}$. So the sect^n is over reinforced

Acceptable value of $x_u = 220.8 \text{ mm}$

Lever arm $= d - 0.42 x_u = 460 - 0.42 \times 220.8 = 367.26 \text{ mm}$

$M_u = 0.36 f_{ck} b x_u (d - 0.42 x)$

$= 0.36 \times 20 \times 230 \times 220.8 (460 - 0.42 \times 220.8)$

$= 134.2 \text{ kN.m}$

* The actual stress in steel f_s, when the section is subjected to $M_{u,lim}$ is determined by equating total tension & total compression.

Then

$0.36 f_{ck} b x_u = 0.87 f_y' A_{st}$

$\Rightarrow 0.36 \times 20 \times 230 \times 220.8 = 0.87 \times f_y' \times 1256.64$

$\Rightarrow f_y' = 336.45 \text{ N/mm}^2$

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Problem: 3 Type-4

A singly reinforced rectangular beam is subjected to a bending moment of 36kNm at working loads. The width of the beam is 200mm. Find the depth & steel area for balanced design. The materials are M20 grade concrete & mild steel reinforcement.

Sol?

Bending moment acting on the beam = 36kNm (at working loads)

Factored moment = $1.5 \times 36 = 54$kNm

width of the beam (b) = 200mm

Characteristic compressive strength of concrete (f_{ck}) = $20 N/mm^2$

Characteristic strength of steel (f_y) = ~~250~~ N/mm^2

Limiting moment of resistance ($M_{u,lim}$) = $0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.42 \frac{x_{umax}}{d}\right) bd^2 f_{ck}$

$\Rightarrow 54 \times 10^6 = 0.36 \times 20 \times 0.53 (1 - 0.42 \times 0.53) \cdot 200 \cdot d^2 \cdot 20$

$\Rightarrow d = 301.86$

Steel area can be found out by using eqn

$$M_u = 0.87 f_y A_{st} d \left(1 - 0.42 \frac{x_u}{d}\right)$$

For balanced section $\frac{x_u}{d} = \frac{x_{u,max}}{d} = 0.53$

substituting $54 \times 10^6 = 0.87 \times 250 \times A_{st} \times 301.86 \times (1 - 0.42 \times 0.53)$

$\Rightarrow A_{st} = 1057.99 mm^2$

Provide $3 - 20\phi + 1 - 16\phi = 1143.54 mm^2$

$D = 302 + 10 + 30 = 342$ mm

Provide overall depth = 350mm.

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Problem:4 Type-4

A rectangular cantilever beam of size 230mm width × 500mm effective depth is subjected to a bending moment of 80 kNm at working loads. Find the steel area required. The materials are M20 grade concrete & HYSD reinforcement of grade Fe415.

Ans

width of the beam(b) = 230mm

effective depth " (d) = 500mm

characteristic compressive strength of concrete (fck) = 20 N/mm²

" strength of steel (fy) = 415 N/mm²

BM due to working loads = 80 kN.m

factored BM (Mu) = 80×1.5 = 120 kNm

Limiting moment of resistance, for 415 grade steel

$M_{u,lim} = 0.138 f_{ck} b d^2$

~~$\Rightarrow 120 \text{ kNm} \times 10^6$~~ $= 0.138 \times 20 \times 230 \times 500^2$

$= 158.7$ kNm

Here $M_u < M_{u,lim}$. So the beam is under reinforced

To find steel area, using eqn

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{b d f_{ck}}\right)$$

$$\Rightarrow 120 \times 10^6 = 0.87 \times 415 \times A_{st} \times 500 \left(1 - \frac{415 A_{st}}{230 \times 500 \times 20}\right)$$

$$\Rightarrow 664.72 = A_{st}\left(1 - 1.80 \times 10^{-4} A_{st}\right)$$

$$\Rightarrow 1.80 \times 10^{-4} A_{st}^2 - A_{st} + 664.72 = 0$$

$$\Rightarrow A_{st} = 772 \text{ mm}^2$$

Provide 4-16ϕ bars in tension zone

$A_{st,provided} = 804.25 \text{ mm}^2$

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Problem: 5 Type-3

Design a singly reinforced rectangular beam for an applied factored moment of 120 kNm. The materials are M20 grade concrete & HYSD reinforcement of grade Fe 415.

Sol^n

Characteristic compressive strength of concrete (f_{ck}) = 20 N/mm²

characteristic strength of steel (f_y) = 415 N/mm²

factored B.M acting on the beam (M_u) = 120 kNm

The beam will be designed so that under the applied moment both materials reach their max^m stresses. Let us assume ratio of effective depth to breadth of the beam equal to 1.5. i.e. $d/b = 1.5$

For a balanced design

$$M_u = 0.36 f_{ck} \frac{x_{u,max}}{d}\left(1 - 0.42\frac{x_{u,max}}{d}\right) b d^2 f_{ck}$$

$$\Rightarrow 120\times10^6 = 0.36\times0.48\times(1-0.42\times0.48)\frac{d}{2}\times d^2\times 20$$

$$\Rightarrow d^3 = 869795514.58$$

$$\Rightarrow d = 443.06\text{ mm}$$

wrong applied
Change it

Increase this depth by 5% & add 40mm effective cover

$$D = 443.06\times1.05 + 40\text{ mm}$$

$$= 505.21\text{ mm}$$

[assume clear cover of beam = 30 mm & dia bar = 20mm]

Provide overall depth (D) of beam = 525 mm (Nearest multiple of 25 mm)

Assume one layer of 20mm dia bars are provided in tension zone

so effective depth (d) = $525 - 30 - 20/2 = 485$ mm

& width of rect^r (b) = $d/1.5 = \frac{485}{1.5} = 323.33\text{ mm} \simeq 325$ mm

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To find the steel area, using eqn

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{b d f_{ck}}\right)$$

$$\Rightarrow 120 \times 10^6 = 0.87 \times 415 \times A_{st} \times 485 \left(1 - \frac{415 \times A_{st}}{325 \times 485 \times 20}\right)$$

$$\Rightarrow 685.29 = A_{st} - 1.316 \times 10^{-4} A_{st}^2$$

$$\Rightarrow 1.316 \times 10^{-4} A_{st}^2 - A_{st} + 685.29 = 0$$

$$\Rightarrow A_{st} = 761.83 mm^2$$

Now Limiting area of steel $A_{st,lim}$ ~~$= 0.87 f_y A_{st}$~~ is obtained by eqn

$$0.36 f_{ck} \cdot b \cdot x_{u max} = 0.87 f_y A_{st,lim}$$

$$\Rightarrow A_{st,lim} = \frac{0.36 f_{ck} b \cdot x_{u,max}}{0.87 f_y}$$

$$= \frac{0.36 \times 20 \times 325 \times 0.48 \times 485}{0.87 \times 415}$$

$$= 1508.79 mm^2$$

Let us provide 3-20ϕ bar in tension zone.

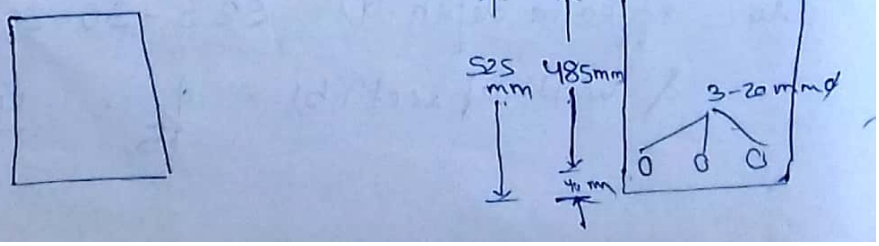
$A_{st,provided} = 3 \times \pi/4 \times 20^2 = 942.47 mm^2$

Thus ~~A_{st} required~~ <

Minm area of steel $A_s = \frac{0.85 bd}{f_y} = \frac{0.85 \times 325 \times 485}{415}$

$= 322.85 mm^2 < 942.47 mm^2$

Thus $A_{st,required} < A_{st,provided} < A_{st,lim}$ ok) ok)



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Doubly Reinforced beams

General features

A doubly Reinforced concrete beams having steel reinforcement both on tensile & compression zones are known as doubly reinforced beams.

Therefore, doubly reinforced beams have moment of resistance more than singly reinforced beams of the same depth for a particular grade of steel & concrete

Necessity of providing doubly-reinforced beam

1) Doubly reinforced beams are provided in order to increase the moment carrying capacity of the section.

2) Minm compression reinforcement is provided to hold the shear reinforcement (stirrups) in position & for increasing the ductility of beam.

3) Most important reason for providing doubly reinforced beams is to ensure safety against reversal of stresses in the structure due to wind forces, seismic forces & temperature stresses

4) When dimensions of the beams are restricted due to some architechtural purpose, then moment of resistance of the section is increased by providing doubly reinforced sections

Analysis of doubly reinforced beams

If the applied moment is greather than the M.R. of a singly reinforced section, there can be three alternatives

(1) If possible, increase the dimensions of the section, preferably depth

(2) Higher grade concrete can be used to increase the MR of the section.

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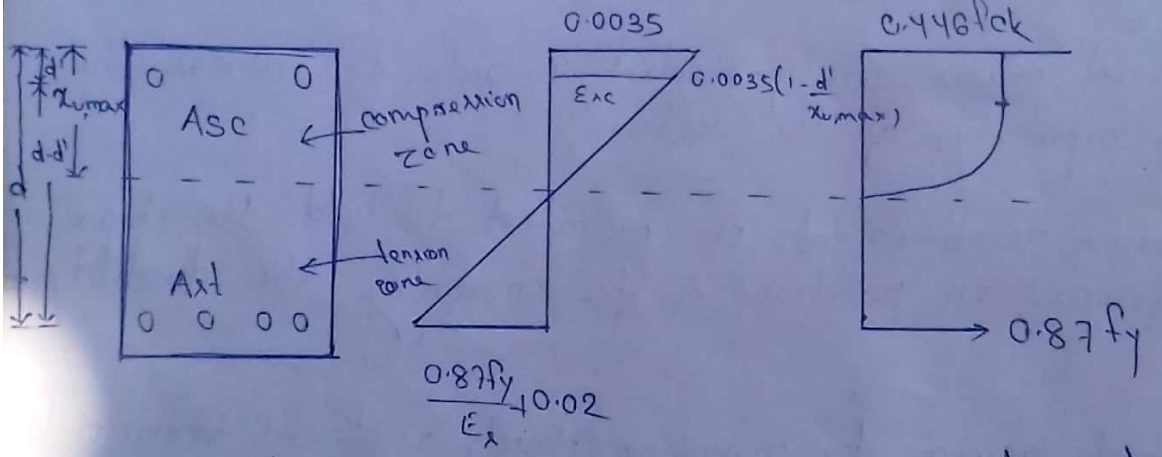
(3) Steel reinforcement may be added in compression zone to increase the M.R. of the section. This is known as doubly reinforced section

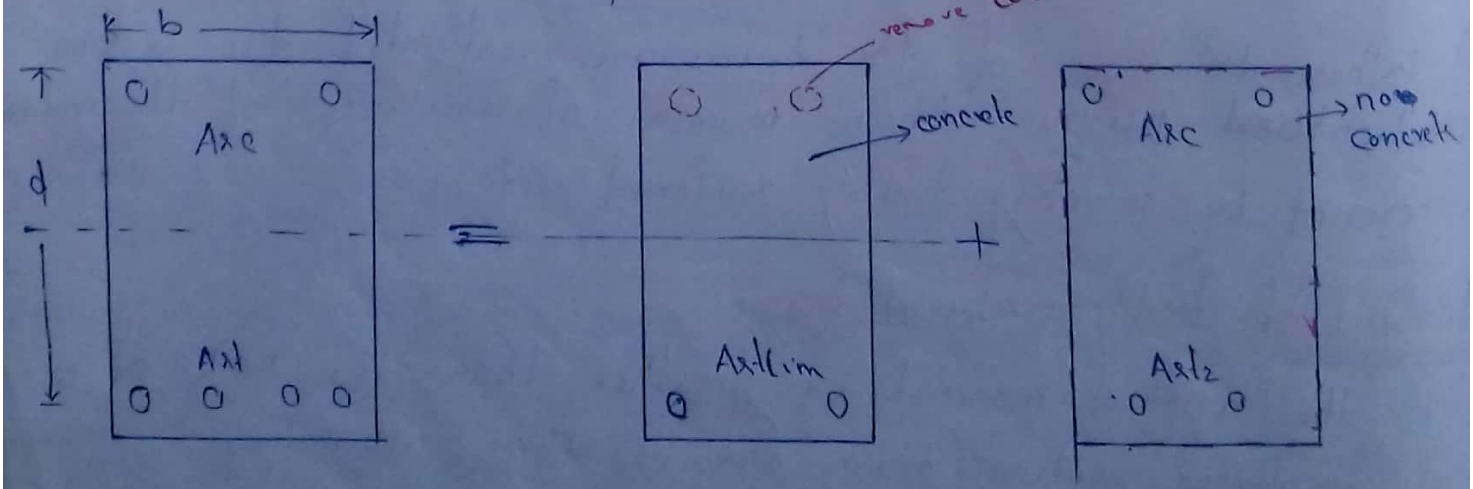
Derivation of formulae:

A doubly reinforced beam section, strain diagram & stress diagram are shown in fig. A doubly reinforced beam subjected to a moment M_u can be expressed as a rectangular sect^n with tension reinforcement $A_{st,lim}$ reinforced for balanced cond^n giving moment of resistance $M_{u,lim}$ + an auxillary sect^n reinforced with comp. reinforcement A_{sc} & tensile reinforcement A_{st2} giving a moment of resistance M_{u2} such that $M_u = M_{u,lim} + M_{u2}$



(a) Section (b) Strain diagram (c) Stress diagram

Doubly reinforced beam



Doubly reinforced sect^n (M_u) = Singly reinforced balanced sect^n ($M_{u,lim}$) + $M_{u2} = M_u - M_{u,lim}$

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Here $M_u = M_{u,lim} + M_{u2}$

$A_{st,lim}$ can be found out from $M_{u,lim}$

A_{st2} & A_{sc} ⟶ " ⟶ $M_{u2} = M_u - M_{u,lim}$

Let d' = Effective cover to compression reinforcement

$d-d'$ = lever arm for additional moment.

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 steel

Considering tension steel

$$M_{u2} = A_{st2} \times 0.87 f_y (d-d') \quad \ldots (1)$$

Considering compression steel

$$M_{u2} = A_{sc} (f_{sc} - f_{cc}) (d-d') \quad \ldots (2)$$

($f_{cc} = 0.446 f_{ck}$)

Now additional tension = additional compression

$$0.87 f_y A_{st2} = A_{sc} (f_{sc} - f_{cc})$$

$$\therefore A_{st2} = \frac{A_{sc} (f_{sc} - f_{cc})}{0.87 f_y} \quad \ldots (3)$$

Any 2 of the above eqns may be used to find out the values of A_{sc} & A_{st2} : The total tensile reinforcement

$$A_{st} = A_{st,lim} + A_{st2}$$

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To solve the above eqns, one needs the value of f_{sc} & f_{cc}.

To find f_{sc}

To find find the strain at the level of compression reinforcement.

refer to fig strain diagram

$$\frac{\varepsilon_{sc}}{0.0035} = \frac{x_{umax} - d'}{x_{umax}}$$

$$\Rightarrow \varepsilon_{sc} = 0.0035 \left(1 - \frac{d'}{x_{u,max}}\right)$$

Using this value of strain, the stress in ~~conc~~ steel can be found out from the representative stress-strain curves for reinforcement from fig 23(A) & (B) of IS456-2000.

To simplify the procedure of finding out the stress in compression reinforcement let us find out the stress for different values of $\frac{d'}{d}$

For mild steel, $x_{umax} = 0.53d$

for $\frac{d'}{d} = 0.2$

strain at the level of compression reinforcement

$$\varepsilon_{sc} = 0.0035 \left(1 - \frac{d'}{x_{u,max}}\right)$$

$$= 0.0035 \left(1 - \frac{d'}{0.53d}\right)$$

$$= 0.0035 \left(1 - \frac{0.2}{0.53}\right) = 0.00218$$

Design stress in mild steel for strain of 0.00109 & above upto failure is 217 N/mm². Therefore, for all values of $\frac{d'}{d}$ from

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0 to 0.2, the stress f_{sc} will be taken as 217 N/mm².

For HYSD steel of grade Fe 415

$$x_{u,max} = 0.48d$$

for $\frac{d'}{d} = 0.05$

strain at the level of compression steel

$$= 0.0035\left(1 - \frac{0.05}{0.48}\right) = 0.0031135$$

Stress in compression steel for this value of strain from fig 23(A) of IS456 2000 is 355 N/mm²

Stress in compression reinforcement (f_{sc}), in N/mm² in Doubly reinforced beams (Table F of SP16)

f_y (N/mm²)	$\frac{d'}{d}$			
	0.05	0.1	0.15	0.20
250	217	217	217	217
415	355	353	342	329
500	424	412	395	370
550	458	441	419	380

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Types of problems:

Two basic types of problems in doubly reinforced beam are discussed below

Type 1: To find out the moment of resistance of a given section.

Referring to fig, the equilibrium of forces yields an equation

total compression = total tension

i.e. $C_1 + C_2 = T$

$$\Rightarrow 0.36 f_{ck} b x_u - A_{sc} \times f_{cc} + A_{sc} \times f_{sc} = 0.87 f_y A_{st}$$

$$\boxed{\Rightarrow 0.36 f_{ck} b x_u + A_{sc} (f_{sc} - f_{cc}) = 0.87 f_y A_{st}}$$

for $\frac{d'}{d}$ upto 0.2, $f_{cc} = 0.446 f_{ck}$ & f_{sc} may be found out from fig 23 (A) & (B) ~~before that~~ by knowing ε_{sc}.

→ find out x_u. Find x_{umax} & type of beam.

→ If it is under reinforced, then moment of resistance of the section can be found out by taking moments of compressive forces about the centroid of tensile reinforcement.

Then $\boxed{M_u = 0.36 f_{ck} b x_u (d - 0.42 x_u) + A_{sc} (f_{sc} - f_{cc}) (d - d')}$

The value of f_{cc} is very small as compared to the value of f_{sc} & can be neglected.

$$\boxed{\text{Then } M_u = 0.36 f_{ck} b x_u (d - 0.42 x_u) + A_{sc} f_{sc} (d - d')}$$

R Moment of resistance can also calculated by taking moments of tensile forces about the centroid of compressive forces

$$\boxed{M_u = 0.87 f_y A_{st1} (d - 0.42 x_u) + 0.87 f_y A_{st2} (d - d')}$$

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→ If it is over-reinforced / balanced, the depth of neutral axis obtained will be more than $x_{u,max}$ i.e. $(x_u > x_{umax})$. In such cases $x_u = x_{u,max}$, shall be used for calculation

$$\text{i.e. } M_u = 0.36 f_{ck} b x_{u,max} (d - 0.42 x_{u,max}) + A_{sc} f_{sc} (d - d')$$

or

$$M_u = 0.87 f_y A_{st1} (d - 0.42 x_{umax}) + 0.87 f_y A_{st2} (d - d')$$

Type 2

To find out reinforcement for flexture for a given section and factored moment

(1) find out $M_{u,lim}$ & reinforcement $A_{st,lim}$ for a given section by using eqns

$$M_{u,lim} = 0.36 f_{ck} b x_{u,max} (d - 0.42 x_{u,max})$$

and $$A_{st,lim} = \frac{M_{u,lim}}{0.87 f_y (d - 0.42 x_{u,max})}$$

(2) ~~find~~ obtain moment $M_{u2} = M_u - M_{u,lim}$

(3) Find compression steel from eqn

$$M_{u2} = A_{sc} f_{sc} (d - d')$$

$$\Rightarrow A_{sc} = \frac{M_{u2}}{f_{sc} (d - d')}$$

(4) Corresponding tension steel A_{st2} may be found out from

$$A_{sc} f_{sc} = 0.87 f_y A_{st2} \Rightarrow A_{st2} = \frac{A_{sc} f_{sc}}{0.87 f_y}$$

or

$M_{u2} = 0.87 f_y A_{st2} (d - d')$ or $A_{st2} = M_{u2} / 0.87 f_y (d - d')$

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5) $A_{st} = A_{st,lim} + A_{st2}$

Problem: 1

Find the factored moment of resistance of a beam section 230mm wide x 460mm effective depth reinforced with 2-16 mm dia bars as compression reinforcement at an effective cover of 40mm & 4-20 mm dia bars as tension reinforcement as shown in fig. The materials are M20 grade concrete & mild steel reinforcement.

Soln

width of the section (b) = 230mm

effective depth of the section (d) = 460mm

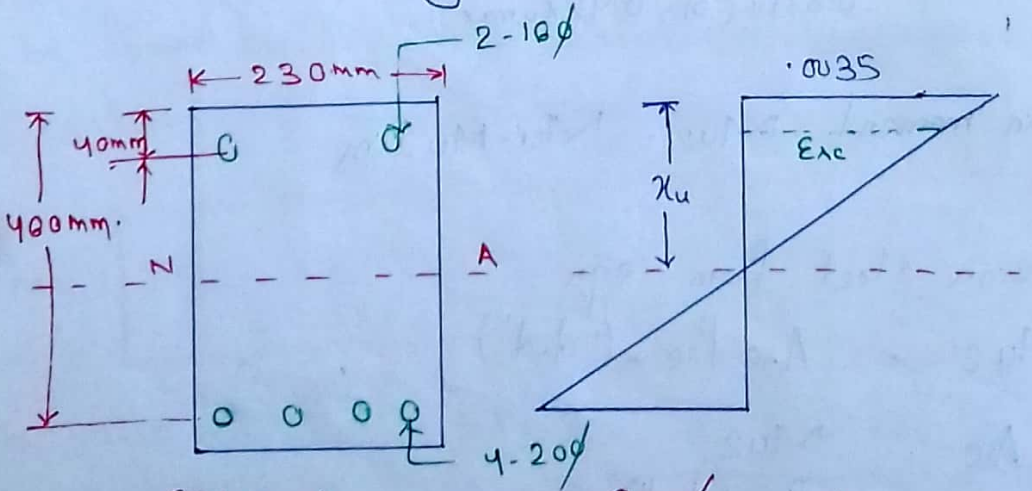
Area of compression reinforcement (A_{sc}) $= 2 \times \frac{\pi}{4} \times 16^2 = 402.12\ mm^2$

Area of tension " (A_{st}) $= 4 \times \frac{\pi}{4} \times 20^2 = 1256.64\ N/mm^2$

effective cover to compression reinforcement (d') = 40mm

characteristic strength of concrete (f_{ck}) = 20N/mm²

characteristic strength of steel (f_y) = 250N/mm²



(a) Section (b) Strain diagram

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Equating total compressive forces to total tensile forces

$$0.36 f_{ck} b x_u + A_{sc}(f_{sc} - f_{cc}) = 0.87 f_y A_{st}$$

~~$\rightarrow 0.36 \times 20 \times 230 \times x_u$~~

for $\frac{d'}{d} = \frac{40}{460} = 0.086 < 0.2$ so $f_{cc} = 0.446 f_{ck}$

$= 0.446 \times 20$

$= 8.92 \text{ N/mm}^2$

Strain at the level of compression reinforcement

$$(\varepsilon_{sc}) = 0.0035\left(1 - \frac{d'}{x_{u,max}}\right)$$

$$= 0.0035\left(1 - \frac{d'}{0.53d}\right)$$ [For Fe250 grade steel $x_{u,max} = 0.53d$]

$$= 0.0035\left(1 - \frac{40}{0.53 \times 460}\right)$$

$$= 0.00293$$

corresponding stress in compression steel can be obtained by ~~interpolation using~~ Fig 23(B) of IS 456 2000

$$f_{sc} = 0.87 f_y = 0.87 \times 250 = 217.5 \text{ N/mm}^2$$

$$\therefore 0.36 \times 20 \times 230 \times x_u + 402.12 \times (217.5 - 8.92) = 0.87 \times 250 \times 1258.64$$

$\Rightarrow x_u = 114.40 \text{ mm}$

Limiting value ($x_{u,max}$) of neutral axis $= 0.53d = 0.53 \times 460 = 243.8 \text{ mm}$

$\therefore x_u < x_{u,max}$. Hence the section is under-reinforced

so $x_u = 114.40 \text{ mm}$

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Taking moments of compressive forces about centroid of tensile reinforcement

$$M_u = 0.36 f_{ck} b x_u (d - 0.42 x_u) + A_{sc} (f_{sc} - f_{cc}) (d - d')$$

$$= 0.36 \times 20 \times 230 \times 114.40 (460 - 0.42 \times 114.40) +$$
$$402.12 \times (217.5 - 8.92) (460 -$$

$$= 113.27 \text{ KN-m}$$

Problem: 2

A rectangular beam of size 230mm wide × 500 mm effective depth is subjected to a factored moment of 200 kNm. Find the reinforcement for flexture. The materials are M20 grade concrete & HYSD reinforcement of grade Fe 415.

Sol^n

width of the section (b) = 230mm

effective depth of the section (d) = 500mm

characteristic strength of concrete (f_{ck}) = 20 N/mm²

characteristic strength of steel (f_y) = 415 N/mm²

Factored bending acting on the beam (M_u) = 200 kNm

Limiting moment of resistance for balanced section ($M_{u,lim}$)

$$= 0.36 f_{ck} b x_{u,max} (d - 0.42 x_{u,max})$$

$$= 0.36 \times 20 \times 230 \times (0.48 \times 500) \times (500 - 0.42 \times 0.48 \times 500)$$

$$= 158.66 \text{ KNm}$$

& Limiting area of steel ($A_{st,lim}$) = $\dfrac{M_{u,lim}}{0.87 f_y (d - 0.42 x_{u,max})}$

$$= \frac{158.66 \times 10^6}{0.87 \times 415 (500 - 0.42 \times 0.48 \times 500)}$$

$$= 1100.80 \text{ mm}^2$$

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Here additional moment taken by the section $(M_{u2}) = M_u - M_{u,lim}$

$= 200 - 158.66$

$= 41.34 \text{ kNm}$

Let the compression reinforcement be provided at an effective cover of 40 mm

$\therefore \frac{d'}{d} = \frac{40}{500} = 0.08$

strain at the level of compression reinforcement (ε_{sc})

$= 0.0035 \left(1 - \frac{d'}{x_{u,max}}\right)$

$= 0.0035 \left(1 - \frac{40}{0.48 \times 500}\right)$

$= 0.00292$

corresponding stress in compression steel can be obtained by fig 23(A) of IS456:2000

$f_{sc} = 0.85 f_y = 0.85 \times 415 = 352.75 \text{ N/mm}^2$

Now, considering compression steel

$M_{u2} = A_{sc} (f_{sc} - f_{cc}) (d - d')$

$\Rightarrow 41.34 \times 10^6 = A_{sc} (352.75 - 0.446 \times 20) (500 - 40)$

$\Rightarrow A_{sc} = 261.38 \text{ mm}^2$

Provide 2-20ϕ bar in compression zone $A_{sc,provided} = 2 \times \pi/4 \times 20^2 = 628.32 \text{ mm}^2$

Now, considering tension steel

$M_{u2} = A_{st_2} \times 0.87 f_y (d - d')$

$\Rightarrow 41.34 \times 10^6 = A_{st_2} \times 0.87 \times 415 (500 - 40)$

$\Rightarrow A_{st2} = 248.91 \text{ mm}^2$

$\therefore A_{st} = A_{st,lim} + A_{st2} = 1100.80 + 248.91 = 1349.71 \text{ mm}^2$

Provide 5-20ϕ bar in tension zone $A_{st,provided} = 5 \times \pi/4 \times 20^2 = 1570.80 \text{ mm}^2$

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For the designed sect^n, determine x_u by equating total compression & total tension

i.e. $0.36 f_{ck} b x_u + A_{sc}(f_{sc} - f_{cc}) = 0.87 f_y A_{st}$

$\Rightarrow 0.36 \times 20 \times 230 \times x_u + 628.32 (352.75 - 0.446 \times 20)$

$= 0.87 \times 415 \times 1570.80$

$\Rightarrow x_u = 212.02 \text{ mm}$

Also $x_{u,max} = 0.48 \times d = 0.48 \times 500 = 240 \text{mm}$

Now $x_u < x_{u,max}$

∴ The beam is under-reinforced. The design is accepted

Q Design a rectangular beam for an effective span of 6m. The super-imposed load is 60 KNm & size of the beam is limited to 30 cm × 60 cm overall. Use M20 mix & Fe415 grade steel.

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Shear, Bond & Development Length (LSM)

Shear in structural elements:

The section of a structural member may be subjected to shear force due to flexture, punching or torsion. Accordingly, the shear may be flexural shear, punching shear or torsional shear.

(1) Flexural shear:

The shear associated with change of bending moment along the span is known as flexural shear. The beams are usually subjected to flexural shear. For beams, the flexural shear is comonly referred to as 'shear'.

(2) Punching shear:

The shear associated with the possibility of punching a thin member by a concentrated load is called punching a shear. A slab carrying a concentrated wall load, a beamless floor slab supported directly by columns (called flat slab) or a footing slab carrying a concentrated column load are subjected to punching shear.

→ The flexural shear is referred to as one-way shear.
→ punching shear " " " " two-way shear

(3) Torsion shear:

When a member is subjected to torsion, it is subjected to torsion shear. In RCC structures, the torsion shear is usually accompanied with flexural shear.

* The beams are the line (uni-dimensional) elements and are usually subjected to flexural shear & ~~However~~, sometimes ~~they are subjected to all the types of shears as in~~ care of with torsion shear also. The slabs are the plate elements and usually subjected to flexural shear. However, sometimes they are subjected to all the types of shears as in case of restrained two-way slabs & the flat slabs.

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* The beams are classified as shallow & deep depending on the ratio of effective span l, to overall depth D, i.e. l/D ratio. According the beams are classified as deep if

$\frac{l}{D} < 2$ for simply supported beams

$\frac{l}{D} < 2.5$, for continuous beams.

The beams are otherwise classified as shallow.

Limit State theory:

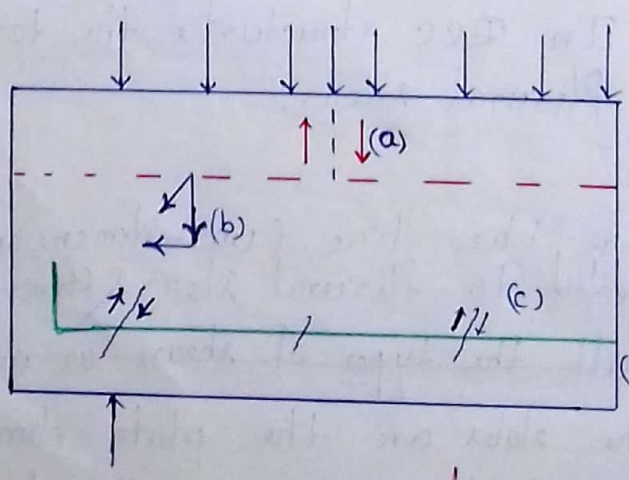
The mechanics of shear transfer in concrete beams without shear reinforcement, when concrete cracks due to shear is illustrated in fig 7.5.

* The shear is resisted by

(1) Above neutral axis shear resistance is provided by the uniform shear stress in uncracked concrete (20 to 40% of total resistance)

(2) Along the crack, shear resistance is provided by the vertical component of force due to the interlocking of aggregates (33 to 50% of total resistance)

(3) At the tensile reinforcement, shear is resisted by the dowel action of the longitudinal bars (15 to 25% of total resistance)



Shear resistance of a cracked beam

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* The behaviour of concrete is thus complicated & the shear stress is found out in limit state theory by a simple formula similar to that of elastic theory where τ_v is defined as nominal shear stress.

$$\text{Then } \tau_v = \frac{V_u}{bd}$$

where τ_v = nominal shear stress

V_u = design shear

b = width of section, which for the case of a flanged beam, is width of the web, bw

d = effective depth

Design shear strength of concrete:

The design shear strength of concrete shall be taken as follows

(1) Without shear reinforcement:

The design shear strength of concrete in beams without shear reinforcement for some cases is given in table 19 of IS456: 2000 (page 73).

* From this table we can observe that design shear strength τ_c in concrete depends on the percentage of tension steel. This is because.

(i) When the amount of tension steel increase, the depth of neutral axis increases & thus, the depth of uncracked concrete increases. This increases the capacity of concrete in shear.

(ii) When the amount of tension steel increases, the cracks formed are smaller, which improves the aggregate interlock. Also because of larger steel area the dowel action is improved. This further improves the capacity of section in shear.

* Again from table, it can be observed that shear strength of concrete (τ_c) is also related to the compressive strength of concrete.

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* When the longitudinal bars are not required to resist moment they are sometimes curtailed or bent up. If the bars are curtailed they create complicated stresses at the point of curtailment, thereby reducing the shear capacity of the section. Therefore while using table 19 of IS456:2000, any longitudinal bars which are terminated within a distance d of the section under consideration, shall not be considered for calculation of $100 A_s / bd$.

* If nominal shear stress (τ_v) does not exceed τ_c found from table 19 the section is safe for shear & shear reinforcements are not required theoretically. However, some minimum shear reinforcement shall be provided in accordance as clause 26.5.1.6.

$$\frac{A_{sv}}{b s_v} \geq \frac{0.4}{0.87 f_y}$$

* If the nominal shear stress (τ_v) exceeds the design shear strength (τ_c). The section shall be suitably reinforced with shear reinforcement in accordance as clause 40.4.

Note: $\tau_v < \tau_c$ → Section is safe for shear and shear reinforcements are not required (But minm shear reinforcement shall be provided as stirrups)

$\tau_v > \tau_c$ → Need to design for shear reinforcemt

2) With shear-reinforcement:

Shear reinforcements are provided against diagonal tension. As the concrete is strong in compression, generally a beam is safe for diagonal compression.

* However for any beam, suitably reinforced against diagonal tension, the nominal shear (τ_v) shall not exceed the max shear stress ($\tau_{c,max}$) values given in table 20 of IS456 2000. The maxm shear stress is given by $\tau_{c,max} = 0.62\sqrt{f_{ck}}$.

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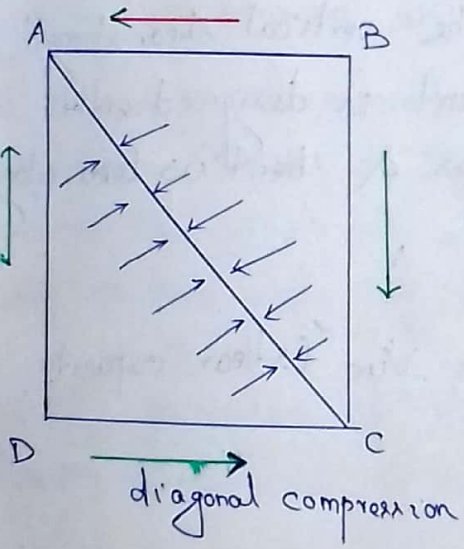
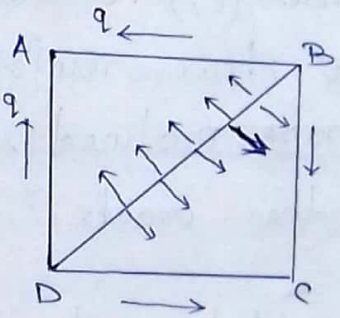
* By this provision the failure of the beam by diagonal compression is prevented. For any section if nominal shear stress (τ_v) exceeds the max^m shear stress ($\tau_{c,max}$) value, the section shall be redesigned either by using richer concrete mix or by increasing the size of the section. For practical purpose, it is recommended that nominal shear stress may not exceed 85% of the $\tau_{c,max}$ value.

Maximum shear stress, $\tau_{c,max}$ (N/mm²)

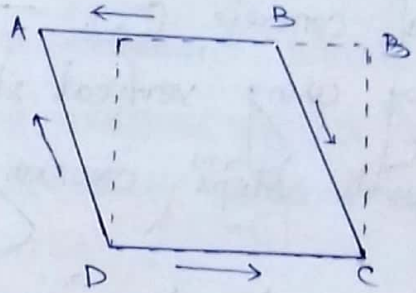
Concrete grade	M15	M20	M25	M30	M35	M40 & above
$\tau_{c,max}$ (N/mm²)	2.5	2.8	3.1	3.5	3.7	4.0

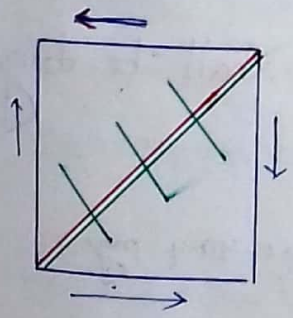


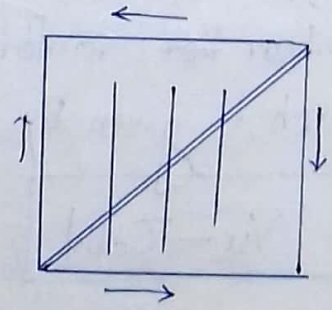
diagonal compression



diagonal tension




shear reinforcement provided diagonally


shear reinforcement provided vertically

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Design for shear:

due to shear stresses, there are cracks in concrete perpendicular to the diagonal tension. As the load increases, these cracks move upward. Most of the cracks stop when they reach the heavily stressed region of compression zone. If there is a further increase of load, a critical diagonal crack may tear through the beam.

As a result, the beam splits into 2 sections & a sudden brittle failure occurs. Thus the diagonal tension failure occurs without giving notice & may cause the beam to collapse completely. Therefore Therefore beam shall be suitably reinforced with shear reinforcements.

* The shear failure of the section occurs on a vertical plane where the nominal shear stress (τ_v) exceeds the vertical shear strength of concrete (τ_c). The shear reinforcement is designed either by using vertical stirrups, inclined stirrups & bent up bars along with stirrups crossing the cracks.

* Concrete alone can resist a shear force V_{uc} (shear capacity of concrete) which is given by

$$V_{uc} = \tau_c b d$$

According to IS:456, the shear ~~for~~ reinforcement shall be designed for a shear force, V_{us}, which is given by

$$\boxed{V_{us} = V_u - V_{uc} = V_u - \tau_c b d}$$ clause 40.4 page-73

where V_{us} = strength of shear reinforcement

V_u = shear force due to applied loads.

$V_{uc} = \tau_c b d$ = shear capacity of concrete without shear reinforcement.

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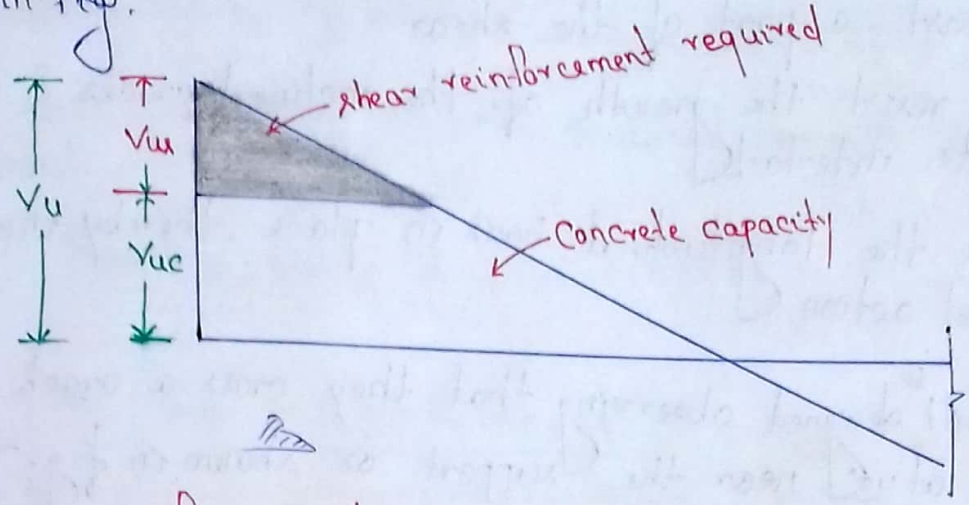
The shearing of shear resistance in simply supported RCC beam is shown in fig.



Shear reinforcement in beams

Shear reinforcements shall be designed to resist a shear force Vus. As concrete is weak in tension, the shear failure is caused by diagonal tension with cracks running at 45° to the beam axis. The shear reinforcement shall be provided by reinforcement which cross the cracks. These shear reinforcement ~~shall be provided by~~ minimize the size of diagonal tension crack & carry diagonal tensio stress from one side of the crack.

* The provision of shear reinforcement is made by any of the following forms

(1) Vertical stirrups
(2) Inclined stirrups
(3) Bent up bars along with stirrups

These are now briefly discussed.

(1) Vertical stirrups

Stirrups are the commonly used shear reinforcement These consist of a series of vertical closed bars spaced along th beam span as shown in fig. The first stirrups shall be provided with in 50mm from the face of supports.

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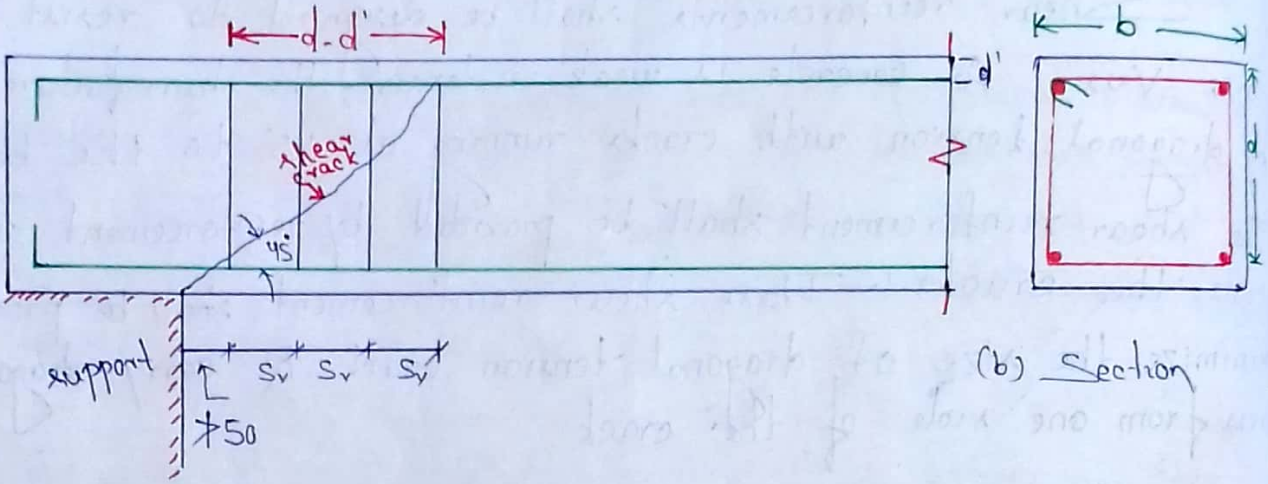
The primary functions of the stirrups are

(i) to resist a part of the shear

(ii) to resist the growth of the inclined cracks & improve aggregate interlock

(iii) to tie the longitudinal bars in place, thereby increase the dowel action.

The stirrups will be designed observing that they cross a crack, say the first crack at 45° near the support as shown in fig. The stirrups are spaced at a distance S_v apart & they cross the crack.



Vertical stirrups as shear reinforcement

Let A_{sv} = area of legs of one stirrup

f_y = characteristic strength of stirrup (or bent up bar) which shall not be greater than 415 N/mm²

$0.87 f_y$ = design tensile strength of shear reinforcement.

V_{us} = strength of shear reinforcement

Now, The horizontal length of crack = $d - d' \simeq d$ (d' neglected)

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No of stirrups crossing the crack $n = d/s_v$

∴ Shear taken by stirrups $= \frac{d}{s_v} \times A_{sv} \times 0.87 f_y$

This shall be equal to the required strength V_{us} of the shear reinforcement.

Equating $V_{us} = \frac{d}{s_v} \times A_{sv} \times 0.87 f_y$

$$\Rightarrow V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

cl. 40.4(a) page 73

(2) Inclined stirrups:

~~It consists of a series of inclined stirrups or~~

A series of inclined stirrups near the support crossing a 45° crack shown in fig.

If A_{sv} is the area of legs of inclined stirrups & $0.87 f_y$ is the design tensile strength in the shear reinforcement, the tensile force in an inclined stirrup $= 0.87 f_y A_{sv}$

For inclined stirrups or a series of bars bent-up at different cross-sections, the spacing (s_v) can be found from following eqn.

$$V_{us} = \frac{0.87 f_y A_{sv} d (\sin\alpha + \cos\alpha)}{s_v}$$

cl. 40.4(b) page. 73

* The inclined stirrups are very effective in carrying flexural shear & restrain the inclined crack very effectively. However in case of stress reversal (due to earthquake loads for example), the direction of crack will be reversed & inclined stirrups become ineffective

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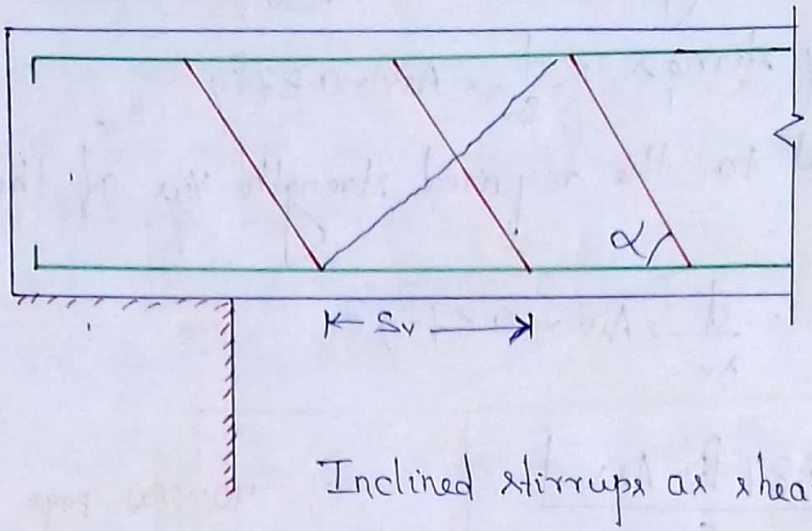


Inclined stirrups as shear reinforcement.

(3) Bent bars.

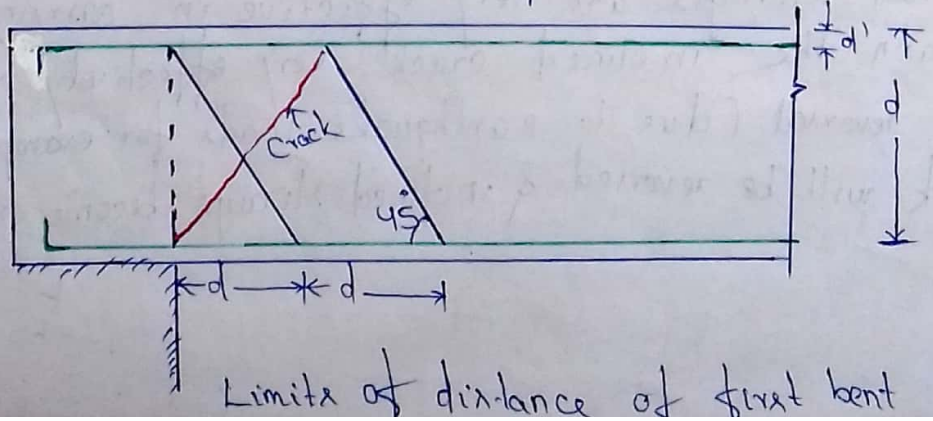
For single bar or single group of parallel bars, all bent up at the same cross-section, The strength of shear reinforcement is given by

$$V_{us} = 0.87 f_y A_{sv} \sin\alpha$$

* Usually the bars are bent at 45° or 60° to the longitudinal axis of the beam.

* Note that if a bar is bent at a distance from the support, less than d the bent portion of the bar will enter into the support which is not practicle. Therefore the first bent, if its contribution to resist the shear at support is to be counted, should start at a distance from support bet^n d & 2d.

* Note that if a second bar or a group of bars is to be bent, this shall have to start at a distance d or less than d from the first bent bar as restricted by the code.



Limits of distance of first bent from support

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Maximum Spacing:

The shear reinforcements are provided to prevent the shear cracks in the beam. The horizontal distance bet^n two successive cracks is approximately equal to the effective depth d. Stirrups shall be provided such that they cross the crack & also no crack shall remain unreinforced.

* The maximum spacing of shear reinforcement measured along the axis of the member shall not exceed
  - ▷ 0.75 d for vertical stirrups
  - ▷ d for inclined stirrups at 45°, — cl 26.5.1.5 page 47

  where d is the effective depth of the section under consideration.

In no case shall the spacing exceed 300 mm.

Minimum shear reinforcement

Minimum shear reinforcement in the form of vertical stirrups shall be provided such that

$$\frac{A_{sv}}{b\,s_v} \geq \frac{0.4}{0.87 f_y}$$ — cl - 26.5.1.6 page(48)

$$\Rightarrow S_v \leq \frac{0.87 f_y A_{sv}}{0.4\,b}$$

where

A_{sv} = total c/s area of stirrup legs effective in shear

S_v = stirrup spacing along the length of the member

b = breadth of the beam or breadth of the web of flanged beam

f_y = characteristic strength of the stirrup reinforcement in N/mm² which shall not be taken greater than 415 N/mm²

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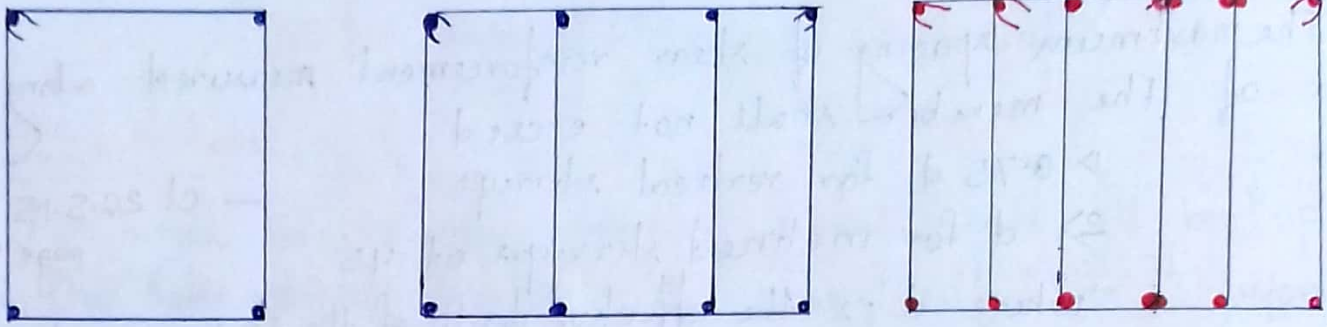
Forms of shear reinforcements

The stirrups are usually 2 legged. However stirrups of 4 legged, six-legged etc also can be formed as shown in fig. More than 2 legged stirrups are used for heavy shear. Unless otherwise specified, stirrups are always 2 legged.



(a) Two-legged stirrup (b) four-legged stirrup (c) Six-legged stirrup

Problem: 1.

A simply supported reinforced concrete beam is 250 mm wide and 500mm effective depth and is reinforced with 5 bars of 18 mm diameter as tensile steel. If the beam is subjected to a factored shear of 62.5 kN at the support, find the nominal shear stress at the support & design the shear reinforcement. Use M20 concrete & Fe 250 steel.

Sol?

Given data:

width of beam (b) = 250mm

effective depth " " (d) = 500mm

Area of tensile steel (A_{st}) = $5 \times \pi/4 \times 18^2$ = 1272.35 mm^2

Factord shear force at the support (V_u) = 62.5kN

Characteristic compressive strength of concrete (f_{ck}) = 20N/mm^2

" strength of steel (f_y) = 250N/mm^2

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Nominal shear stress(τ_v) $= \frac{V_u}{bd} = \frac{62.5 \times 10^3}{250 \times 500} = 0.5\ N/mm^2$

Percentage of tensile steel provided $= \frac{A_{st}}{bd} \times 100 = \frac{1272.35}{250 \times 500} \times 100$

$= 1.02\ \%$

So design strength of concrete (τ_c) from table 19 of IS 456 : 2000 $= 0.62 + \frac{0.67 - 0.62}{1.25 - 1.00} \times (1.02 - 1.00)$

$= 0.6288\ N/mm^2$

As $\tau_v < \tau_c$, the section is safe for shear & shear reinforcements are not required theoretically. However, some minimum shear reinforcement shall be provided in accordance as clause 26.5.1.6 given by

$$\frac{A_{sv}}{b s_v} \geq \frac{0.4}{0.87 f_y}$$

$$\Rightarrow \frac{A_{sv} \times 0.87 f_y}{0.4 b} \geq S_v$$

$$\Rightarrow S_v \leq \frac{A_{sv} \times 0.87 f_y}{0.4 b}$$

[using 6mm diameter of Fe 250 grade, two legged stirrups]

$$\Rightarrow S_v \leq \frac{\left(2 \times \pi/4 \times 6^2\right) \times 0.87 \times 250}{0.4 \times 250}$$

$$\Rightarrow S_v \leq 122.99\ mm \simeq 123\ mm$$

The spacing of stirrups (S_v) shall not exceed

i) $0.75d = 0.75 \times 500 = 375\ mm$

ii) 300 mm

iii) 123 mm as calculated above

Provide 6mm diameter two legged stirrups @ 120 mm c/c.

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Problem: 2

A simply supported reinforced concrete beam is 250mm wide and 500 mm deep to the centre of tensile reinforcement & is reinforced with 4 bars of 20mm diameter as tensile steel. If the beam is subjected to a factored shear force of 95KN at the support, design the shear reinforcement consisting of stirrups. Use M20 concrete & Fe415 steel.

Sol^n

Width of the section (b) = 250mm

effective depth (d) = 500mm

Area of tensile steel (A_{st}) = $4 \times \pi/4 \times 20^2$ = 1256.64 mm^2

factored shear force at the support (V_u) = 95KN

characteristic compressive strength of concrete (f_{ck}) = 20 N/mm^2

" strength of steel (f_y) = 415 N/mm^2

Nominal shear stress (τ_v) = $\frac{V_u}{bd} = \frac{95 \times 10^3}{250 \times 500}$ = 0.76 N/mm^2

Percentage of steel provided (p_t) = $\frac{A_{st}}{bd} \times 100$

= $\frac{1256.64}{250 \times 500} \times 100$

= 1.01%

So design shear strength of concrete (τ_c) from table 19 of IS 456:2000 = $0.62 + \frac{0.67 - 0.62}{1.25 - 1.00} \times (1.01 - 1.00)$

= 0.622 N/mm^2

Now $\tau_v > \tau_c$, therefore shear reinforcement shall be designed.

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Shear resistance of concrete $(V_{uc}) = \tau_c bd = 0.622 \times 250 \times 500$

$= 77.75\,KN$

shear to be resisted by stirrups

$V_{us} = V_u - \tau_c bd = 95\,KN - 77.75\,KN$

~~$= 95 \times 10^3 - (0.622 \times 250 \times 500)$~~

$= 17.25\,KN$

Using 6mm diameter two-legged vertical stirrups. (Fe415 grade steel)

spacing of stirrups

$$(S_v) = \frac{0.87 f_y A_{sv} d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times (2 \times \pi/4 \times 6^2) \times 500}{17.25 \times 10^3}$$

$= 591.79\,mm$

The spacing of stirrups shall not exceed

a) $0.75d = 0.75 \times 500 = 375\,mm$

b) 300 mm

c) due to minm shear reinforcement required

$$= \frac{0.87 f_y A_{sv}}{0.4 b} = \frac{0.87 \times 415 \times (2 \times \pi/4 \times 6^2)}{0.4 \times 250} = 204.17\,mm$$

d) required spacing $= 591.79\,mm$

provide 6mm ϕ two legged stirrups about 200 mm c/c.

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Problem: 3

A simply supported reinforced concrete beam is 250mm wide 500mm effective depth and is reinforced with 5 bars of 18mm dia as tensile steel. If the beam is subjected to a factored shear of 140kN at the support & two of the main bars are cranked up at 45°, find the spacing of 2 legged 6mm diameter stirrups at support. Use M20 grade concrete & Fe250 steel

Sol^n

Width of the section (b) = 250mm

effective depth " (d) = 500mm

Area of tensile steel $(A_{st}) = 5 \times \pi/4 \times 18^2 = 1272.35 mm^2$

Factored shear force at the support $(V_u) = 140 kN$.

Characteristic strength of concrete $(f_{ck}) = 20 N/mm^2$

" " " steel $(f_y) = 250 N/mm^2$.

total c/s area of stirrup legs $(A_{sv}) = 2 \times \pi/4 \times 6^2 = 56.55 mm^2$

Nominal shear stress $(\tau_v) = \frac{V_u}{bd} = \frac{140 \times 10^3}{250 \times 500} = 1.12 N/mm^2$

percentage of tensile steel available after cranking up two bars

$$= \frac{A_{st}}{bd} \times 100 = \frac{(3 \times \pi/4 \times 18^2) \times 100}{250 \times 500} = 0.61\%$$

So design shear strength of concrete (τ_c) from table 19 of IS 456: 2000

$$= 0.48 + \frac{0.56 - 0.48}{0.75 - 0.50} \times (0.61 - 0.50)$$

$$= 0.515 N/mm^2$$

& $\tau_{cmax} = 2.8 N/mm^2$

Here $\tau_v > \tau_c < \tau_{c,max}$, therefore shear reinforcement shall be designed.

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Shear resistance of the 2 cranked up bars

$= 0.87 f_y A_{sv} \sin\alpha = 0.87 \times 250 \times 2 \times \pi/4 \times 16^2 \times \sin 45^\circ$

$= 78.27\ kN$

Shear resistance of concrete $(V_{uc}) = \tau_c b d = 0.5152 \times 250 \times 500$

$= 64.40\ kN$

shear resistance to be provided by shear reinforcement

$V_{us} = V_u - V_{uc} = 140 - 64.40 = 75.6\ kN$

Shear resistance to be provided by bent up bars

$= 50\% V_{us} = 50\% \times 75.6$

$= 37.8\ kN$

Shear resistance to be provided by stirrups $= 50\% V_{us}$

$= 50\% \times 75.6$

$= 37.8\ kN$

Spacing of 2 legged 6 mm diameter vertical stirrups (S_v)

$$= \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87 \times 250 \times 2 \times \pi/4 \times 6^2 \times 500}{37.8 \times 10^3}$$

$= 162.69\ mm$

The spacing of stirrups shall not exceed

1) $0.75d = 0.75 \times 500 = 375\ mm$

2) 300 mm

3) due to min^m shear reinforcement required

$$= \frac{0.87 f_y A_{sv}}{0.4 b} = \frac{0.87 \times 250 \times (2 \times \pi/4 \times 6^2)}{0.4 \times 250} = 122.99\ mm$$

4) required spacing = 162.69 mm

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Provide 6mm ϕ two legged stirrups about 120 mm c/c

Problem: 4

Determine the shear capacity of the beam section in each of the following cases

(a) Size 230mm x 500 effective depth reinforced with 3 no 16mm diameter Fe 415 grade reinforcement & 6mm ϕ M.S. Stirrups @ 130mm c/c. Concrete is of M20 grade.

(b) Size 230mm x 720mm effective depth reinforced with 5 no 16 mm diameter Fe 415 grade reinforcement (3 straight + 2 bent at 45°) with 8mm # stirrups at 300 mm c/c Fe 415 grade. Consider M20 grade concrete.

(c) section same as (b) but stirrups 10mm # Fe 415 grade at 300mm c/c.

Soln

(a) Width of section (b) = 230mm

effective depth of section (d) = 500mm

Area of tensile steel (A_{st}) $= 3 \times \pi/4 \times 16^2 = 603.19 mm^2$

spacing of stirrup (S_v) = 130mm

total c/s area of stirrup legs (A_{sv}) $= 2 \times \pi/4 \times 6^2 = 56.55 mm^2$

Percentage of tensile steel (P_t) $= \frac{100 A_{st}}{bd} = \frac{603.19}{230 \times 500} \times 100$

$= 0.525$

So design shear strength of concrete (τ_c) from table 19 of IS456:2000 $= 0.48 + \frac{0.56 - 0.48}{0.75 - 0.50} \times (0.525 - 0.50) = 0.488$ N/mm²

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Shear to be resisted by vertical stirrups (V_{us1})

$$= \frac{0.87 f_y A_{sv} d}{S_v} = \frac{0.87 \times 415 \times 100.53 \times 720}{300}$$

$$= 87.11 \text{ kN}$$

shear to be resisted by bent up bar (V_{us2}) $= 0.87 f_y A_{sv} \sin\alpha$

$$= 0.87 \times 415 \times 2 \times \pi/4 \times 16^2 \times \sin 45^\circ$$

$$= 102.66 \text{ kN}$$

but $V_{us2} \not> V_{us1}$

$\therefore V_{us2} = 87.11 \text{ kN}$

Total shear capacity $= V_{uc} + V_{us1} + V_{us2}$

$= 68.724 + 87.11 \text{ kN} + 87.11 \text{ kN}$

$= 242.94 \text{ kN}$

(c) Same data as (b) question except

spacing of stirrup (S_v) = 300mm

total c/s area of stirrup legs (A_{sv}) $= 2\pi/4 \times 10^2 = 157.08$ mm^2

Shear resistance of concrete (V_{uc}) $= \tau_c bd = 56.12$ kN ---- (same as b)

Shear to be resisted by vertical stirrups (V_{us1})

$$= \frac{0.87 f_y A_{sv} d}{S_v} = \frac{0.87 \times 415 \times 157.08 \times 720}{300}$$

$$= 136.11 \text{ kN}$$

shear to be resisted by bent up bars (V_{us2}) = 102.66 kN ---- same as (b)

Here $V_{us2} < V_{us1}$

$\therefore V_{us2} = 102.66$ kN

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∴ Total shear capacity = $V_{uc} + V_{us1} + V_{us2}$

= 56.12 KN + 136.11 KN + 102.66 KN

= 294.89 KN

Problem: 5

A simply supported normall tee beam of 4.5m clear span is loaded with characteristic load of 40KN/m. It is reinforced with 4no. 20mm dia bars at support. The section of the beam is 230mm wide × 560mm effective depth. Design the shear reinforcement at the support. The materials are M20 grade concrete & HYSD reinforcement of grade Fe415.

Sol^n

clear span of the beam = 4.5m

characteristic load acting on the beam = 40KN/m

Area of tensile steel (A_{st}) = $4 \times \frac{\pi}{4} \times 20^2$ = 1256.64 mm^2

width of the section (b) = 230mm

effective depth of the section (d) = 560mm

characteristic compressive strength of the concrete (f_{ck}) = 20 N/mm^2

characteristic strength of steel (f_y) = 415 N/mm^2

(a) checking the max^m shear stress at the face of the support:

The factored load = 1.5 × 40 = 60KN/m

shear force at the support (V_u) = $\frac{60 \times 4.5}{2}$ = 135KN

Nominal shear stress (τ_v) = $\frac{V_u}{bd} = \frac{135 \times 10^3}{230 \times 560}$ = 1.05 N/mm^2

Max^m shear stress ($\tau_{c,max}$) for M20 grade concrete = 2.8 N/mm^2 > 1.05 N/mm^2 OK

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(b) Shear stress at the critical section

The beam is normal tee beam, i.e., the beam ends are confined by compressive reaction. The critical section for shear at a distance d from the face of the support.

Critical shear (V_u) = $135 - 0.56 \times 60$

= 101.4 KN

& Nominal shear stress (τ_v) = $\frac{101.4 \times 10^3}{230 \times 560}$ = 0.787 N/mm²

(c) Design shear strength of section:

Area of tensile steel (A_{st}) = $4 \times \pi/4 \times 20^2$ = 1256.64 mm²

Percentage of tensile steel (p_t) = $\frac{A_{st}}{bd} \times 100$

= $\frac{1256.64}{230 \times 560} \times 100$

= 0.976

Design shear strength of concrete (τ_c) from table 19 of IS456:2000 = $0.56 + \frac{0.62 - 0.56}{1.00 - 0.75} \times (0.976 - 0.75)$

= 0.614 N/mm²

Now $\tau_v > \tau_c$. So shear design is necessary

(d) Minimum shear reinforcement:

Assuming 8 mm # 2 legged stirrups.

The spacing of min^m shear reinforcement is given by

$S_v \leq \frac{0.87 f_y A_{sv}}{0.4 b}$

$\leq \frac{0.87 \times 415 \times 2 \times \pi/4 \times 8^2}{0.4 \times 230}$

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$S_v \leq 392.53\text{mm}$

) Simplified approach

At d from face, $V_u = 101.4\text{KN}$, $\tau_v = 0.787\ \text{N/mm}^2$, & $\tau_c = 0.614\ \text{N/mm}^2$

Shear capacity of concrete $V_{uc} = \tau_c b d = 0.614 \times 230 \times 560$

$= 79.08\text{KN}$

The shear resisted by stirrups $(V_{us}) = V_u - V_{uc}$

$= 101.4 - 79.08 = 22.32\text{KN}$

Using 8mm # 2legged stirrups, the spacing is given by

$$S_v = \frac{0.87 f_y A_{sv} d}{V_{us}}$$

$$= \frac{0.87 \times 415 \times 2 \times \pi/4 \times 8^2 \times 560}{22.32 \times 10^3}$$

$= 910.67\text{mm}$

spacing should not exceed

i) $0.75d = 0.75 \times 560 = 420\text{mm}$

ii) 300mm

(iii) due to min^m shear reinforcement = 392.53mm

(iv) 910.67mm as designed above

So provide 8mm # @ 300mm c/c.

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Bond & development length:

While analysing & designing the reinforced concrete structures the basic assumption is that there is a perfect bond betⁿ concrete & ie. there is absolutely no slippage betⁿ the concrete & steel. The grip of the reinforcement & concrete due to adhesion or bearing is termed as bond.

* The magnitude of the bond stress at a point is called local bond stress which varies with the bending moment. Local bond is therefore, also termed as the flexural bond.

* To transfer the design force from steel to concrete or vice versa, the reinforcement is anchored or embeded in concrete for some length. A length of reinforcement embedded in concrete so that it can develop the stress by bond is termed as development length. The anchorage length is thus equal to development length.

Development length : pull out test.

To find out development length, consider a pull out test as indicated in fig.

* A rod of diameter ϕ is embedded in concrete. An axial pull 'P' is applied & increased uniformly.

* The pull out test indicates that the maximum stress is induced in the material at the point where the bar enters the concrete & it diminishes to zero at the end of the bar. However for calculation of development length average bond resistance is considered.

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Let τ_{bd} = average design bond stress in concrete.

σ_s = working stress in bar.

ϕ = diameter of bar.

Then equating bond resistance of concrete = strength of bar

$$\tau_{bd} \times (\pi \phi) \times L_d = \sigma_s \times \frac{\pi \phi^2}{4}$$

$$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}}$$

where L_d is known as development length.

The factors influencing the stress transfer by bond are

1> Adhesion of concrete & steel

2> Shear strength of concrete

3> interlocking of surface ribs of HYSD (CTD or TMT) bars with concrete

Code Provision

According to IS:456, the bond stresses are assumed to be uniform over the effective surface area of the bar. Thus according to clause 26.2, the calculated tension or compressio in any bar at any section shall be developed on each side of the section by an appropriate development length or end anchorage or by a combination thereof, where the development length is defined as

$$L_d = \frac{\phi \sigma_s}{4 \tau_{bd}} \quad \ldots\ldots \text{clause 26.2.1. (page 42)}$$

where

ϕ = nominal dia of the bar

σ_s = stress in the bar at the sect^n considered at design load

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τ_{bd} :- design bond stress given in clause 26.2.1.1

Design bond stress (τ_{bd}) for plain bars in tension

Grade of concrete	M20	M25	M30	M35	M40 & above
Design bond stress τ_{bd} (N/mm²)	1.2	1.4	1.5	1.7	1.9

Note 1 : τ_{bd} shall be increased by 25% for bars in compression

Note 2 : For HYSD bars, the value of τ_{bd} shall be increased by 60%

* It is also stated that development length includes the anchorage values of the hook in tension reinforcements. For bars of section other than circular, the development length should be sufficient to develop the stress in the bar by bond.

bars in bundled in contact (not in syllabus)

Bars larger than 36 mm diameter shall not be bundled except in columns. The bundled bars are shown in fig(1)

* The effective perimeter of a single bar in bond is its actual perimeter $\pi\phi$. When the bars are bundled, effective perimeter having a bond with concrete is decreased. This is shown in fig(2)

* Thus total effective perimeter for a group of bars is less than the sum of perimeters of individual bars. Therefore the development length of each bar of bundled bars shall be increased by 10% for 2 bars in contact, 20% for 3 bars in contact & 33% for 4 bars in contact, for the individual bar.

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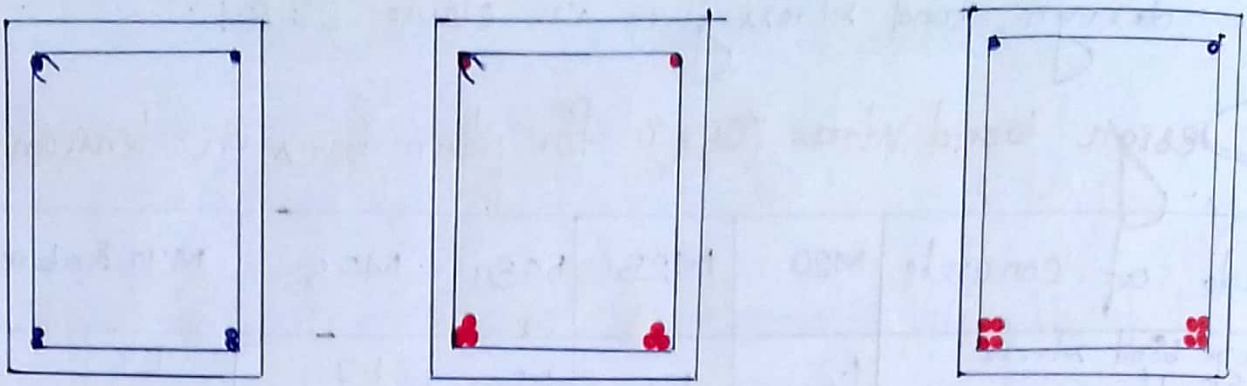
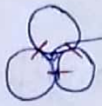


(a) Two-bars in contact (b) Three bars in contact four bars in contact

Fig(1) bundled bars



Ineffective perimeter in bond.

fig(2) bundled bars

Anchoring reinforcement

The bars are anchored in tension, compression or shear as shown below.

(1) Anchoring bars in tension. (cl 26.2.2. page 43)

(i) Deformed bars may be used without end anchorages provided that development length requirement is satisfied. Hooks should be normally provided for plain bars in tension.

(ii) Bends: The anchorage value of bend shall be taken as 4 times the diameter of the bar for each 45° bend subjected to a max^m of 16 times the diameter of the bar.

(iii) Hooks: The anchorage value of a standard U type hook shall be equal to 16 times the diameter of the bar.

Standard hook & bend are shown in fig

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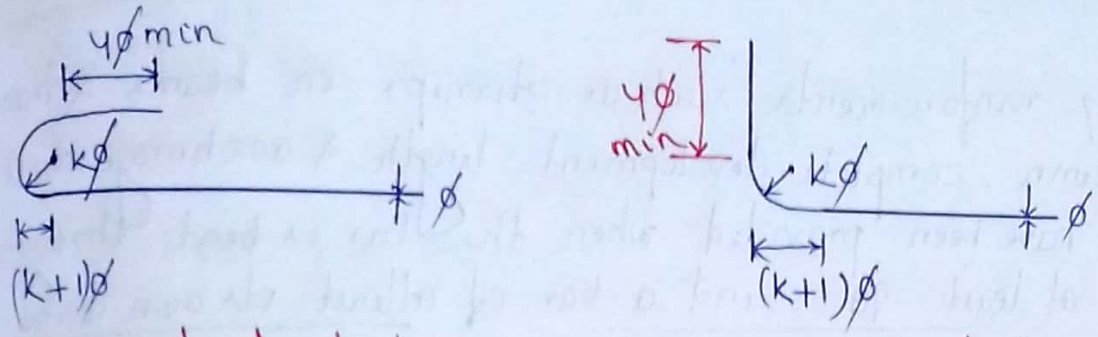

(a) Standard hook (b) Standard 90° bend

Standard hook & bend

* The value of k shall be taken as follows

Type of steel	Minimum value of k
mild steel	2
Cold worked steel	4

(2) 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 & straight lengths beyond bends if provided for bars in compression, shall be considered for development length.

(3) Anchoring bars in shear:

(i) Inclined bars:

The development length shall be as for bars in tension. This length shall be measured as under:

In the tension zone, from the end of the sloping or inclined portion of the bar.

In the compression zone, from the mid depth of the beam

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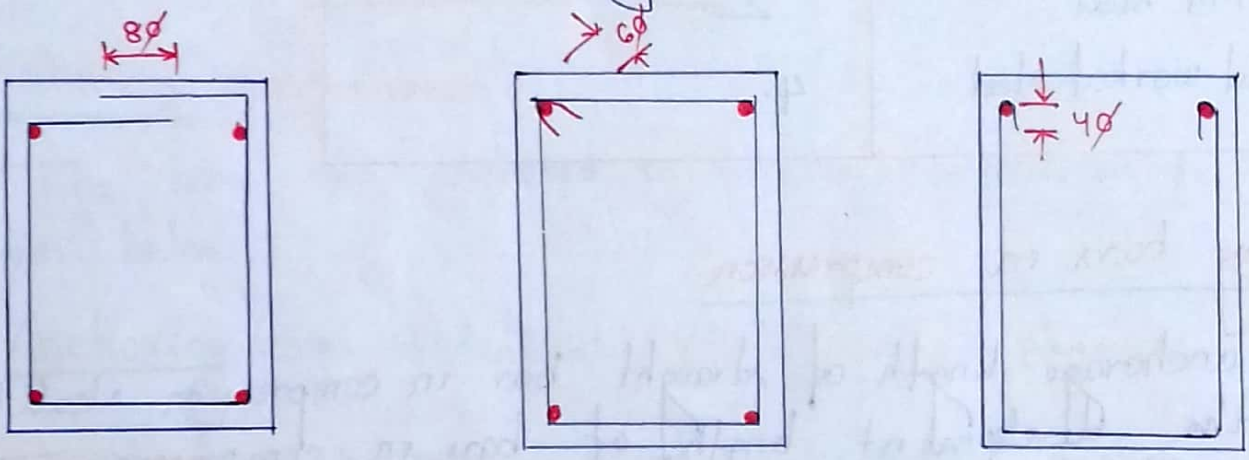
(ii) Stirrups:

For secondary reinforcements such as stirrups in beams & transverse ties in a column, complete development length & anchorage shall be deemed to have been provided when the bar is bent through an angle of at least 90° round a bar of atleast its own diameter & is continued beyond the end of the curve for a length of at least 8ϕ or when the bar is bent through an angle of 135° & is continued beyond the end of the curve for a length of at least six diameters (6ϕ) or when the bar is bent through an angle of 180° and is continued beyond the end of the curve for a length of atleast 4ϕ.



Anchoring vertical stirrups

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Problem: 1

Calculate the anchorage length in tension and compression for
(a) a single mild steel bar of diameter ϕ in concrete of grade M20
(b) an HYSD bar of grade Fe415 of diameter ϕ in concrete of grade M20.

Soln

(a) Mild steel bar

(1) Tension

Design stress $(\sigma_s) = 0.87 f_y = 0.87 \times 250 = 217.5 N/mm^2$

design bond stress (τ_{bd}) for M20 grade concrete $= 1.2 N/mm^2$

anchorage length = development length

$$= \frac{\phi \times \sigma_s}{4\tau_{bd}} = \frac{\phi \times 0.87 f_y}{4\tau_{bd}} = \frac{\phi \times 217.5}{4 \times 1.2}$$

$$= 45.3\phi$$

$$\simeq 46\phi$$

(2) Compression

(i) Flexural compression (beam bars)

design bon stress $(\sigma_s) = 0.87 f_y = 0.87 \times 250 = 217.5 N/mm^2$

design bond stress $(\tau_{bd}) = 1.2 \times 1.25$ (for compression)
$= 1.5 N/mm^2$

anchorage length = development length (L_d)

$$= \frac{\phi \sigma_s}{4\tau_{bd}} = \frac{\phi \times 217.5}{4 \times 1.5}$$

$$= 36.25\phi \simeq 37\phi$$

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(ii) Direct compression (column bars)

design stress $(\sigma_s) = 0.75 f_y = 0.75 \times 250 = 187.5 N/mm^2$

design bond stress (τ_{bd}) $= 1.2 \times 1.25$ (for compression)
$= 1.5 N/mm^2$

$$L_d = \frac{\phi \sigma_s}{4\tau_{bd}} = \frac{\phi \times 187.5}{4 \times 1.5} = 31.25\phi \simeq 32\phi$$

(b) HYSD bar of grade Fe415 & M20 grade concrete

(1) Tension

Design ~~bond~~ stress $(\sigma_s) = 0.87 f_y = 0.87 \times 415 = 361.05 N/mm^2$

design bond stress $(\tau_{bd}) = 1.2 \times 1.6$ (for HYSD bars) $= 1.92 N/mm^2$

Anchorage length = development length

$$L_d = \frac{\phi \sigma_s}{4\tau_{bd}} = \frac{\phi \times 361.05}{4 \times 1.92} = 47.01\phi$$
$$\simeq 47\phi$$

(2) Compression

(i) flexural compression (beam bars)

design stress, $(\sigma_s) = 0.87 f_y = 0.87 \times 415 = 361.05 N/mm^2$

design bond stress $(\tau_{bd}) = 1.2 \times 1.6$ (for HYSD bar) $\times 1.25$ (for compression)
$= 2.4 N/mm^2$

$$L_d = \frac{\phi \sigma_s}{4\tau_{bd}} = \frac{\phi \times 361.05}{4 \times 2.4} = 37.61\phi$$
$$\simeq 38\phi$$

(ii) Direct compression (column bars)

design stress $(\sigma_s) = 0.75 f_y = 0.75 \times 415 = 311.25 N/mm^2$

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design bond stress (τ_{bd}) = 1.2 × 1.6 (for HYSD bar) × 1.25 (for compression)

= 2.4 N/mm²

$$L_d = \frac{\phi \sigma_s}{4\tau_{bd}} = \frac{\phi \times 311.25}{4 \times 2.4} = 32.42\phi \simeq 33\phi$$

Problem: 2

Calculate the anchorage length of a group of 3 bundled bars in contact & of equal diameter in tension. Concrete grade is M20 & mild steel reinforcement

Soln

Design stress (σ_s) due to tension = $0.87 f_y$ = 0.87 × 250 = 217.5 N/mm²

Design bond stress (τ_{bd}) for M20 grade concrete = 1.2 N/mm²

Anchorage length = development length = $\frac{\phi \sigma_s}{4\tau_{bd}} = \frac{\phi \times 0.87 f_y}{4\tau_{bd}}$

$= \frac{\phi \times 217.5}{4 \times 1.2}$

$= 45.3\phi$ for single bar.

For 3 bars in bundle, the development length for each bar

$L_d = 45.3\phi \times 1.2 = 54.36\phi \simeq 55\phi$.

Problem: 3

The main reinforcement of one-way footing is shown in fig. Verify the anchorage of main bars. The point of maximum stress is assumed to occur at the face of wall. The materials are M20 grade concrete & HYSD reinforcement of grade Fe415.

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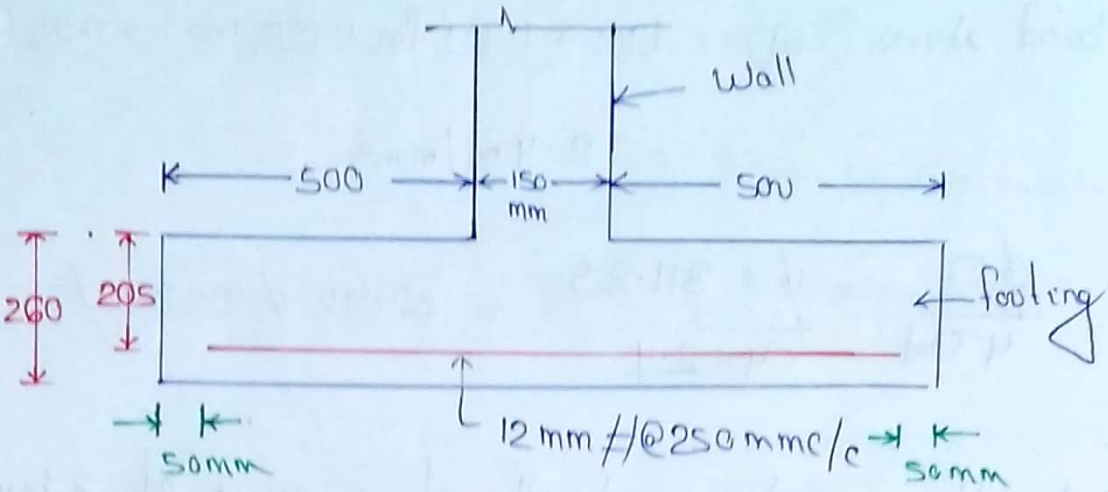

Soln

The anchorage available = 500 − 50 = 450 mm

The anchorage required $(L_d) = \frac{\phi \sigma_s}{4\tau_{bd}} = \frac{\phi \times 0.87 f_y}{4\tau_{bd}}$

$= \frac{\phi \times 0.87 \times 415}{4 \times 1.2 \times 1.6}$

$= 47.01\phi$

$= 47.01 \times 12 = 564.12$ mm

$\simeq 564$ mm

The arrangement is thus not satisfactory. To have proper anchorage ement in this case hook or bend may be used. The other alternative is to use the lower size diameter.

Problem: 4

Check the anchorage of the bent bar of fig(1), if shear taken by them is 50kN. 2 bars are bent about 560mm from support. In tension zone, sufficient length is available. In compression zone, the available anchorage is calculated in fig(1). The materials are M20 grade concrete & HYSD reinforcement of grade Fe415.

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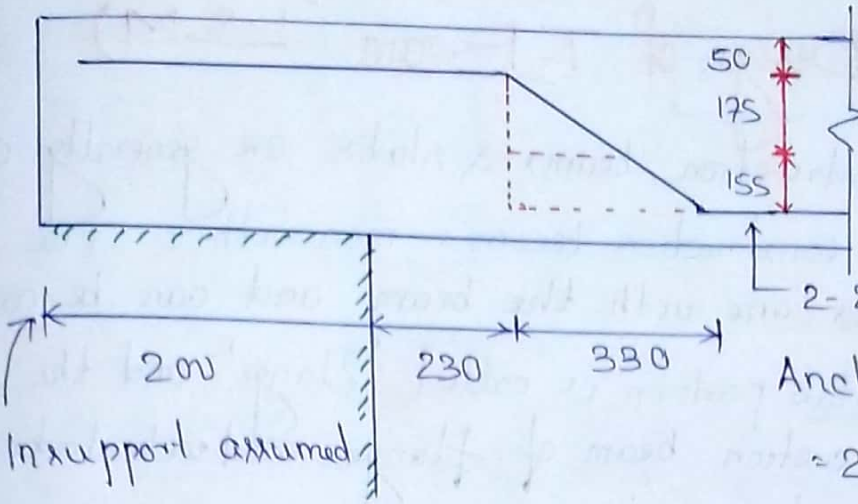


Anchorage length
$= 200 + 230 + 175\sqrt{2}$
$= 677\ mm$

Sol^{n}

Shear taken by two bent bars = 50kN

stress in bar $= \frac{50 \times 10^3}{2 \times 314.0 \times 45^\circ} = 112.60\ N/mm$

development length $(L_d) = \frac{\# \times \sigma_s}{4 \tau_{bd}} = \frac{\# \times 112.60}{4 \times 1.2 \times 1.6}$

$= 14.66\ \#$

$= 14.66 \times 20 = 293.20\ mm$

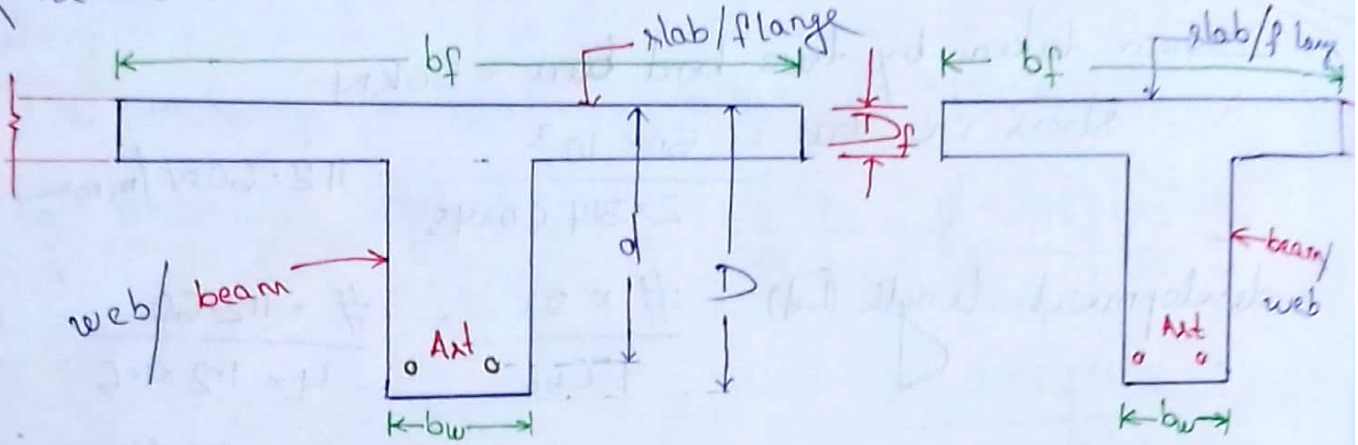
Anchorage provided = 677mm

The effect of 45° bend in compression zone is neglected.
The anchorage of bent bar is thus satisfactory.

Scanned with CamScanner

Analysis & Design of T-Beam (LSM)

In R.C. Construction, beams & slabs are generally cast at one time. Hence the construction becomes monolithic. The slab above the beam becomes one with the beam and can be considered for compression. The slab portion is called 'flange' and the beam 'web'. Such a joint action beam of flange and web looks like a letter 'T' & hence it is known as a T-beam.



where b_f = effective width of flange

b_w = breadth of web

d = effective depth of beam

D_f = thickness of flange (slab)

* The flanged beam analysis & design are analogous to doubly reinforced rectangular beam. In doubly reinforced beams, additional compressive force is provided by ~~additional~~ adding reinforcement in compression zone, whereas in flanged beams, this is provided by the slab concrete.

* The beam of fig(a) can be thought of as a singly reinforced beam (b) plus beam (c) which provides additional compressive force

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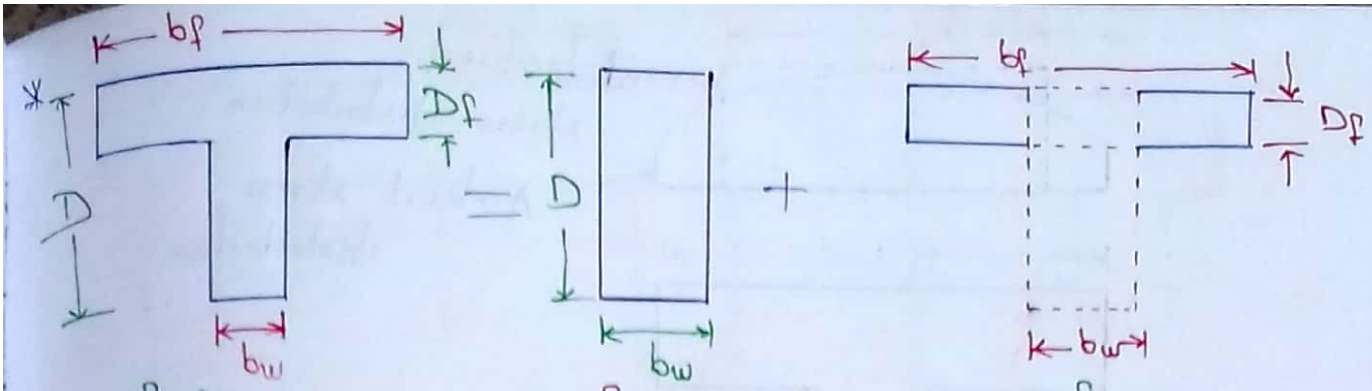

fig(a) fig(b) fig(c)

* A flanged beam also can be doubly reinforced. However, doubly reinforced flanged beams are rare as a large amount of compressive resistance is provided by the slab.

* The moment of resistance of a tee beam of fig(a) is a sum of the moment of resistance of beam b) plus the moment of resistance of beam (c).

* Similarly the steel area required for beam (a) shall be equal to the sum of the steel ~~are~~ required for the beam b) & the steel area required for beam (c).

Effective width of flange

Effective width instead of actual width of a flanged beam is used for calculating the moment of resistance as given in clause 23.1 of the code.

* The actual width of a flanged beam is equal to the distance betⁿ the middle points of the adjacent spans of the slab.

* The effective width is ~~equal~~ used to allow for the fact that the compressive stress in the flange higher above the rib than at some distance away from the rib. The effect width concept makes use of constant compressive stress over the entire effective width such that the force of compression remains same.

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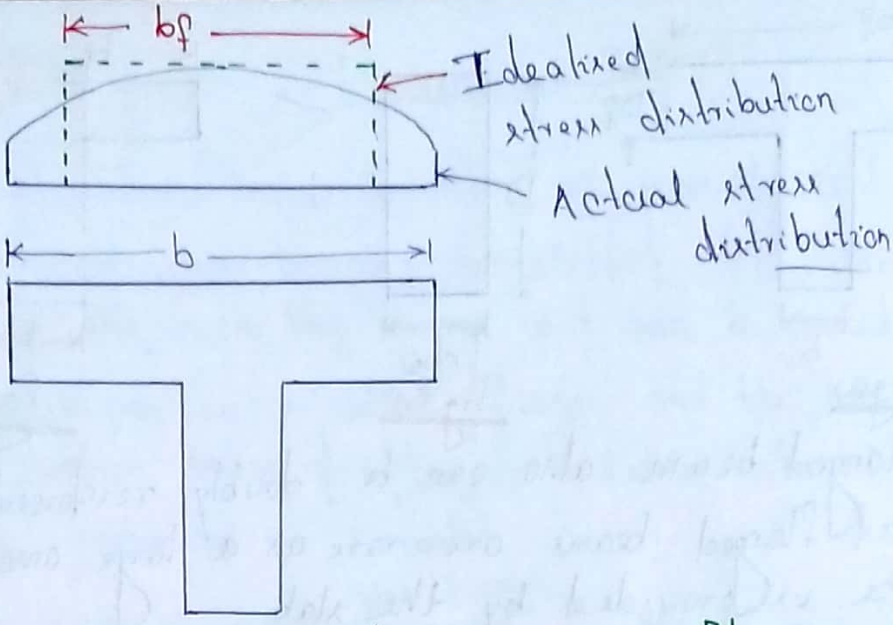

Stress distribution over flange width.

Clause 23.1.2. of the recommends that effective width of flange of beams may be taken as follows

(a) for T-beams, lesser of

(i) $b_f = \frac{l_o}{6} + b_w + 8D_f$

(ii) $b_f = x_1 + x_2 + b_w$

(b) for isolated T-beam, lesser of

(i) $b_f = \frac{l_o}{(l_o/b)+4} + b_w$

(ii) b

where b_f = effective width of flange

b_w = breadth of the riber web.

b = actual width of flange

l_o = distance betⁿ points of zero moment in a beam, for continuous beams l_o may be taken as 0.7 times effective span.

x_1, x_2 = half the clear distance betⁿ two adjacent beams as shown in fig.

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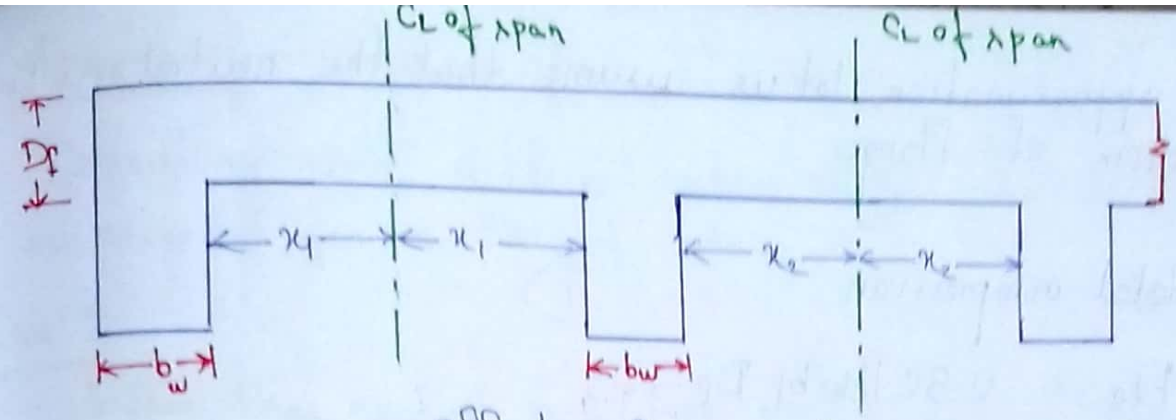

Effective flange width

Min^m & Max^m reinforcement

clause 26.5.1. of the code requires that min^m tension reinforcement should not be less than that given by the following

$$\frac{A_s}{b_w d} = \frac{0.85}{f_y}$$

where A_s = min^m area of tension reinforcement

b_w = breadth of the web of a flanged beam

d = effective depth

f_y = characteristic strength of reinforcement in N/mm^2

* Max^m area of tension steel or compression steel should not exceed $0.04 b_w D$.

~~Position of neutral axis~~

For a flanged beam, the neutral axis either (a) lies in flange or (b) lies in web. For a given section, to decide whether the neutral axis lies in flange or web, the flange force and the total tension may be compared as explained below.

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As a first approximation, let us assume that the neutral axis lies at the bottom of flange.

Now total compression

$$F_{fc} = 0.36 f_{ck} b_f D_f$$

& Total ~~compression~~ tension $(F_{tx}) = 0.87 f_y A_{st}$

Then

(i) If $F_{fc} > F_{tx}$; neutral axis lies in the flange

(ii) If $F_{fc} = F_{tx}$; neutral axis lies at the bottom of the flange.

(iii) If $F_{fc} < F_{tx}$; neutral axis lies in the web.

* By equating total compression with total tension, the actual depth of neutral axis can be found out

ie $0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$

$$\Rightarrow x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}$$

To find out the type of the beam, $x_{u,max}$ shall be found and compared with actual value of neutral axis x_u

Then

(i) If $x_u < x_{u,max}$; the section is under reinforced

(ii) If $x_u = x_{u,max}$; the section is ~~over reinforced~~ balanced.

(iii) If $x_u > x_{u,max}$; the section is over-reinforced & take $x_u = x_{u,max}$

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Moment of resistance of a T- beam

Depending upon different values of x_u & D_f, 4 different cases are arised for a flanged beam.

Case: 1 When the neutral axis lies within the flange. ($x_u < D_f$) — Only in the syllabus

Case: 2 → Neutral axis lies in web ($x_u > D_f$): the section is balanced (Limiting value of moment of resistance)

Case: 3 → Neutral axis lies in the web, section is under-reinforced

Case: 4 → Neutral axis " " " " & the section is over-reinforced

Case: 1

When the neutral axis lies within the flange ($x_u < D_f$)

When the neutral axis lies in the flange, the size of the compression zone becomes $b_f \times x_u$. As concrete does not resist any tension, the width of tension zone has no effect on the MR of the section. Therefore, this beam can be thought of as a rectangular beam of dimensions $b_f \times d$.

(1) For a singly reinforced flanged beam.

(i) Equating total compression & total tension & find out the actual depth of N.A. (x_u)

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b_f}$$

(ii) Then find the maxm depth of neutral axis ($x_{u,max}$) & compare bet^n x_u & x_{umax}. Then state the type of section.

(iii) For under-reinforced section

$$M_u = 0.87 f_y A_{st} (d - 0.42 x_u)$$

or $$M_u = 0.36 f_{ck} b_f x_u (d - 0.42 x_u)$$

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(2) For a doubly reinforced flanged beam

(ii) M_u.

(iii) For balanced or over-reinforced section

$$M_{u,lim,T} = 0.36 f_{ck} b_f x_{u,max} (d - 0.42 x_{u,max})$$

or $$M_{u,lim,T} = 0.87 f_y A_{st,lim} (d - 0.42 x_{u,max})$$

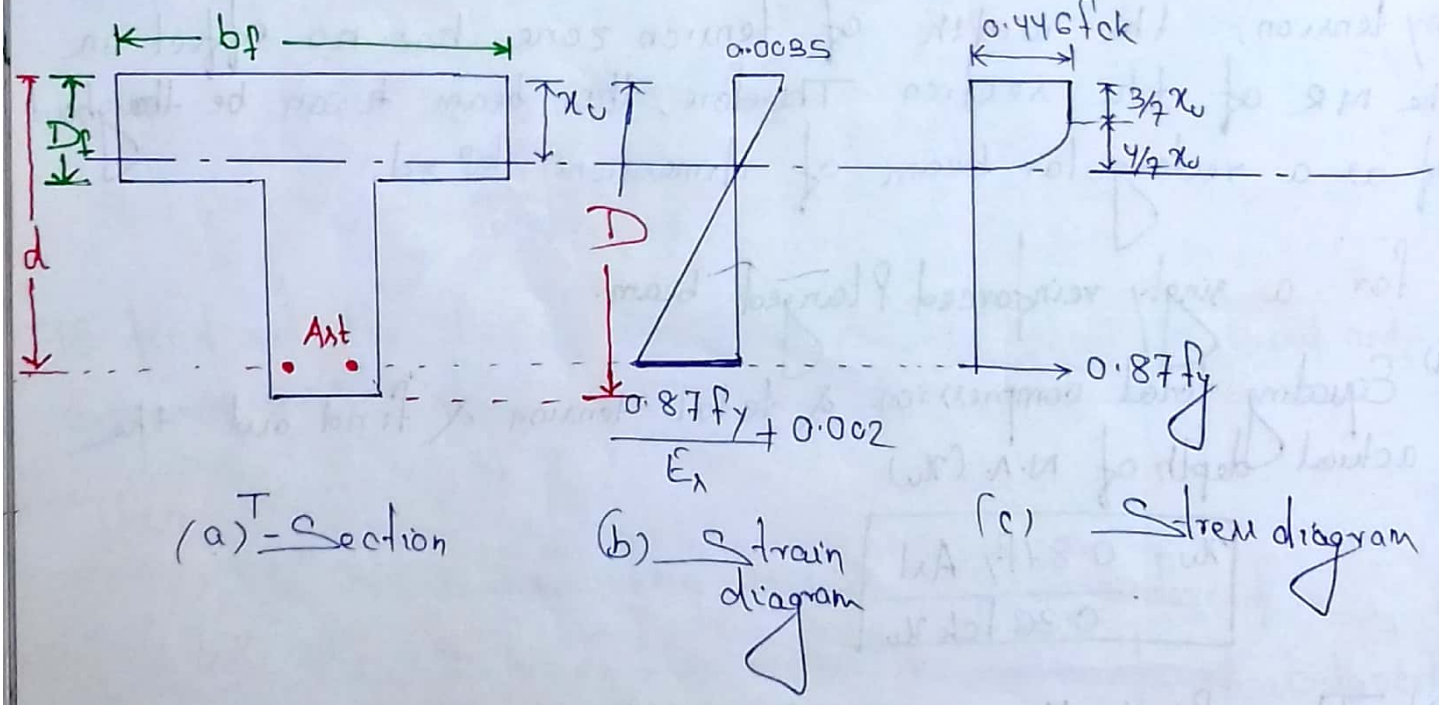
(2) For doubly reinforced flanged beam

(i) $M_u - M_{u,lim,T} = f_{sc} A_{sc} (d - d')$

(ii) $A_{st1} = A_{st,lim}$

(iii) $A_{st2} = \frac{A_{sc} f_{sc}}{0.87 f_y}$

(iv) $A_{st} = A_{st1} + A_{st2}$



(a) T-Section (b) Strain diagram (c) Stress diagram

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Problem: 1

A tee beam of effective flange width 1200 mm, thickness of slab 100mm, width of rib 300mm and effective depth of 560mm is reinforced with 4 no 25mm diameter bars. Calculate the factored moment of resistance. The materials are M20 grade concrete & HYSD reinforcement of grade Fe415.

Soln

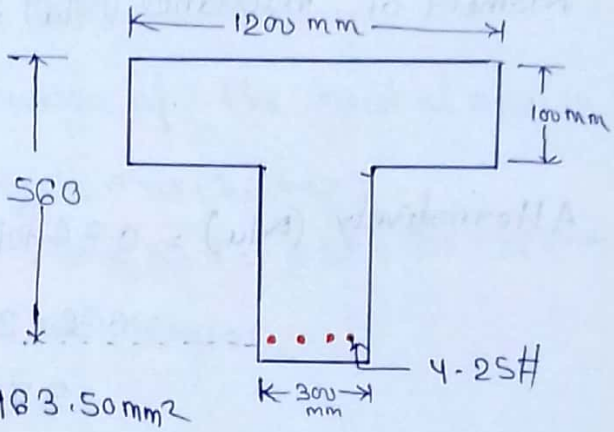
Given data:

effective flange width (b_f) = 1200 mm

thickness of slab (D_f) = 100 mm

width of rib (b_w) = 300mm

effective depth (d) = 560mm



Area of steel (A_{st}) = $4 \times \pi/4 \times 25^2$ = 1963.50 mm^2

characteristic compressive strength of concrete (f_{ck}) = 20 N/mm^2

characteristic strength of steel (f_y) = 415 N/mm^2

To find whether the neutral axis lies in the flange or the web, flange compression & tensile force are compared.

Total (compression) $F_{tc} = 0.36 f_{ck} b_f D_f = 0.36 \times 20 \times 1200 \times 100 = 864000$ N

$= 864$ kN

Total tension (F_{ts}) $= 0.87 f_y A_{st} = 0.87 \times 415 \times 1963.50 = 708921.68$ N

$= 708.92$ kN

As $F_{tc} > F_{ts}$

∴ Neutral axis lies in flange.

To find the actual depth of neutral axis (x_u) Equating the total compression to total tension

i.e. $0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$

$\Rightarrow 0.36 \times 20 \times 1200 \times x_u = 0.87 \times 415 \times 1963.50 \Rightarrow x_u = 82.05$ mm < 100 mm (OK)

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Maxm depth of neutral axis ($x_{u,max}$) = 0.48 d [for Fe 415 grade steel]

= 0.48 × 560

= 268.8 mm

As $x_u < x_{u,max}$

∴ Section is under-reinforced.

Moment of resistance (M_u) = $0.87 f_y A_{st} (d - 0.42 x_u)$

= 0.87 × 415 × 1963.50 (560 − 0.42 × 82.05)

= 372.56 kNm

Alternatively (M_u) = $0.36 f_{ck} b_f x_u (d - 0.42 x_u)$

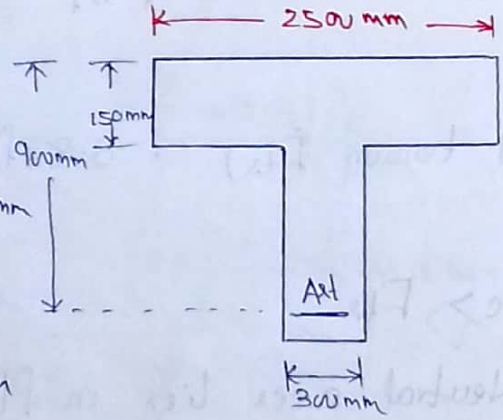
= 0.36 × 20 × 1200 × 82.05 (560 − 0.42 × 82.05)

= 372.56 kNm

Problem: 2

A tee-beam having an effective flange width of 2500 mm is required to resist an ultimate moment of 1200 kNm. The thickness of the flange is 150 mm, the width of the rib is 300 mm and effective depth is 900 mm. Using M20 concrete and Fe250 steel, determine the area of reinforcement required.

Soln



Given data:

effective flange width (b_f) = 2500 mm

Thickness of flange (D_f) = 150 mm

Width of rib (b_w) = 300 mm

effective depth (d) = 900 mm

Ultimate moment resisted by the beam (M_u) = 1200 kNm

Characteristic compressive strength of concrete (f_{ck}) = 20 N/mm²

" strength of steel (f_y) = 250 N/mm²

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Assuming the neutral axis to lie within the flange ($x_u < D_f$)

Moment of resistance (M_u) $= 0.36 f_{ck} \times b_f x_u (d - 0.42 x_u)$

$\Rightarrow 1200 \times 10^6 = 0.36 \times 20 \times 2500 \times x_u (900 - 0.42 \times x_u)$

$\Rightarrow 66666.67 = x_u (900 - 0.42 x_u)$

$\Rightarrow 0.42 x_u^2 - 900 x_u + 66666.67 = 0$

$\Rightarrow x_u = 76.83\,mm < D_f = 150\,mm$

Hence our assumption about the position of the neutral axis is correct.

Also maxm depth of neutral axis ($x_{u,max}$)

$= 0.53 d = 0.53 \times 900 = 477\,mm$

Equating total compression to total tension

$0.36 f_{ck} b_f x_u = 0.87 f_y A_{st}$

$\Rightarrow 0.36 \times 20 \times 2500 \times 76.83 = 0.87 \times 250 \times A_{st}$

$\Rightarrow A_{st} = 6358.34\,mm^2$

$\therefore$ Provide 14 bars of 25 mm diameter ($6874\,mm^2$).

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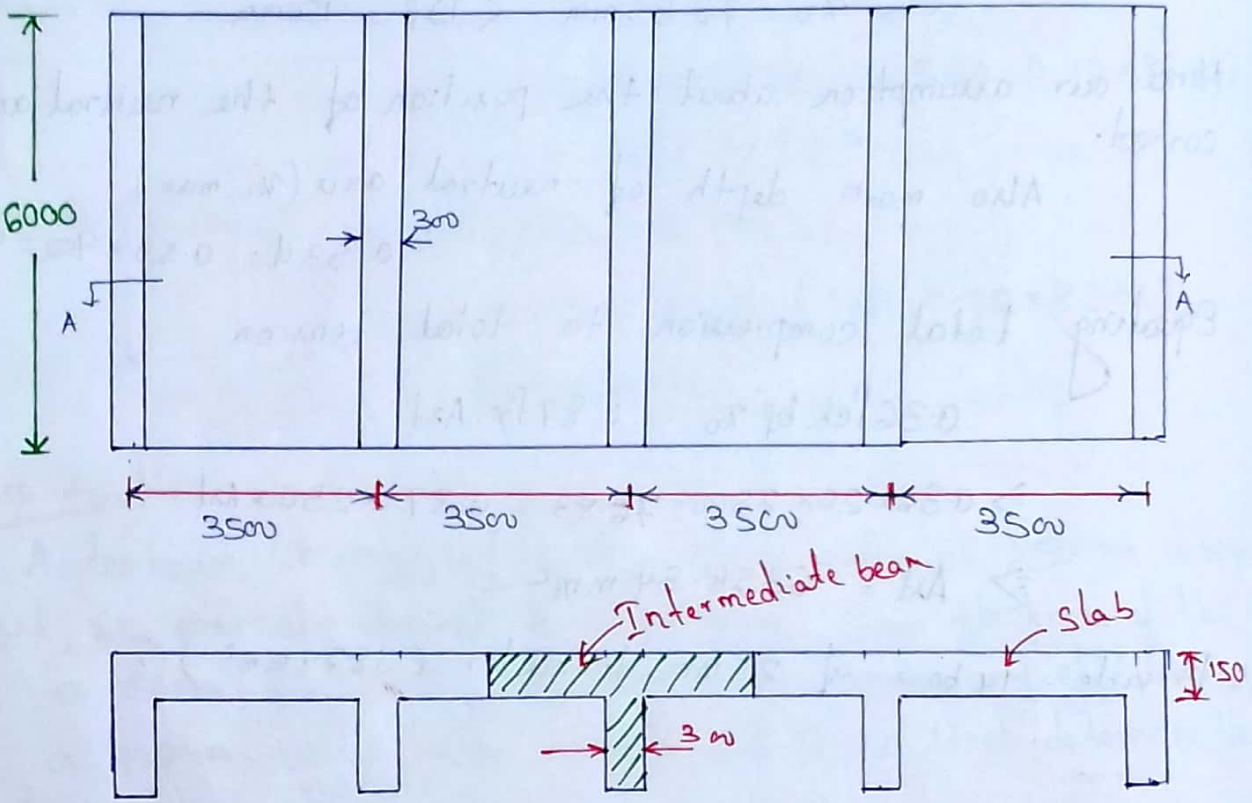
Problem:3

A T-beam floor consists of 15cm thick RC slab monolithic with 30 cm wide beams. The beams are spaced at 3.5m centre to centre and their effective span is 6m as shown in fig. If the super imposed load on the slab is 5KN/m², design an intermediate beam. Use M20 mix & Fe250 grade steel.



Section A A

T-beam floor

Sol^n

Given data.

Thickness of slab/flange (D_f) = 15cm

Width of rib/web (b_w) = 30 cm

effective span of beam (l) = 6m

c/c distance ~~bet^n bet^n~~ of beam = 3.5m

super imposed load on the slab = 5KN/m²

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characteristic compressive strength of concrete (f_{ck}) = 20 N/mm²

" strength of steel (f_y) = 250 N/mm²

Dead load of slab = $0.15 \times 35 \times 25 = 3.75$ kN/m²

superimposed load on slab = 5 kN/m²

Total load on slab = 8.75 kN/m²

Load per meter run of beam = load on slab per unit area × c/c distance betn beams

$= 8.75 \times 3.5 = 30.63$ kN/m

effective width of flange (b_f) = $\frac{l_o}{6} + b_w + 6D_f$

$= \frac{6000}{6} + 300 + 6 \times 150$

$= 2200$ mm

Maxm value of flange width = 3500 mm > 2200 mm

∴ Effective width of flange (b_f) = 2200 mm

Let us adopt overall depth (D) of beam equal to 400 mm & effective cover equal to 40 mm so that effective depth is 360 mm

Dead load of web of beam = width of web × depth of web × concrete density

$= 0.30 \times 0.25 \times 25 = 1.875$ kN/m

Total load on beam per meter run (w) = 30.6 + 1.875 = 32.475 kN/m

Factored maxm B.M. = $\frac{wl^2}{8} = \frac{(1.5 \times 32.475) \times 6^2}{8} = 219.2$ kNm

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Let us assume that neutral axis lies in the flange, that is, then depth of neutral axis (x_u) is

$$x_u = \frac{0.87 f_y A_{st}}{0.36 \times f_{ck} \times b_f} = \frac{0.87 \times 250 \times A_{st}}{0.36 \times 20 \times 2200} = 0.0137 A_{st}$$

Now factored B.M = ~~0.87 fy~~ Force of tension $\times z$

$$\Rightarrow 219.2 \times 10^6 = 0.87 f_y A_{st} (d - 0.42 x_u)$$

$$\Rightarrow 219.2 \times 10^6 = 0.87 \times 250 \times A_{st} (360 - 0.42 \times 0.0137 A_{st})$$

$$\Rightarrow 1007816.092 = 360 A_{st} - 5.754 \times 10^{-3} A_{st}^2$$

$$\Rightarrow 5.754 \times 10^{-3} A_{st}^2 - 360 A_{st} + 1007816.092 = 0$$

$$\Rightarrow A_{st} = 2937.40 \text{ mm}^2$$

& $x_u = 0.0137 \times 2937.40 = 40.24 \text{mm} < 150 \text{mm}$ (Ok)

$\therefore$ NA lies in the flange

Use 5 - 28 mm bar reinforcement in tension side

($A_{st,provided} = 3078.76 \text{mm}^2 > 2937.40 \text{ mm}^2$) OK

Min^m area of tension steel $A_s = \frac{0.85}{f_y} \times b_w d$

$$= \frac{0.85}{250} \times 300 \times 360$$

$$= 367.2 \text{ mm}^2 < 3078.76 \text{mm}^2$$

(Ok)

Max^m area of tension steel $A_t = 0.04 b_w D$

$$= 0.04 \times 300 \times 400$$

$$= 4800 \text{ mm}^2 > 3078.76 \text{mm}^2$$

(Ok)

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Control of deflection

A structural member deflects when, it carries a load
According to clause 23.2 IS:456, the deflection of structure

see-[clause - 23.2]

* The first limit is specified to avoid psychological upsetting of the occupants
* The second limit is intended to avoid damage to partitions and finishes.

In the clause 23.2.1 of IS:456 suggested for checking that deflect^n is not excessive.

These are

(i) Control of span/effective depth ratio and is discussed in clause 23.2.1 which shall be used modified for.

(ii)

Span/effective depth ratio

This method is based on the calculat^n of deflect^n and tests on practical beams/slabs. This is a semi-empirical method and is covered in clause 23.2.1 of IS:456. Accordingly the basic span/effective depth ratios for diff. support cond^{n}s are given and shall be modified for

(1) Amount of tension steel
(2) the amount of compression steel.
(3) the type of beam.

* For beams and slabs, the vertical deflect^n limits, as stated above may generally be assumed to be satisfied provided that the span to depth ratios are not greater than the values obtained as below.

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Clause - 23.2.1 (a), (b), (c), (d), (e):

Now the allowable $\frac{span}{d}$ ratio is given by

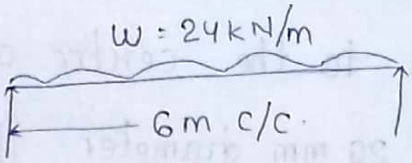
$$\left(\frac{span}{d}\right)_{allowable} = basic\left(\frac{span}{d}\right)_{ratio} \times modification\ factor\ for\ tension\ reinforcement \times modification\ for\ compression\ reinforcement \times reduction\ factor\ for\ flanged\ beam$$

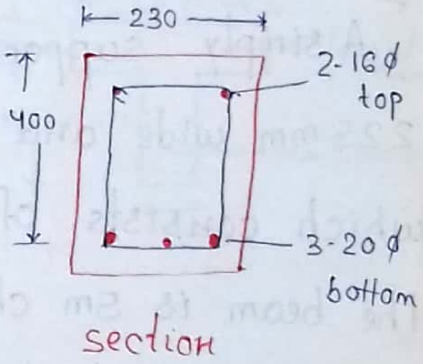
The actual $\frac{span}{depth}$ ratio shall be less than or equal to the allowable $\frac{span}{d}$ ratio.

Problem-1

A simply supported R.C.C. beam is shown in fig. If the span of the beam is 6m, check the deflection of the beam using the rules. The area required in tension is 920 mm^2 and 120 mm^2 in compression. The materials are M20 grade concrete and HYSD reinforcement of grade Fe 415.

Solution





section

For simply supported beam basic $\frac{span}{d}$ ratio = 20

A_{st} = 942.47 mm^2

$$f_s = 0.58 f_y \times \frac{A_{st,required}}{A_{st,provided}}$$

$$= 0.58 \times 415 \times \frac{920}{942.47}$$

$$= 234.96 \ N/mm^2$$

$$P_t = \frac{100 \times 942.47}{230 \times 400} = 1.02$$

So modification factor = 1.0

A_{sc} = 402.12 mm^2

$$P_c = \frac{100 \times 402.12}{230 \times 400} = 0.44$$

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modification factor = 1.13

Then permissibe $\frac{\text{span}}{d}$ ratio = $20 \times 1.0 \times 1.13 = 22.6$

actual $\frac{\text{span}}{d}$ ratio = $\frac{6000}{400} = 15 < 22.6$

The beam is satisfactory in respect of deflection.

Problem: 2

A simply supported beam reinforced on the tension side is 225 mm wide and 400mm deep to the centre of tensile steel which consists of 3 bars of 20 mm diameter. If the span of the beam is 5m check if the beam is satisfactory from deflection control. Use M 20 concrete and Fe 415 steel.

Solution

Basic $\frac{\text{span}}{d}$ ratio for simply supported beams = 20

percentage of tensile steel provided = $\frac{3 \times \frac{\pi}{4} \times 20^2 \times 100}{225 \times 400} = 1.05\%$.

modification factor = 0.97

$\therefore$ permissible $\frac{\text{span}}{\text{depth}}$ ratio = $20 \times 0.97 = 19.4$

Actual $\frac{\text{span}}{\text{depth}}$ ratio = $\frac{5000}{400} = 12.5 < 19.4$

Hence the beam is satisfactory from deflection consideration.

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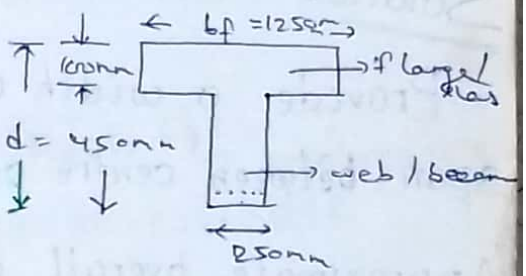
Problem - 3

A tee beam with $b_f = 1250$ mm, $b_w = 250$ mm, $D_f = 100$ mm $d = 450$ mm requires 1370 mm^2 area of tension reinforcement. It is reinforced with 5-20 ϕ in tension. Check the beam in respect of deflection. The materials are M20 grade concrete and reinforcement of grade Fe 415. The beam is simply supported over a span of 5.6m.



Solution

$b_f = 1250$ mm

$b_w = 250$ mm

$D_f = 100$ mm

$d = 450$ mm

$A_{st} = 5 \times \frac{\pi}{4} \times 20^2 = 1570.79\ mm^2$

$p_t = \frac{100 \times 1570.79}{1250 \times 450} = 0.279$

$f_s = 0.58 \times 415 \times \frac{1370}{1570.79} = 209.93\ N/mm^2$

So modification factor = 1.8

Also $\frac{b_w}{b_f} = \frac{250}{1250} = 0.2$

Reduction factor = 0.8

Basic $\frac{span}{d}$ ratio = 20

permissible $\frac{span}{d}$ ratio = $20 \times 1.8 \times 0.8 = 28.8$

Actual $\frac{span}{d}$ ratio = $\frac{5600}{450} = 12.44 < 28.8$

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Design of slab

* Slabs are plate elements having the depth D much smaller than its span and width. They usually carry a uniformly distributed load and form the floor or roof of the building.

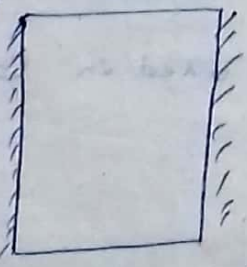
* Like beams, slabs are horizontal except in the case of stair cases and ramps

* Like beams, slabs also may be simply supported, cantilever or continuous depending upon the support conditions. They are classified according to the system of supports used as under

1) One-way spanning slabs
2) Two-way " "
3) Flat slabs supported directly on columns without beams.
4) Grid slabs.
5) Circular & other shapes
6) Ribbed & Waffle slabs.

(1) One-way spanning slabs

* The slabs supported on 2 opposite supports is a one-way spanning slab. Inshort, a slab which transfers its load on one of the set of two opposite edge supports is called one-way slab. In fig (i) by virtue of the geometry, the slab is a one-way spanning slab, where it is supported only on 2 opposite edge supports



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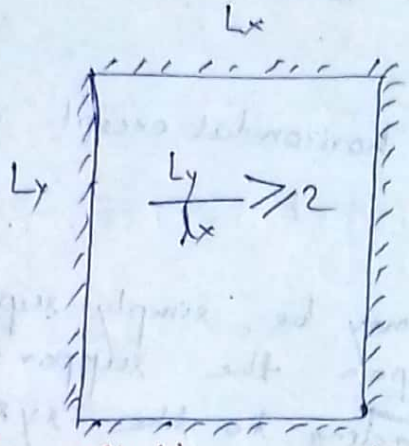
* The slab where $\frac{L_y}{l_x} \geq 2$, is called one-way slab. provi that it is supported on all four edges. In one way slab
* In the load on the slab is taken as carried by the short span.

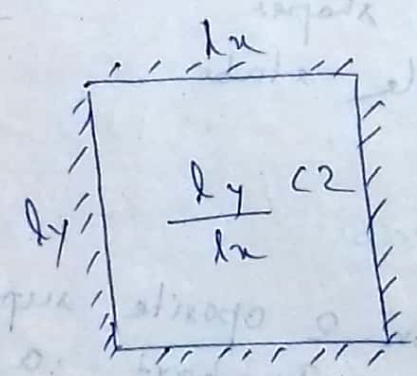


lx = span in shorter dirn
ly = " " longer dirn

Ex Generally all the cantilever slabs are one way slabs. chajjas and verandahs are ex of one way slab

(2) Two way slab

if the slab is supported on all four edges and if $\frac{l_y}{l_x} < 2$ the tendency of the slab is to bend in both dir^ns, such slabs are called two way spanning slabs



* In 2 way slabs, load will be carried in both the dir^ns. So main reinforcement is provided in both dir^n for 2 way slabs

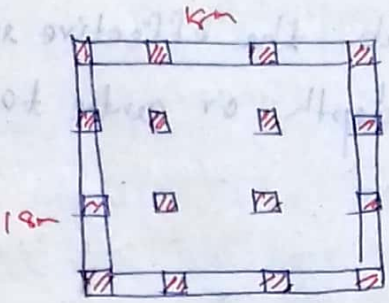
Ex: These types of slabs are used in Construction floors of multistoreyed building

Flat slabs

Sometimes, beam free areas are required to reduce the floor height & to permit large amount of light which might be obstructed by the larger depth of the beams spanning on comparatively larger spans. (5-6 metres) Flat slabs are used in such cases

* Flat slabs are thicker slabs supported directly on columns.

* for office building in commercial areas, flat slabs are beneficial.



(a) Flat slab

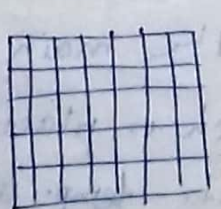
Grid slabs

Sometimes, in large halls, public places, marriage halls, auditoriums etc a large column free area is required. In these cases, large depth beams may be permitted only on periphery.

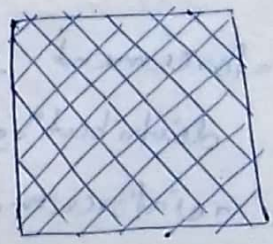
* In such cases grid slabs and grid beams are provided.

~~The grid slabs and grid beams are pr~~

The grid slabs may have rectangular (or square) grid or diagrid.



Rectangular grid



Circular slabs

The slab may be circular by virtue of its shape. Cover slab of a circular water tank is an excellent example of such slab.

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Ribbed & waffle slabs

Thin ribs spanning in one dir^n & placed not more than 15 c/c constitute ribbed slabs. If such ribs are provided in both dir^{n}s, the waffle slab is obtained.

One-way spanning slab

The following are the design consideration for one-way slab.

(1) Effective span

* For simply supported slab, the effective span is equal to clear span plus effective depth or centre to centre of supports which ever is less.

(2) General

* A simply supported one-way slab. The reinforcement in the dir^n of span is the moment steel & is designed in the same way as beams. Slabs are usually not designed as doubly reinforced except for some special cases.
* The moment steel is known as main reinforcement and is placed in the first layer near to the extreme tension fibre, keeping clear cover as per requirement to get max^m effective depth.
* The reinforcement lar to the main reinforcement is known as distribut^n steel which is placed in second layer. This reinforcement resists temp. and shrinkage stresses, keeps the main reinforcement in posn & distributes the concentrated or non uniform loads throughout the slab.

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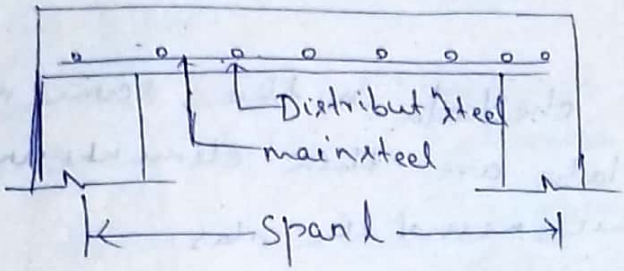
The spacing of slab bars is given by

$$\text{spacing} = \frac{\text{area of one bar} \times 1000}{\text{required area in mm}^2 \text{ per metre}}$$

Diagram - 380 (fig 10.3.(a))



Simply supported one-way slab

(3) Reinforcement requirements

a) Minm reinforcement: The reinforcement in either dirn in slabs shall not be less than following

0.0015 bD for mild steel bars.

0.0012 bD " HYSD bars.

b) maxm diameter: The diameter of reinforcing bars shall not exceed $\frac{1}{8}$ th of the total thickness of the slab.

c) According to SP: 34, for main bars the minm dia shall be 10mm for plain bars and 8 mm for deformed bars. For distributn bar, the minm dia shall be 6 mmm in any case.

(4) Shear stress

In normal cases the shear in slabs is not critical. ~~However~~

cl - 40.2.1.1

→ It can be seen from the above values that higher shear stresses are permitted for thinner slabs. This is due to the observatn made that thin plates can resist more shear per unit area. If shear is critical in slabs usually the shear reinforcements are not provided but

the depth of slab is increased

* According to clause 40.2.3, the nominal shear stress should not exceed $0.5\,\tau_{c,max}$

$$\tau_v \not> 0.5\,\tau_{c,max}.$$

Deflection

This shall be checked in the same manner as beams. The slabs are thin elements and deflect may govern the thickness of the slab.

Cracking

To ensure that the cracking of the slabs is not excessive, spacing of the reinforcements shall be limited to the following

For main bars, spacing $\not> 3d$
$\not> 300\,mm$

For secondary bars, spacing $\not> 5d$
$\not> 450\,mm$

d = effective depth of slab

Cover

page - 382 H J Sheehan

8) Development Length - 382 "

Design a simply supported roof slab for a room 8m × 3.5m clear in size if the super-imposed load is 5 kN/m². Use M25 mix and Fe415 grade steel.

Soln

Depth of slab:

The first trial of depth of slab can be arrived at by considering deflection criterion.

Let us provide a bearing of slab = 250 mm

short span bet^n centres of bearings = 3.5 + 0.25
= 3.75 m

long span bet^n " " " = 8 + 0.25 = 8.25 m

Assuming 0.3 % steel ~~modificat^n factor~~ as a first trial with service stress of 0.58 f_y = 0.58 × 415 = 240 N/mm². So modified factor = 1.45

Basic $\frac{\text{span}}{d}$ ratio = 20

Then permissible $\frac{\text{span}}{d}$ ratio = 20 × 1.45 = 29

~~The depth~~ d =

Effective depth required = $\frac{\text{span}}{20 \times \text{modificat}^n \text{ factor}}$

= $\frac{3.75}{29}$

= 129.31 ≃ 130 mm

Considering mild exposure & ~~max~~ dia of reinforcement to be 10mm

overall depth (D) = 130 + 15 + 5 = 150 mm

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Loads

Consider a 1m wide strip of the slab parallel to its shorter span.

DL of the slab = $0.15 \times 1.0 \times 25 = 3.75$ KN/m

super imposed load = $5 \times 1 = 5$ KN/m

floor finish = $1 \times 1 = 1$ KN/m [Assume floor finish = 1 KN/m²]

Total load = 9.75 KN/m

factored load = $1.5 \times 9.75 = 14.625$ KN/m

Effective short span is taken as the lesser of the following

(i) Distance bet^n centre of bearing = $3.5 + 0.25$
$= 3.75$ m

(ii) clear span + effective depth
$= 3.5 + 0.13 = 3.63$ m

∴ Effective span in shorter dir^n = 3.63 m (l_x)

Moment & shear

Max^m BM at centre of shorter span = $\frac{W_u l_x^2}{8}$

$= \frac{14.625 \times 3.63^2}{8}$

$= 24.08$ KN-m

Max^m shear force = $\frac{W_u l_x}{2}$ (Based on clear span in short dir^n)

$= \frac{14.625 \times 3.5}{2} = 25.99$ KN

~~$= 20.544$ kN.~~

Depth of the slab is given by B.M = $0.138 f_{ck} b d^2$

$\Rightarrow d = \sqrt{\frac{BM}{0.138 f_{ck} b}} = \sqrt{\frac{24.08 \times 10^6}{0.138 \times 25 \times 1000}}$

$= 83.54$ mm ($d_{provided} = 130$ mm (OK)

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Effective depth required for flexture

=

Design for flexture

Area of tension steel is given by

$$M = 0.87 f_y A_{st} d\left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

$$\Rightarrow 24.08 \times 10^6 = 0.87 \times 415 \times A_{st} \times 130 \times \left(1 - \frac{A_{st} \times 415}{1000 \times 130 \times 25}\right)$$

$$\Rightarrow 513.03 = A_{st} - 4 \times 10^{-5} A_{st}^2$$

$$\Rightarrow A_{st} = 524.01 \text{ mm}^2$$

spacing of 10mm ϕ bars $= \frac{\pi/4 \times 10^2 \times 1000}{524.01}$

$= 149.88$ mm

provide 10mm ϕ bar @ 140mm c/c

($A_{st,provided} = 560.99$ mm²)

Bend alternate bars at 0.1L from the face (ok)

of support where moment reduces to less than half of its max^m value.

check for spacing & dia of main reinforcement

Temperature / distribut^n reinforcement is equal to 0.12% of the gross concrete area provided in the longitudinal dir^n

$$A_{st} = 0.12\% \times b \times D = 0.12\% \times 1000 \times 150$$

$$= 180 \text{ mm}^2$$

spacing of 6mm HYSD steel @ $= \frac{\pi/4 \times 6^2}{180} \times 1000$

$= 157.07$ mm

provide 6mm ϕ bars @ 150mm c/c [$A_{st,provided} = 188$ mm² (ok)]

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Check for shear

Shear force at the edge of the support $(V_u) = \frac{w_u \cdot l_x}{2} = \frac{14.625 \times 3.5}{2} = 25.59\ kN$

Nominal shear stress $(\tau_v) = \frac{V_u}{bd} = \frac{25.59 \times 10^3}{1000 \times 130} = 0.20\ N/mm^2$

$0.1968 \cong 0.20\ N/mm^2$

Percentage of steel available near support $= \frac{100 \times A_{st}}{bd} = \frac{100 \times \frac{560.99}{2}}{1000 \times 130} = 0.22\%$

Design shear strength of concrete $(\tau_c) = 0.34\ N/mm^2$

$0.5\tau_{c\,max} = 0.5 \times 3.1 = 1.55\ N/mm^2$ (OK)

for 150mm thick slab $k = 1.3$

for solid slabs, the design shear strength for concrete $(\tau_c') \leq k \times \tau_c = 1.3 \times 0.34$

$= 0.442\ N/mm^2$ (OK)

$\therefore \tau_v < \tau_c'$ (OK)

The design is safe against shear.

Check for development length

Moment of resistance of the sect^n after 50% bent up of steel

$$M_1 = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{bdf_{ck}}\right)$$

$$= 0.87 \times 415 \times \left(\frac{560.99}{2}\right) \times 130 \left(1 - \frac{\left(\frac{560.99}{2}\right) \times 415}{1000 \times 130 \times 25}\right)$$

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$M_1 = 12.694$ KN-m

$= 12.$

$V = 25.59$ kN

let us assume $l_0 = 8\phi$. [l_0 = anchorage length beyond the centre of support & not exceeding 9 or 12 times the bar dia]

Bend $l_d = \frac{0.87 f_y \phi}{4 \times \tau_{bd}} = \frac{0.87 \times 415 \phi}{4 \times 1.4 \times 1.6} = 40.29\phi$

As per code

[Assume → 8φ]

$$L_d \leq 1.3 \frac{M_1}{V_u} + l_0$$

$$40.29\phi \leq \frac{1.3 \times 12.694 \times 10^6}{25.59 \times 10^3} + 8\phi$$

$\Rightarrow 32.29\phi \leq 644.86$

$\Rightarrow \phi \leq 19.97$ mm

ϕ provided = 10 mm .... (safe)

Check for deflection

Basic $\frac{span}{d}$ ratio = 20

% of tensile reinforcement $= \frac{100 \times A_{st}}{bd} = \frac{100 \times 560.99}{1000 \times 130}$

$= 0.43\%$

$f_s = 0.58 \times f_y \times \frac{A_{st,required}}{A_{st,provided}} = 0.58 \times 415 \times \frac{524.01}{560.91}$

$= 224.83$ N/mm²

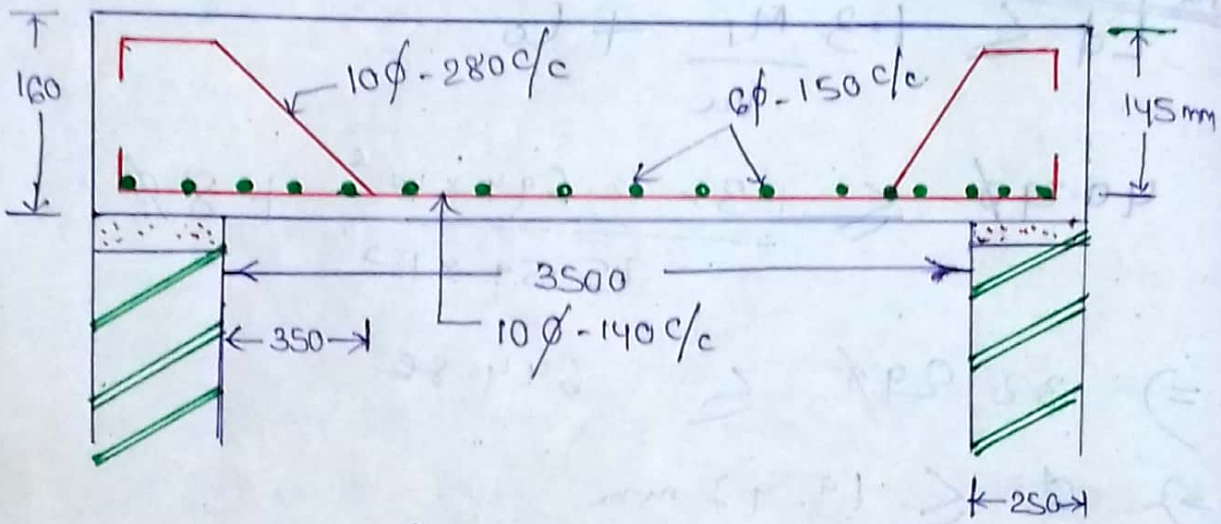
modificat^n factor (γ) = 1.3 at a service stress of 240 N/mm² in fe 415 grade steel

$\frac{\times 415}{0 \times 25}$) permissible $\frac{span}{d}$ ratio = 20 × 1.3 = 26

actual $\frac{span}{d}$ ratio $= \frac{3630}{130} = 27.92 < 26.$ KG

∴ There is need to increase the depth of slab.
Let us increase the effective depth to 145mm
& overall depth to 160mm.

The details of reinforcement are shown in fig.



Reinforcement in one-way slab.

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Two-way slabs:

The two-way spanning slab occurs when the slab is supported on all four edges. The two way actn can be well studied by considering a thin flexible membrane supported on all four edges.

* If ly is much greater than lx, the membrane deflectn is in dirn of lx. Now keeping lx const, ly is reduced & the deflectn of the membrane is observed.

* When ly = 2lx; from this point it can be observed the dirn of ly also. If ly is further reduced, the deflectn decreases in the dirn of lx & increases in the dirn of ly.

* When ly = lx, the membrane deflects as a saucer. When ly ≥ 2lx, the cases is a one way spanning & When ly < 2lx or $\frac{ly}{lx} < 2$ the case is a two way spanning slab.

* The two-way slabs can be simply supported or restrained slabs.

* In simply supported slabs, the corners can lift away from the support while for restrained slabs, the corners are held down by edge beams

* The corners are held down by beam of the stiffness of the beam and therefore at corners, torsion is induced. The slab should be capable of resisting torsion at corners. Also the edge beams shall be designed for torsion.

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$= 4 + 0.2$ or $4 + 0.015$ which ever is lesser

$\therefore l_x = 4.15\ m$

similarly $l_y = 6 + 0.2$ or $6 + 0.141$ which ever is lesser

$\therefore l_y = 6.141\ m$

span ratio $= \frac{l_y}{l_x} = \frac{6.141}{4.15} = 1.48 < 2$

so it is a two way slab.

so for $\frac{l_y}{l_x} = 1.48$, $\alpha_x = 0.103$ $\alpha_y = 0.047$

Loads consider 1m wide strip

DL of slab $= 0.17 \times 1 \times 25 = 4.25\ KN/m$

floor finish $= 1 \times 1 \times$ [illegible] $= 1\ KN/m$

Live load $= 3 \times 1 = 3\ KN/m$

Total load $= 8.25\ KN/m$

factored load $= 1.5 \times 8.25 = 12.38\ KN/m$

~~Now~~ Moments along short span & along long span is given by

$M_x = \alpha_x w l_x^2 = 0.103 \times 12.38 \times 4.15^2$

$= 21.96\ KN-m$

$M_y = \alpha_y w l_x^2 = 0.047 \times 12.38 \times 4.15^2$

$= 10.02\ KN-m$

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required
The effective depth of the slab is given by
equating $M_{u,lim}$ to M_x

$$0.138 f_{ck} b d^2 = 21.96 \times 10^6$$

$$\Rightarrow 0.138 \times 20 \times 1000 \times d^2 = 21.96 \times 10^6$$

$$\Rightarrow d = 89.19 \text{ mm} < d_{short, provided} = 150 \text{ mm}$$

$d_{long, provided} = 141$ mm

(Ok)

Area of steel along short span

$$M_x = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

$$\Rightarrow 21.96 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \times \left(1 - \frac{A_{st} \times 415}{1000 \times 150 \times 20}\right)$$

$$\Rightarrow 405.48 = A_{st} - 1.38 \times 10^{-4} A_{st}^2$$

$$A_{st} = 431.13 \text{ mm}^2$$

spacing of 10mm ϕ bars $= \frac{\pi/4 \times 10^2 \times 1000}{431.13} = 182.17$ mm

provide 10mm ϕ bars @ 180mm c/c

Actual area of steel provided $= \frac{\pi/4 \times 10^2 \times 1000}{180} = 436.33$ mm²

Actual % of steel provided $= \frac{100 \times 436.33}{1000 \times 150} = 0.29\%$

percentage of steel originally assumed $= 0.3\%$

∴ The design is safe from serviceability cond^n,

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area of steel for along span long span

$$M_y = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

$$\Rightarrow 10.02 \times 10^6 = 0.87 \times 415 \times A_{st} \times 141 \times \left(1 - \frac{A_{st} \times 415}{1000 \times 141 \times 20}\right)$$

$$\Rightarrow 196.83 = A_{st} - 1.47 \times 10^{-4} A_{st}^2$$

$$\Rightarrow 1.47 \times 10^{-4} A_{st}^2 - A_{st} + 196.83 = 0$$

$$\Rightarrow A_{st} = 202.88 \text{ mm}$$

spacing of 8 mm ϕ bars $= \frac{\pi/4 \times 8^2 \times 1000}{202.88} = 247.75$ mm

provide 8mm ϕ bars @ 240mm c/c

Actual area of steel provided

$$= \frac{\pi/4 \times 8^2 \times 1000}{245} = 208.33 \text{ mm}^2$$

The code requires that the min^m area of steel should be $0.12\% \; b \times D = 0.0012 \times 1000 \times 170 = 204 \text{mm}^2$

A_{st}, in long span

A_{st} in short span

curtail alternate bars at $\frac{1}{10}$th of effective span in each dir^n as with clause D-2.1 of the code. Provide 50% of the ~~+ve moment reinforc~~ maximum +ve steel at top near the supports to resist bending moment due to partial fixity. This steel is provided in c/l from the face of the supports

The bars cannot be bent or curtailed because if 50% of long span bars are curtailed the remaining bars will be less than the min^m.

* At top or support, provide 50% of bars of respective span to take into account any possible -ve moment created due to [illegible] nature of slab

Check for shear force at short edges

Max^m shear force intensity in either dir^n can be taken as $\frac{1}{2} w L_x$ where L_x is clear short span

$$\text{Max}^m \text{ SF} = \frac{1}{2} w \times L_x = \frac{1}{2} \times 12.38 \times 4$$

$$= 24.76 \text{ kN/m}$$

$$\text{Nominal shear stress } \tau_v = \frac{24.76 \times 1000}{1000 \times 150}$$

$$= 0.17 \text{ N/mm}^2$$

% of tensile steel near support

$$= \frac{100 A_{st}}{bd}$$

$$= 100 \times$$

Check for development length

long span $V_u = 12.38 \times \frac{4.15}{2} = 25.69 \text{ kN}$

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

$$= 0.87 \times 415 \times 208.33 \times 141 \left(1 - \frac{208.33 \times 415}{1000 \times 141 \times 20}\right)$$

$$= 10.28 \text{ kN-m}$$

Assuming $\phi = 8$

l_d = development length $= \frac{0.87 f_y \phi}{4 \times 2 b d}$

$$= \frac{0.87 \times 415 \times \phi}{4 \times 1.2 \times 1.6}$$

$$= 47.01 \phi$$

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As per code

$$Ld \leq 1.3\frac{M_1}{V} + l_0$$

$$\Rightarrow 47.01\phi \leq \frac{1.3 \times 10.28 \times 10^6}{25.69 \times 10^3} + 8\phi$$

$$\Rightarrow 39.01\phi \leq 520.20$$

$$\Rightarrow \phi \leq 13.34 \text{ mm} \quad \ldots (\phi_{provid} = 8\text{mm}) \ldots (Ok)$$

Short span $V_u = 12.38 \times \frac{4.15}{2} = 25.69$ kN

$$M_{U1} = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

$$= 0.87 \times 415 \times 436.33 \times 150 \left(1 - \frac{436.33 \times 415}{1000 \times 150 \times 20}\right)$$

$$= 23.62 \text{ kN-m}$$

Assuming $l_0 = 8\phi$

As per code

$$Ld \leq 1.3\frac{M_1}{V} + l_0$$

$$\Rightarrow 47.01\phi \leq \frac{1.3 \times 23.62 \times 10^6}{25.69 \times 10^3} + 8\phi$$

$$\Rightarrow 39.01\phi \leq 1195.25$$

$$\Rightarrow \phi \leq 30.64 \text{ mm} \quad \ldots (\phi_{provided} = 10\text{mm}) \ldots (Ok)$$

check for shear

This is critical along long span.

$$\text{Shear stress } (\tau_v) = \frac{V_u}{bd} = \frac{25.69 \times 10^3}{1000 \times 141}$$

$$= 0.182 \text{ N/mm}^2$$

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% of steel near support $= \frac{100 A_{st}}{bd}$

$= \frac{100 \times 208.33}{1000 \times 141}$

$= 0.148\%$

$\tau_c = 0.28 \text{ N/mm}^2$

for a slab thickness of 170mm $k = 1.25$

$\tau'_c = 0.28 \times 1.25 = 0.35 \text{ N/mm}^2 > \tau_v$ ...(ok)

∴ The slab is safe in shear

Check for deflection

The deflection shall be checked along short span

Basic $\frac{\text{span}}{d}$ ratio $= 20$

% of tensile reinforcement $= \frac{100 \times A_{st}}{bd}$

$= \frac{100 \times 436.33}{1000 \times 150}$

$= 0.29\%$

service stress $= 240 \text{ N/mm}^2$ in fe 415 grade steel

modification factor $= 1.45$

permissible $\frac{\text{span}}{d}$ ratio $= 20 \times 1.45 = 30$

actual $\frac{\text{span}}{d}$ ratio $= \frac{4150}{150} = 27.65 < 30$

...(ok)

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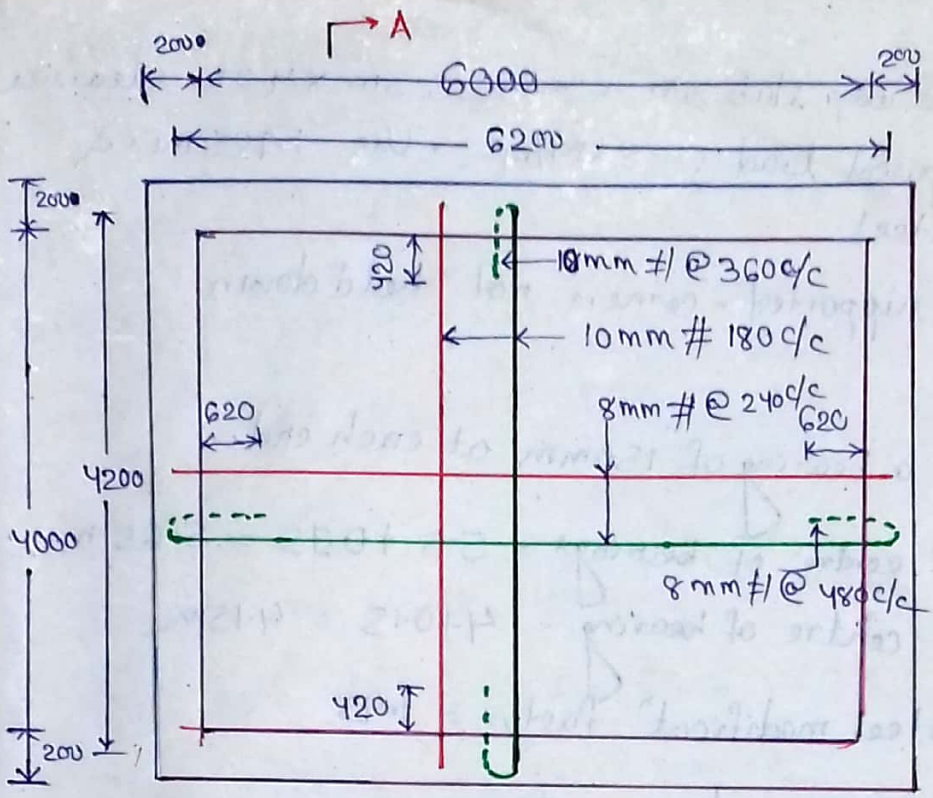
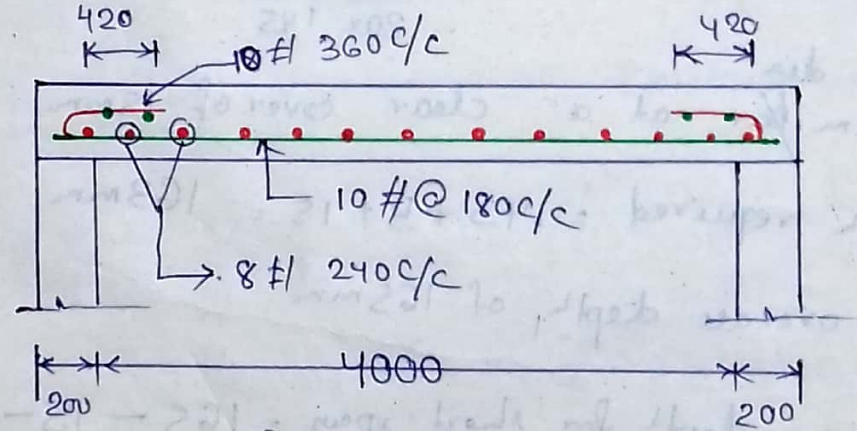
Fig - 41
HJSch



A (a) Plan



(b) Section A-A

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Q Design a cantilever slab to carry a live load of $3000\ N/m^2$. The overhang of the slab is 1.25m. Use M20 concrete & Fe415 steel.

Sol^n

Calcut^n Depth of slab

Assume 0.2% steel, modification factor = 1.65

$$\text{Basic } \frac{\text{span}}{\text{effective depth}} = 7$$

$$\text{Effective depth required} = \frac{\text{span}}{7 \times \text{modificat}^n \text{ factor}}$$

$$= \frac{1250}{7 \times 1.65} = 108.23\ mm$$

Providing 10 mm ϕ bars at a clear cover of 15mm.

Effective cover to reinforcement = 15+5 = 20mm

Overall depth required = 108.23 + 20 = 128.23 mm

Provide an overall depth of 130mm & Reduce the depth to 100 mm

Actual effective depth (d) = 130 − 20 = 110mm

Design load (W_u) & Factored Bending moment (M_u)

DL of slab = $\left(\frac{0.13+0.1}{2}\right) \times 1 \times 25 = 2.875\ kN/m$

finishing load = 1+1 = 1 kN/m² (Assume Finishing load = 1 kN/m²)

Live load = 3 × 1 = 3 kN/m²

Total load on the slab = 5.875 kN/m

factored load (W_u) = 1.5 × 5.875 = 8.8125 kN/m

Max^m factored B.M (M_u) = $\frac{W_u l^2}{2} = \frac{8.8125 \times 1.25^2}{2}$

$= 6.885\ kN\text{-}m$

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Minᵐ effective depth required.

Equating $M_{u,lim}$ to M_u

$$0.138 f_{ck} b d^2 = 6.885 \times 10^6$$

$$\Rightarrow 0.138 \times 20 \times 1000 \times d^2 = 6.885 \times 10^6$$

$$\Rightarrow d^{\#} = 49.95 \text{ mm} < d_{provided} = 110 \text{ mm (OK)}$$

& spacing

Calculatⁿ of Area of steel

Area of steel is given by

$$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

$$\Rightarrow 6.885 \times 10^6 = 0.87 \times 415 \times A_{st} \times 110 \left(1 - \frac{A_{st} \times 415}{1000 \times 110 \times 20}\right)$$

$$\Rightarrow 1.886 \times 10^{-4} A_{st}^2 - A_{st} + 173.36 = 0$$

$$\Rightarrow A_{st} = 179.43 \text{ mm}^2$$

~~Calculate spacing for ste~~

Let us take 10mm ϕ bars for main reinforcement

Spacing of main bars $= \frac{\pi/4 \times 10^2}{179.43} \times 1000 = 437.72$ mm

Check for spacing

Maxᵐ spacing of bars shall not exceed

(i) $3d = 3 \times 110 = 330$ mm

(ii) 300 mm

Hence provide 10mm ϕ bars @ 300 mm c/c.

Actual Area of main bars provided $= \frac{\pi/4 \times 10^2}{300} \times 1000 = 261.80$ mm²

Calculate distribution area of steel.

Provide 0.12% temperature reinforcement in the transverse dirⁿ, that is,

$$= \frac{0.12}{100} \times \left(\frac{130 + 110}{2}\right) \times 1000 = 144 \text{ mm}^2$$

Let us take 8 mm ϕ bars for distributⁿ reinforcement

Spacing of distributⁿ bars $= \frac{\pi/4 \times 8^2 \times (1000)}{144} = 349.06$ mm

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Check for spacing

The spacing of the distribution bars shall not exceed

(i) $5d = 5 \times 110 = 550$ mm

(ii) 300 mm

Hence provide 8mm ϕ bars @ 300mm c/c.

Actual area of distribut^n bars provided $= \frac{\frac{\pi}{4} \times 8^2}{300} \times 1000 = 167.55$ mm^2

Check for shear

S.F. at the support $(V_u) = w_u l = 88125 \times 1.25 = 11.02$ kN

Nominal shear stress $(\tau_v) = \frac{V_u}{bd} = \frac{11.02 \times 10^3}{1000 \times 110} = 0.10$ N/mm^2

percentage of steel $= \frac{100 \times A_{st}}{bd} = \frac{100 \times 261.8}{1000 \times 110} = 0.23\%$

Shear strength of M20 concrete $(\tau_c) = 0.35$ N/mm^2

but shear strength of slab $(\tau_c') = k \times \tau_c$

$= 1.3 \times 0.35 = 0.455$ N/mm^2

$> \tau_v$ (OK)

The cantilever slab is safe in shear and no shear reinforcement is necessary.

Check for development length

development length (l_d) is given by $= \frac{\phi \sigma_s}{4 \tau_{bd}} = \frac{0.87 f_y \phi}{4 \tau_{bd}}$

$= \frac{0.87 \times 415 \times \phi}{4 \times 1.2 \times 1.6}$

$= 47.01 \phi$

$= 47.01 \times 10 = 470.01$ mm

$\simeq 470$ mm

The main bars must be embeded into the beam/lintel upto a distance $L_d = 470$ mm from the fixed edge

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Check for deflection

Basic $\frac{span}{d}$ ratio = 7

Percentage of steel = $\frac{100 \times Ast}{bd} = \frac{100 \times 261.8}{1000 \times 110}$

$= 0.24$

service stress $(f_s) = 0.58 f_y \times \frac{Ast, required}{Ast, provided}$

$= 0.58 \times 415 \times \left(\frac{179.43}{261.8}\right)$

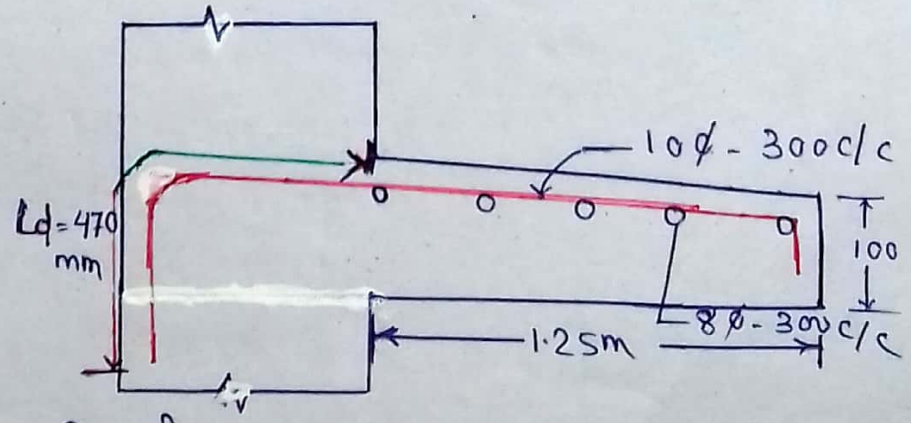
$= 164.96$ (say 190)

So modification factor = 2.0

permissible $\frac{span}{d}$ ratio = $7 \times 2.0 = 14.0$

actual $\frac{span}{d}$ ratio = $\frac{1250}{110} = 11.36 < 14.10$ (OK)

Hence the cantilever slab is satisfactory from deflectn consideratn.



Reinforcement in cantilever slab

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Column:

Assumptions in limit state of collapse : compression

Assumptions cl. 39.1

* A column may be defined as an element used primarily to support axial compressive loads and with a height of at least 3 three times its least lateral dimensions.

* A compression member whose effective length exceeds three times its least lateral dimension is termed as column.

* If the effective length is less than 3 times its lateral dimension, it is termed as pedestal.

* The pedestal may be designed as an unreinforced compression block, however, minimum reinforcement according to the code requirement shall be provided.

* The columns in a building usually carry axial compressive loads. In addition to the axial loads, columns usually carry some moments also.

* A column with only axial load is practically impossible. The moment in column ~~with only axial load~~ may be due to gravity loads, wind loads or earthquake loads.

* A column may be classified based on different criterial such as

(a) Shape of c/s
(b) Slenderness ratio
(c) type of loading
(d) pattern of lateral reinforcement

* The shape of a column may be square, rectangular, circular, polygon or any other shape depending on architectural requirement.

polygon < Hexagonal, octagonal

* Tee shape, ell shape or swastik columns are used to match with the architectural requirement.

* Some of the popular shapes of colum

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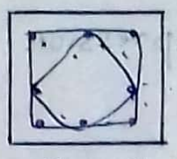
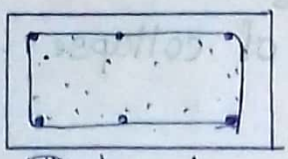


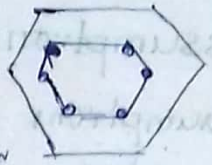
Square



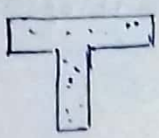
Rectangular



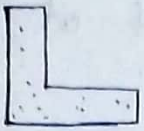
Circular



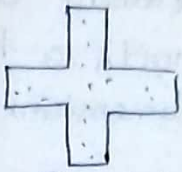
Hexagonal.



Tee



Ell



Swastik (Cruciform)

* A column may be classified as short or long column depending on its effective slenderness ratio

* The ratio of effective length of column to least lateral dimension is referred to as effective slenderness ratio.

* A short column are those which fail primarily due to material Short columns do have buckling effect, but of a limited value

* Long columns are those which fail due to material failure and also due to buckling.

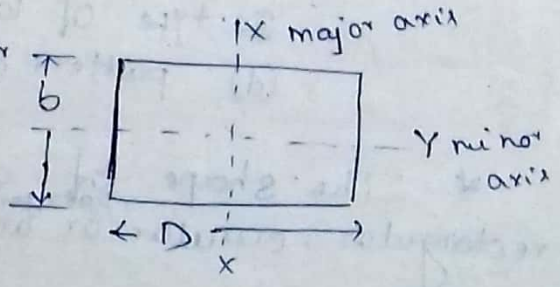
* According to IS: 456, a column may be considered as short when both slenderness ratios $\frac{Lex}{D}$ and $\frac{Ley}{b}$ are less than 12.

Lex = effective length in respect of major axis

Ley = " " " " " minor "

D = depth in respect of the major axis.

b = width of the member.



* It shall be otherwise cosidereded as a slender or a long column. However maxm slenderness ratio of a column should not exceed 60.

* Effective length of a column is defined as the length betⁿ the point of contraflexture of a bucked column.

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$\frac{l_{ex}}{D} > 12 \quad \frac{l_{ey}}{b} < 12$

$\frac{l_{ex}}{D} < 12 \quad , \frac{l_{ey}}{b} > 12$

$\frac{l_{ex}}{D} > 12 \quad , \frac{l_{ey}}{b} > 12$



* A column may be classified as follows based on types of loading

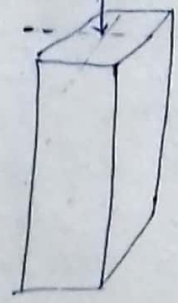
a) axially loaded column

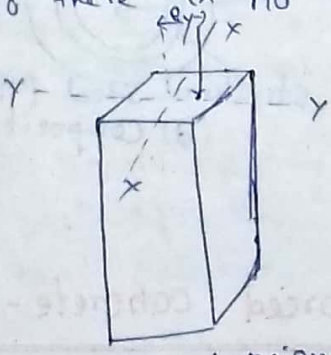
b) a column subjected to axial load and uniaxial bending

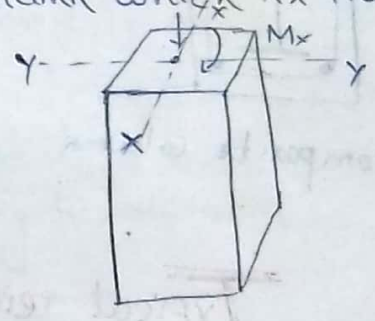
c) a column subjected to axial load & biaxial bending

* Axially loaded columns are those in which the line of act^n of resultant thrust of the loads supported by a column coincides with the C.G. of the column c/s.

* As per IS:456-2000 there is no column which is truely axially loaded.

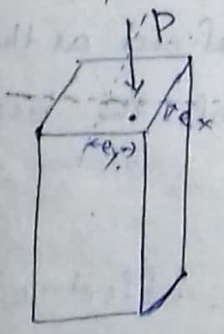


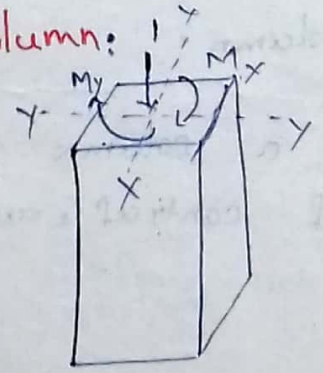


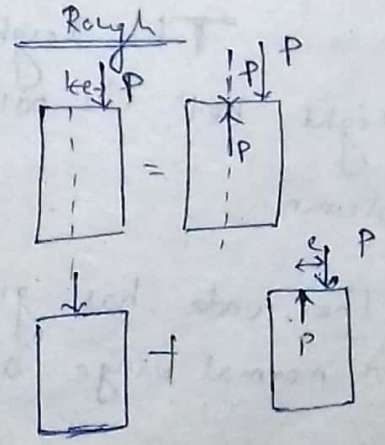


uniaxially eccentrically loaded column

Axially loaded column:







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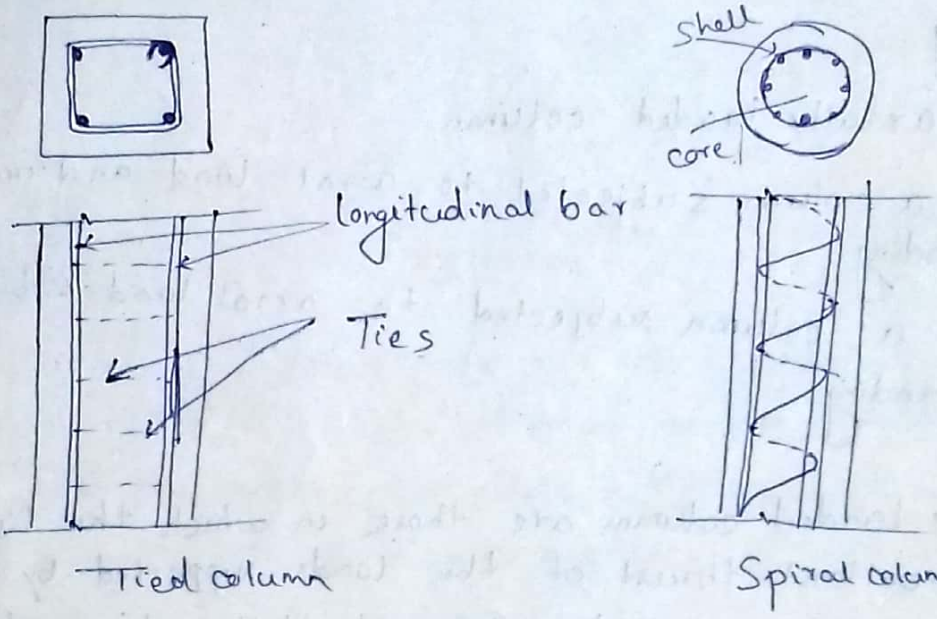
* A reinforced concrete column can also be classified according to the manner in which the longitudinal bars are laterally supported that is

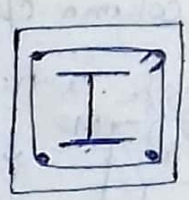
a) tied column
b) spiral "
c) Composite "



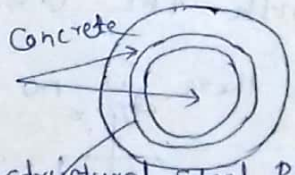
Tied column

Spiral column

(c)



composite column



(d) Composite column

Typical reinforced concrete - column

Effective height of a column:

The height of a column is defined as the height betⁿ the points of contraflexure of the bucked column.

* The code has given certain values of the effective height for normal usage assuming idealized end conditions :

~~cl no~~ - Table No - 28.

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These are briefly discussed as follows:

Tied Column:

* Fig shows a typical tied column where the transverse ties are used as lateral reinforcement restraint.

The function of the longitudinal & transverse ties are as follows:

(i) When the load commences on the column, they support the longitudinal bars so

* RC columns concrete has a high compressive strength & a low tensile strength. Hence theoretically concrete should need no reinforcement when it is subjected to compression.

* Though a column is mainly a compression member, it is liable to some moment due to eccentricity of loads or transverse loads or due to its slenderness ratio.

* Such moments may occur in any dirn and so it is necessary to maintain the posn of the longitudinal reinforcement near all faces of the column.

* These reinforcements provide form the longitudinal steel.

* In order to maintain the position of the longitudinal reinforcement in the form of lateral ties or spirals at close pitch.

* The transverse reinforcement also exists in confining the concrete.

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Specification for min^m & max^m reinforcement

26.5.3

* As per IS 456 a reinforced concrete column shall have min^m steel longitudinal reinforcement & the c/s area of such reinforcement shall not be be less than 0.8% nor more than 6% of the c/s area of the column required to transmit all the loading.

* The object of stipulating a min^m percentage of steel is to ~~provide~~ make provision to prevent buckling of the column due to any accidental eccentricity of the load on it.

* The object of stipulating a max^m percentage of steel is to provide reinforcement within such a limit to avoid congestion of reinforcements which would make it very ~~of steel is to~~ difficult to place concrete & consolidate it.

Clear cover

for slab = 15mm
for beam = 25mm
for column = 40mm
" found^n = 40-60mm
for wall = 20mm

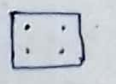
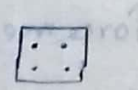
If the size of the column ≤ 200mm provide: 25mm & bars ≯ 25mm
If the size of the column > 200mm : 40mm

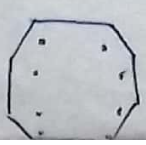
IS Recommendations Regarding longitudinal Reinforcement

Cl - 26.5.3.1

Min^m Number of bars

1) Square column (4)
2) Rectangular " (4)
3) Circular " (6)
4) polygonal column
   1) Hexagonal column (6)
   2) octagonal " (8)

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Transverse reinforcement 26.5.3.2

* The longitudinal reinforcement should be laterally tied by transverse links to provide a restraint against outward buckling of each of the longitudinal bars.

$\nless$ 12mm

transverse $\nless$ 6mm dia

Unsupported length of column: 25.1.3

* The unsupported length of a column shall be taken as the clear distance betn slabs, beams or other members that provid lateral support to the column.

* If the haunches in the beams at the columns or capital/drop panel in flat slab constructn are present, the unsupported length shall be measured from the bottom of the hunch or capital/drop

* It is the distance from the above floor to below the beam/slab.

* The unsupported length, l of a compression member shall be taken as the clear distance betn end restraints

Slenderness limits for column 25.3

* The unsupported length betn end restraints shall not exceed 60 times the least lateral dimension of a column.

* If, in any given plane, one end of a column is unrestrained, its unsupported, its unsupported length, l shall not exceed $\frac{100b^2}{D}$

where b = width of that c/s

D = depth of the c/s measured in the plane under consideratn

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Minᵐ eccentricity

cl-25.4

All columns shall be designed for minᵐ eccentricity, equal to the unsupported length of column/sec plus lateral dimension subjected to a minᵐ of 20mm.

Short Axially loaded members in compression

cl-39.3

when minᵐ eccentricity $\not>$ 0.05 × lateral dimension the members may be designed by the following eqn

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

where P_u = axial load on the member

f_{ck} = characteristic compressive strength of the concrete.

A_c = Area of concrete

f_y = charateristic strength of the compression reinforcement.

A_{sc} = area of longitudinal reinforcement for column

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Problem-1

A short column of size 230 mm × 350 mm is subjected to a factored load of 1500 kN. If the unsupported length of the column is 3.2 m, find out design moments due to min^m eccentricity. Can we apply simplified formula of design in this case?

Soln

Min^m eccentricity about x-axis e_x, and about y axis e_y are calculated and compared with respective sides D and b of the column as follows.

$$e_x = \frac{L}{500} + \frac{D}{30} = \frac{3200}{500} + \frac{350}{30}$$

$$= 18.06 \text{ mm} < 20 \text{ mm}$$

$\therefore e_x = 20 \text{ mm}$

$0.05 D = 0.05 \times 350 = 17.5 \text{ mm} < e_x$

$$e_y = \frac{L}{500} + \frac{b}{30} = \frac{3200}{500} + \frac{230}{30} = 14.06 \text{ mm} < 20 \text{ mm}$$

$e_y = 20 \text{ mm}$

$0.05 b = 0.05 \times 230 = 11.5 \text{ mm} < e_y$

$M_{ux} = M_{uy} = 1500 \times 0.02 = 30 \text{ kN-m}$

Design the column for

$P_u = 1500 \text{ kN}$

$M_{ux} = 30 \text{ kN-m}$

$M_{uy} = 30 \text{ kN-m}$

As $e_x > 0.05 D$ and $e_y > 0.05 b$, simplified formula of design cannot be applied.

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Problem: 2

Design the reinforcement in a column of size 400mm × 600mm subjected to an axial load of 2000 kN under service dead and live load ~~2000 kN~~ The column has an unsupported length of 3m and is effectively held in position & restrained against rotation at both ends. Use M20 grade concrete & Fe 415 grade steel

Page-94

Soln

Size of the column = 400 × 600 mm

b = 400mm D = 600mm

factored load (P_u) = 1.5 × 2000 kN = 3000 kN

unsupported length of the column (l) = 3m

effective length with respect to major / x-axis (l_{ex}) = 0.65 l

= 0.65 × 3m = 1.95

" " " " " minor / y-axis (l_{ey}) = 0.65 l

= 1.95m

slenderness ratio with respect to major axis (x-axis)

$$= \frac{l_{ex}}{D} = \frac{1.95}{0.6} = 3.25 < 12$$

slenderness ratio with respect to minor axis (y axis)

$$= \frac{l_{ey}}{b} = \frac{1.95}{0.4} = 4.875 < 12$$

So the column may be considered as short.

In the dirn of the longer dimension, min^n eccentricity is greater than the following

i) $e_{x,min} = \frac{L}{500} + \frac{D}{30} = \frac{3000}{500} + \frac{600}{30}$

$= 26mm$ ~~> 20mm~~

ii) 20mm

$\therefore e_{x,min} = 26mm$

In the dirn of the shorter dimension, min^m eccentricity is greater than the following

i) $e_{y,min} = \frac{L}{500} + \frac{b}{30} = \frac{3000}{500} + \frac{400}{30}$

$= 19.33$

ii) 20mm

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$\therefore e_{y,min} = 20mm$

$0.05b = 0.05 \times 400 = 20mm$

$0.05D = 0.05 \times 600 = 30mm$

As $e_{x\,min} \neq 0.05D$ & $e_{y,min} \neq 0.05b$

The column can be designed as axially loaded column by using the eqn

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

$$\Rightarrow P_u = 0.4 f_{ck} (A_g - A_{sc}) + 0.67 f_y A_{sc}.$$

$$\Rightarrow 3000 \times 10^3 = 0.4 \times 20 (240000 - A_{sc}) + 0.67 \times 415 \times A_{sc}$$

$$\Rightarrow A_{sc} = 2238.39 mm^2$$

provide ~~6 bars~~ of 20mm dia & 2 bars of 16 mm dia

($A_{sc\,provided} \neq 2287.07 mm^2$)

Lateral ties

a) Dia of ties shall not be less than,

(i) $\frac{1}{4}$ x dia of largest longitudinal bar $= \frac{1}{4} \times 20 = 5mm$

(ii) 6mm

provide 6mm ϕ ties

b) Pitch / spacing of ties shall not exceed,

i) least lateral dimension of the column = 400mm

ii) 16 x dia of the smallest longitudinal reinforcement

$= 16 \times 16 = 256mm$

(iii) 300mm

let us provide 6mm dia bar @ 250mm c/c spacing

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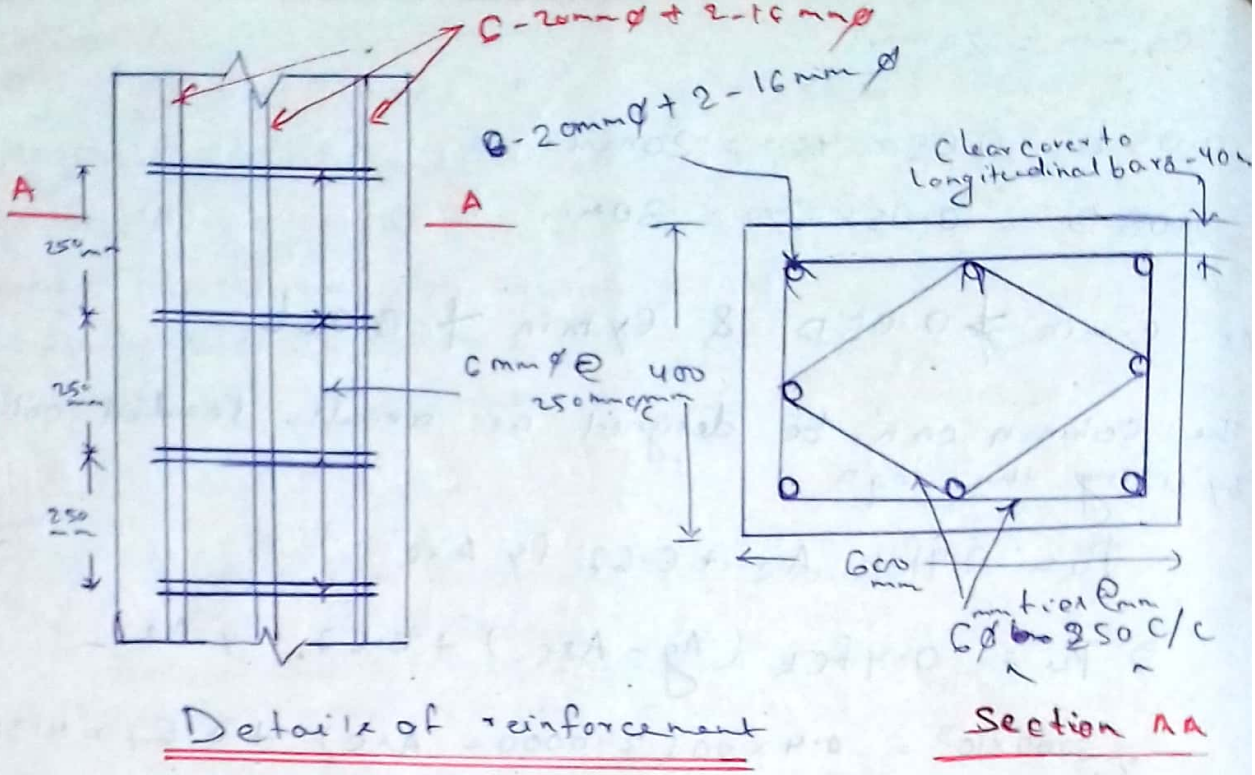

Details of reinforcement

Section AA

Problem - 3

A reinforced concrete short column 400mm × 400 mm has to carry an axial load of 1200 KN. Find the area of steel required. Use M20 concrete and Fe 415 steel.

Soln

Size of the column = 400 mm × 400 mm

i.e. b = D = 400 mm

Characteristic compressive strength of the concrete (fck) = 20 N/mm²

Characteristic strength of compression reinforcement (fy) = 415 N/mm²

Axial load acting on the column = 1200 kN

Factored / ultimate load (Pu) = 1.5 × 1200 = 1800 kN

Ultimate load on the column, given by the eqn

$$P_u = 0.4 f_{ck} A_c + 0.67 \times f_y A_{sc}$$

$$\Rightarrow 1800 \times 10^3 = 0.4 \times 20 \times (160000 - A_{sc}) + 0.67 \times 415 \times A_{sc}$$

$$\Rightarrow 1800 \times 10^3 = 8(160000 - A_{sc}) + 278.05 A_{sc}$$

$$\Rightarrow 1800 \times 10^3 = 1280000 - 8 A_{sc} + 278.05 A_{sc}$$

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⇒ $A_{sc} = 1925.57 mm^2$

provide 4 bars of 20mm ϕ & 4 bars of 16mm ϕ

($A_{sc,provided} = 2060.88 mm^2$)

As per code 15:456:2000

$A_{sc,min} = 0.8\% \times A_g = 0.8\% \times (400 \times 400) = 1280 mm^2$

$A_{sc,max} = 6\% \times A_g = 6\% \times (400 \times 400) = 9600 mm^2$

$\therefore A_{sc,min} < A_{sc,provided} < A_{sc,max}$ (OK)

Lateral ties

a) Dia of ties shall not be less than

i) $\frac{1}{4}$ × dia of Largest longitudinal bar = $\frac{1}{4} \times 20$ = 5mm

ii) 6mm

provide 6mm ϕ ties

b) Pitch/spacing of ties shall not exceed,

i) least Lateral dimension of the column = 400mm

ii) 16 × smallest dia of the longitudinal reinforcement = 16 × 16 = 256mm

iii) 300mm

Let us provide 6mm dia bar @ 250mm c/c spacing

Diagram: similar

Problem

A short RCC column is to carry a factored load of 1900kN. If the column is to be square, design the column. Assume emin < 0.05D. The materials are M20 grade concrete & mild steel.

Soln

Here emin < 0.05D. But emin = 20mm. Therefore the side of the column shall be min^m $\frac{20}{0.05}$ = 400mm i.e 400mm × 400mm

As there is no restriction on size of the column, we may assume min^m percentage of steel to have economy in sect^n. Assume 0.8 percent steel.

Then Asc = 0.008 Ag

Ac = Ag − Asc = 0.992 Ag

ultimate load on the column given by

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$$

$$\Rightarrow 1900 \times 10^3 = 0.4 \times 20 \times 0.992 A_g + 0.67 \times 250 \times 0.008 A_g$$

$$\Rightarrow 1900 \times 10^3 = 7.936 A_g + 1.34 A_g$$

$$\Rightarrow 1900 \times 10^3 = 9.276 A_g$$

$$\Rightarrow A_g = 204829.67 \text{ mm}^2$$

If the column is to be a square, the side of the column = 452.58 mm

Adopt ~~450 × 450~~ mm size column, 455mm × 455mm

Gross area of concrete provided = 455×455 = 207025 mm^2

" " " required = 452.8×452.8 = 205027.84 mm^2

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Longitudinal area of steel required

$= 0.8\% \times 205027.84$

$= 1640.22\ mm^2$

provide 6 no 20 mm dia bars giving $A_{sc} = 6 \times \frac{\pi}{4} \times 20^2$

$= 1884.95\ mm^2$

The section is shown in fig. Note that the distance betⁿ the bars exceed 300mm on two parallel sides. Therefore, the arrangement of reinforcement should be changed

Provide 4 nos of 18 mm diameter bar plus 4 no. 16mm dia bars giving

$A_{sc,provided} = 4\left(\frac{\pi}{4} \times 18^2 + \frac{\pi}{4} \times 16^2\right) = 1822.12\ mm^2$

Lateral ties

a) Dia of ties shall not be less than

i) $\frac{1}{4} \times$ dia of largest longitudinal bar

$= \frac{1}{4} \times 18 = 4.5\ mm$

ii) 6 mm

provide 6 mm ϕ ties

(b) Pitch/spacing of ties shall not exceed,

i) least lateral dimension of the column

$= 455\ mm$

ii) 16 × smallest dia of the longitudinal reinforcement $= 16 \times 16 = 256\ mm$

iii) 300 mm

Let us provide 6mm ϕ ties about 250 mm c/c. This is shown in fig (b).

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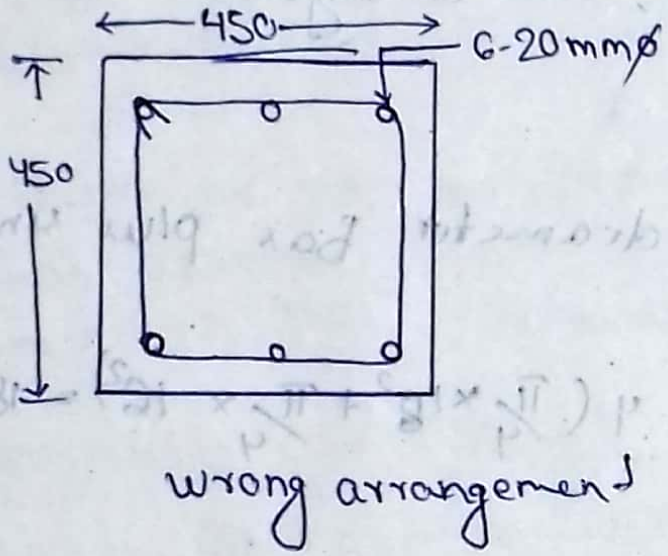
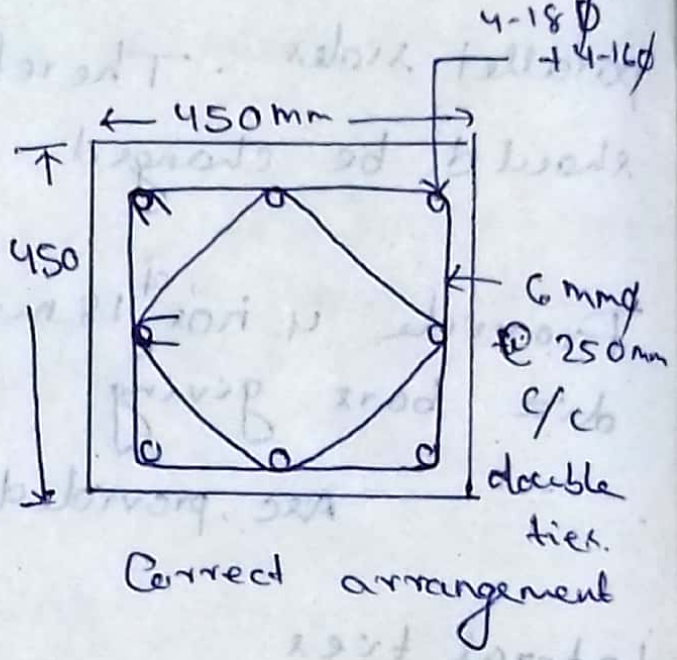
Note that distance betⁿ corner bars in one face equal to 455 - 80 - 18 = 357. This is more than 48 ϕ_{tr} (48 × 6 = 288 mm). Therefore two sets of closed ties shall be used. Note that if this distance would be less than (48 × 6mm) open ties for internal bars would be sufficient



Wrong arrangement



Correct arrangement

Q A 4m high column is effectively held in position at both ends and restrained against rotation at one end. It's size is restricted to 50cm. Calculate the reinforcement if it is required to carry a factored axial load of 1500 kN. Use M20 mix and Fe 415 grade steel.

Soln

Effective height of a column effectively held in posⁿ at both ends and restrained against rotatⁿ at one end. $= 0.80L = 0.8 \times 4000 = 3200$ mm

slenderness ratio $= \frac{l_{eff}}{D} = \frac{3200}{500} = 6.4 < 12$

∴ It is a short column

Minm eccentricity $\frac{l}{500} + \frac{D}{30}$ is greater than the following

i) $\frac{L}{500} + \frac{D}{30} = \frac{4000}{500} + \frac{500}{30} = 24.67$ mm

ii) 20mm

$e_{min} = 24.67$ mm

Also $0.05D = 0.05 \times 500 = 25$ mm

As $e_{min} \not> 0.05D$

The strength of the column is given by

$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$\Rightarrow 1500 \times 10^3 = 0.4 \times 20 ([400 \times 400] - A_{sc}) + 0.67 \times 415 A_{sc}$

$\Rightarrow 1800 \times 10^3 = 8[160000 - A_{sc}) + 278.05 A_{sc}$

$\Rightarrow 1800 \times 10^3 = 1280000 - 8A_{sc} + 278.05 A_{sc}$

$\Rightarrow 520000 = 270.05 A_{sc}$

$\Rightarrow A_{sc} = 1925.56$ mm

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As per min^m provide

As per IS : 456 2000

$A_{sc,min} = 0.8\% \times A_g = 1280\ mm^2$

$A_{sc,max} = 6\% \times A_g = 9600\ mm^2$

provide 6 bars of 22mm dia ($A_{sc,provided} = 2280.80\ mm^2$)

$\therefore A_{sc,min} < A_{sc,provided} < A_{sc,max}$ (ok)

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Axially loaded spiral columns:

The code gives the following expression

$$\frac{\text{volume of spiral in one loop}}{\text{volume of core for a length } p} \geq 0.36\left(\frac{A_g}{A_c} - 1\right) \times \frac{f_{ck}}{f_y}$$

$$= \frac{\pi (D_c - \phi_{sp}) a_{sp}}{\pi/4 \, D_c^2 \times p}$$

$$= \frac{\pi D_c \times a_{sp}}{\pi/4 \times D_c^2 \times p} \quad \text{neglecting the value of } \phi_{sp}$$

$$= \frac{4 a_{sp}}{p D_c}$$

A_c = Area of the core $= \pi/4 \times D_c^2$

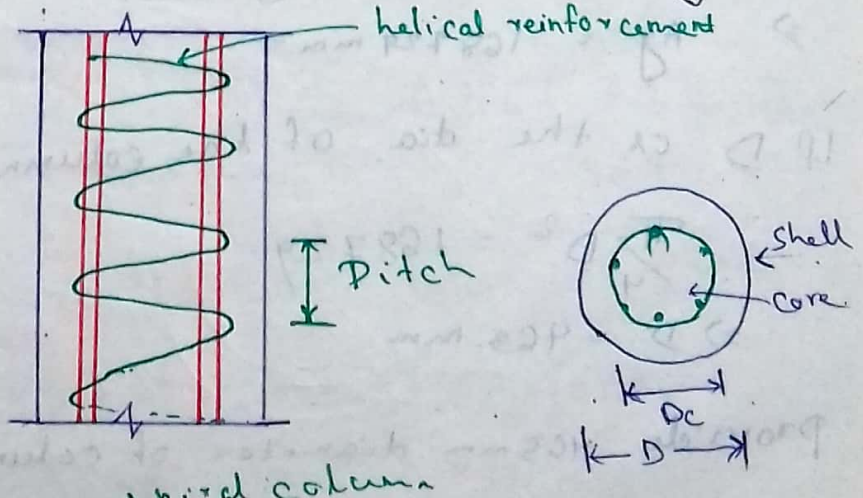
D_c = Diameter of the core, outer to outer of spiral

ϕ_{sp} = diameter of spiral wire

a_{sp} = area of spiral wire $= \pi/4 \times \phi_{sp}^2$

p = pitch of helix

$$\frac{4 a_{sp}}{p D_c} \geq 0.36\left(\frac{A_g}{A_c} - 1\right) \times \frac{f_{ck}}{f_y}$$



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Q. Design a circular short column to carry an axial working load of 1200kN. Assume $e_{min} < 0.05D$

Use (a) lateral ties
(b) helical reinforcement

* The materials are M20 grade concrete & HYSD reinforcement of grade Fe 415.
For lateral reinforcement mild steel may be used

(b) Helical reinforcement
factored load (P_u) = $1.5 \times 2000 = 1800$kN
The column with helical reinforcement can support 1.05 times the load of a similar member with lateral ties.

Therefore

$$P_u = 1.05[0.4 f_{ck} A_c + 0.67 f_y A_{sc}]$$

$$\Rightarrow 1800 \times 10^3 = 1.05 [0.4 \times 20 \times$$

Assume min^m steel = 0.8 % of gross sect^{n}al area of the column

Then $A_{sc} = 0.008 A_g$

$A_c = 0.992 A_g$

substituting we have

$$1800 \times 10^3 = 1.05 [0.4 \times 20 \times 0.992 A_g + 0.67 \times 415 \times 0.008 A_g]$$

$$\therefore 1800 \times 10^3 = 10.668 A_g$$

$$\Rightarrow A_g = 168729 \text{ mm}^2$$

If D is the dia of the column

$$\frac{\pi}{4} D^2 = 168729$$

$$\Rightarrow D = 463 \text{ mm}$$

Provide 465mm diameter of column

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Area of longitudinal steel = $0.008 \times A_g = 0.008 \times 168729$

$= 1349.83 \text{ mm}^2$

Use 6-18 mm diameter bars giving

$A_{sc} = 1526.81 \text{ mm}^2$

Assume 6 mmϕ MS bars for helix at 40 mm clear cover.

~~Dc~~

The code requires that :

$$p_s = \frac{\text{volume of spiral in one loop}}{\text{volume of core for a length } p} \geq 0.36\left(\frac{A_g}{A_c} - 1\right)\frac{f_{ck}}{f_y}$$

A_g = gross concrete area $= \frac{\pi}{4} \times 465^2 =$ ~~169822.71~~ mm^2

Core Diameter $D_c = 465 - 2 \times 40 + 2 \times 6 = 397$ mm

A_c = ~~core~~ area of the core $= \pi/4 \times 397^2$

$$\therefore \frac{A_g}{A_c} = \frac{465^2}{397^2} = 1.37$$

$$\therefore 0.36\left(\frac{A_g}{A_c} - 1\right)\frac{f_{ck}}{f_y} = 0.36\left(1.37 - 1\right) \times \frac{20}{250}$$

$$= 0.010656$$

$$p_s = \frac{\text{volume of spiral in one loop}}{\text{volume of core for a length } p} = \frac{\pi \times a_{sp} \times (D_c - d_{sp})}{\pi/4 \times D_c^2 \times p}$$

$$= \frac{4\, a_{sp}}{D_c\, p}$$

$$\therefore p_s = \frac{4 \times \pi/4 \times 6^2}{397 \times p} \geq 0.36\left(1.37 - 1\right) \times \frac{20}{250}$$

$$= \frac{4 \times \pi/4 \times 6^2}{397 \times p}$$

As per code

$$\frac{4 \times \pi/4 \times 6^2}{397 \times p} \geq 0.010656$$

$$\Rightarrow 26.73 \geq p \quad \Rightarrow p \leq 26.73 \text{ mm}$$

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The code requires that pitch P

i) $\not> 75$ mm

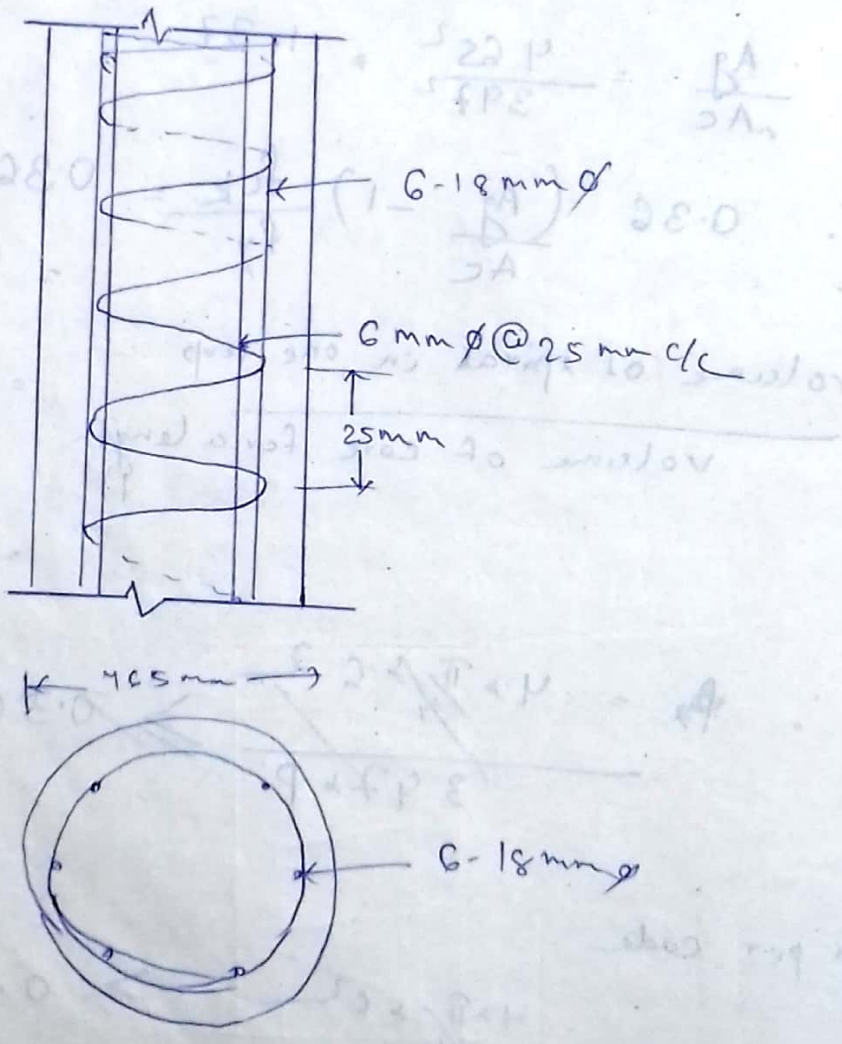
ii) $\not> \frac{1}{6} \times 397 = 66.16$ mm

iii) $\not< 25$ mm

iv) $\not< 3 \times 6 = 18$ mm

Adopt 6mm helix with pitch @ 25mm c/c

* Helical reinforced columns are very ductile as compared with the columns with lateral ties. Hence, columns with helical reinforcement are most desirable in highly seismic zones.



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Footing

The loads from the supporting members of the structure like columns, walls, etc, known as super structure should be safely transmitted to the soils

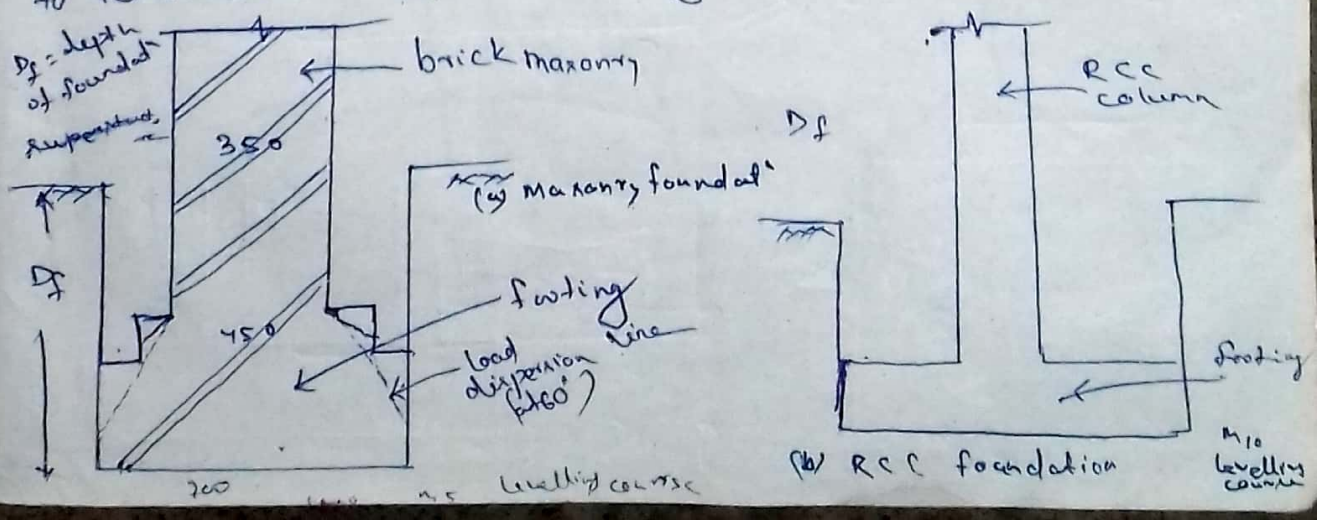
* The permissible pressure on soils is much less than that on the concrete column or a wall. Therefore it becomes necessary to spread the load over a sufficiently larger area.

* To transmit the load from the masonry wall for example, the area is increased slowly by providing a no of masonry steps so that the load dispersion lines remain within the steps as shown in fig (a)

* For the concrete column, the loaded area is suddenly increased by providing an interfacing element known as footing bet^n the column & the soil. It is possible to increase the area suddenly like this because the bearing strength of concrete is sufficiently larger.

* In general a spread constructed in brickwork, stone masonry or concrete under the base of a wall or column for the purpose of distributing the load over a larger area is known as footing.

* Usually lean concrete (MS, M7.5, M10, PCC) is provided below the footing and above the soil to give more protect^n to reinforcement & to properly level the ground.



Classification of foundations:

The found^{n} are classified as

1) flexible & rigid
2) Shallow & deep

(1) flexible & rigid foundation:

This classification is based on the rigidity of the footings. Flexible footing is free to bend & follow the theoretical settlement profile such as rubber pad. Soil bearing pressure on such footings is always uniform. The rigid footing on the other hand does not bend and settles uniformly such as RCC footing.

(2) Shallow and Deep foundation

This classification is based on the ratio of depth of foundat^{n} Df to the width of found^{n} B

* The foundation is classified as.

Shallow if $\frac{Df}{B} \leq 2$

deep if $\frac{Df}{B} > 2$

In most practical cases for shallow foundat^{n} usually $\frac{Df}{B} = 0.25 - 1.0$ where as for deep foundation this ratio is usually 5-20

* Spread, footings, combined footings, mat footing, raft etc are the examples of shallow found^{n}, where as piles, piers, wells are the example of deep found^{n}.

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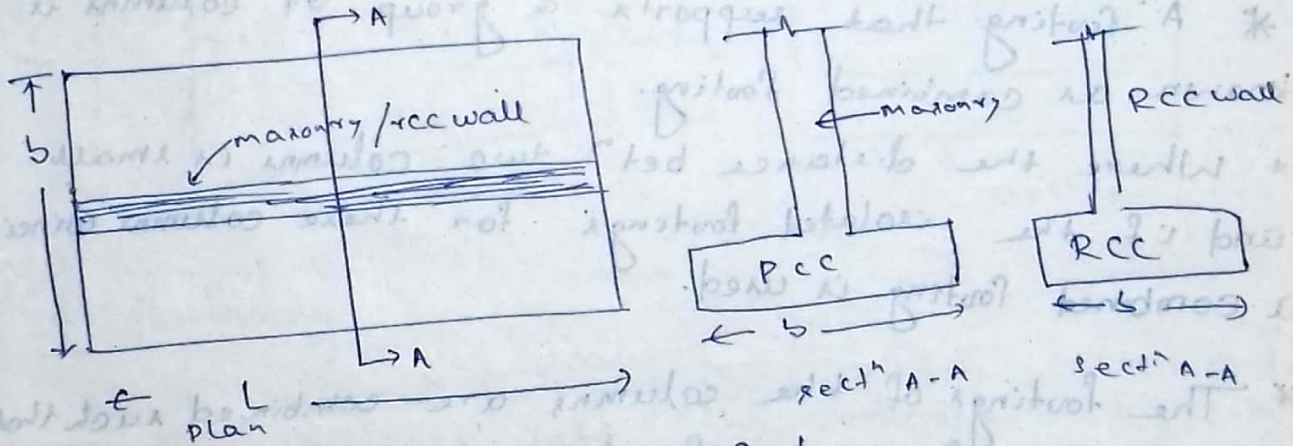
Types of footings

Some of the common footings usual in general building construction are as follows:-

(1) Continuous wall footing
(2) Isolated footing
(3) Combined "
(4) Strap "
(5) Strip footing
(6) Raft foundation
(7) pile foundation

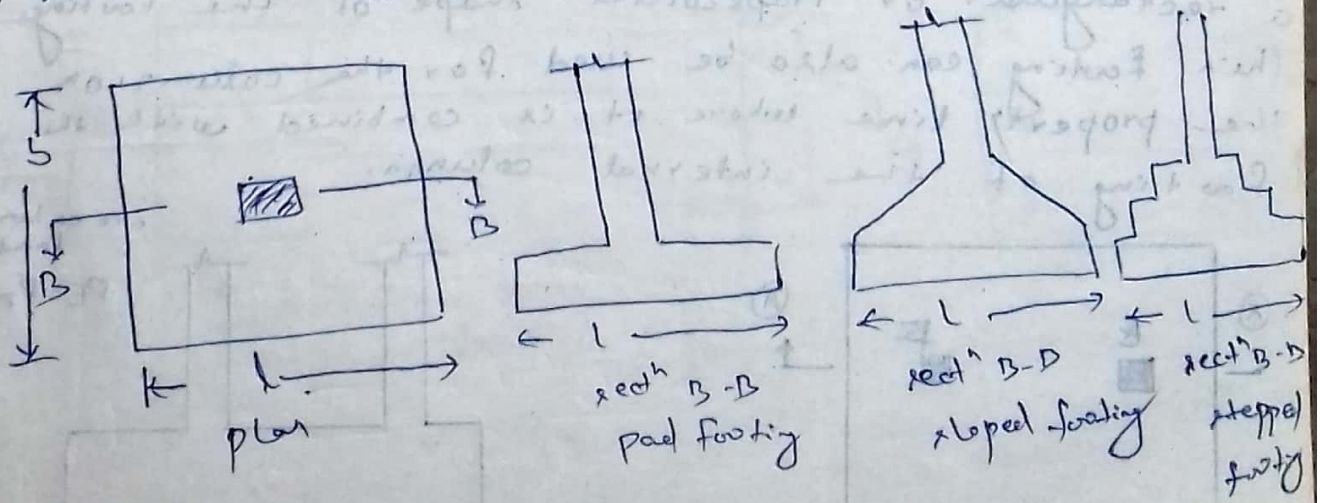
(1) Continuous wall footing

A footing that supports a continuous long masonry or rcc wall is known as continuous footing. In this case, the width of the footing is very small than the length of the footing



(a) Continuous wall footing

(2) Isolated footing



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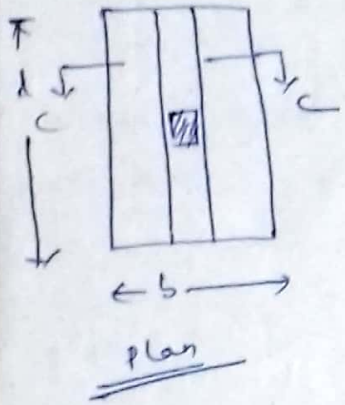
An individual footing under a single column is known as an isolated footing

* where good soil & sufficient area is available, these footings are economical. These may be pad, stepped, stepped or with isolated beam & slab type footings



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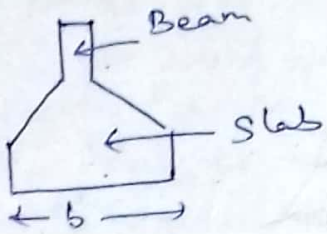


rectⁿ cc

(b) column footting

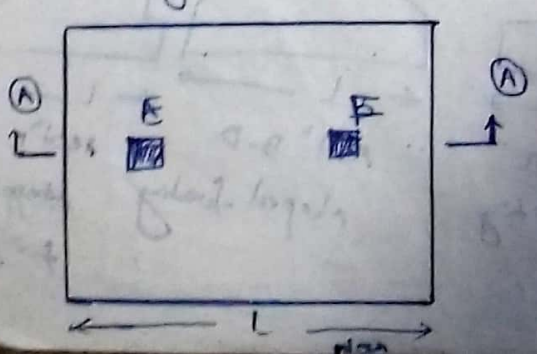
Isolated footing

(3) Combined footing

* A footing that supports a group of columns is known as combined footing.

* Where the distance betⁿ two columns is small and if the isolated footings for these columns coincide a combined footing is used.

* The footings of the columns are combined such that the centre of gravity of column loads and that of the combined footing coincide. This may result in a rectangular or trapezoidal shape of the footing. This footing can also be used for the column on the property line where it is combined with the footing of the internal column.



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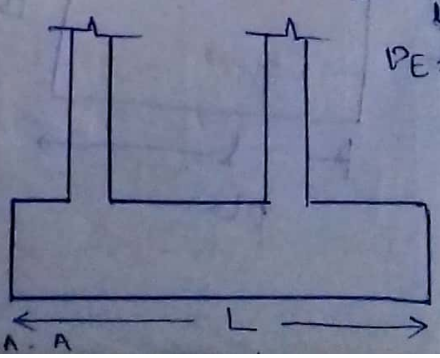
are column loads.

$P_E = P_F$



rectⁿ A-A

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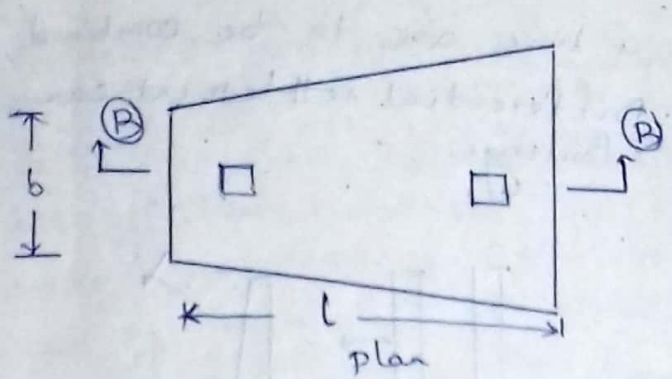
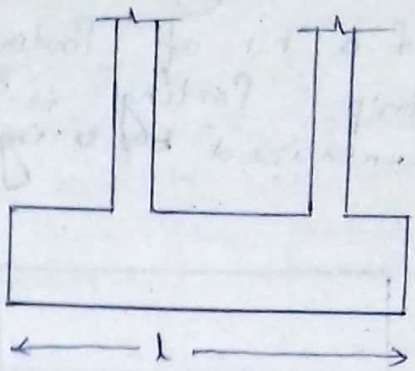
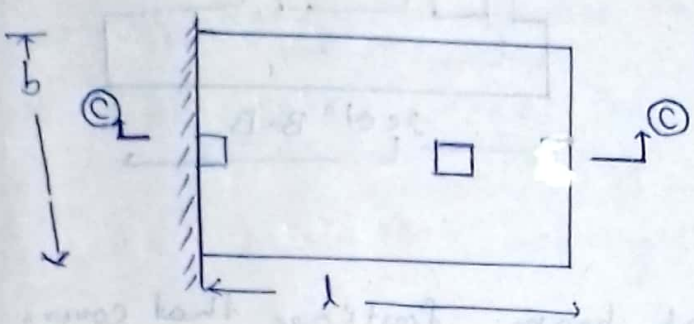
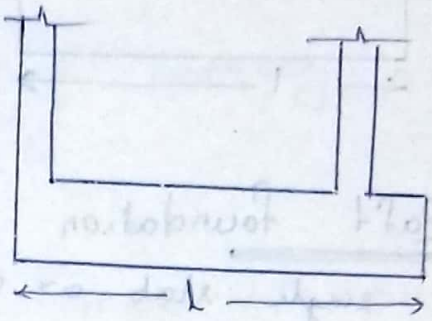
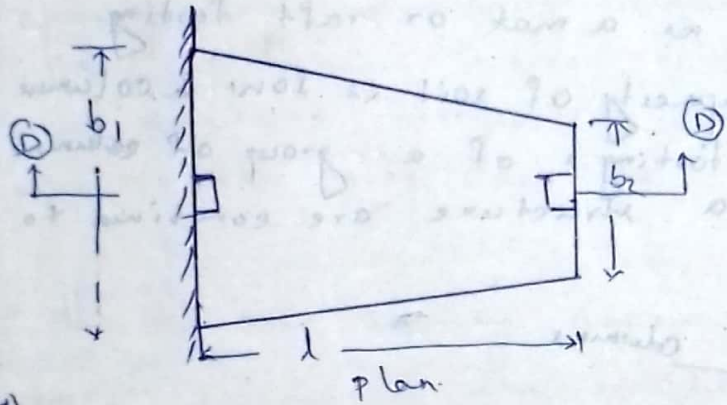
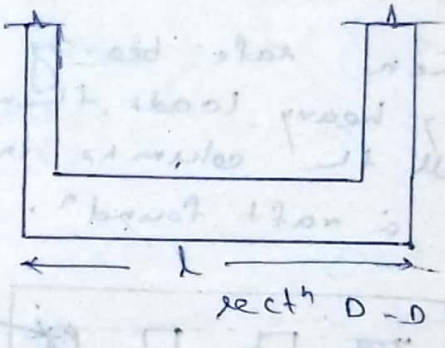






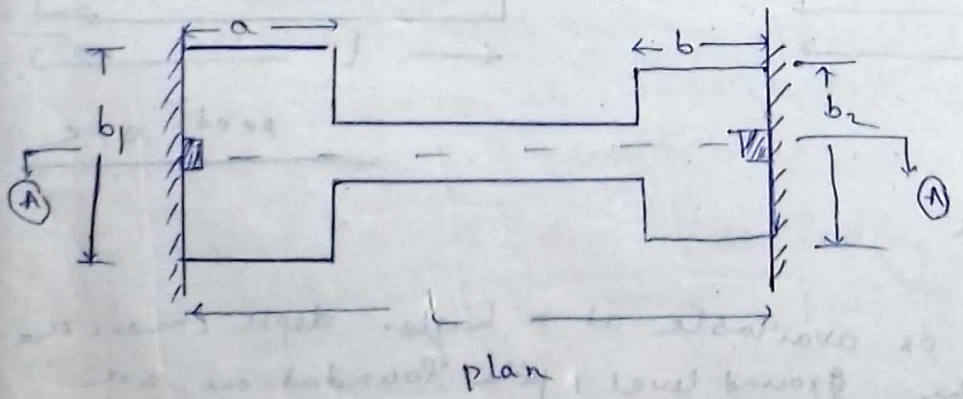


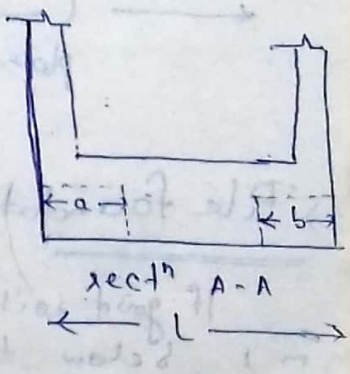




4) Strap footing

If a combined footing is required due to site condⁿ but the distance bet^n the columns is large, a strap footing is used for economy





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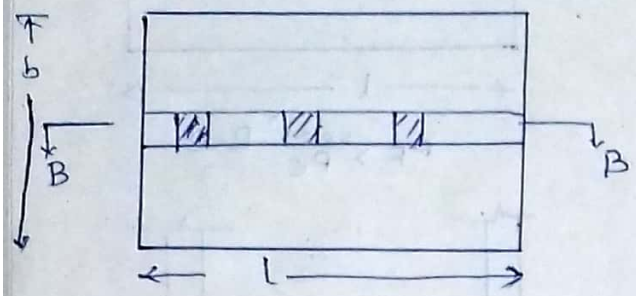
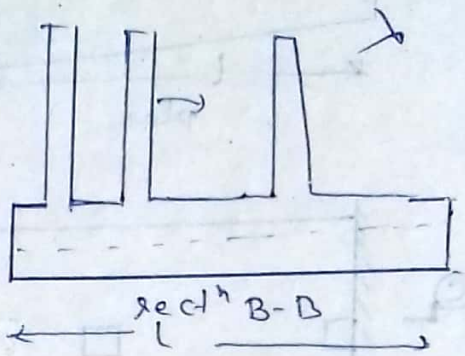
5) Strip footing

If a no of footings in a line are to be combined, a strip footing is used. Differential settlement can be minimized by using such footings.

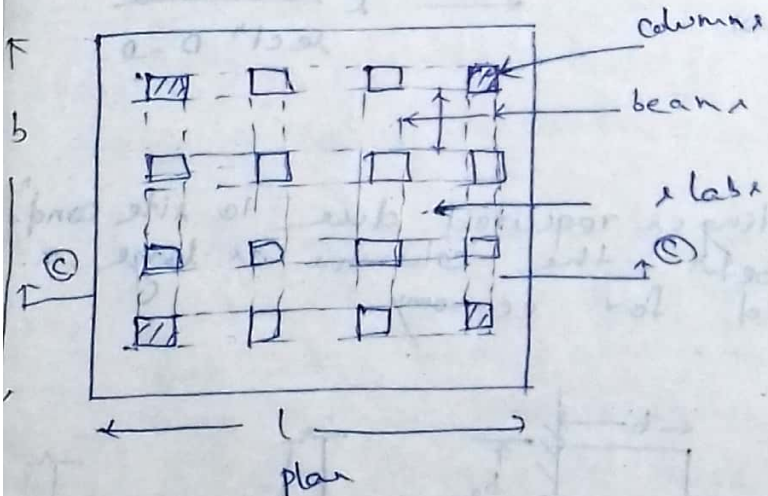


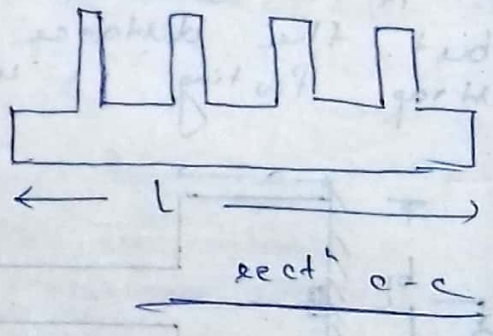


6) Raft foundation

A single slab or a slab beam footing that covers the entire stratum beneath, the entire area of the superstructure is known as a mat or raft footing.

* When safe bearing capacity of soil is low & columns carry heavy loads, then footings of a group of columns or all the columns in a structure are combined to form a raft found^n.





7) Pile foundat^n

If good soil is available at a higher depth (more than 3 m) below the ground level, pile foundations are economical.

* Pile transfer the loads from columns to the hard soil by ~~friction~~ end bearing and to the surrounding

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soil by friction.

Bearing Capacity of Soil.

* The size of foundation depends on the permissible bearing capacity of the soil. Total load per unit area under the footing must be less than the permissible bearing capacity of the soil to prevent excessive settlement. In general, foundations have to resist vertical load, horizontal load and moments.

* Two terms used for bearing capacity
  1) Gross bearing capacity
  2) Net bearing "

* Gross bearing capacity is the total safe bearing ~~capacity and net bearing capacity~~ pressure at the bottom of the footing including the load of superstructure, weight of the footing and that of the earth lying over the footing.

* Net safe bearing capacity is the gross bearing capacity minus the weight per unit area dug out of trench.

Depth of Foundation

* IS: 1080-1985 requires that in all soils a minm depth of 50 cm is necessary. However, if good rock is met at smaller depth, only removal of top soil may be sufficient.

* An estimate of depth of footing below ground level may be obtained by using the Rankine formula. that is,

$$h = \frac{P}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

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h = minimum depth of found^n

p = gross bearing capacity

γ = density of soil

ϕ = angle of repose of soil

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Problem-1

Determine the area and depth of foundation of a square column carrying 1000KN vertical load. The gross bearing capacity of soil is $100 KN/m^2$, density is $17 KN/m^3$ and angle of repose is 29°.

Solution

Load on column W = 1000KN

Aproximate area of footing $= \frac{W}{P} = \frac{1000}{100} = 10 m^2$

Minimum depth of foundation is given by

$$h = \frac{P}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

$$= \frac{100}{17}\left(\frac{1-\sin 29^\circ}{1+\sin 29^\circ}\right)^2$$

$$= 0.71 m$$

Weight of foundation including earth $= 17 \times 10 \times 0.71 = 120.7$

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Total load on foundation = 1000 + 120.7 = 1120.7 kN

Area of foundation required = $\frac{1120.7}{100}$ = 11.21 m^2

Let us revise the area of foundation due to increased self weight of foundation and earth

weight of foundation and eath = 17 × 11.21 × 0.71 = 135.07 kN

Total load on foundation = 1000 + 135.07 = 1135.07 kN

Area of foundation required = $\frac{1135.07}{100}$ = 11.35 m^2

Adopt a foundation having an area of 3.4m × 3.4m (11.56m^2) at a depth of 0.71m from the ground level.

Isolated Footings

* Isolated footings are most commonly used footings for R.C.C. columns because of the simplicity and economy.

* For any column footings, if provision of isolated footing is not possible due to geometry or any other reason, only then other types of footings may be thought.

* The isolated footings for the column may be a pad footing (constant thickness) or a sloped footing.

* It may be axially loaded or eccentrically loaded.

* Sometimes a pedestal is used to achieve economical design.

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Problem: 2

Design a square spread footing to carry a column load of 1000KN from a 40cm square tied column containing 20mm bars as the longitudinal steel. The bearing capacity of soil is 100KN/m². Consider base of footing at 1m below the ground level. The unit weight of earth is 20KN/m³. Use fck = 15N/mm², fy = 415N/mm² and load factor of 1.5.

Solution

Size of footing

Axial load = 1000KN

Aproximate area of footing = $\frac{1000}{100} = 10m^2$

weight of footing including earth = $10 \times 1 \times 20 = 200KN$

Total load on foundation = 1000 + 200 = 1200KN

Actual area of the footing = $\frac{1200}{100} = 12m^2$

provide 3.5m × 3.5m square footing giving total area = $12.25m^2 > 12m^2$

Bending moment

The net earth pressure acting upward due to factored loads is.

$$p = \frac{1000 \times 1.5}{12.25} = 122.45 KN/m^2$$

(where 1.5 is the partial safety factor

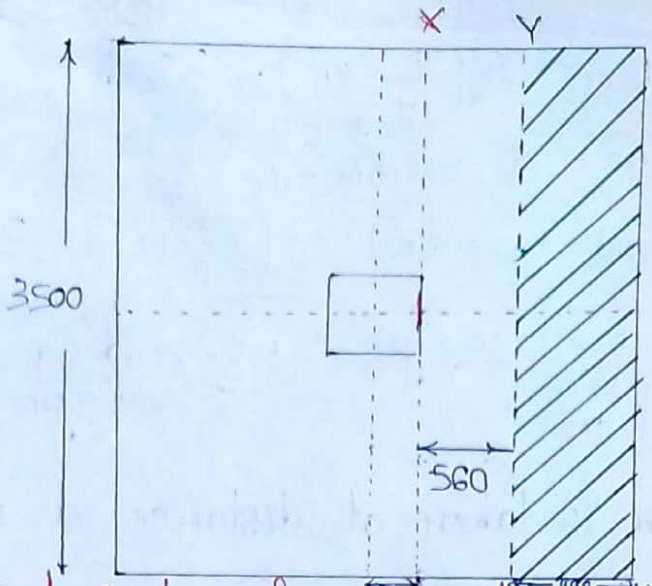

Fig (1) Critical section for moment and one-way shear.

Bending moment about an axis x-x passing through the face of the column as shown in fig (1)

$$= 122.45 \times 3.5 \times \left(\frac{3.5-0.4}{2}\right)^2 \times \frac{1}{2}$$

$$= 514.83 \text{ KN-m}$$

The effective depth required is

$$BM = 0.138 f_{ck} b d^2$$

$$\Rightarrow 514.83 \times 10^6 = 0.138 \times 15 \times 3500 \times d^2$$

$$\Rightarrow d = 266.57 \text{ mm}$$

provide D = 600mm
d = 550mm

Adopt 560mm effective depth and ~~600mm~~ 610 overall depth. Increase depth is taken due to shear consideration.

Area of tension steel is given by

$$B.M = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{b d f_{ck}}\right)$$

$$\Rightarrow 514.83 \times 10^6 = 0.87 \times 415 \times A_{st} \times 560 \left(1 - \frac{A_{st} \times 415}{3500 \times 560 \times 15}\right)$$

$$\Rightarrow 2546.29 = A_{st} - 1.411 \times 10^{-5} A_{st}^2$$

$$\Rightarrow 1.411 \times 10^{-5} A_{st}^2 - A_{st} + 2546.29 = 0$$

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or Ast = $2645 mm^2$

spacing of 10 mm ϕ bars = $\frac{\left(\frac{\pi}{4}\times 10^2\right)\times 3500}{2645} = 103.92$ mm

provide 10mm ϕ bars @ 100 mm c/c

Actual area of steel provided = $\frac{\left(\frac{\pi}{4}\times 10^2\right)\times 3500}{100} = 2748.89\ mm^2$

percentage of steel p = $\frac{100\,Ast}{bd} = \frac{100\times 2748.89}{3500\times 560} = 0.14\%$ $> 2645 mm^2$ (ok)

Shear one-way action.

The critical section is taken at distance d away from the face of column as shown in fig. 1

Shear force (V_u)

$= 122.45\times 3.5\left[\left(\frac{3.5-0.4}{2}\right)-0.56\right]$

$= 424.29$ KN

Nominal shear stress (τ_v) = $\frac{V_u}{bd}$

$= \frac{424.29\times 10^3}{3500\times 560}$

$= 0.216\ N/mm^2$

percentage of tensile steel = $\frac{100\,Ast}{bd}$

$= \frac{100\times 2645}{3500\times 560}$

$= 0.135$

So design shear strength of M15 concrete with 0.14% steel

$\tau_c = 0.28\ N/mm^2$

$> \tau_v$ OK

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Shear two-way action

The critical section is taken at a distance 0.5d away from the face of column as shown in fig (2)

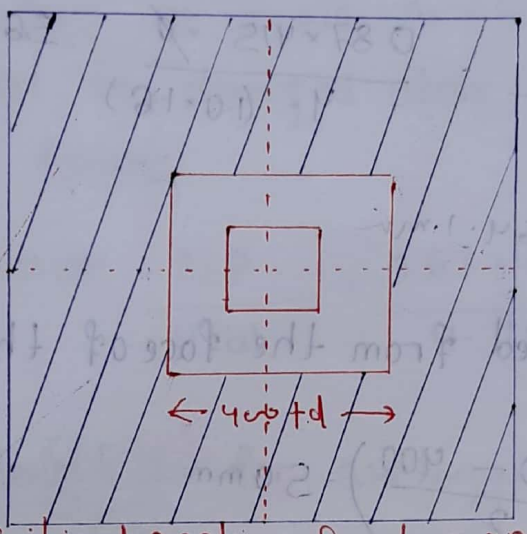

Fig (2) Critical section for two-way shear

Shear force $(V_u) = 122.45 \left[12.25 - (0.40+0.56)^2 \right]$

$= 1387.17 \text{ KN}$

Nominal shear stress $(\tau_v) = \frac{V_u}{b_0 d}$

$= \frac{1387.17 \times 10^3}{4(400+560) \times 560}$

$= 0.645 \text{ N/mm}^2$

Design

Shear strength of M15 concrete

$\tau'_c = k_s \tau_c$

$k_s = (0.5 + \beta_c)$

$\beta_c = \frac{\text{length of shorter side of column}}{\text{length of longer side of column}} = 1$

$k_s = 0.5 + 1 = 1.5$

$k_s = 1.5 \not> 1$

$\therefore k_s = 1.0$

$\tau'_c = 1 \times 0.25 \sqrt{f_{ck}}$

$= 1 \times 0.25 \sqrt{15} = 0.97 \text{ N/mm}^2 > 0.645 \text{ N/mm}^2$

OK

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This shows that a footing having an effective depth 266 mm would not be safe in shear.

Development of reinforcement

Development length for 10mm bars

$$L_d = \frac{\sigma_s \phi}{4\tau_{bd}} = \frac{0.87 \times 415 \times \phi}{4 \times (1.0 \times 1.6)} = 56.41\,\phi$$

$\therefore L_d =$ ~~560mm~~ 564.1 mm

Actual embedment provided from the face of the column is

$$= \left(\frac{3500 - 400}{2}\right) - 50\,mm$$

$$= 1500\,mm > L_d \quad (OK)$$

Load transfer from column to footing

Nominal bearing stress in the column concrete

$$\sigma_{br} = \frac{P_u}{A_c} = \frac{1.5 \times 1000\,kN \times 10^3}{400 \times 400} = 9.38\,N/mm^2$$

where 1.5 is the partial safety factor

Allowable bearing stress $= 0.45 f_{ck} = 0.45 \times 15 = 6.75\,N/mm^2$

$< 9.4\,N/mm^2$ (NG)

Thus the column load cannot be transferred by bearing alone

To carry the excess load P $= 9.4 \times 400 \times 400 - 6.75 \times 400 \times 400$

$= 424000\,N$

Required area of steel $A_s = \frac{424000}{0.67 \times 415} = 1525\,mm^2$

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Minimum As = 0.5% of column area

$= 0.5\% \times 400 \times 400$

$= 800\ mm^2$

$< 1525\ mm^2$ OK.

provide 5-20mm bars as dowels. As = $1570\ mm^2$. The stress in 20mm dowels must be developed above and below the junction of column and footing.

Ld for compression $= \frac{\sigma_s \phi}{47 bd} = \frac{0.67 \times 415 \phi}{4 \cdot (1.6 \times 1.25 \times 1)} = 34.75\phi$

$= 34.75 \cdot 20$

$= 695mm$

The available length L_1 for anchorage is

$L_1 = 600 - 35 - 2 \cdot 10 - 20$ (50)

$= 525mm < 695mm$ → 520mm < 695mm → 510mm < 695mm NG.

Let us provide smaller diameter bars as dowels so that the available vertical length in the footing is sufficient for anchorage.

Use 16mm bars as dowels

$\frac{34.75 \times 12}{} = 417mm < \frac{L_1}{520mm}$ (OK)

Development length $= 34.75 \times 16 = 560mm$

$\simeq L_1$ OK

provide 8-16mm bars As = $1600\ mm^2$

$> 1525\ mm^2$ OK

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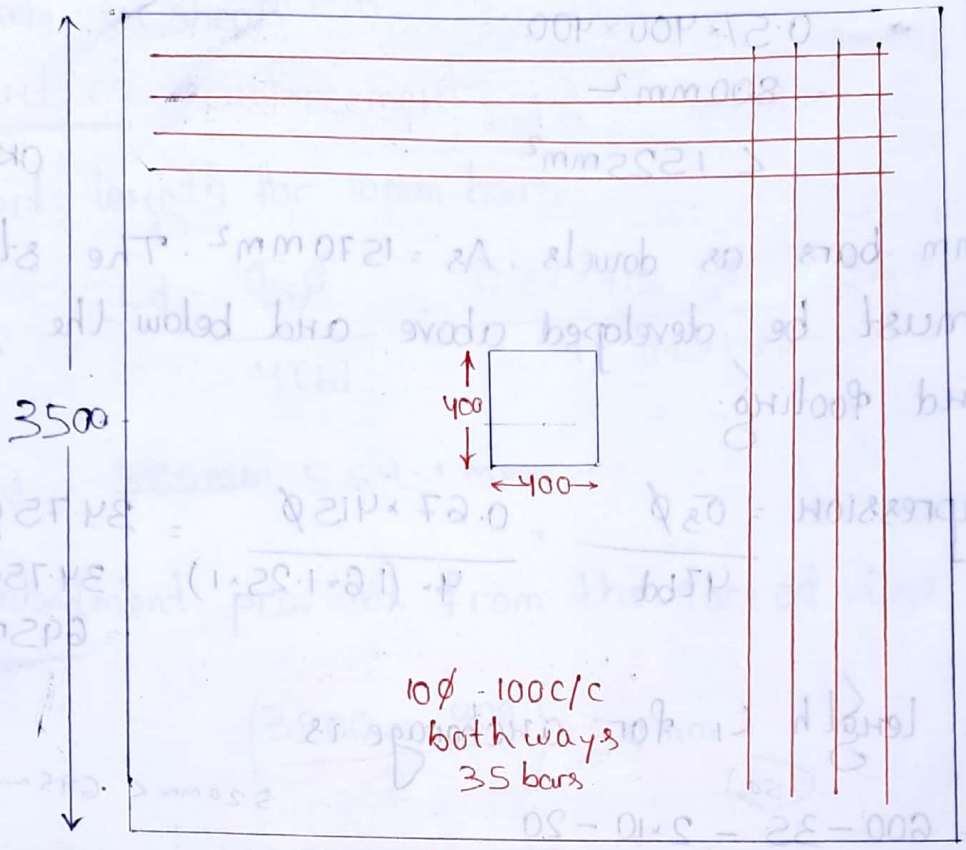
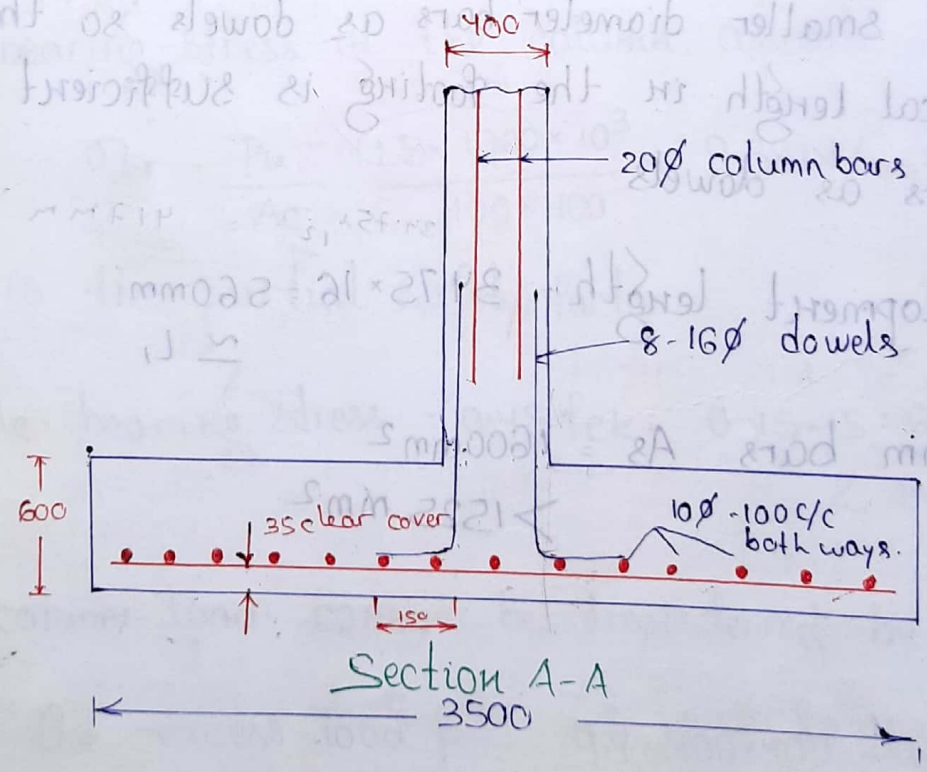


Plan of reinforcement



Sectional elevation showing reinforcement

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1.1 Properties of fluid

Fluid Mechanics

Branch of science dealing with the property of fluids (including both liquids or gases) at rest or in motion. behaviours of

Fluids [Are substances which continue in motion under the application of a shear stress and cannot sustain any shear stress when at rest].

- at rest → Fluid Statics
- in motion (where pressure forces not considered) → Fluid kinematics
- in motion (where pressure forces are considered) → Fluid Dynamics.

(Shear stress - force acting to cause deformation of a material by slippage along a plane or planes parallel to the imposed stress.)

Properties of Fluids

I) **Density** (Mass Density) : Ratio of $\frac{\text{Mass}}{\text{Volume}}$. **MASS DENSITY**

(Mass per unit volume is called density)

Mass = The quantity of matter contained in a body.

Weight = The force acted upon the object due to the pull of gravity.

ρ (rho) :- Symbol. — (i)

kg/m^3 :- Unit (SI) — (ii)

$\rho = \frac{\text{Mass of fluid}}{\text{Volume of fluid}}$.·. Mathematical Equation - (iii)

ρ_{liquid} = constant (if pressure changes and temperature changes are not considered).

ρ_{gas} = function of pressure and temperature.

$\rho_{water} = 1000 kg/m^3 \triangleq 1 gm/cm^3$

II) **Specific Gravity** — Specific Weight (Weight Density) **WEIGHT DENSITY**

Weight Density (water) $= 9.81 \times 1000$ N/m³ SI Units.

Ratio of $\frac{\text{weight}}{\text{Volume}}$.

Mathematical Equation

$w = \frac{\text{Weight of fluid}}{\text{Volume of fluid}} = \frac{\text{Mass of fluid} \times \text{Acceleration due to gravity}}{\text{Volume of fluid}}$

$= \frac{\text{Mass of fluid} \times g}{\text{Volume of fluid}} = \frac{\rho \text{ (Mass density)} \times g}{}$

$\boxed{w = \rho g}$

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Specific Volume

SPECIFIC VOLUME

Specific Volume is the volume of fluid occupied by a unit mass of water.

Eqn:- $$\text{Specific Volume} = \frac{\text{Volume of fluid}}{\text{Mass of fluid}} = \frac{1}{\frac{\text{Mass of fluid}}{\text{Volume of fluid}}} = \frac{1}{\rho}.$$

Units:- M^3/kg (SI units).

Specific Gravity

SPECIFIC GRAVITY (Dimensionless)

Specific Gravity of a Fluid is defined as the ratio of weight density of the fluid to the weight density of a standard fluid.

Liquids] standard fluid is water.

Gases] standard fluid is Air.

$$S\,(\text{liquids}) = \frac{\text{Weight density (density) of liquid}}{\text{Weight density (density) of water}}. \quad (1000 \times 9.81)\,N/m^2$$

$\Rightarrow$ Weight Density of Liquid = S (specific gravity) $\times$ weight density of water

= S (Specific Gravity) $\times 1000 \times 9.81\ N/m^3$.

Density of a Liquid = S (Specific Gravity) $\times$ 1000 kg/m³

$$S\,(\text{Gases}) = \frac{\text{Weight density of a Gas}}{\text{Weight density of Air}}.$$

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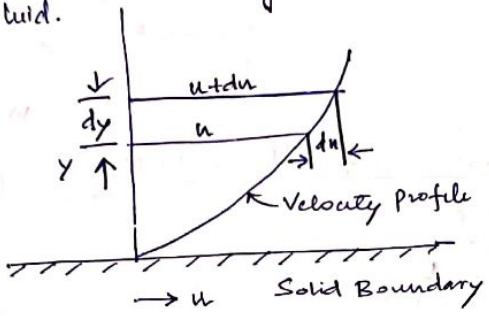
III. Viscosity

Property which defines the resistance of a fluid to flow.

It is the property by virtue of which it offers resistance to the relative motion between two liquid layers or the relative motion of the immersed objects through them.

It is the property of a fluid which offers resistance to the movement of one layer of fluid over another adjacent layer of fluid.



Top layer causes a shear stress on the adjacent lower layer while the lower layer causes a shear stress on the adjacent top layer. This shear stress is proportional to the rate of change of velocity w.r.t y.

Denoted by the symbol τ (tau)

$$\tau \propto \frac{du}{dy}.$$

$$\boxed{\tau = \mu \frac{du}{dy}}$$

(μ (mu) is known as the coefficient of proportionality / dynamic viscosity / constant)

$\frac{du}{dy}$ = velocity gradient / rate of shear strain / rate of shear deformation.

$$\boxed{\mu = \frac{\tau}{du/dy}}$$

Coefficient of viscosity is defined as the shear stress required to produce a unit rate of shear strain.

Units :-

$$\mu = \frac{\text{Shear Stress}}{\frac{\text{Change in velocity}}{\text{Change in distance}}} = \frac{F/A}{(L/T) \times (1/L)} = \frac{F/A}{1/T.}$$

$$= \frac{F/L^2}{1/T}$$

$$= \frac{F \times T}{L^2}$$

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(1 kg = 9.81 N)

$$\mu = \frac{f \times T}{L^2}$$

MKS units

f = kg-f (9.8 N)

L = m

$\mu = \frac{\text{kgf sec}}{m^2}$

CGS unit

F = dyne (10^{-5} N)

L = cm

$\mu = \frac{\text{dyne sec}}{cm^2}$ — Poise

SI unit

F = N

$L = m^2$

$\mu = \frac{N \text{ sec}}{m^2}$

($\because N/m^2 = Pa$) (Pascals)

μ = Pa sec

Conversion from Poise to $\frac{N \text{ sec}}{m^2}$ (CGS unit to SI unit).

1 poise = $\frac{1 \text{ dyne sec}}{cm^2}$

$= 1 \frac{\frac{gm\,cm^1}{sec^2} \cdot sec}{cm^2}$ ($N = m.a$)

$= \frac{1 \frac{gm\,cm}{sec^2} \cdot sec}{cm^2}$ $\Rightarrow$ $= \frac{10^{-3} kg \times 10^{-2} m}{sec^2} \cdot sec$ / $(10^{-2})^2 m^2$

$= \frac{1 \cdot gm}{cm \cdot sec}$

$= \frac{1 \cdot 10^{-3} kg}{10^{-2} m \, sec}$

$= \frac{10^{-5} \frac{kgm}{sec^2} \cdot sec}{10^{-4} m^2}$

$= \frac{10^{-1} N \text{ sec}}{m^2}$

$= \frac{1}{10} \frac{N \text{ sec}}{m^2}$

$= \frac{1}{10} \frac{Ns}{m^2}$

$= \frac{1}{10}$ Pa sec ($\because N/m^2 = Pa$ (Pascal)).

Centipoise

1 centipoise = $\frac{1}{100}$ poise

Water] viscosity at 20°c

= 1 centipoise

= $\frac{1}{100}$ poise

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Kinematic Viscosity

Ratio = $\frac{\text{Dynamic Viscosity}}{\text{Density}}$ (Symbol ν (nu)).

Mathematical Equation = $\nu = \frac{\text{Dynamic Viscosity}}{\text{Density}} = \frac{\mu}{\rho}$

Units = $\nu = \frac{\mu}{\rho} = \frac{\frac{\text{Force} \times \text{Time}}{(\text{Length})^2}}{\frac{\text{Mass}}{(\text{Length})^3}}$

$= \left(\frac{F \times T}{L^2}\right) \times \left(\frac{L^3}{M}\right)$

$= \left(\frac{MLT^{-2} \times T}{L^2}\right)\left(\frac{L^3}{M}\right)$

$= M^0 L^2 T^{-1}$

$= \frac{(\text{Length})^2}{\text{Time}}$

MKS Units / SI.

$\frac{m^2}{sec}$

CGS Units

$\frac{cm^2}{sec}$ (Stoke)

$1 \text{ stoke} = 1 \frac{cm^2}{sec} = \frac{1 \times (10^{-2})^2 m^2}{sec.} = 10^{-4} \frac{m^2}{sec}$

$\text{Centistoke} = \frac{1}{100} \text{ stoke} = 10^{-6} \frac{m^2}{sec.}$

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NEWTONS LAW OF VISCOSITY

Newtons law of viscosity states that the shearstress (τ) on a fluid element layer is directly proportional to the rate of shear strain.

Mathematical Equation

$$\dot{\tau} \propto \frac{du}{dy}$$

$$\Rightarrow \tau = \mu \frac{du}{dy}$$ where τ = shear stress (τ) on an element layer of fluid

μ = proportionality constant
Coefficient of dynamic Viscosity / viscosity

$\frac{du}{dy}$ = velocity gradient / rate of shear strain / rate of shear deformation.

Fluids
- Newtonian fluids — Obey Newtons law
- Non Newtonian fluids — Disobey Newtons law

Viscosity ~ Temperature

(i) $\text{Viscosity}]_{\text{Liquids}} \propto \frac{1}{\text{temperature}}$

Viscosity of Liquids decrease with increase in temperature.
Viscous forces in a liquid is due to (i) cohesive forces (between molecules)
(ii) Molecular momentum transfers (depending on the transfer of momentum between molecules)

In liquid cohesive forces between molecule dominate contributing to imparting viscosity to the liquid. With increase in temperature these cohesive forces decrease hence the viscosity decreases with increase in temperature for a liquid.

(ii) $\text{Viscosity}]_{\text{gas}} \propto \text{temperature}$.

In gases viscosity is due to molecular momentum transfer

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predominantly. As this increases with increase in temperature, viscosity of gases increase with increase in temperature.

(i) For liquids

$$\mu = \mu_0 \left(\frac{1}{1 + \alpha t + \beta t^2} \right)$$

μ = Viscosity of liquid at $t°C$, in poise
μ_0 = Viscosity of liquid at $0°C$, in poise
α, β = constants specific to the liquid.

For water,
$\mu_0 = 1.79 \times 10^{-3}$ poise
$\alpha = 0.03368$
$\beta = 0.000221$

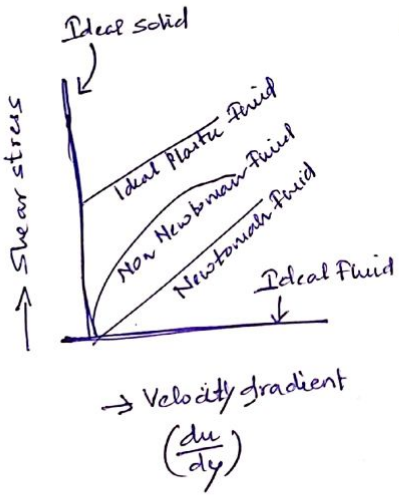
(ii) For a gas.

$$\mu = \mu_0 + \alpha t - \beta t^2$$

μ = Viscosity of a liquid at $t°C$, in poise.
μ_0 = Viscosity of a liquid at $0°C$, in poise.
α, β = constants specific to the liquid.

For air,
$\mu_0 = 0.000017$, $\alpha = 0.000000056$, $\beta = 0.1189 \times 10^{-9}$

Fluids
- Ideal fluid (i) Incompressible (ii) 0 Viscosity (No viscosity)
- Real fluid (i) with Viscosity.
- Newtonian fluid
  (Obeys Newton Law $\tau \propto \frac{du}{dy}$
  Shearstress $\propto$ velocity gradient
- Non Newtonian fluid
  Shearstress is not proportional to velocity gradient.
- Ideal Plastic fluid
  Shear stress > Yield value
  Shear stress $\propto \frac{du}{dy}$ (velocity gradient)



(Types of fluid).

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1. Ideal Fluids

Fluid which is incompressible and have no viscosity is known as ideal fluid.

It is an imaginary fluid as all fluids have some viscosity.

2. Real Fluids

A fluid which possesses viscosity, is known as real fluid. All fluids are real fluid as they have viscosity.

3. Newtonian Fluids

A real fluid in which shear stress is directly propotional to rate of shear strain / velocity gradient is known as newtonian fluid. Ex:- water, air, glycerol & thin motor oil.

4. Non-Newtonian Fluids

A real fluid in which the shear stress is not propotional to the rate of shear strain / velocity gradient, is known as non-newtonian fluids.
Ex:- blood, saliva, corn flour mixed with water

5. Ideal Plastic Fluid

A fluid in which shear stress is more than the yield value & shear stress is propotional to the rate of shear strain / velocity gradient, is known as ideal plastic fluid.
Ex:- Mayonnaise, Toothpaste.

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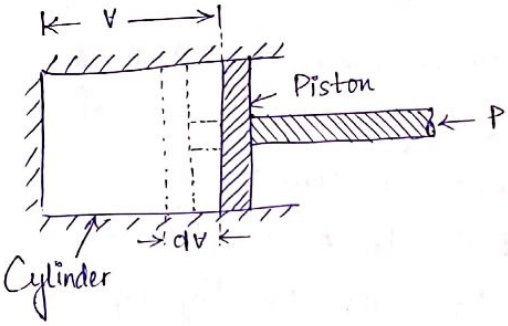
Compressibility & Bulk Modulus

→ Compressibility :- The capacity of something to be flattened or reduced in size by pressure is known as compressibility.

→ Compressibility is a measure of the relative volume change of a fluid or ~~fee~~ as a response to a pressure change.

→ Compressibility is the reciprocal of the bulk modulus of elasticity, K which is defined as the ratio of compressive stress to volumetric strain.

→ Bulk modulus is the relative change in the volume of a body produced by a unit compressive or tensile ~~force~~. stress acting uniformly over its surface.



A cylinder is fitted with a piston.

Let V = Volume of gas enclosed in cylinder

p = Pressure of gas when volume is V.

Let the pressure is increased to p+dp, the volume of gas decreases from V to V-dV.

Then, increase in pressure = dp kg-f/m²

Decrease in volume = dV

∴ Volumetric Strain = $-\frac{dV}{V}$

-ve sign denotes the volume decrease with increase

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in pressure.

$\therefore$ Bulk Modulus $K = \frac{\text{Increase of pressure}}{\text{Volumetric Strain}}$

$= \frac{dp}{-\frac{dv}{V}} = \frac{-dp}{dv} V$

Compressibility $= \frac{1}{K}$

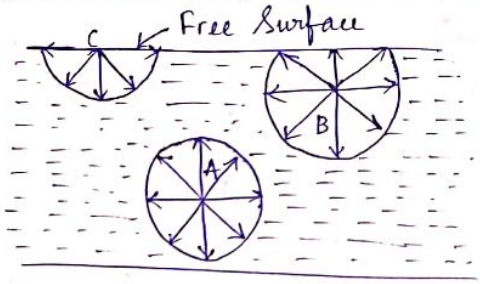
Surface Tension

It 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 liquids such that the contact surface behaves like a membrane under tension.

It is denoted by 'σ'.

Unit - MKS unit → kgf/m

- SI → N/m.



→ Three molecules A, B & C of a liquid in a mass of liquid are considered.

→ Molecule A is attracted in all directions equally by the surrounding molecules of liquid, which results in zero force on molecule A.

→ Molecule B which is situated near free surface, is acted upon by upward & downward forces which are unbalanced. The resultant force on molecule B will act in downward direction.

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→ Molecule C, which is situated on the free surface of liquid experience a resultant downward force.

→ The molecules on the free surface experiences a downward force.

Surface tension on liquid droplet :-

A spherical droplet of liquid of radius 'r' is taken. On the entire surface of droplet, tensile force acts.

Let, σ = surface tension of the liquid.

p = pressure intensity inside the droplet (in excess of outside pressure intensity)

d = dia. of droplet

When droplet is cut into two halves. The forces acting on one half will be i) tensile force due to surface tension acting around the circumference of cut portion and given by,

$= \sigma \times$ circumference

$= \sigma \times \pi d$.

ii) Pressure force on area $\frac{\pi}{4} d^2 = p \cdot \frac{\pi}{4} d^2$

These forces will be equal & opposite under equilibrium condition, i.e,

$$p \times \frac{\pi}{4} d^2 = \sigma \times \pi d$$

$$\Rightarrow p = \frac{\sigma \times \pi d}{\frac{\pi}{4} \times d^2} \Rightarrow \boxed{p = \frac{4\sigma}{d}}$$

→ Decrease in dia. of droplet, pressure intensity inside

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the droplet increases.

Surface Tension on a Hollow Bubble

→ A hollow bubble like soap bubble has two surfaces in contact with air, one inside & other outside. Those two surfaces are surface to surface tension.

Pressure force on area = $p \times \frac{\pi}{4} d^2$

Force due to surface tension = $2(\sigma \times \pi d)$

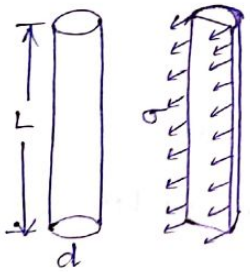
$$p \times \frac{\pi}{4} d^2 = 2(\sigma \times \pi d)$$

$$\Rightarrow p = \frac{2\pi\sigma d}{\frac{\pi}{4} d^2}$$

$$\Rightarrow \boxed{p = \frac{8\sigma}{d}}$$

Surface Tension on a Liquid Jet

A liquid jet of diameter 'd' & length 'L' is taken.



let,

p = pressure intensity inside the liquid jet above the outside pressure.

σ = surface tension of the liquid.

Consider the equilibrium of set jet,

Force due to pressure = p × area of semi-jet

$= p \times L \times d$

Force due to surface tension = $\sigma \times 2L$

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Equating the two forces,

$$P \times L \times d = \sigma \times 2L$$

$$\Rightarrow \boxed{P = \frac{\sigma \times 2L}{Ld}}$$

Capillarity

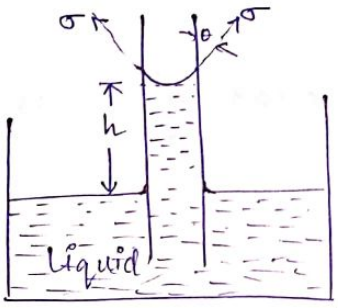
The phenomenon of rise or fall of a liquid surface in a small tube relative to the adjacent general level of liquid when the tube is held vertically in the liquid, is known as capillarity.

→ The rise of liquid surface is known as capillary rise and the fall of liquid surface is known as capillary depression.

→ Expressed in cm or mm of liquid.

→ Capillarity depends upon the sp. wt. of the liquid, dia. of the tube & surface tension of the liquid.

Expression for Capillary Rise.



A glass tube of small dia 'd' with open ends both sides is inserted in a liquid like water.

h = ht. of liquid in the tube.

Under state of equilibrium, the wt. of liquid of ht. 'h' is balanced by the force at the surface of the liquid in the tube. But the force at surface of the liquid in the tube is due to surface tension

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Let

σ = surface tension of liquid.

θ = Angle of contact between liquid & glass tube

Wt. of liquid of ht. h in the tube $= [(\text{Area of tube}) \times h] \times \rho \times g$

$= \frac{\pi}{4} \times d^2 \times h \times \rho \times g$ —— (1)

where,

ρ = density of liquid.

Vertical component of the surface tensile force $= (\sigma \times \text{circumference}) \times \cos\theta$

$= (\sigma \times \pi d) \times \cos\theta$ —— (2)

Equating eqn. (1) & (2), we get

$$\frac{\pi}{4} d^2 \times h \times \rho \times g = \sigma \times \pi d \times \cos\theta$$

$$\Rightarrow h = \frac{\sigma \times \pi d \times \cos\theta}{\frac{\pi}{4} d^2 \times \rho \times g}$$

$$\Rightarrow h = \frac{4\sigma\cos\theta}{\rho g d}$$

Value of θ between water & clean glass is approx. equal to zero.

$\cos\theta = \cos 0^\circ = 1$

$$\boxed{h = \frac{4\sigma}{\rho g d}}$$

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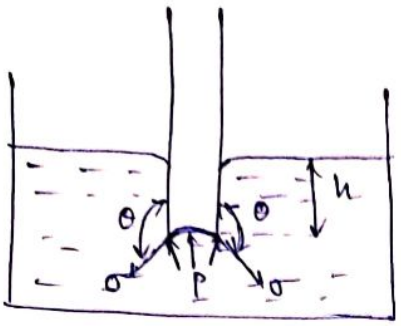
Expression for Capillary Fall



When the glass tube is dipped in mercury, the level of mercury in the tube will be lower than the general level of the outside liquid.

Let h = Ht. of depression in tube.

In equilibrium, two forces act on mercury inside the tube. First is surface tension acting in downward direction i.e $\sigma \times \pi d \times \cos\theta$

Second is hydrostatic force acting upward & is equal to intensity of pressure at a depth $h \times$ Area.

$$= p \times \frac{\pi}{4} d^2 = \rho \times g \times h \times \frac{\pi}{4} d^2$$

Equating the two eqn., we get

$$\sigma \times \pi d \times \cos\theta = \rho g h \times \frac{\pi}{4} d^2$$

$$\Rightarrow \boxed{h = \frac{4\sigma \cos\theta}{\rho g d}}$$

Value of $\theta = 128^\circ$ (for mercury)

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1.2 Pressure & Its Measurements

Pressure

The force applied perpendicular to the surface of an object per unit area over which the force is distribution, is known as pressure.

→ Let dF is the force acting on the area dA in the normal direction. The ratio of $\frac{dF}{dA}$ is known as intensity of pressure or simply pressure and denoted by p.

$$p = \frac{dF}{dA}.$$

Unit :- MKS unit → Kgf/m^2

SI unit → N/m^2 or N/mm^2.

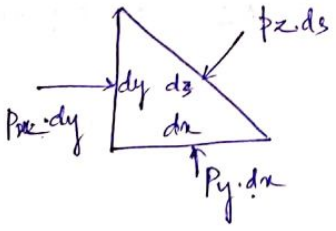
N/m^2 → Pascal

KPa → $1000\ N/m^2$

bar → $10^5\ N/m^2$.

Pascal's law

It states that the pressure intensity at a point in a static fluid is equal in all directions.



$$p_x = p_y = p_z.$$

As per Pascal's law, the intensity of pressure will be equally transmitted in all directions.

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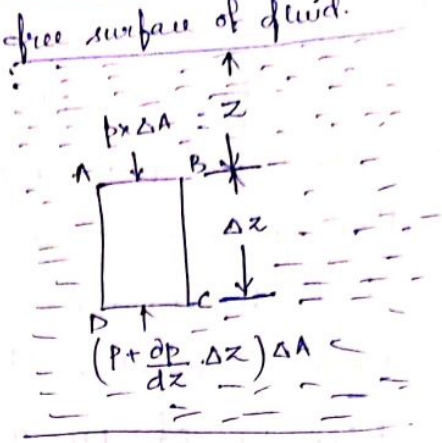
Pressure Variation in a fluid at Rest



Pressure at any point in a fluid at rest is obtained by hydrostatic law which states that the rate of increase of pressure in the vertically downward direction must be equal to the sp. wt. of fluid at that point.

The fluid element ABCD is considered.

Let ΔA = cross-sectional area of element

Δz = Ht. of fluid element

p = pressure on face AB.

z = distance of fluid element from free surface.

Forces acting on the fluid element are :-

i) Pressure force on AB = $p \times \Delta A$

ii) Pressure force on CD = $\left(p + \frac{\partial p}{\partial z}\Delta z\right) \times \Delta A$

iii) Weight of fluid element = $\rho g (\Delta A \times \Delta z)$

iv) Pressure force on surfaces BC & AC are equal & opposite

For equilibrium of fluid, we have.

$$p \times \Delta A - \left(p + \frac{\partial p}{\partial z} \cdot \Delta z\right) \times \Delta A + \rho \times g \times (\Delta A \times \Delta z) = 0.$$

$$\Rightarrow p\Delta A - p\Delta A - \frac{\partial p}{\partial z} \cdot \Delta z \Delta A + \rho g \Delta A \Delta z = 0$$

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$$\Rightarrow -\frac{\partial p}{\partial z}\Delta z \Delta A + \rho g \Delta A \Delta z = 0$$

$$\Rightarrow \frac{\partial p}{\partial z}\Delta z \cdot \Delta A = \rho g \Delta A \Delta z$$

$$\Rightarrow \frac{\partial p}{\partial z} = \rho g$$

$$\Rightarrow \frac{\partial p}{\partial z} = w \qquad (\because \rho g = w) \quad \text{---} \quad \text{---} \quad (1).$$

Rate of increase of pressure in a vertical direction is equal to wt. density of fluid at that point, is known as hydrostatic law.

$$\Rightarrow \partial p = \rho g \partial z.$$

By integrating, we get.

$$p = \rho g z$$

$$z \text{ (pressure head)} = \frac{p}{\rho \times g}.$$

Q.1 A hydraulic press has a ram of 30cm diameter & plunger of 4.5 cm diameter. Find the wt. lifted by the hydraulic press when the force applied at the plunger is 500 N.

Pressure intensity due to plunger $= \frac{f}{a} = \frac{500}{0.00159}$ N/m^2

Pressure intensity at ram $= \frac{W}{A}$

$W = 22.22$ KN.

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Q. 2. A hydraulic press has a ram of 20cm dia. & plunger of 3cm dia. It is used for lifting a wt. of 30 KN. Find the force required at the plunger.

$\frac{W}{A}$ = pressure intensity at ram.

$\frac{30\times10^3}{\frac{\pi}{4}\times0.2^2}$ = pressure intensity at ram.

Pressure intensity due to plunger = $\frac{F}{a} = \frac{F}{\frac{\pi}{4}\times(0.03)^2}$

$F = 675.2\ N$.

Q. 3. Calculate the pressure due to column of 0.3m of a) water b) an oil of sp. gravity 0.8 & c) Mercury of sp. gravity 13.6. $\rho_w = 1000\ kg/m^3$.

Ans: Water, $p = \rho g z = 0.2943\ N/cm^2$

Oil, $p = 0.2354\ N/cm^2$

Mercury, $p = 4.002\ N/cm^2$

Q. 4 The pressure intensity at a point in a fluid is given $4.9\ N/cm^2$. Find the corresponding ht. of fluid when it is a) water & b) an oil of sp. gr. 0.8.

Ans: a) 5m of water b) 6.25m of oil.

Q. 5. An oil of sp. gr. 0.8 is contained in a vessel. At a point the ht. of oil is 20 m. Find the corresponding ht. of water at that point.

Ans: 16m

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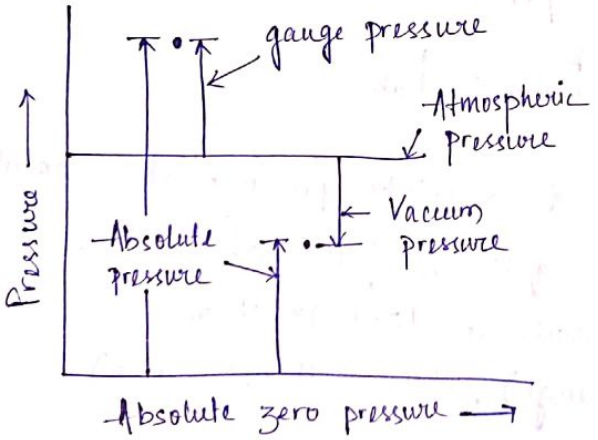
Q.6 An open tank contains water upto a depth of 1.5m and above it an oil of sp. gr. 0.8 for a depth 2m. Find pressure intensity i) at the interface of two liquids, ii) at the bottom of tank.

Ans :- i) 1.57 N/cm² ii) 3.04 N/cm².

Q.7 The dia. of small piston & large piston of a hydraulic jack are 2cm & 10cm respectively. A force of 60N is applied on the small piston. Find the load lifted by the large piston when a) the pistons are at the same level & b) small piston is 20cm above the large piston. The density of the liquid in the jack is given as 1000 kg/m².

Ans :- a) 1500N b) 1520.5N.

Absolute, Gauge, Atmospheric & Vacuum Pressures



1. Absolute pressure

It is a pressure which is measured with reference to absolute zero pressure.

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2. Gauge Pressure :-

It is a pressure which is measured with the help of a pressure measuring instrument, in which the atmospheric pressure is taken as datum and on scale is marked as zero.

3. Vaccum Pressure :-

It is the negative pressure or pressure measured below atmospheric pressure.

4. Atmospheric Pressure :-

It is the pressure in the surrounding atmosphere.

Mathematically,

i) Absolute pressure = atmospheric pressure + Gauge pressure

$$P_{ab} = P_{atm} + P_{gauge}$$

ii) Vacuum pressure = Atmospheric pressure − Absolute pressure

$$= P_{atm} - P_{ab}$$

Q.8 Determine the gauge & absolute pressure at a point which is 2m below the free surface of water. Take atmospheric pressure as $10.1043\ N/cm^2$.

Ans :- $1.962\ N/cm^2$ (gauge), $12.066\ N/cm^2$ (abs)

Q.9. What are the gauge pressure & absolute pressure at a point 3m below the free pressu surface of a liquid having density of $1.53 \times 10^3\ kg/m^3$ if the atmospheric pressure is equivalent to 750mm of mercury.

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The sp. gr. of mercury is 13.6 & density of water is 1000 kg/m³.

Ans:- $z_1 = 3m$

$\rho_1 = 1.53 \times 10^3 \text{ kg/m}^3$.

$z_0 = 750\text{mm of Hg}$.

$= \frac{750}{1000} = 0.75\text{ m of Hg}$.

$P_{atm} = \rho_0 \times g \times z_0$

$= (13.6 \times 1000) \times 9.81 \times 0.75$

$= 100062 \text{ N/m}^2$

Pressure at a depth of 3m from free surface $= \rho_1 g z_1$

$= (1.53 \times 1000) \times 9.81 \times 3$

$= 45028 \text{ N/m}^2$.

gauge pressure $= p = 45028 \text{ N/m}^2$.

Absolute pressure = Gauge pressure + atmospheric pressure

$= 45028 + 100062 = 145090 \text{ N/m}^2$.

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→ Fluid Pressure is measured by following devices :-

i) Manometers

ii) Mechanical Gauges.

Manometers

→ Manometers are defined as the device used for measuring the pressure at a point in a fluid by balancing the column of fluid by the same or another column of fluid.

They are classified as :-

i) Simple manometer ii) Differential Manometer.

Mechanical Gauges

→ Mechanical gauges are defined as the device used for measuring the pressure by balancing the fluid column by the spring or dead weight.

The commonly used mechanical pressure gauges are :-

i) Diaphragm pressure gauge.

ii) Bourdon tube pressure gauge.

iii) Dead wt. pressure gauge.

iv) Bellows pressure gauge.

Simple Manometers :-

It consist of a glass tube having one of its end connected to a point where pressure is to be measured and other end remain open to atmosphere.

The commonly used simple manometers are :-

i) Piezometer ii) U-tube manometer iii) Single column Manometer

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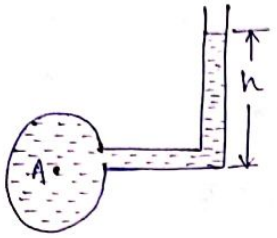
Piezometer

→ It is simplest form of manometer used for measuring gauge pressures. One end of this manometer is connected to the point where pressure is to be measured and other end open to atmosphere.



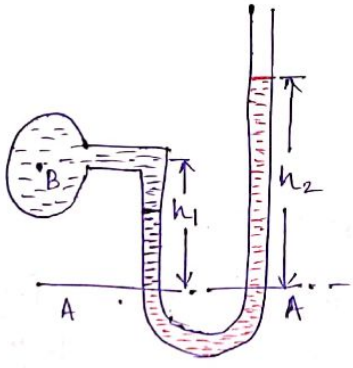
At point A, the ht. of liquid is h, in piezometer tube, then pressure at A,

$$= \rho \times g \times h \ \frac{N}{m^2}$$

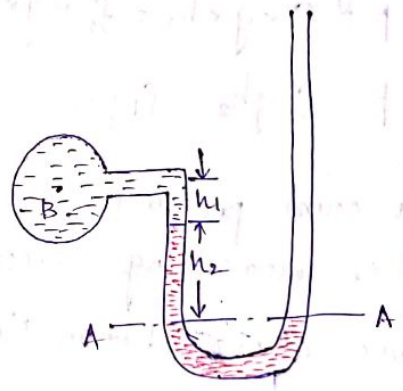
U-tube Manometer

It consist of glass tube bent in U-shape, one end is connected to a point at which pressure to be measured and other remains open to the atmosphere.

The tube contains mostly mercury or any other liquid whose sp. gravity is greater than the specific gravity of liquid whose pressure is to be measured.



for gauge pressure



for vacuum pressure.

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For gauge pressure :-

Let B is the point where pressure is to be measured whose value is P

h_1 = ht. of light liquid above datum line.

h_2 = ht. of heavy liquid about datum line.

S_1 = sp. gr. of light liquid.

S_2 = sp. gr. of heavy liquid.

ρ_1 = density of light liquid = $1000 \times S_1$

ρ_2 = density of heavy liquid = $1000 \times S_2$

As pressure is same in horizontal surface. Hence Pressure above datum line in the left column is equal to the right column of U-tube manometer.

Pressure above datum line in left column = $p + \rho_1 \times g \times h_1$

Pressure above datum line in right column = $\rho_2 \times g \times h_2$

Equating above two pressures,

$$p + \rho_1 \times g \times h_1 = \rho_2 \times g \times h_2$$

$$\Rightarrow p = \rho_2 g h_2 - \rho_1 g h_1$$

For vaccum pressure :-

For measuring vaccum pressure :-

Pressure above datum line in left column = $\rho_1 g h_1 + \rho_2 g h_2 + p$

Pressure head in right column = 0.

Equating above two pressures,

$$\rho_1 g h_1 + \rho_2 g h_2 + p = 0$$

$$\Rightarrow p = -(\rho_1 g h_1 + \rho_2 g h_2)$$

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Single Column Manometer

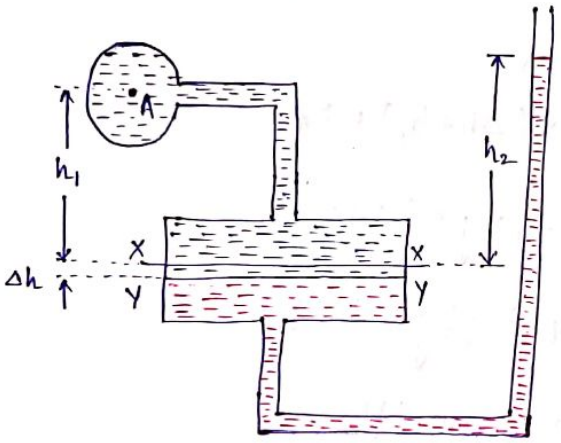
→ It is modified U-tube manometer in which a reservoir, having large cross-sectional area as compared to area of tube is connected to one limb of the manometer.

There are two types of single column manometer and they are :-

i) Vertical Single Column Manometer

ii) Inclined Single Column Manometer

i) Vertical Single Column Manometer



Let x-x be the datum line in the reservoir & in the right limb, when the pipe is not connected to manometer.

When it is connected to a pipe, due to high pressure at A, heavy liquid in reservoir will be pushed downward & will rise in right limb.

Let Δh = Fall of heavy liquid in reservoir.

h_2 = rise of heavy liquid in right limb.

h_1 = ttl. of center of pipe above x-x.

p_A = pressure at A which is to be measured.

A = cross sectional area of reservoir.

a = cross sectional area of right limb.

S_1 = sp. gravity of liquid in pipe.

S_2 = sp. gravity of heavy liquid in reservoir &

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right limb.

ρ_1 = density of liquid in pipe.

ρ_2 = density of liquid in reservoir.

Fall of heavy liquid in reservoir will cause rise of heavy liquid level in right limb.

$$A \times \Delta h = a \times h_2$$

$$\Delta h = \frac{a \times h_2}{A} \quad \text{---------- (1)}$$

Now, consider Y-Y as datum,

pressure in right limb above Y-Y = $\rho_2 \times g \times (h_2 + \Delta h)$

pressure in left limb above Y-Y = $\rho_1 \times g \times (\Delta h + h_1) + p_A$

Equating above two pressures,

$$\rho_2 \times g \times (h_2 + \Delta h) = \rho_1 \times g \times (\Delta h + h_1) + p_A.$$

$$\Rightarrow p_A = \rho_2 \times g \times (h_2 + \Delta h) - \rho_1 \times g \times (\Delta h + h_1)$$

$$\Rightarrow p_A = \Delta h [\rho_2 g - \rho_1 g] + h_2 \rho_2 g - h_1 \rho_1 g$$

Putting Δh value from eqn-(1),

$$\Rightarrow p_A = \frac{a \times h_2}{A} [\rho_2 g - \rho_1 g] + h_2 \rho_2 g - h_1 \rho_1 g$$

As A is very large as compared to a, so $\frac{a}{A}$ is very small and can be neglected.

then, $\boxed{p_A = h_2 \rho_2 g - h_1 \rho_1 g}$

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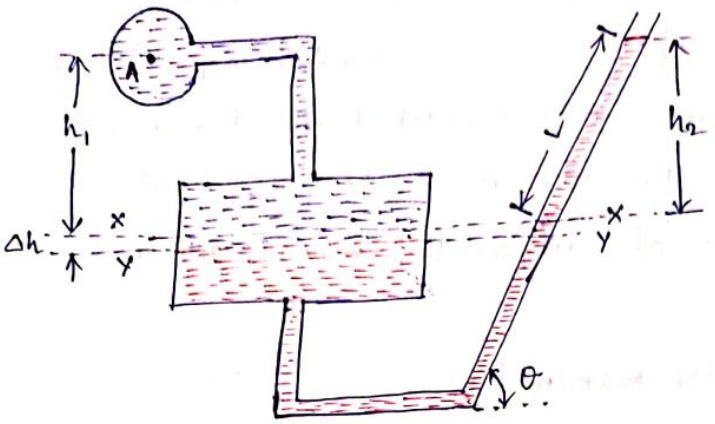
Inclined Single Column Manometer



This manometer is more sensitive. As the tube is inclined, the distance moved by heavy liquid in the right limb will be more.

Let L = Length of heavy liquid moved in right limb from x-x.

θ = inclination of right limb with horizontal.

h_2 = vertical right of heavy liquid in right limb from x-x

$= L \sin\theta$.

Pressure in left limb above

$$p_A = h_2 \rho_2 g - h_1 \rho_1 g.$$

Substituting $h_2 = L \sin\theta$

$$\boxed{p_A = L \sin\theta \times \rho_2 \times g - h_1 \times \rho_1 \times g}$$

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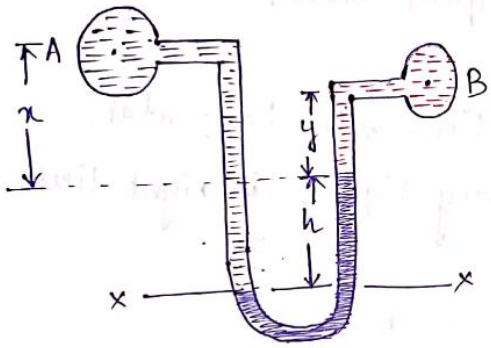
Differential Manometers

It is used for measuring difference of pressures between two points in a pipe or in two different pipes. It consist of a U-tube containing heavy liquid, whose two ends are connected to the points whose difference of pressure is to be measured. There are two types of differential manometers and they are :-

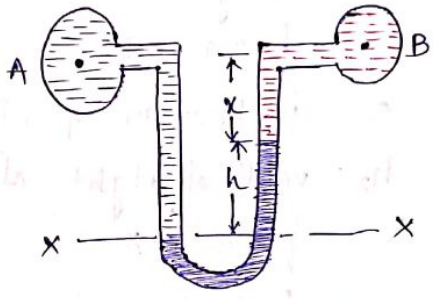
i) U-tube differential manometer.

ii) Inverted U-tube manometer.

U-tube Differential Manometer



a) Two points at different levels



b) Two points at same level

a) Two points at different level

Points A & B contains liquid at different level and of different sp.grarity. Point A & B are connected through a U-tube differential manometer. Pressure at A & B are p_A & p_B.

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Let h = difference of mercury level in U-tube.

y = distance of center of B, from the mercury level in right limb.

x = distance of center of A, from mercury level in ~~left~~ right limb.

ρ_1 = density of liquid at A.

ρ_2 = " " " B.

ρ_g = density of heavy liquid or mercury.

Taking x-x as datum,

Pressure at left limb above x-x line $= p_A + \rho_1 g(h+x)$

Pressure at right limb above x-x line $= p_B + \rho_2 g y + \rho_g g h$

Equating above two pressures, we have

$$\rho_1 g(h+x) + p_A = \rho_g g h + \rho_2 g y + p_B$$

$$\Rightarrow p_A - p_B = \rho_g g h + \rho_2 g y - \rho_1 g(h+x)$$

$$= \rho_g g h + \rho_2 g y - \rho_1 g h - \rho_1 g x$$

Difference of pressure at A & B.

$$= g h(\rho_g - \rho_1) + \rho_2 g y - \rho_1 g x$$

b) Two points at same level containing same liquid

Pressure above x-x in right limb $= \rho_g \times g \times h + \rho_1 g x + p_B$

Pressure above x-x in left limb $= \rho_1 g(h+x) + p_A$.

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Equating above two pressures,

$$\rho_g \times g \times h + \rho_1 g x + p_B = \rho_1 g (h+x) + p_A$$

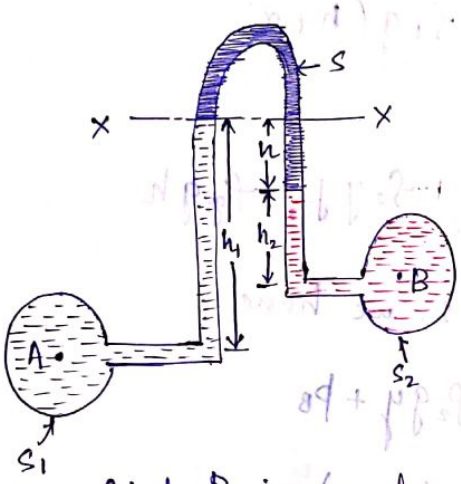
$$\Rightarrow p_A - p_B = \rho_g g h + \rho_1 g x - \rho_1 g (h+x)$$

$$= \rho_g g h + \rho_1 g x - \rho_1 g h - \rho_1 g x$$

$$= \rho_g g h - \rho_1 g h$$

$$\boxed{p_A - p_B = g h (\rho_g - \rho_1)}$$

Inverted U-tube Differential Manometer



It is an inverted U-tube, containing a light liquid. The two ends of two are connected to the points whose difference of pressure is to be measured. It is used for measuring difference of low pressures.

(Let p_A is greater than p_B)

Let, h_1 = ht. of liquid in left limb below datum line X-X.

h_2 = ht. of liquid in right limb.

h = differene of light liquid.

ρ_1 = density of liquid at A.

ρ_2 = density of liquid at B.

ρ_s = density of light liquid.

p_A = pressure at A.

p_B = pressure at B.

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Taking x-x as datum line,

Pressure at left limb below x-x = $p_A - \rho_1 g h_1$

Pressure at right limb below x-x = $p_B - \rho_2 g h_2 - \rho_3 g h$

Equating above two pressures,

$$p_A - \rho_1 g h_1 = p_B - \rho_2 g h_2 - \rho_3 g h$$

$$\Rightarrow \boxed{p_A - p_B = \rho_1 g h_1 - \rho_2 g h_2 - \rho_3 g h}$$

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1.3 Pressure Exerted On an immersed Surface

Total Pressure :-

The force exerted by a static fluid on a surface either plane or curved when the fluid comes in contact with the surface. This force is known as total pressure. It always acts normal to the surface.

Centre of Pressure :-

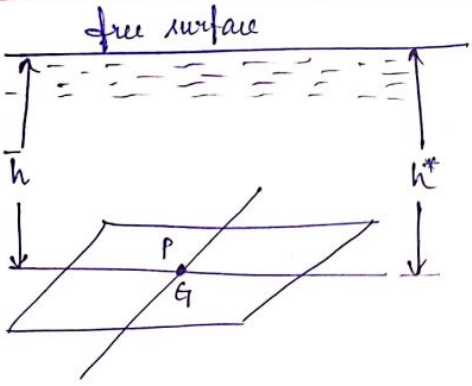
It is the point of application of total pressure on the surface.

Total Pressure on Immersed Surface :-

When a body is immersed in water, total pressure is exerted on the body by the liquid. The position of immersed surface may be :-

i) Horizontal ii) Vertical iii) Inclined

Horizontal Plane Surface Submerged in Liquid



- A plane horizontal surface is immersed in static fluid.
- → As every point of the surface is at equal depth from free surface of liquid ; pressure intensity will be equal on the entire surface.

$$p = \rho g h$$

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let A = total area of surface.

Total force, F = p × Area

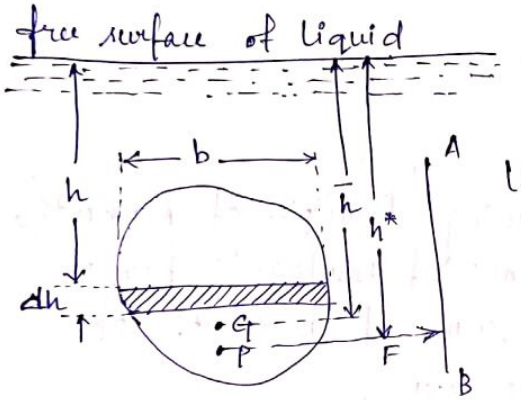
$= \rho g h \times A = \rho g A \bar{h} = \rho g A h^*$ $(\because h = \bar{h} = h^*)$

where,

$\bar{h}$ = depth of C.G from free surface of liquid

h^* = depth of center of pressure from free surface of liquid.

Vertical Plane Surface Submerged in Liquid



Let a plane surface submerged in liquid vertically.

let, A = Total area of surface.

$\bar{h}$ = distance of C.G of area from free surface of liquid.

G = center of gravity of plane surface

P = center of pressure.

h^* = distance of center of pressure from free surface of liquid.

Total Pressure

Take a strip of thickness dh & width b at a depth h from free surface of liquid.

Pressure intensity on strip = p = ρgh

Area of strip, dA = b × dh

Total pressure force on strip, F = p × Area

$= \rho g h \times b \times dh$

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Total pressure force on whole surface,

$$F = \int dF = \int \rho g h \times b \times dh$$

$$= \rho g \int h \times b \times dh$$

But $\int b \times h \times dh = \int h \times dA$

= moment of surface area about free surface of liquid.

= Area of surface x distance of C.G from free surface.

$$= A \times \bar{h}$$

$$F = \rho g A \bar{h} \quad (N)$$

b) Center of Pressure

It is calculated by using 'Principle of Moments', which states that the moment of resultant force about an axis is equal to sum of moments of the components about same axis.

The resultant force F, is acting at P at a distance h^* from free surface of liquid.

Hence moment of force F about free surface of liquid $= F \times h^*$ ——— (1)

Moment of force dF, acting on strip about free surface.

$$= \rho g h \times b \times dh \times h$$

Sum of moment of all such surfaces about free surface of liquid

$$= \int \rho g h \times b \times dh \times h$$

$$= \rho g \int b \times h \times h \, dh$$

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$= \rho g \int b h^2 dh$ $\quad (b \times dh = dA)$

$= \rho g \int h^2 dA$

But $h^2 \int dA = \int b h^2 dh$

= moment of inertia of surface about free surface of liquid.

$= I_0$

∴ Sum of moments about free surface

$= \rho g I_0$ ——————— (2)

Equating (1) & (2), we get

$F \times h^* = \rho g I_0$

But, $F = \rho g A \bar{h}$

$\rho g A \bar{h} \times h^* = \rho g I_0$

$\Rightarrow h^* = \frac{\rho g I_0}{\rho g A \bar{h}} = \frac{I_0}{A \bar{h}}$ ——————— (3)

By the theorem of parallel axis, we get

$I_0 = I_G + A \bar{h}^2$

I_G = moment of inertia of area about an axis passing through the C.G of area & parallel to free surface of liquid.

Substituting I_0 in eqn. (3), we get

$h^* = \frac{I_G + A \bar{h}^2}{A \bar{h}} = \frac{I_G}{A \bar{h}} + \bar{h}$

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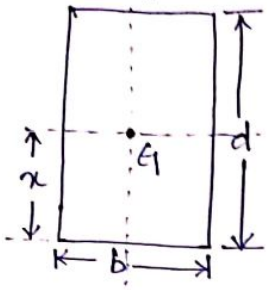
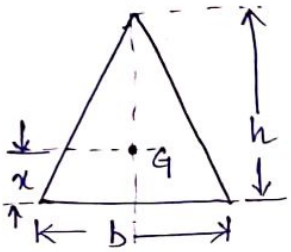
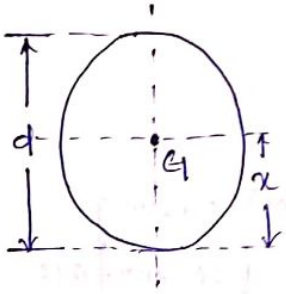
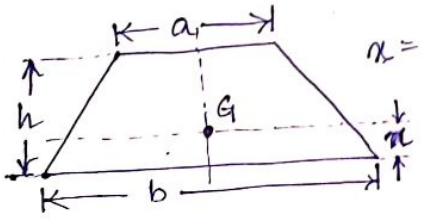
Plane Surface	C.G from the base	Area	Moment of inertia about an axis passing through C.G & parallel to base (I_G)	Moment of inertia about base (I_o)
1. Rectangle 	$x = \frac{d}{2}$	bd	$\frac{bd^3}{12}$	$\frac{bd^3}{3}$
2. Triangle 	$x = \frac{h}{3}$	$\frac{bh}{2}$	$\frac{bh^3}{36}$	$\frac{bh^3}{12}$
3. Circle 	$x = \frac{d}{2}$	$\frac{\pi d^2}{4}$	$\frac{\pi d^4}{64}$	-
4. Trapezium 	$x = \left(\frac{2a+b}{a+b}\right)\frac{h}{3}$	$\frac{(a+b)}{2} \cdot h$	$\left(\frac{a^2+4ab+b^2}{36(a+b)}\right) \cdot h^3$	-

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2.0 Kinematics of Fluid Flow

It is that branch of science which deals with motion of particles without considering the forces causing the motion.

2.1 Basic equation of fluid flow & their application

Rate of Discharge or Flow (Q) :-

It is defined as the quantity of fluid flowing per second through a section of a pipe or a channel.

For incompressible fluid (or liquid) the rate of flow or discharge is expressed as volume of fluid flowing across the section per second. For compressible fluids, the rate of flow is usually expressed as wt. of fluid flowing across the section.

i) For liquid, unit → m^3/s or lt/s

ii) For gases, unit → kgf/s or N/s.

Discharge, $\boxed{Q = A \times V}$

where A = cross-sectional area of pipe

V = average velocity of fluid across the section.

Equation of Continuity of a Liquid Flow

If an incompressible liquid is flowing continuously through a pipe or a channel the quantity of liquid passing per second is the same at all sections. This is known as equation of continuity of a liquid flow.

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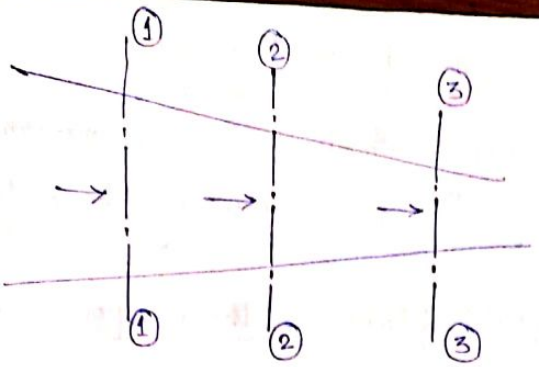

Consider three sections of the pipe through which some liquid is flowing.

Let,

A_1 = cross-sectional area of pipe at section 1-1.

V_1 = velocity of liquid at section 1-1.

A_2, V_2 = corresponding values at section 2-2.

A_3, V_3 = corresponding values at section 3-3.

We know,

Total quantity of liquid passing through section 1-1,

$$Q_1 = A_1 V_1$$

Similarly, Total quantity of liquid passing through section 2-2,

$$Q_2 = A_2 V_2$$

and total quantity of liquid passing through section 3-3,

$$Q_3 = A_3 V_3$$

From law of conservation of mass, we know total quantity of liquid passing through the section 1-1, 2-2 & 3-3 is same. Therefore,

$$Q_1 = Q_2 = Q_3$$

$$A_1 V_1 = A_2 V_2 = A_3 V_3$$

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Energy of a liquid in Motion

Energy may be defined as the capacity to do work. Energy exists in many forms, the following are important forms of energy and they are :-

i) Potential Energy
ii) Kinetic Energy
iii) Pressure Energy.

Potential Energy of a liquid Particle in Motion

It is the energy possessed by the virtue of its position. If a liquid particle is z meters above the horizontal datum, the potential energy of the particle be z metre-kilogram per kg of liquid. The potential head of liquid, at the point will be z meters of liquid.

Kinetic Energy of a liquid Particle in Motion

It is the energy, possessed by a liquid particle, by virtue of its motion or velocity. If a liquid particle is flowing with a mean velocity of v meter per second, then the kinetic energy of the particle will be $\frac{v^2}{2g}$ m-kg per kg of the liquid. Velocity head of the liquid, at that velocity, will be $\frac{v^2}{2g}$ metres of the liquid.

Pressure Energy of a liquid Particle in Motion

It is the energy, possessed by a liquid particle

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by virtue of its existing pressure. If a liquid particle is under a pressure of p KN/m^2 (i.e kPa), then the pressure energy of the particle will be $\frac{p}{w}$ m kg per kg of the liquid, where w is the sp. wt. of the liquid. Pressure head of the liquid under that pressure will be $\frac{p}{w}$ meters of the liquid.

Total Energy of the Liquid Particle in Motion

The total energy of a liquid, in motion, is the sum of its potential energy, kinetic energy & pressure energy.

Total energy, $E = Z + \frac{v^2}{2g} + \frac{p}{w}$ m of liquid.

Total Head of a Liquid Particle in Motion

The total head of a liquid particle, in motion, is the sum of its potential head, kinetic head & pressure head.

Total head, $H = Z + \frac{v^2}{2g} + \frac{p}{w}$ m of liquid.

Euler's Equation of Motion

Forces due to gravity and pressure are taken into account.

Consider a stream line in which flow takes place in s-direction. Take a cylindrical element of cross-section dA & length ds.

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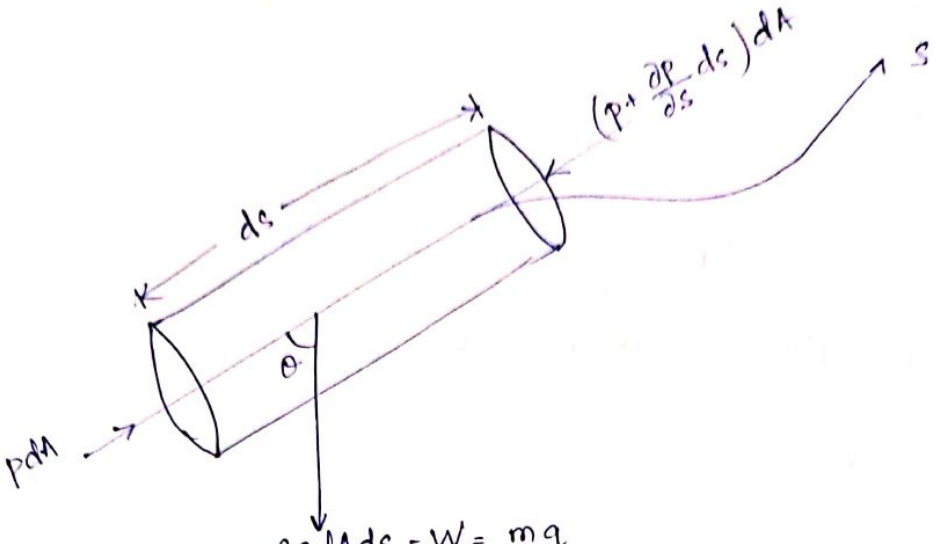

The forces acting on the cylindrical element are as follows :-

i) Pressure force pdA in the direction of flow.

ii) Pressure force $\left(p+\frac{\partial p}{\partial s}ds\right)dA$ opposite to the direction of flow.

iii) Weight of element $\rho g dA ds$.

Let θ is the angle between the direction of flow & the line of action of weight of element.

Resultant force = mass of fluid element × Acceleration in the direction s.

$$pdA - \left(p+\frac{\partial p}{\partial s}ds\right)dA - \rho g dA ds\cos\theta = \rho dA ds \times a_s$$

$$\Rightarrow pdA - pdA - \frac{\partial p}{\partial s}ds dA - \rho g dA ds\cos\theta = \rho dA ds \times a_s$$

$$\Rightarrow -\frac{\partial p}{\partial s}ds dA - \rho g dA ds\cos\theta = \rho dA ds \times a_s \quad \text{——— (1)}$$

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a_s = acceleration in direction of s.

$a_s = \frac{dv}{dt}$, where v is function of s & t.

$= \frac{ds}{dt}\frac{\partial v}{\partial s} \cdot \frac{\partial s}{\partial t} + \frac{\partial v}{\partial t} \cdot \frac{\partial t}{\partial t}$

$= \frac{\partial v}{\partial s} \cdot v + \frac{\partial v}{\partial t}$ $\left(\because \frac{\partial s}{\partial t} = v\right)$

As the flow is steady, $\frac{\partial v}{\partial t} = 0$

$a_s = v \cdot \frac{\partial v}{\partial s}$

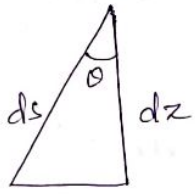
Putting value of a_s in eqn. (1), we get

$\Rightarrow -\frac{\partial p}{\partial s} ds\, dA - \rho g\, dA\, ds \cos\theta = \rho g\, dA\, ds \times v \frac{\partial v}{\partial s}$

Dividing by $\rho\, dA\, ds$,

$\Rightarrow -\frac{\partial p}{\rho \partial s} - g\cos\theta = v\frac{\partial v}{\partial s}$

$\Rightarrow \frac{\partial p}{\rho \partial s} + g\cos\theta + v\frac{\partial v}{\partial s} = 0.$



$\cos\theta = \frac{dz}{ds}$

$\Rightarrow \frac{\partial p}{\rho \partial s} + g\frac{dz}{ds} + v\frac{\partial v}{\partial s} = 0.$

$\Rightarrow \frac{dp}{\rho ds} + g\frac{dz}{ds} + v\frac{dv}{ds} = 0$

$\Rightarrow \boxed{\frac{dp}{\rho} + g\,dz + v\,dv = 0.}$ → Eulers eqn. of motion.

$\Rightarrow$

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Integrating the above equation, we get.

$$\int \frac{dp}{\rho} + \int g\,dz + \int v\,dv = \text{constant}$$

If flow is incompressible ρ is constant.

$$\Rightarrow \frac{p}{\rho} + gz + \frac{v^2}{2} = \text{constant}$$

$$\Rightarrow \boxed{\frac{p}{\rho g} + z + \frac{v^2}{2g} = \text{constant}} \leftarrow \text{Bernoulli's Eqn.}$$

where,

$\frac{p}{\rho g}$ = pressure energy per unit wt. of fluid or pressure head

$\frac{r^2}{2g}$ = kinetic energy per unit wt. or kinetic head

z = potential energy per unit wt. or potential head.

Assumptions

Following assumptions are made in the derivation of Bernoulli's Eqn :-

i) The flow is ideal i.e viscosity is zero.

ii) Flow is steady

iii) Flow is incompressible

iv) Flow is irrotational.

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Statement of Bernoulli's Theorem :-

It is states that in a steady, ideal flow of an incompressible fluid, the total energy at any point of the fluid is constant. The total energy consists of pressure energy, Kinetic energy and potential energy or datum energy.

Limitations of Bernoulli's Equations :-

i) The Bernoulli's equation has been derived under the assumption that the velocity of every liquid particle, across any cross-section of a pipe, is uniform. But, in actual practice, it is not so. The velocity of liquid particle in the center of pipe is maximum & gradually decreases towards the wall of the pipe due to the pipe function. Thus, while using the Bernoulli's equation, only the mean velocity of the liquid should be taken into account.

ii) The Bernoulli's equation has been derived under the assumption that no external force, except the gravity force, is acting on the liquid. But, in actual practice, it is not so. There are always some external forces (such as pipe friction) acting on the liquid, which affect the flow of the liquid. Thus, while using the Bernoulli's equation, all such external forces should be neglected. But if some energy is supplied to, or, extracted from the flow, the same should also be taken into account.

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iii) The Bernoulli's equation has been derived, under the assumption that there is no loss of energy of the liquid particle while flowing. But, in actual practice, it is rarely so. In a turbulent flow, some kinetic energy is converted into heat energy. And in a viscous flow, some energy is lost due to shear forces. Thus, while using Bernoulli's eqn., all such losses should be neglected.

iv) If the liquid is flowing in a curved path, the energy due to centrifugal force should also be taken into account.

Practical Applications of Bernoulli's Equation

Bernoulli's eqn. is applied in all problems of incompressible fluid flow where energy ~~considered~~ considerations are involved. Its application is involved in following measuring devices :-

i) Venturimeter
ii) Orifice meter.
iii) Pitot-tube

Venturimeter :-

It is an apparatus for finding out the discharge of a liquid flowing in a pipe. A venturimeter, in its simplest form, consists of following three parts :-

a) Convergent cone b) Throat c) Divergent cone

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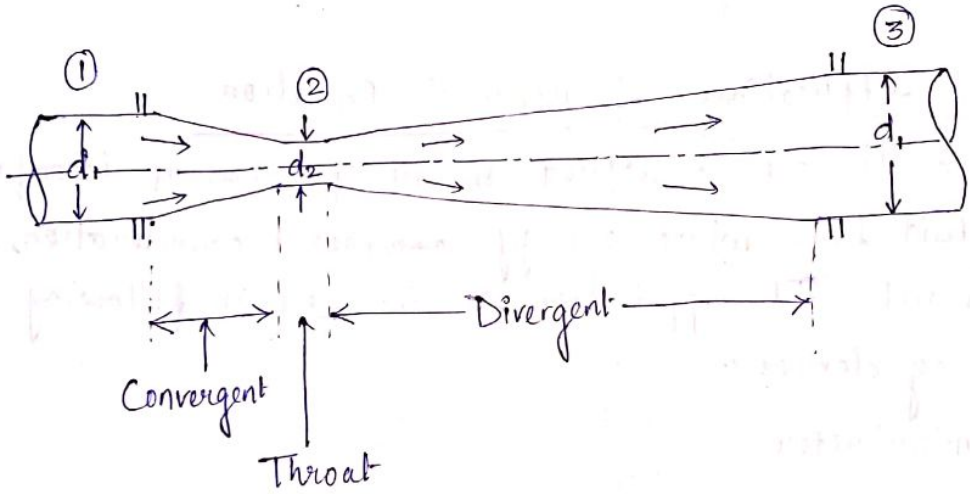
a) Convergent Cone

It is a short pipe which converges from a diameter d_1 (dia. of the pipe in which the venturimeter is fitted) to a smaller diameter d_2. The convergent cone is also known as inlet of the venturimeter. The slope of the converging side is between 1 in 4 or 1 in 5.

b) Throat

It is the smallest portion of circular pipe in which the diameter d_2 is kept constant.



c) Divergent Cone

It is a pipe, which diverges from a dia d_2 to a large diameter d_1. The divergent cone is also known as outlet of the venturimeter. The length of the divergent cone is about 3 to 4 times than that of the convergent cone.

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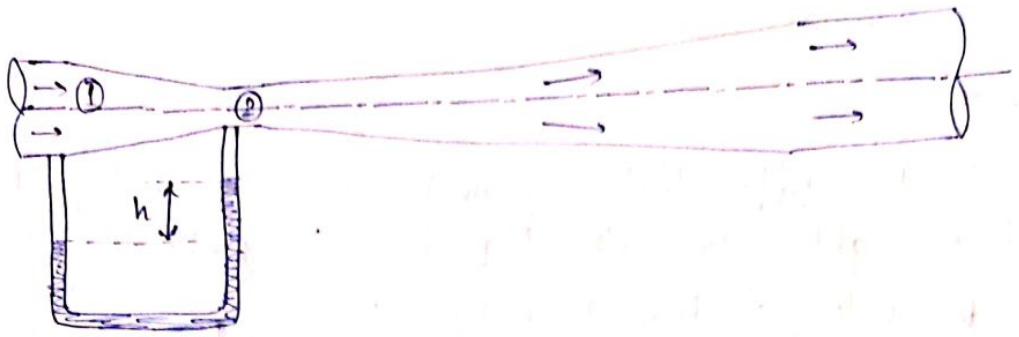
Discharge through a venturimeter



Consider a venturimeter through which some liquid is flowing.

Let

p_1 = pressure at section 1.

v_1 = velocity of water at section 1.

z_1 = datum head at section 1,

a_1 = area of venturimeter at section 1,

p_2, v_2, z_2 & a_2 = corresponding values of section 2.

$$Q = C_d \times \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \times \sqrt{2gh}$$

C_d = co-efficient of venturimeter & its value is less than 1.

Value of 'h' given by differential U-tube manometer

Case-I

Let the differential manometer contains a liquid which is heavier than the liquid flowing through the pipe. Let,

S_h = Sp. gravity of heavier liquid

S_o = Sp. gravity of liquid flowing through pipe

x = diff. of heavier liquid column in U-tube

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$$h = x\left[\frac{S_h}{S_0} - 1\right]$$

Case II

If the differential manometer contains a liquid which is lighter than the liquid flowing through a pipe, the value of h is given by,

$$h = x\left[1 - \frac{S_l}{S_0}\right]$$

where,

S_l = Sp. gravity of lighter liquid in U-tube

S_0 = Sp. gravity of fluid flowing through pipe

x = difference of lighter liquid column in U-tube.

Case III

~~Let~~ This case is related to inclined venturimeter having differential U-tube manometer. Let the differential manometer contains heavier liquid then h is,

$$h = x\left[\frac{S_h}{S_0} - 1\right]$$

Case IV

In this case inclined venturimeter in which differential manometer contains a liquid which is lighter than the liquid flowing through the pipe, the value of h is,

$$h = x\left[1 - \frac{S_l}{S_0}\right]$$

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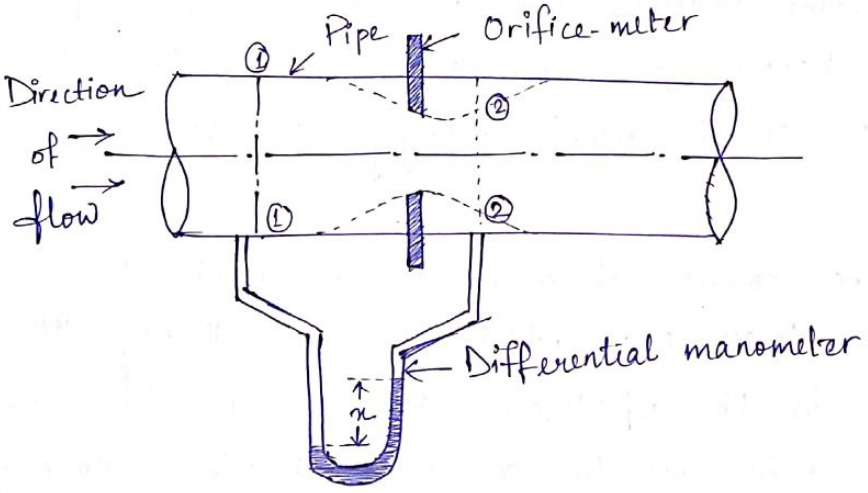
Orifice Meter or Orifice Plate

→ It is a device used for measuring the rate of flow of a fluid through a pipe. It is cheaper than venturimeter.

→ 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.

→ Orifice diameter is kept 0.5 times the diameter of the pipe, though it may vary from 0.4 to 0.8 times the pipe diameter.



→ A differential manometer is connected at section (1), at a distance of about 1.5 to 2 times the pipe dia. upstream from orifice plate, and at section (2), which is at a distance of about half the dia. of orifice on the downstream side from orifice plate.

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Let p_1 = pressure at section (1),

v_1 = velocity at section (1),

a_1 = area of pipe at section (1), &

p_2, v_2, a_2 = corresponding values at section (2).

Applying Bernoulli's eqn. at section (1) & (2), we get

$$Q = \frac{C_d \, a_0 a_1 \sqrt{2gh}}{\sqrt{a_1^2 - a_0^2}}$$

Pitot - Tube

→ It is a device used for measuring the velocity of flow at any point in a pipe or channel.

→ Based on the principle that if the velocity of flow at a point becomes zero, the pressure increases due to conversion of the kinetic energy into pressure energy.

→ Pitot tube consist of a glass tube, bent at right angle. The lower end which is bent through 90° is directed in the upstream direction. Liquid rises up in the tube due to conversion of kinetic energy into pressure energy.

→ Velocity is determined by measuring the rise of liquid in the tube.

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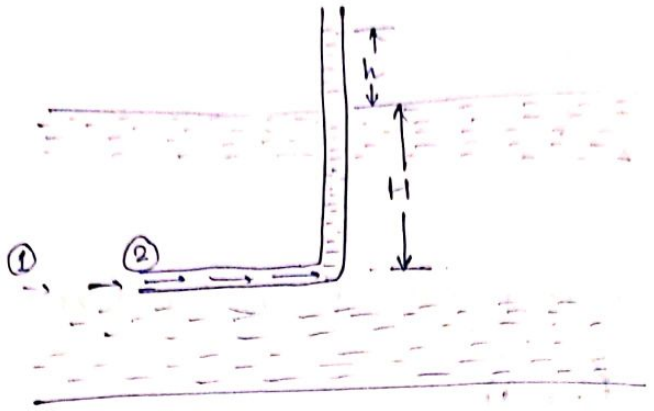

Let, p_1 = intensity of pressure at point (1)

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 liquid

h = rise of liquid in the tube above free surface.

Velocity at a point,

$$v = C_v\sqrt{2gh}$$

where,

C_v = coefficient of pitot tube.

Velocity of flow in a pipe by pitot-tube

For velocity at any point in a pipe by pitot-tube, following arrangements are adopted:-

i) Pitot-tube along with a vertical piezometer tube.

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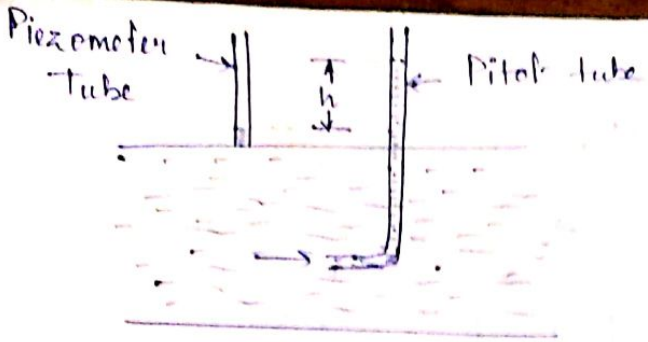
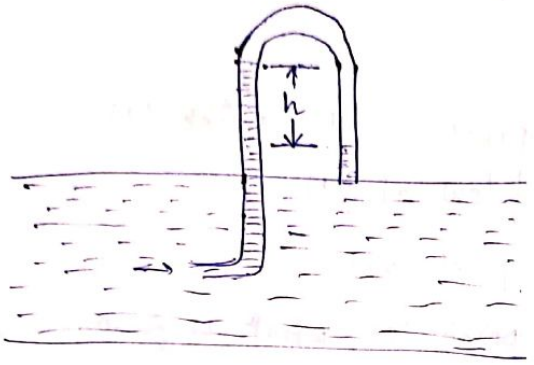
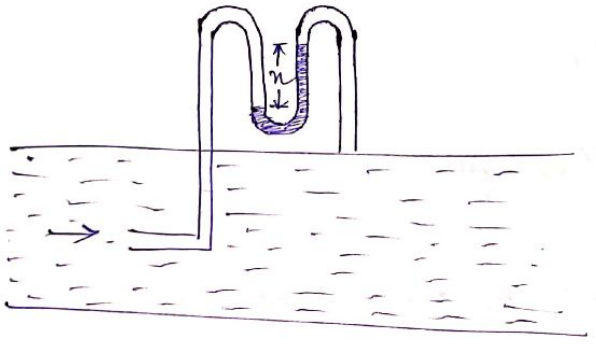


ii) Pitot tube connected with piezometer tube.



iii) Pitot-tube & vertical piezometer ~~or~~ tube connected with a differential U-tube manometer



iv) Pitot-~~tub~~ static tube consist of two concentric tubes one inside the other with some annular space in between. The outlet of these two tubes are connected ~~on~~ to the differential manometer where the difference of pressure head 'h' is measured by knowing the difference of the level of manometer liquid say x. Then $h = x\left[\frac{S_1}{S_0} - 1\right]$

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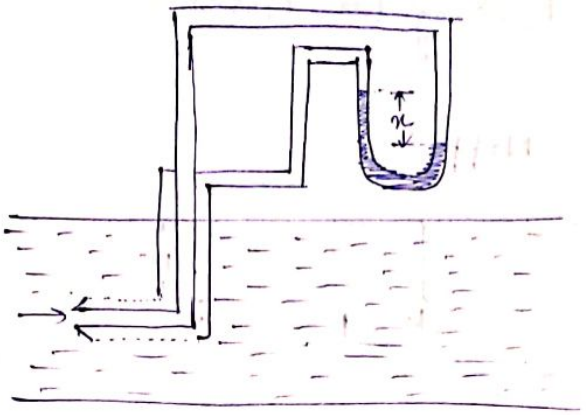

2.2 Flow Over Notches

→ Notch is a device used for measuring the rate of flow of a liquid through a small channel or a tank.

→ It may be defined as an opening in the side of a tank or a small channel in such a way that the liquid surface in the tank or channel is below the top edge of the opening.

Classification of Notches :-

Notches are classified as :-

1. According to the shape of the opening :-
   a) Rectangular notch
   b) Triangular notch
   c) Trapezoidal notch
   d) Stepped notch

2. According to the effect of the sides on the nappe :-
   a) Notch with end contraction
   b) Notch with out end contraction or suppressed notch.

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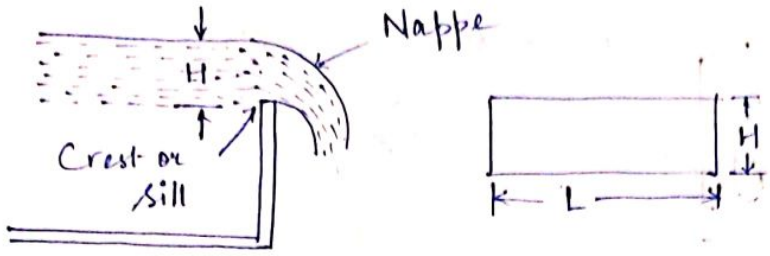
Discharge over a Rectangular Notch



H = head of water over the crest
L = length of the notch.

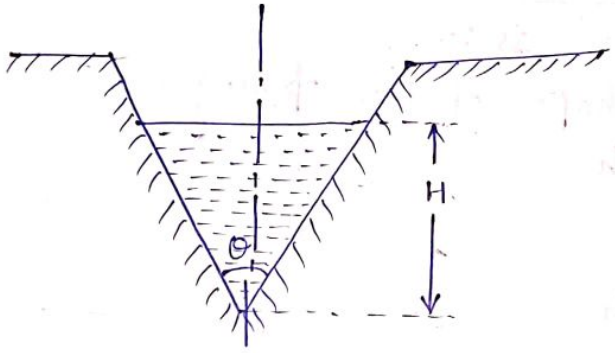
Discharge,

$$Q = \frac{2}{3} C_d \times L \times \sqrt{2g} [H]^{3/2}$$

where,

C_d = coefficient of discharge.

Discharge over a Triangular Notch.



Let, H = head of water above the V-notch.

θ = angle of notch.

Discharge,

$$Q = \frac{8}{15} C_d \tan\frac{\theta}{2} \sqrt{2g} \times H^{5/2}$$

where, C_d = coefficient of discharge

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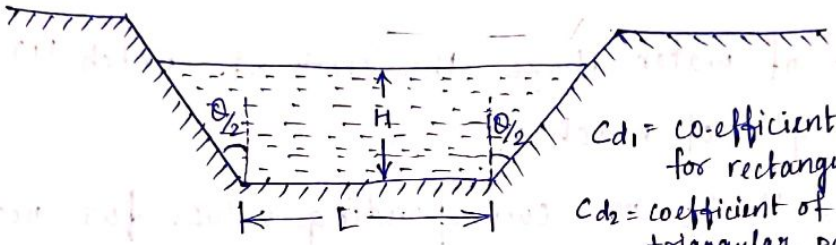
For right angled V- notch, if $C_d = 0.6$

$\theta = 90^\circ \quad \therefore \tan\frac{\theta}{2} = 1$

$$Q = \frac{8}{15} \times 0.6 \times 1 \times \sqrt{2 \times 9.81} \times H^{5/2}$$

$$= 1.417\, H^{5/2}$$

Discharge Over a Trapezoidal Notch.



C_{d_1} = co-efficient of discharge for rectangular portion

C_{d_2} = coefficient of discharge for triangular portion.

A trapezoidal notch is a combination of a rectangular and triangular notch.

Total discharge will be equal to the sum of the discharge through a rectangular notch and discharge through a triangular notch.

Total discharge,

$$Q = \frac{2}{3} C_{d_1} L \sqrt{2g} \times H^{3/2} + \frac{8}{15} C_{d_2} \times \tan\frac{\theta}{2} \times \sqrt{2g} \times H^{5/2}$$

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 discharge through the different rectangular notches.

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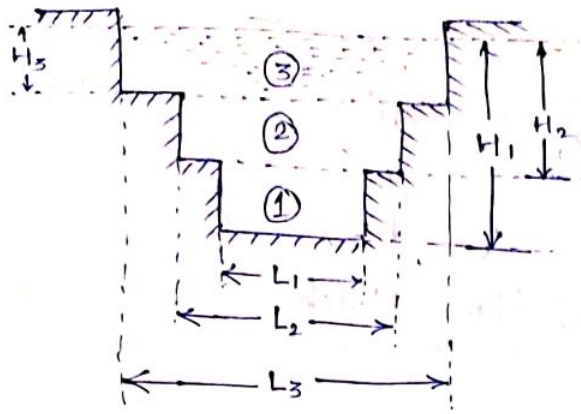

Let, H_1 = ht. of water above the crest of notch (1).

L_1 = length of notch 1,

H_2, L_2 & H_3, L_3 are corresponding values for notches 2 & 3 respectively.

C_d = co-efficient of discharge for all notches.

Total discharge, $Q = Q_1 + Q_2 + Q_3$

$$Q = \frac{2}{3} \times C_d \times L_1 \times \sqrt{2g} \left[H_1^{3/2} - H_2^{3/2} \right] + \frac{2}{3} C_d \times L_2 \times \sqrt{2g} \left[H_2^{3/2} - H_3^{3/2} \right]$$

$$+ \frac{2}{3} C_d \times L_3 \times \sqrt{2g} \times H_3^{3/2}$$

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Flow Over Weirs

Weir :-

A structure, used to dam up a stream or river, over which the water flows, is called a weir.

The conditions of flow, in the case of a weir, are practically the same as those of a rectangular notch.

The only difference between a notch & a weir is that the notch is of a small size & the weir is of a bigger size.

Notch is usually made in a plate, whereas a weir is usually made of masonry or concrete.

Types of Weirs :-

There are many types of weirs depending upon their shape, nature of discharge, width of crest and nature of crest.

But the following are important from the subject point of view :-

1. According to the shape :-
   a) Rectangular weir
   b) Cippoletti weir
2. According to the nature of discharge :-
   a) Ordinary weir
   b) Submerged or drowned weir
3. According to width of crest :-
   a) Narrow-crested weir.
   b) Broad-crested weir

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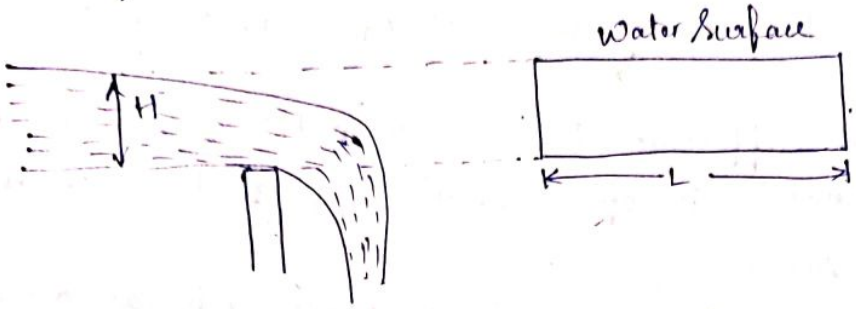
4. According to the nature of crest :-
   a) Sharp-crested weir.
   b) Ogee weir.

Discharge over a Rectangular Weir :-

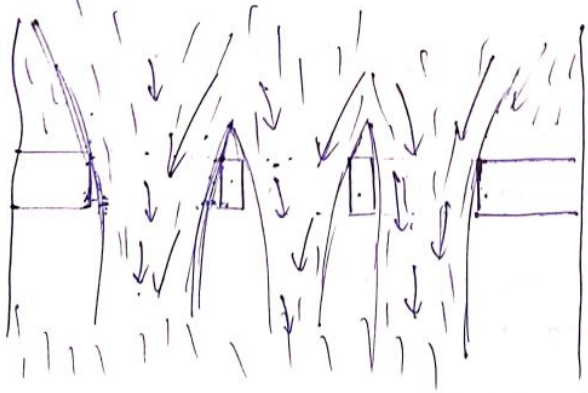


Let, H = ht. of water above crest of weir
L = length of weir
C_d = coefficient of discharge

$$Q = \frac{2}{3} \times C_d \times L \times \sqrt{2g} \times H^{3/2}$$

Francis's Formula for Discharge Over a Rectangular Weir (Effect of end contraction)

Francis found that the length of stream of liquid, while flowing over a weir, gets contracted at the ends of the sill.



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This end contraction of the stream of liquid is known as lateral contraction or side contraction.

Amount of end contraction depends upon the condition of sides of the channel & top of the sill as well as velocity of liquid.

An approximate value of end contraction at each end is $\frac{1}{10}$th of the height of the liquid above the sill of weir.

If their are two end contractions only the effective length of the weir is $(L-0.2H)$.

$$Q = \frac{2}{3} C_d \times (L-0.2H)\sqrt{2g} \times H^{3/2}$$

If the total length of weir is divided into number of bays or spans by vertical posts, the no. of end contraction will be twice the number of bays or spans, into which the weir is divided.

$$Q = \frac{2}{3} C_d \times (L-0.1nH)\sqrt{2g} \times H^{3/2}$$

where,

n = no. of end contractions

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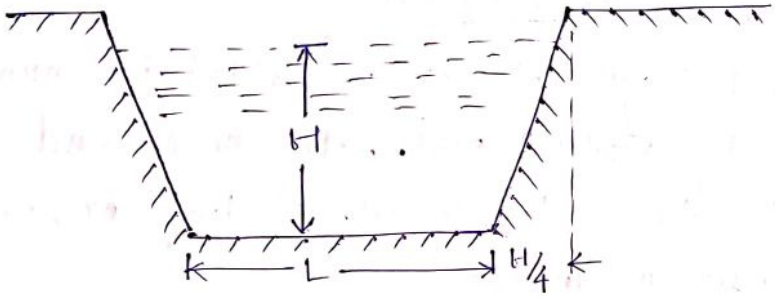
Discharge Over a Cippoletti Weir

Cippoletti weir is a trapezoidal weir, having 1 horizontal to 4 vertical side slopes.



The purpose of slope, on the sides is to obtain increased discharge through the triangular portion of the weir, which would have been decreased due to end contraction in case of rectangular weirs.

Advantage of a ~~Cipot~~ Cippoletti weir is that the factor of end contractions is not required, while using Francis formula.

So, $$Q = \frac{2}{3} C_d L \sqrt{2g}\, H^{3/2}$$

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Discharge Over Narrow-Crested Weir

Let,

b = width of the crest of weir

H = ht. of water above the weir crest

If 2b is less than H, the weir is called narrow-crested weir. But if 2b is more than H it is called a broad-crested weir.

Unless specified, the given weir is taken as narrow-crested weir.

$$Q = \frac{2}{3} C_d \times L \times \sqrt{2g} \times H^{3/2}$$

where,

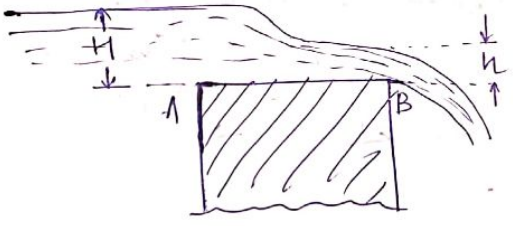
Q = discharge over weir

C_d = coefficient of discharge

L = length of weir

H = ht. of water level above the crest of weir

Discharge over a Broad-crested weir



Let, H = head of water on the upstream side of weir.

h = head of water on the downstream side of weir.

C_d = coefficient of discharge.

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L = length of weir.

Discharge,

$$Q = C_d \cdot L \cdot h \sqrt{2g(H-h)}$$

For max. discharge,

$$h = \frac{2}{3}H.$$

$$Q_{max} = 1.71\, C_d \cdot L \cdot H^{3/2}$$

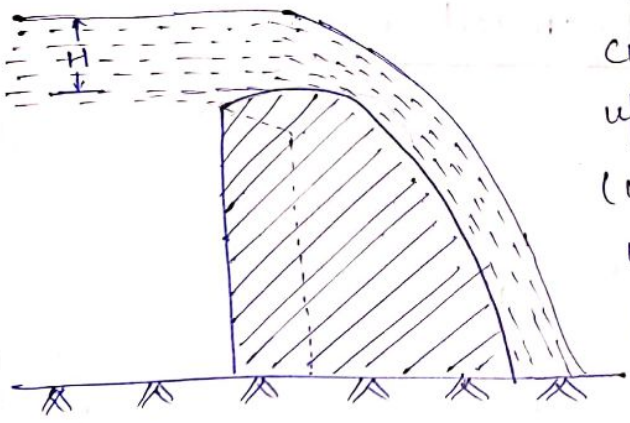
Discharge over narrow-crested weir

For narrow-crested weir,

$2L < H$.

$$Q = \frac{2}{3} \times C_d \times L \times \sqrt{2g} \times H^{3/2}$$

Discharge over an Ogee Weir



In ogee weir, the crest of the weir rises upto max. ht. of 0.115 H (where H is the ht. of water above inlet of weir) and then falls.

Discharge for an Ogee weir :-

$$Q = \frac{2}{3} \times C_d \times L \times \sqrt{2g} \times H^{3/2}$$

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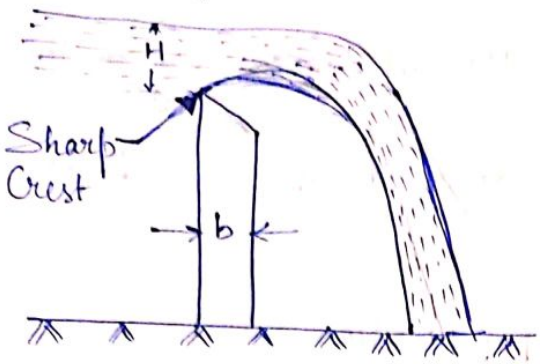
Discharge over a sharp-crested weir.



It's a special type of weir, having a sharp crest. Water flows over the crest, comes in contact with the crest line & then springs up from the crest & falls as a trajectory.

In sharp-crested weir, the thickness of weir is kept less than half of the ht. of water on the weir,

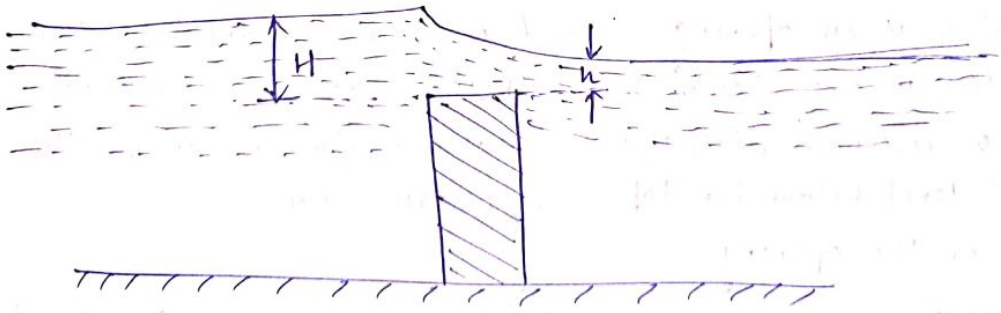
$$b < \frac{H}{2}$$

Discharge,

$$Q = \frac{2}{3} \times C_d \times L \times \sqrt{2g} \times H^{3/2}.$$

Discharge over a Submerged or Drowned Weir.

→ When water level on the downstream side of a weir is above the crest of the weir, then the weir is called sub-merged or drowned weir.



Total discharge, over the weir is obtained by dividing the weir into two parts. The portion between upstream & downstream water surface may be treated as free weir & portion between downstream water

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surface & crest of weir as drowned weir.

Let,

H = ht. of water on the upstream side of the weir.

h = ht. of water on downstream side of the weir.

Q_1 = discharge over upper portion

$$= \frac{2}{3} \times C_d \times L \times \sqrt{2g} \times [H-h]^{3/2}$$

Q_2 = discharge through drowned portion

$$= C_{d_2} \times L \times h \times \sqrt{2g(H-h)}$$

Total discharge,

$$Q = Q_1 + Q_2$$

$$= \frac{2}{3} \cdot C_d \cdot L \sqrt{2g} \cdot (H-h)^{3/2} + C_{d_2} \cdot L \cdot h \cdot \sqrt{2g(H-h)}$$

Difference Between Notch & Weir

Notch	Weir
→ A notch is an opening provided in one side of a tank or reservoir, with u/s liquid level below the top edge of the opening.	→ A weir is a structure constructed across a river or canal to store water on the upstream side.
→ The bottom edge of notch over which water flows is known as sill or crest	→ The top of weir over which water flows is known as crest.

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→ A notch is usually made of a metallic plate.	→ A weir is made of cement-concrete or masonry.
→ A notch is used to measure small discharge of small stream or canal.	→ A weir is used to measure large discharge of rivers & large canals.
→ Notches are of small size.	→ Weirs are of bigger size.

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2.3 Types of flow through the pipe.

Uniform Flow :-

A flow in which the velocities of liquid particles at all sections of a pipe or channel are equal, is called a uniform flow.

This term is generally applied to flow in channels.

Non-uniform flow :-

A flow in which the velocities of liquid particles at all sections of a pipe or channel are not equal, is called non-uniform flow.

Laminar flow :-

A flow in which each liquid particle ~~does not have a~~ has a definite path, and the paths of individual particles donot cross each other,

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is called laminar flow or streamline flow.

Turbulent flow :-

A flow in which the each liquid particle does not have a definite path, and the path of individual particles crosses each other, is known as turbulent flow

Steady flow :-

A flow, in which the quantity of liquid flowing per second is constant, is called steady flow.

A steady flow may be uniform or non-uniform.

Unsteady flow :-

A flow, in which the quantity of liquid flowing per second is not constant, is called unsteady flow.

Reynold's Number

It is defined as the ratio of inertia force of a flowing fluid & the viscous force of the fluid.

Reynold's Number,

$$R_e = \frac{V \times L}{\nu}$$

where,

~~V = volume, L = Linear dimension~~

~~ν = Kinematic Viscosity.~~

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In case of pipe flow, the linear dimension L is taken as diameter, d. Hence Reynold's number for pipe flow,

$$Re = \frac{V \times d}{\nu} \text{ or } \frac{\rho V d}{\mu}$$

where,

Re = Reynolds number
ρ = fluid density (kg/m^3)
V = velocity (m/s)
L = length of travel (m)
μ = fluid dynamic ~~visos~~ viscosity ~~(kg~~

Applications of Reynold's Number

i) Reynold's Number plays an important part in the calculation of the friction factor in a few of the equations of fluid mechanics, including the Darcy-Weisbach eqn.

ii) It is used when modelling the movement of organisms swimming through water.

iii) Atmospheric air is considered to be a fluid. Hence, the Reynold's number can be calculated for it. This makes it possible to apply it in wind tunnel testing to study the aerodynamic properties of various surfaces.

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iv) It plays an important part in the testing of wind lift on aircraft, especially in cases of supersonic flights where the high speed causes a localized increase in the density of air surrounding the air craft.

24 Losses of head of a liquid flowing through pipes

Loss of Head due to Friction :-

→ **Darcy - Weisbach Formula**

The loss of head in pipes due to friction is calculated from Darcy - Weisbach eqn.,

$$h_f = \frac{4 \cdot f \cdot L \cdot v^2}{d \times 2g}$$

where,

h_f = loss of head due to friction.

f = co-efficient of friction which is a function of ~~Reyon~~ Reynolds 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.

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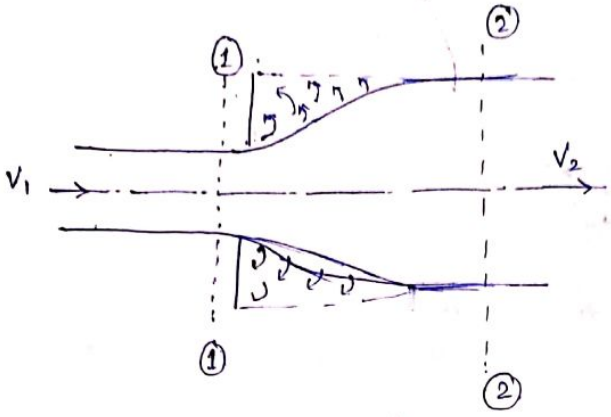
Loss of Head Due to Sudden Enlargement



Let,

P_1 = pressure intensity at section 1-1.

V_1 = velocity of flow at section 1-1.

A_1 = area of pipe at section 1-1.

P_2, V_2 & A_2 = Corresponding values at section 2-2.

Due to sudden change of diameter of the pipe from D_1 to D_2, the liquid flowing from smaller pipe is not able to follow the abrupt change of boundary. Thus the flow separates from the boundary and turbulent eddies are formed. The loss of head takes place due to formation of these eddies.

Loss of head,

$$h_e = \frac{V_1^2 - V_2^2}{2g}$$

where,

h_e = loss of head due to sudden enlargement.

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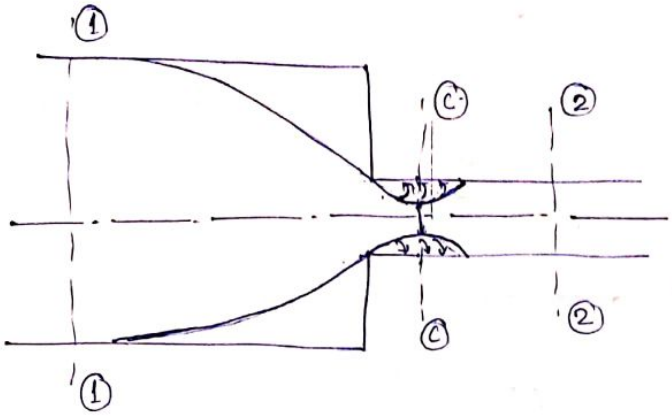
Loss of Head due to Sudden Contraction



Water or liquid flows in a pipe which has sudden contraction. Considering two sections 1-1 & 2-2 before & after contraction. As liquid flows from large pipe to smaller pipe, the area of flow goes on decreasing and becomes minimum at section c-c. This section c-c is called Vena-contracta.

Let,

A_c = area of flow at section c-c.

V_c = velocity of flow at section c-c.

A_2 = area of flow at section 2-2.

V_2 = velocity of flow at section 2-2.

h_c = loss of head due to sudden contraction.

$$h_c = \frac{V_2^2}{2g}\left[\frac{1}{C_c} - 1\right]^2$$

where,

C_c = coefficient of contraction.

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If the value of C_c is not given then the head loss due to contraction is given as,

$$= 0.5 \frac{V_2^2}{2g} \quad \text{or} \quad h_c = 0.5 \frac{V_2^2}{2g}$$

Loss of Head at the Entrace of a Pipe

It is the loss of energy which occurs when a liquid enters a pipe which is connected to a large tank or reservoir. This loss is similar to the loss of head due to sudden contraction. This loss depends on form of entrance.

For sharp edge entrance, this loss is slightly more than a rounded or bell mouthed entrance.

This loss is denoted by,

$$h_i = 0.5 \frac{V^2}{2g}$$

Loss of Head at the Exit of Pipe.

It is the loss of head due to the velocity of liquid at outlet of the pipe which is dissipated either in the form of free jet or it is lost in the tank or reservoir.

The loss is given by,

$$h_o = \frac{V^2}{2g}$$

where, V = velocity at outlet of pipe.

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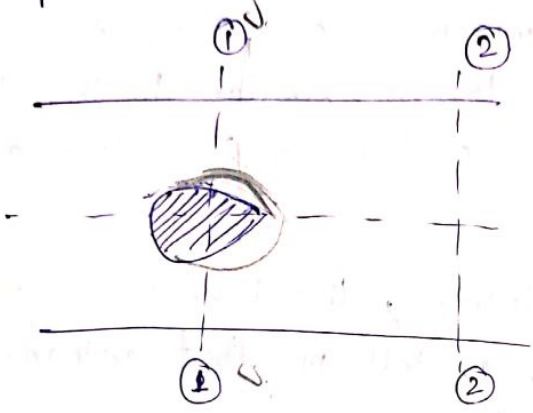
Loss of Head Due to an Obstruction in a Pipe

When obstruction is present in a pipe, the loss of energy takes place due to reduction of the area of the cross-section of the pipe at the place of obstruction. There is sudden enlargement of the area of flow beyond the obstruction due to which loss of head takes place.



Consider a pipe section,

Let a = Max. area of obstruction

A = Area of pipe

V = velocity of liquid in pipe

A-a = area of flow of liquid at section 1-1.

V_c = velocity of liquid at vena-contracta.

Head loss due to obstruction,

$$= \frac{(V_c - V)^2}{2g} = \frac{V^2}{2g}\left(\frac{A}{C_c(A-a)} - 1\right)^2$$

where,

C_c = coefficient of contraction.

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Loss of Head due to Bend in Pipe.

When there is bent in pipe, the velocity of flow changes, due to which the separation of the flow from the boundary & also formation of eddies takes place. Thus energy is lost. Loss of head in pipe due to bend is expressed as,

$$h_b = \frac{kV^2}{2g}$$

where,

h_b = loss of head due to bend

V = velocity of flow

K = coefficient of bend.

Value of K depends on :-

i) Angle of bend ii) Radius of curvature of bend

iii) Dia. of pipe.

Loss of Head in Various Pipe Fittings

Loss of head in the various pipe fittings such as valves, couplings etc is expressed as,

$$= \frac{kV^2}{2g}$$

where,

V = velocity of flow

K = coefficient of pipe fitting.

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Hydraulic Gradient line

It is defined as the line which gives the sum of pressure head $(\frac{p}{w})$ and datum head (z) of a flowing fluid in a pipe with respect to some reference line or it is the line which is obtained by joining the top of all vertical ordinates, showing the pressure head $(\frac{p}{w})$ of a flowing fluid in a pipe from the center of pipe. It is written as H.G.L (Hydralic Gradient line).

Total Energy line

It is defined as the line which gives the sum of pressure head, datum head & Kinetic head of a flowing fluid in the pipe with respect to some reference line.

It is also defined as the line which is obtained by joining the top of all vertical ordinates showing the sum of pressure head & Kinetic head from center of pipe. It is written as T.E.L (Total energy line).

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2.5 Flow Through the Open Channels

Discharge Through Open Channel By Chezy's Formula :-

$$Q = \text{Area} \times \text{Velocity}$$
$$= A \times V$$
$$= A \times C\sqrt{mi}$$

where,

A = area of flow of water

C = Chezy's constant

m = hydraulic mean depth of hydraulic radius.

$$= \frac{A}{P}$$

$$= \frac{\text{Area of flow of water.}}{\text{Wetted perimeter of the cross-section}}$$

i = slope of bed.

Discharge Through Open Channel By Manning's Formula :-

$$Q = A \times V$$
$$= A \times C\sqrt{mi}$$

$$C = \frac{1}{N} m^{1/6}$$

N = manning's constant

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Flow in open channel is defined as flow of liquid with free surface. A free 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 channel.

Best Economical Section

A section of a channel is said to be most economical when cost of construction of the channel is minimum. Cost of construction depend upon the excavation & lining. To keep cost minimum, the wetted perimeter for a given discharge should be minimum.

A section is said to be most economical as the discharge, passing through a most economical section of a channel for given cross-sectional area (A), slope of bed (i), and resistance co-efficient, is maximum.

As, $Q = A \times V$

$= AC\sqrt{mi}$

$Q = AC\sqrt{\frac{A \times i}{P}}$

Discharge will be maximum, when wetted perimeter P is minimum.

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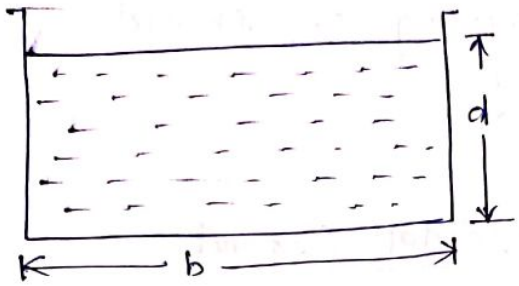
It is used for determining best section of a channel i.e best dimension of a channel for a given area.

Most Economical Rectangular Channel



For most economical section the wetted perimeter should be minimum.

Area of flow = $A = b \times d$

Wetted perimeter, $P = d + b + d = b + 2d$.

$$b = \frac{A}{d}$$

Putting in P, $P = b + 2d$

$$= \frac{A}{d} + 2d$$

For most economical section, P should be minimum,

$$\boxed{\frac{dP}{d(d)} = 0}$$

$$\frac{d}{d(d)}\left[\frac{A}{d} + 2d\right] = 0 \Rightarrow -\frac{A}{d^2} + 2 = 0$$

$$\Rightarrow 2 = \frac{A}{d^2}$$

$$\Rightarrow A = 2d^2$$

$A = b \times d$

$\therefore b \times d = 2d^2 \Rightarrow b = 2d.$

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Hydraulic mean depth, $m = \frac{A}{P} = \frac{b \times d}{b + 2d}$

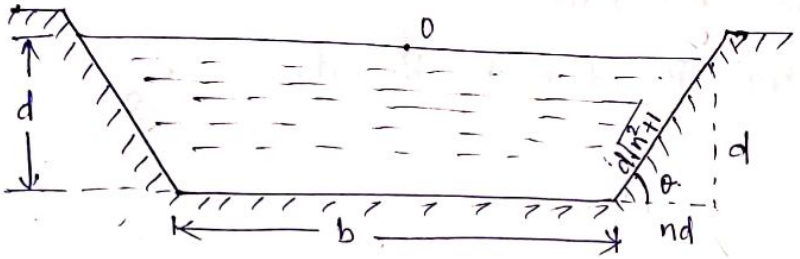
$= \frac{2d \times d}{2d + 2d}$

$= \frac{2d^2}{4d} = \frac{d}{2}$

For most economical rectangular channel,

$\boxed{b = 2d}$ or $\boxed{m = \frac{d}{2}}$

Most Economical Trapezoidal Channel



For most economical section, wetted perimeter should be minimum.

If the semi-circle is drawn with O as center and radius equal to depth of flow d, the three sides of most economical trapezoidal section will be tangential to the semi-circle.

Conditions for most economical trapezoidal section are :-

i) $\frac{b + 2nd}{2} = d\sqrt{n^2 + 1}$

ii) $m = \frac{d}{2}$

iii) A semi-circle drawn from O with radius equal to

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depth of flow will touch the three side of the channel.

Best side slope for Most Economical Trapezoidal Section

Best side slope is at 60° to the horizontal or the value of n for the best side slope is,

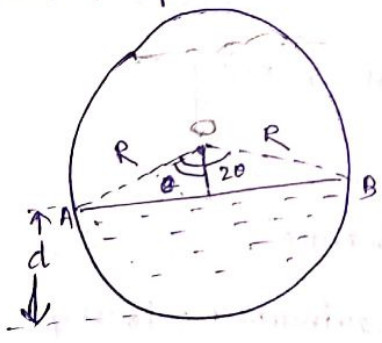
$$n = \frac{1}{\sqrt{3}}$$

For a slope of 60°, the length of sloping side is equal to width of trapezoidal section.

$$P = 3b.$$

Flow Through Circular Channel.

The ~~when~~ liquid flows through a circular pipe, when the level of liquid in the pipe is below the top of the pipe is classified as open channel flow. The rate of flow is determined from the depth of flow & angle subtended by the liquid surface at the center of circular channel.



d = depth of water

2θ = angle subtended by water surface AB at the center in radians,

R = radius of channel.

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$Q = A \times V$

$= A \times C\sqrt{mi}$

$m = \frac{A}{P} = \frac{R^2\left(\theta - \frac{\sin 2\theta}{2}\right)}{2R\theta} = \frac{R}{2\theta}\left(\theta - \frac{\sin 2\theta}{2}\right)$

Wetted perimeter, $P = \frac{2\pi R}{2\pi} \times 2\theta = 2R\theta$

Most Economical Circular Section

In case of circular channels, the area of flow cannot be maintained constant. With the change of depth of flow in a circular channels, ~~the area of flow cannot be maintained~~ of any radius, the wetted area & wetted perimeter changes.

In circular channels, for most economical section two separate conditions are obtained. They are:-

i) Condition ~~with~~ for maximum velocity.

ii) Condition for maximum discharge.

Condition for Maximum Velocity for Circular Section

The depth of flow for maximum velocity

$$\boxed{d = 0.81D}$$

where, D = diameter of circular channel.

Hydraulic mean depth for maximum velocity

$$\boxed{m = 0.3D}$$

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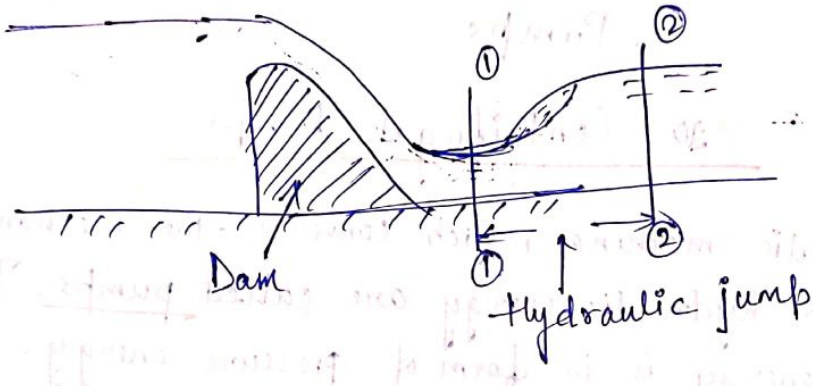
Condition for Maximum Discharge for Circular Section

Depth of flow for max. discharge.

$$d = 0.95D$$

Hydraulic Jump or Standing Wave.



When water flows over a dam, on the down-stream side the ht. of water or depth of water increases rapidly over short length. Because at section 1-1, the flow is shooting flow as the depth of water at section 1-1 is less than critical depth.

Shooting flow is an unstable type of flow & does not continue on the downstream side. Then the shooting flow will convert into a streaming flow & hence depth of water will increase.

This sudden increase of depth of water is called hydraulic jump.

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The rise of water level, which takes place due to transformation of the unstable shooting flow (super-critical) to the stable streaming flow (sub-critical flow).

At hydraulic jump, loss of energy takes place due to eddy formation & ~~turbulati~~ turbulence occurs.

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3.0 Pumps

Pumps are classified on basis of its mechanical configuration & their working principle.

3.1 Types of pumps :-

1) Dynamic pumps / Kinetic pumps
2) Displacement pumps / positive displacement pumps

Dynamic Pumps :-

These pumps impart velocity & pressure to the fluid as it moves past or through the pump impeller & subsequently, convert some of that velocity into additional pressure. It is called kinetic pumps.

Dynamic pumps are sub-divided into two major groups and they are :-

i) Centrifugal pumps
ii) Vertical pumps

i) Centrifugal Pumps :-

It is a rotating machine in which flow & pressure are generated dynamically. The energy changes occur by virtue of two main parts of the pump, the impeller & the volute or casing. The function of the casing is to collect the liquid discharged by the impeller & to convert some of the kinetic

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(velocity) energy into pressure energy.

Centrifugal pumps are subdivided into several categories and they are :-

a) Radial Flow :-

The impeller discharges fluid at right angles to the shaft axis. In this centrifugal pump in which the pressure is developed wholly by centrifugal force. The radial type pumps are used for the application of high head & low discharge.

b) Mixed Flow :-

The flow direction is partly axial & partly radial. Hence has a result the flow is diagonal. The mixed flow type pumps are used for the application of medium head & high discharge.

Axial Flow :-

The flow through impeller is parallel to shaft axis low head & very high discharge. The axial flow type pumps are used for the application of medium head & high discharge.

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Vertical Pumps:

These pumps were originally developed for well pumping. The bore size of well limits the outside diameter of the pump & so controls the overall pump design.

Vertical pumps can be subdivided into three major categories :-

a) Line Shaft Pumps :-

The driver is mounted on the discharge head for these type of motors. The line-shafting extend through the column to the bowl assembly & transmit torque to the pump rotor.

b) Submersible Pumps :-

These are close coupled pumps driven by a submersible motor & designed for submerged installation in a wet well. The motor is mounted below the bowl assembly & is directly coupled to the pump rotor shaft.

Displacement Pumps :-

Positive displacement pumps, the moving element (piston, plunger, rotor, lobe or gear) displaces the liquid from the pump casing (or cylinder) and, at the same time, raises the pressure of the liquid. So displacement pumps does not develop pressure, it only

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produces a flow of fluid.

Classification of Displacement Pumps :-

The mainly positive displacement pumps are sub-divided into three categories as follow :-

i) Reciprocating pumps
ii) Rotary pumps
iii) Pneumatic pumps

Reciprocating pumps :

In a reciprocating pumps, a piston or plunger moves up & down. During the suction stroke, the pump cylinder fills with fresh liquid, & the discharge stroke displaces it through a check valve into the discharge line.

Reciprocating pumps can develop very high pressures. Plunger, piston & diaphragm pumps are under these type of pumps.

a) Plunger / piston type pumps :-

The plunger contains the cross head, driven by a cams haft arrangement. The capacity of pump can be adjusted by changing the stroke, the rotating speed of the pump, or both. The stroke of the pump is changed by the eccentric pin setting.

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These types pumps used for the application of sewage, sludge, scum, clasifier thickener underflow. It can be applied for transfer & for metering service. Such pumps are available in single and multi cylinder models

b) Diaphragm pumps :-

These type of pumps are quite versatile, handling a wide variety of fluids like food additives, chemicals, dry powders, slurries, pharmaceutical products, & wastewater etc. The advantages in diaphragm pumps is the absence of seals or packing, meaning they can be used in applications requiring zero leakage.

Rotary Type Pumps :

The pump rotor of rotary pumps displaces the liquid either by rotating or by a rotating & orbiting motion. The rotary pump mechanisms consisting of a casing with closely fitted cams, lobes or vanes, that provide a means for conveying a fluid.

Vane, gear & lobe pumps are positive displacement rotary pumps.

a) Rotary Lobe Pumps :

Lobe pumps contains two elastomer-coated rotors that are driven by an integral gear box & synchronized by timing gears. The rotors run without touching each other or the casing. The liquid is

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drawn through the inlet port into the pockets between the lobes & chamber walls. Because liquid cannot escape between the two rotors, it discharges in the direction of rotation of the outer lobes through the discharge nozzle.

b) Screw Pumps:-

These pumps are a special type of rotary positive displacement pump in which the flow through the pumping elements is truly axial. Screw pumps are high-volume, non-clog, atmospheric-head devices that can pump a variety of solids & debris in raw waste water without screening. Screw pumps, however, have a practical limitation as to pumping head.

c) Progressive Cavity Pumps :-

A progressive cavity pump is designed specifically to transfer abrasive & viscous fluids with a high solid, fiber & air content. A hard steel screw rotor rotates & orbits within an elastomer stator.

Pneumatic Pumps :-

Compressed air is used to move the liquid in pneumatic pumps. In pneumatic ejectors, compressed air displaces the liquid from a gravity fed pressure vessel through a check valve into the discharge line in a series of surges spaced by the time required for the tank or receiver to fill again.

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32 Centrifugal Pumps

→ A hydraulic machine which converts the mechanical energy into hydraulic energy are called pumps. The hydraulic energy is in form of pressure energy.

→ If mechanical energy is converted into pressure energy by means of centrifugal force acting on the fluid, the hydraulic machine is called centrifugal pump.

Working Principle

The flow in centrifugal pumps is in radial outward direction. Centrifugal pump works on the principle of forced vortex flow. when means that when a certain mass of liquid is rotating by an external torque, the rise in pressure head of the rotating liquid takes place.

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The rise in pressure head at any point of the rotating liquid is proportional to square of tangential velocity of the liquid at that point (i.e rise in pressure head = $\frac{v^2}{2g} = \frac{\omega^2 r^2}{2g}$

$V = \omega r$

where,

V = tangential velocity

ω = angular velocity

r = radius of wheel.

At the outlet of impeller, where radius is more, the rise in pressure head will be more & the liquid will be discharged at the outlet with a high pressure head. Due to this high pressure head, the liquid can be lifted to high level.

Vortex flow :- It is the flow of a liquid along a curved path or the flow of a rotating mass of liquid.

Parts of Centrifugal Pump :-

i) Impeller ii) Casing iii) Suction pipe with a foot valve & strainer. iv) Delivery pipe.

Impeller :-

The rotating part of a centrifugal pump is called impeller. It consist of a series of backward

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curved vanes. The impeller is mounted on a shaft which is connected to the shaft of an electric motor.

ii) Casing :-

It is an air-tight passage surrounding the impeller & it is designed in such a way that the kinetic energy of water discharged at the outlet of the impeller is converted into pressure energy before water leaves the casing & enters the delivery pipe.

Following three types of casings are commonly used :-

a) Volute casing
b) Vortex casing
c) Casing with guide blades

a) Volute Casing :-

Volute casing surrounds the impeller. It is spiral type in which area of flow gradually increases. The increase in area of flow decreases the velocity of flow. The decrease in velocity increases the pressure of water flowing through the casing. It has been observed that in case of volute casing, the efficiency of pump increases slightly as a large amount of energy is lost due

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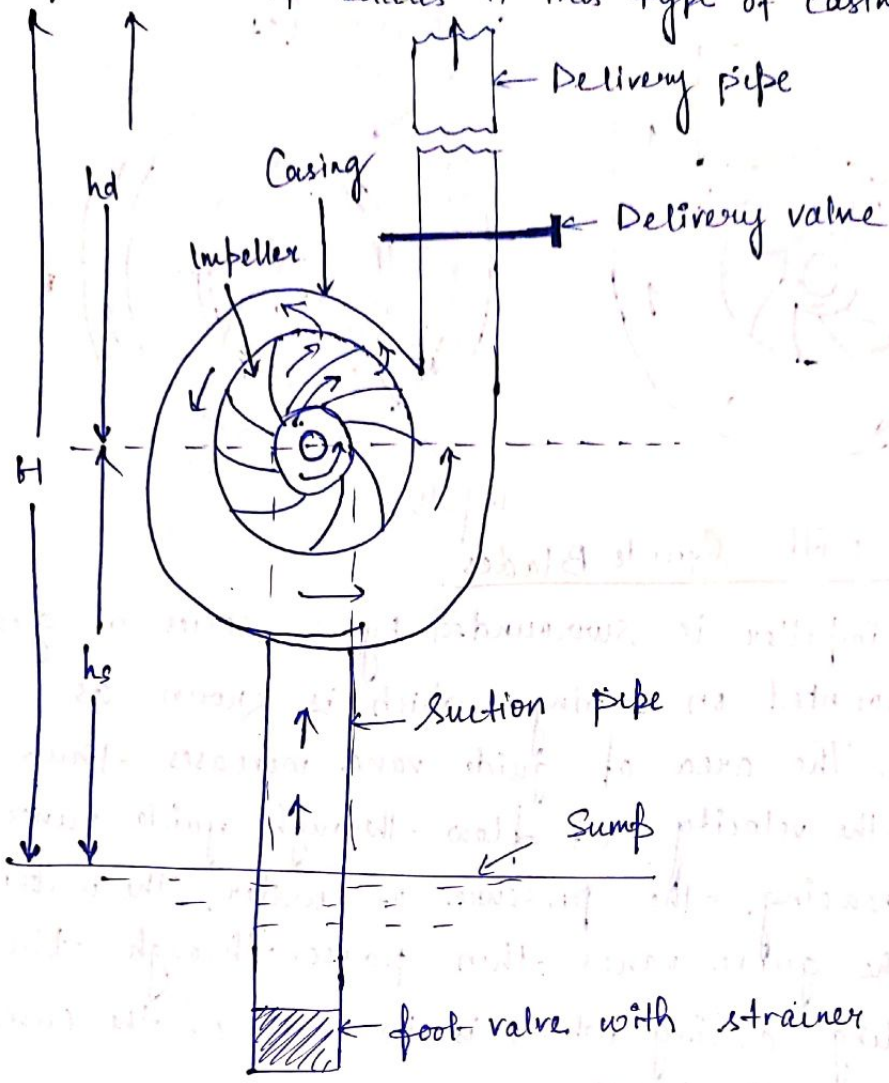
to formation of eddies in this type of casing.



b) Vortex Casing :-

If a circular chamber is introduced between the casing & ~~tim~~ the impeller, or the casing is known as vortex casing. With the circular chamber, the loss of energy due to ~~fto~~ formation of eddies is reduced. Thus it increases the efficiency of pump than the efficiency when only volute casing is provided.

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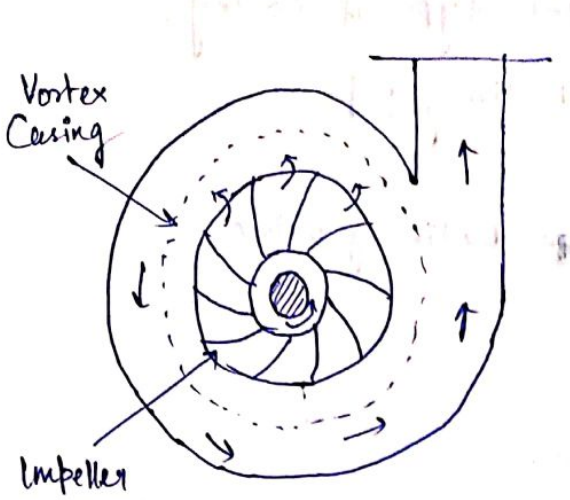

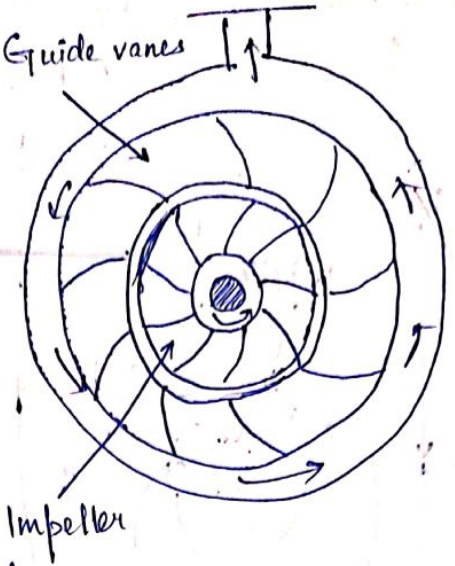

ii) Casing with Guide Blades

The impeller is surrounded by a series of guide blades mounted on a ring which is known as diffuser. The area of guide vanes increases, thus reducing the velocity of flow through guide vanes and increasing the pressure of water. The water from the guide vanes then passes through the surrounding casing which is in most of the cases concentric with impeller.

iii) Suction Pipe with a foot valve & Strainer :-

→ The pipe whose one end is connected to the inlet of the pump & other end dips into water in a sump is known as suction pipe.

→ A foot valve is a non-return valve or one-way type of valve is fitted at the lower end of the suction pipe. The foot valve opens only in the upward direction. A strainer is also fitted at the

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lower end of the suction pipe.

iv) Delivery Pipe

A pipe whose one end is connected to the outlet of the pump & other end delivers the water at a required ~~length~~ height is known as delivery pipe.

Suction Head (h_s)

It is the vertical height of the center line of the centrifugal pump above the water surface in the tank or pump from which water is to be lifted. This height is also called suction lift & denoted by h_s.

Delivery Head (h_d)

The vertical distance between the center line of the pump & the water surface in the tank to which water is delivered is known as delivery head. It is denoted as h_d.

Static Head (H_s)

The sum of ~~static~~ suction head & delivery head is known as static head. It is represented as H_s.

$$H_s = h_s + h_d.$$

Manometric Head (H_m)

It is the head against which a centrifugal pump has to work.

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Efficiencies of a Centrifugal Pump :-

The power transmitted from shaft of the electric motor to the shaft of the pump & then to the impeller. From impeller, the power is given to the water. Thus power is decreasing from shaft of the pump to the impeller & then to water.

Following are important efficiencies :-

a) Manometric efficiency, η_{man}

b) Mechanical efficiency, η_m

c) Overall efficiency, η_o

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.

$$\eta_{man} = \frac{\text{Manometric Head}}{\text{Head imparted by impeller to water}}$$

$$\boxed{\eta_{man} = \frac{g H_m}{V_{w_2} u_2}}$$

Mechanical Efficiency (η_m)

The power at the shaft of the centrifugal pump is more than power available at impeller of the pump. The ratio of the power available at the impeller to the power at the shaft of the

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Centrifugal pump is known as mechanical efficiency.

$$\eta_m = \frac{\text{Power at the impeller}}{\text{Power at the shaft}}$$

$$\eta_m = \frac{\frac{W}{g}\left(\frac{V_{w_2} u_2}{1000}\right)}{S.P}$$

where,

W = wt. of water

u_2 = Tangential vel. of impeller at outlet

$= \frac{\pi D_2 N}{60}$

D_2 = dia. of impeller at outlet

$S.P$ = shaft power.

Overall Efficiency (η_o)

It is the ratio of power output of the pump to the power input to the pump.

$$\eta_o = \eta_{man} \times \eta_m$$

Discharge of a Centrifugal Pump.

Centrifugal pump lifts water from a lower level to a higher level.

D = diameter of impeller at inlet,

V_f = velocity of flow at inlet,

b = width of impeller at inlet,

D_1, b_1, V_{f_1} = corresponding value at outlet.

Discharge, $Q = \pi D b V_f = \pi D_1 b_1 V_{f_1}$

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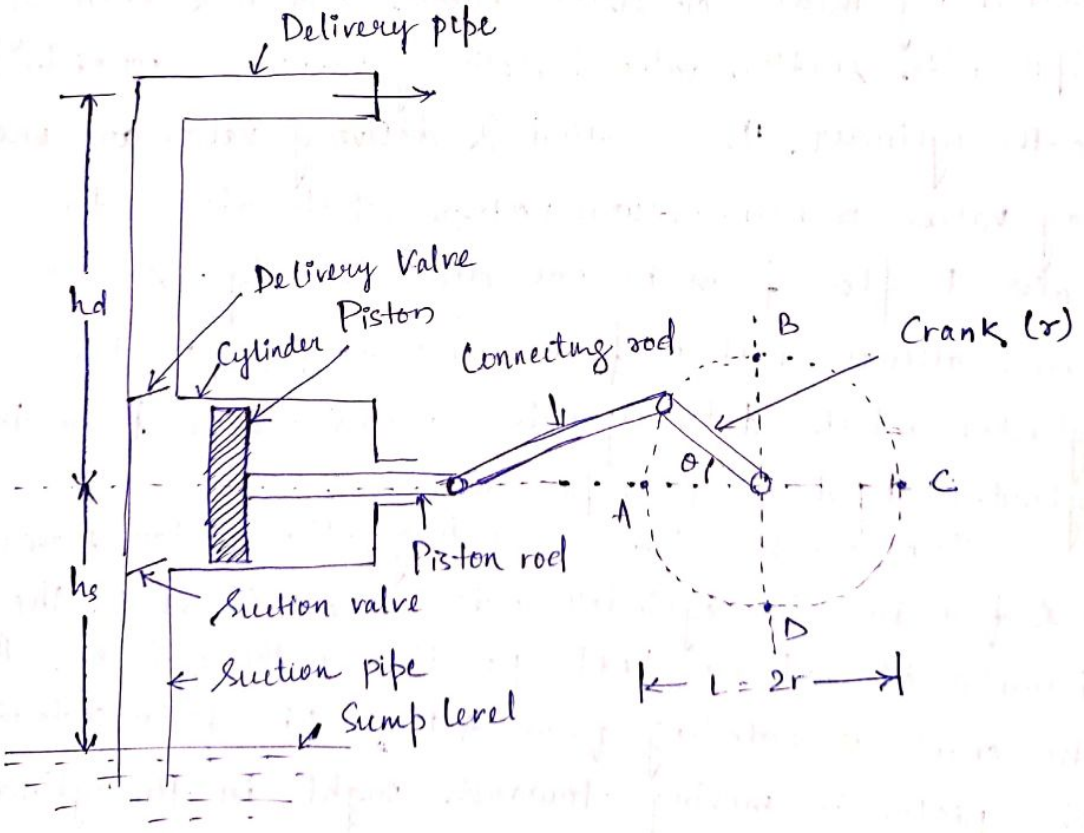
3.3 Reciprocating Pumps.

When the mechanical energy is converted into hydraulic energy (or pressure energy) by sucking the liquid into a cylinder in which a piston is reciprocating (moving backward & forward) which exerts the thrust on the liquid & increases its hydraulic energy (pressure energy), the pump is known as reciprocating pump.

Main parts of reciprocating pump :-



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i) A cylinder with piston, piston rod, connecting rod & crank.
ii) Suction pipe
iii) Delivery pipe
iv) Suction valve
v) Delivery valve.

Working of a Reciprocating Pump :-

A piston moves forwards & backward in a close fitted cylinder. The movement of the piston is obtained by connecting the piston rod to crank by means of a connecting rod. The crank is rotated by means of electric motor. Suction & delivery pipes with suction valve & delivery valve are connected to the cylinder. The suction & delivery valves are one way valve or non-return valves, which allow the water to flow in one direction only. Suction valve allows water from suction pipe to the cylinder which delivery valve allows water from the cylinder to delivery pipe only.

When crank starts rotating, the piston moves to & fro in the cylinder. when crank is at A, the piston is at extreme left position in the cylinder. As the crank is rotating from A to C (i.e from 0° to 180°) the piston is moving towards right in the cylinder. The movement of piston towards right create a partial vaccum in the cylinder. But on the surface

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of the liquid in the sump atmospheric pressure is acting, which is more than pressure inside the cylinder. Thus, the liquid is forced in the suction pipe from the sump. This liquid opens the suction valve & enters the cylinder.

When crank is rotating from C to A (i.e from $\theta = 180°$ to $\theta = 360°$), the piston from extreme right position starts moving towards left in the cylinder. The movement of the piston towards left increases the pressure of the liquid inside the cylinder more than atmospheric pressure. Hence suction valve closes & delivery valve opens. The liquid is forced into the delivery pipe & is raised to a required height.

Discharge of a Reciprocating Pump.

In a single acting reciprocating pump (i.e A pump, in which the water is acting on one side of the piston only).

let, L = Length of the stroke or piston in meters

A = cross-sectional area of the piston in m^2.

N = no. of revolutions per minute of the Crank.

r = radius of crank

Discharge of pump per second, Q = Discharge in one revolution × No. of revolution per second

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$= A \times L \times \frac{N}{60}$

$Q = \frac{ALN}{60}$ m^3/s

If the pump is a double acting reciprocating pump the discharge is taken to be double the discharge than that of a single acting pump. This is due to the reason that, in a double acting pump, the water is pumped, the water sucked on one side of the piston & delivery from the other side during the same stroke. These two processes are reversed during return stroke. Therefore the discharge of a double acting reciprocating pump,

$Q = \frac{2ALN}{60}$ m^3/s

As a matter of fact, the discharge of a double acting reciprocating pump is not double of a single acting pump, because some area is blocked by the connecting rod. However this area being small as compared to the discharge is neglected.

Slip of a Pump

When the actual discharge is less than the theoretical discharge. This difference between theoretical and actual discharge is known as slip of pump.

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Negative Slip of Pump

When actual discharge of a reciprocating pump is more than theoretical discharge, the coefficient of discharge will be more than unity, the corresponding slip is known as negative slip of pump.

This happens when the suction pipe is long & delivery pipe is short and pump is running at high speeds. This causes the delivery valve to open before complete suction stroke & some water is pushed into the delivery pipe, before the piston commences its delivery stroke.

$$\text{Slip} = Q_{th} - Q_{act}$$

$$\text{Percentage slip} = \frac{Q_{th} - Q_{act}}{Q_{th}} \times 100 = \left(1 - \frac{Q_{act}}{Q_{th}}\right) \times 100$$

$$= (1 - C_d) \times 100$$

where,

C_d = coefficient of discharge.

Classification of Reciprocating Pump :-

The reciprocating pumps may be classified as :—

a) According to the water bearing being in act contact wide one side or both side of piston.

b) According to the number of cylinders provided.

c) According to the existance of air vessels.

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According to the contact of water :-

i) Single acting pump.

ii) Double acting pump.

According to no. of cylinders provided :-

i) Single cylinder pump.

ii) Double cylinder pump.

According to existance of air vessel :-

i) With air vessel

ii) Without air vessel.

Air Vessel :-

It is a closed chamber containing compressed air in the top postion & liquid at the bottom of chamber. There is opening at base of chamber through which the liquid may flow into the vessel or out from the vessel.

Work Done in Single Acting Reciprocating Pump :-

Power required to drive the pump, in KW.

$$P = \frac{\text{Work done per sec}}{1000} = \frac{\rho g \times ALN \times (h_s + h_d)}{60 \times 1000}$$

$$= \frac{\rho g \times ALN\,(h_s + h_d)}{60{,}000} \text{ KW}$$

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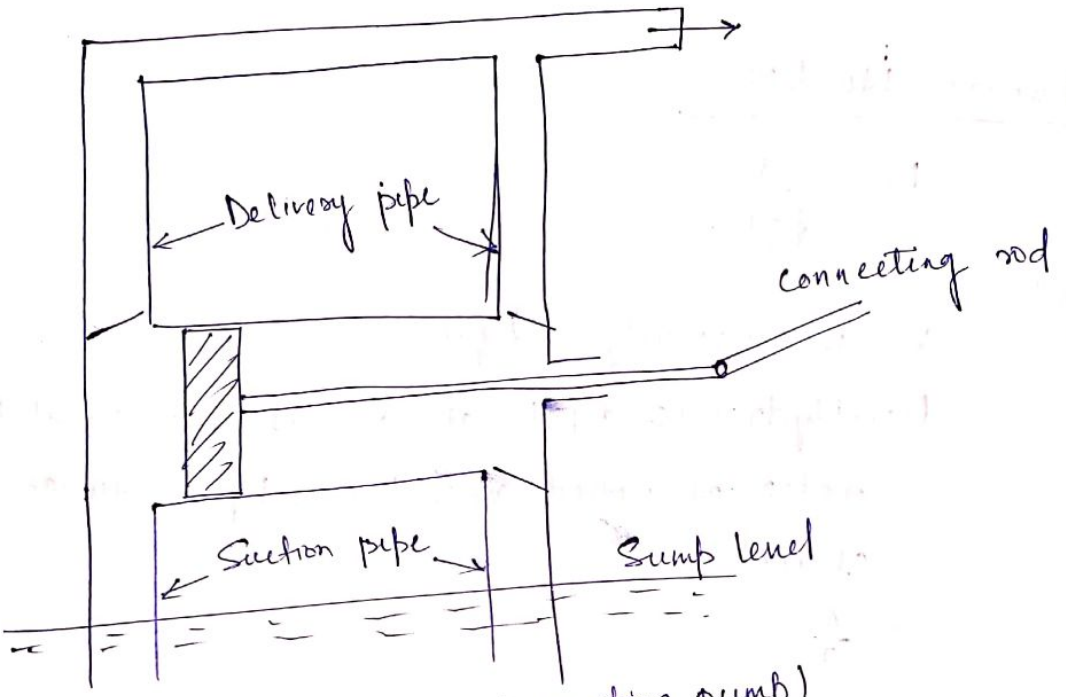
Work done by Double Acting Reciprocating Pump

Power required to drive the double-acting pump,

$$P = \frac{\text{Work done per sec}}{1000}$$

$$= 2\rho g \times \frac{ALN}{60} \times \frac{(h_s + h_d)}{1000}$$

$$= \frac{2\rho g \times ALN \times (h_s + h_d)}{60000}$$



(Double acting reciprocating pump)

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Froude's Number

$$Fe = \frac{V}{\sqrt{gD}}$$

where,

V = mean velocity of flow

D = hydraulic depth of channel & is equal to ratio of wetted area to the top width of channel

$$= \frac{A}{T}$$

→ Flow is said to be sub-critical if froude's Number (Fe) is less than 1.0.

→ Flow is said to be super-critical (shooting) if Fe > 1.0.

→ Flow is said to be critical if Fe = 1.

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Part - B

Irrigation Engg.

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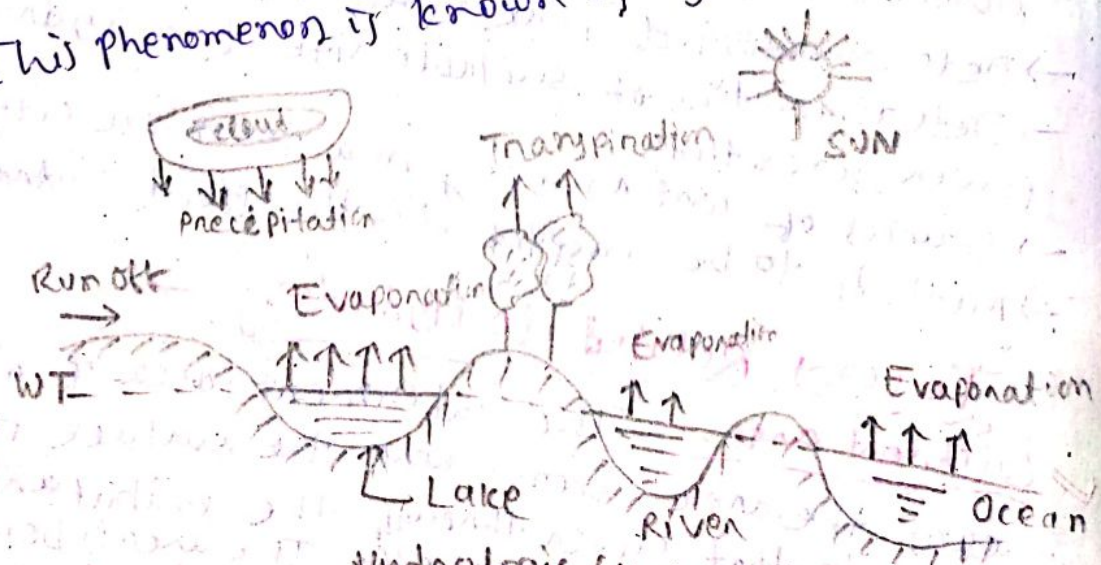
1.1 - Hydrologic cycle:-

The water ~~from~~ of the universe always changes from one state to other under the effect of the Sun. The water from the surface sources like lake, rivers, ocean, etc. converts to vapour by evaporation due to solar heat. The vapour goes on accumulating continously in the atmosphere. This vapour is again condensed due to the sudden fall of temperature & pressure. Thus clouds are formed. These clouds again causes the precipitation.

Some of vapour is converted to ice at the peak of the mountains. The ice again melts in summer & flows of rivers to meet the sea or ocean. These process of evaporation, precipitation & melting of ice go on continously like an endless chain & thus a balance is maintained in the atmosphere.

This phenomenon is known as hydrologic cycle.



Hydrologic cycle.

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1.2 Rain-fall or Precipitation :-

Defⁿ :- From the principle of hydrolic cycle we have seen that water goes on evaporation continuously from the water surface on earth (e.g. river, lake, sea, ocean, etc.) by the effect of sun. The water vapour goes on collecting in the atmosphere upto a certain limit. When this limit exceeds & the temperature & pressure fall to a certain value, the water vapour will get condensed & thereby cloud is formed. Ultimately droplets are formed & returned to earth in the form of rain, snowfall, hail etc. is known as Precipitation.

Types of Precipitation :-

Depending upon the various atmospheric conditions the precipitation may be of following types :-

1. Cyclonic Precipitation :-

⇒ This type of precipitation is caused by the difference of pressure within the air mass on the surface of the earth. If low pressure is generated at some place the warm moist air from the surrounding area rushes to the zone of low pressure with violent force. The warm moist air rises up with whirling motion & get condensed at higher altitude & ultimately heavy rainfall occurs.

→ This may be of two types :-

(a) Frontal Precipitation :-

When the moving warm moist air mass is obstructed by the zone of cold air mass, the warm moist air rises up to higher altitude where it gets condensed & heavy rainfall occurs. This is known as frontal precipitation.

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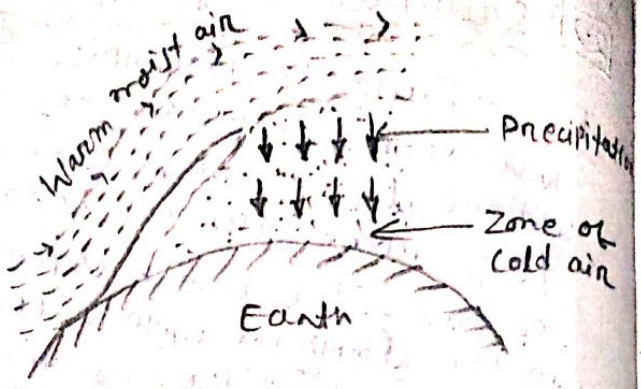
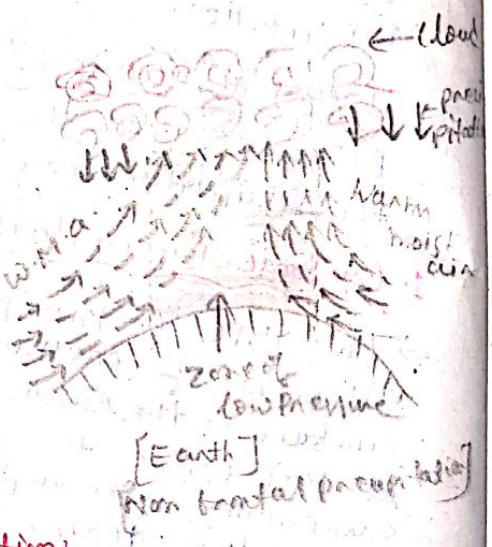


fig.

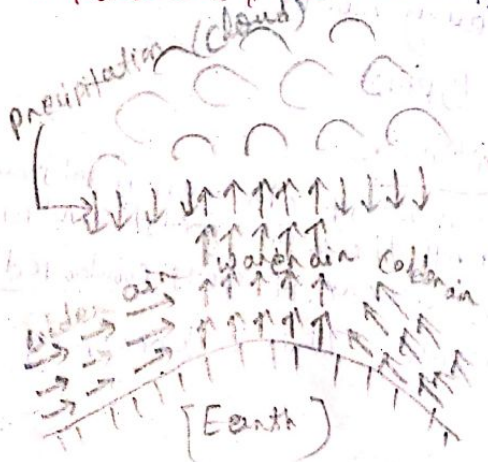
(b) Non-Frontal Precipitation:—

When the warm moist air mass rushes to the Zone of low pressure area, a pocket is formed and the warm moist air rises up like a chimney towards higher altitude.

→ At higher altitude this air mass gets condensed & heavy rainfall occurs. This is known as non frontal precipitation.



2. Convective Precipitation:—



→ In tropical countries when on a particular hot day, the ground surface gets heated un-equally the warm air is lifted to high altitude & the cooler air takes its place with higher velocity.

→ Thus the warm moist air when obstructed by some mass condensed at the high altitude causes heavy rainfall. This is known as convective precipitation.

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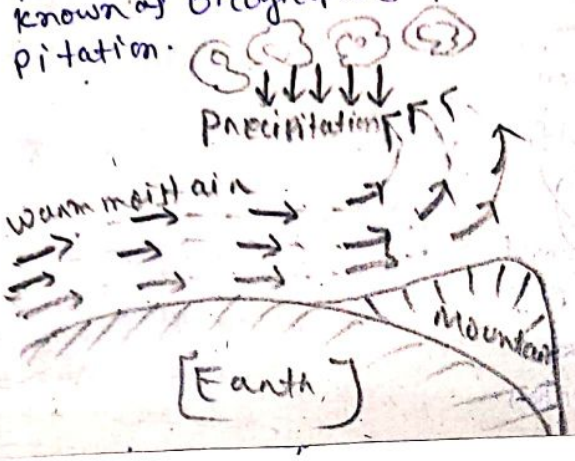
(3) Orographic Precipitation :—

The moving warm moist air when obstructed by some mountain rises up to a high altitude.
→ Then it gets condensed and precipitation occurs. This is known as orographic precipitation.



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Intensity of Rainfall :—

→ If intensity of rainfall is more, the corresponding run-off will be more. → Again if the intensity of rainfall is low, the corresponding run-off will also be low.

* → Intensity of Rainfall is a measure of the amount of rain that falls over time.

→ The intensity of rain is measured in the height of the water layer covering the ground in a period of time.

* → It means that if the rain stays where it falls, it would form a layer of a certain height

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HYETOGRAPH :-

The graphical representation of rainfall and runoff is known as hyetograph.

→ The graph is represented with intensity of rainfall in (cm/hr) as ordinate and time (in hrs) as abscissa.

→ The infiltration capacity curve is drawn on this graph to show the amount of infiltration loss (shown by dotted portion).

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→ The upper portion indicates the effective rainfall (shown by hatched lines). The centroid of the effective rainfall is ascertained on the graph for the determination of total run-off at any specified period.

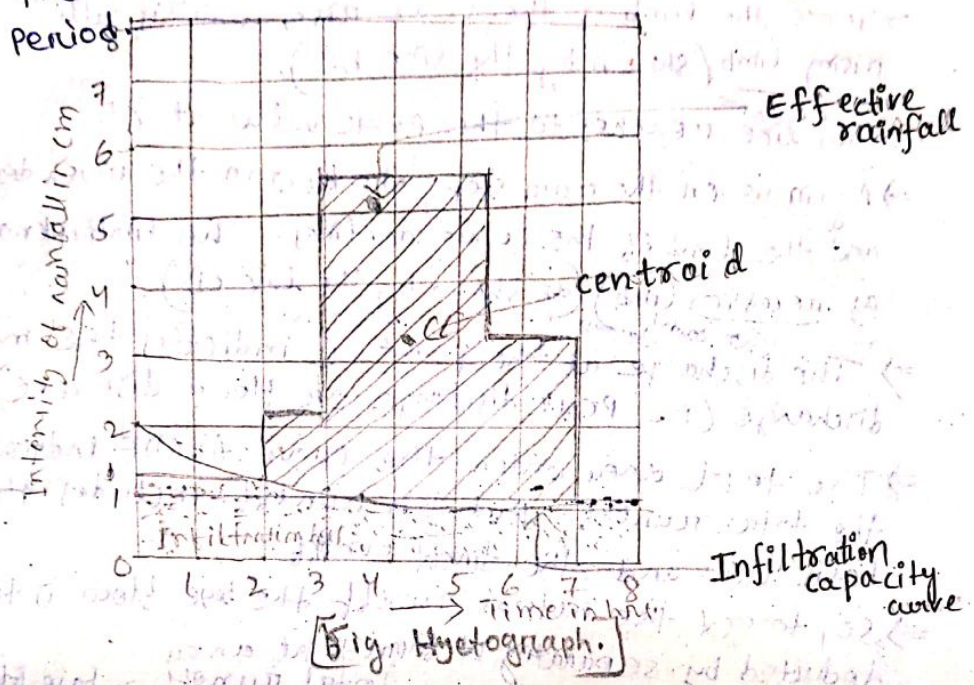

Fig. Hyetograph.

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1.3 MEASUREMENT OF RAINFALL

→ The instrument which is used to measure the amount of rainfall is known as raingauge.

⇒ The principle of raingauge is that the amount of rainfall is a small area will represent the amount of rainfall in a large area provided the meteorological characteristics of both small & large area are similar.

→ The raingauges are of following types:—

(1) Non-recording type Raingauge:-

⇒ Simson's raingauge is a non-recording type of raingauge which is most commonly used.

⇒ It consists of metal casing of diameter 127mm which is set on a concrete boundation.

⇒ A glass bottle of capacity about 100mm of rainfall is placed within the casing.

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⇒ A funnel with brass rim is placed on the top of the bottle.

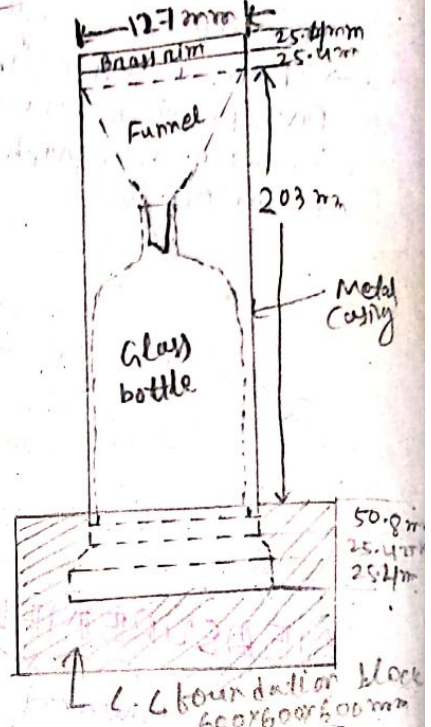
⇒ The rainfall is recorded at every 24 hours.

⇒ Generally, the measurement is taken at 8.30 AM. everyday.

⇒ In case of heavy rainfall the measurement should be taken 2 or 3 times daily so that bottle does not overflow.

⇒ To measure the amount of rainfall the glass bottle is taken off & 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 replaced with a new bottle of same capacity.

[Simson's raingauge]



2. Recording type 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.

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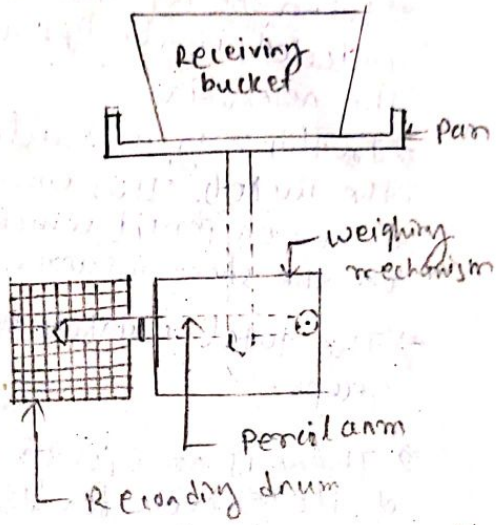
(a) Weighing Bucket Raingauge:-

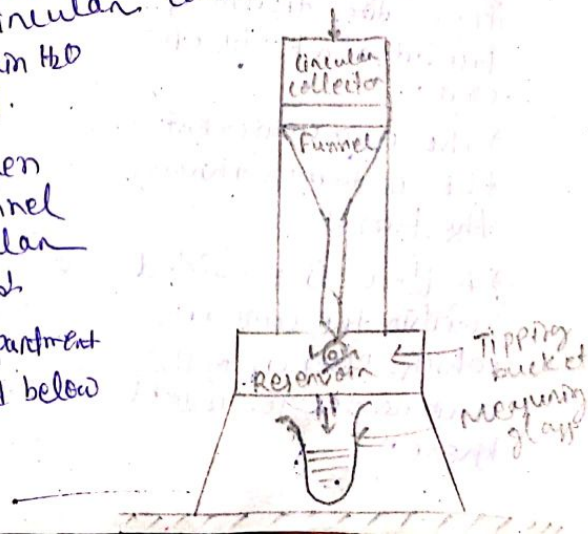
→ This type of raingauge consists of a receiving bucket which is placed on pan.
→ The pan is again fitted with some weighing mechanism.
→ A pencil arm is pivoted with the weighing mechanism in such a way that the movement of the bucket can be traced by a pencil on the moving recording drum.
→ So, when 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 ~~vs~~ time & hence it is some time called Integrating raingauge.
→ The graph is known as the mass curve of rainfall.



Weighing bucket raingauge

(b) Tipping Bucket Raingauge:-

⇒ It consist of a circular collector of diameter 30cm in which the rain H2O is initially collected.
⇒ The rainwater then passes through a funnel fitted to the circular collector & gets collected in two compartment tipping buckets pivoted below the funnel.



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⇒ When 0.25mm rain water is collected in one bucket then it tips and discharges 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 rainwater is collected, it also tips and discharges 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.

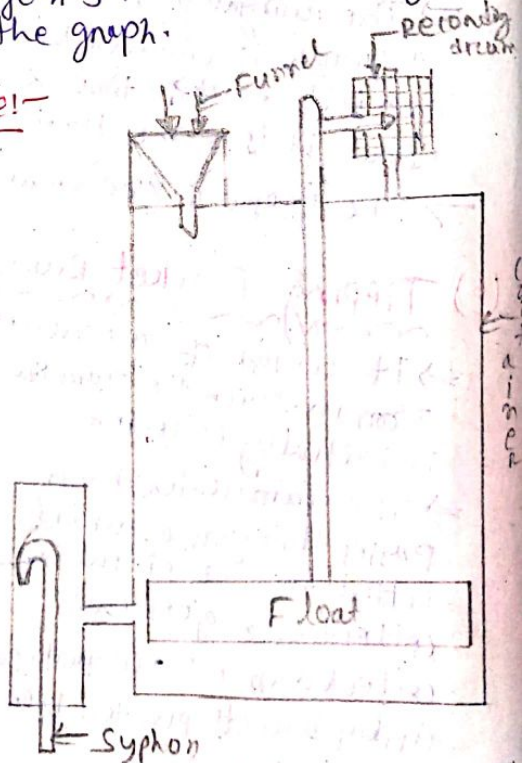
⇒ Sometimes a measuring glass is provided to verify the results shown by the graph.

(c) Float type Raingauge:-

⇒ In this type of rain gauge, a funnel is provided at one end of a rectangular container & 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 as the rain water gets collected there.



Float type raingauge

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→ The float consists of a rod which contains a pen arm 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 & the container goes on emptying gradually.

1.4 Concept of Catchment area:-

→ Catchment area of a river means the area from where the surface runoff flows to that river through the stream, spring etc. the line that gives a boundary to the area is called as watershed line.

→ There are two different shapes of catchment area.

i) Fan shaped:-

→ If the catchment area is large and fan shaped, the runoff will be more.

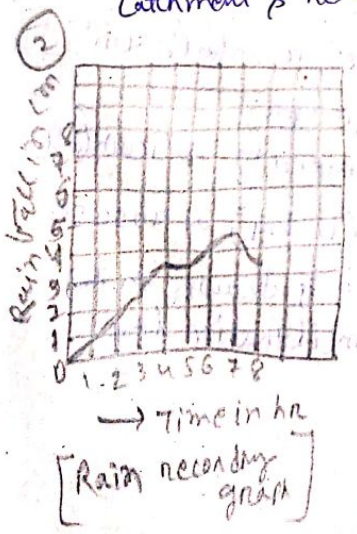
→ If the catchment area is large like the shape of a fan, then it is called as fan shaped catchment.



[big. Fan shaped]

(ii) Fern shaped:-

If catchment area is small & narrow then it is known as fern shaped catchment & here the runoff is low.



[Rain recording graph]



[Fern shaped]

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Estimation of flood discharge :- (Peak flow)

The peak flow of flood discharge may be estimated by the methods :-

(a) Dicken's Formula :-

$$Q = C \times A^{3/4}$$

where, Q = Discharge in cumec (m^3/sec)

A = catchment area in km^2

C = A constant depending upon the factors affecting the flood discharge

= 11.5 (avg value of C is considered as 11.5)

(b) Ryve's Formula :-

$$Q = C \times A^{2/3}$$

where Q = Discharge in cumec

A = catchment area in km^2

C = A (constant)

(Avg value of C is considered as 6.8)

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Run-off :- When it rains, some portion of rain water infiltrates into the soil, some is intercepted by vegetation, some evaporates & the remaining portion flows over the ground surface to join the rivers, streams, lakes etc. This portion of water which flows over the ground surface is known as surface run off or run-off.

→ run-off is also designated by rainfall excess or effective rainfall.

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2.0 Water Requirement of Crops

2.1 Definition of Irrigation :-

Defⁿ:- Irrigation may be defined as the science of artificial application of water to the land, in accordance with the 'Crop requirements' throughout the 'crop period' for full-fledged nourishment of the crops.

OR

"Irrigation may be defined as the process of artificially supplying water to soil for rising crops."

→ It is practically a science of planning & designing a water supply system for the agricultural land to protect the crops from bad effect of drought or low rainfall.

→ It includes the construction of weirs, dams, barrages & canal systems for the regular supply of water to the culturable land.

Necessity of irrigation :-

→ Throughout the crop period adequate quantities of water is required near the root zone of the plants for their growth.

→ ~~At times~~ During the crop period the rainfall may not be adequate to fulfil the water requirement.

→ The intensity of rainfall is practically uncertain & beyond the control of human power & it may not be well distributed throughout the crop season.

→ So irrigation becomes absolutely necessary to fulfil the water requirement of crops.

→ Following are some factors by which irrigation is necessary.

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(a) Insufficient rainfall:—
When the seasonal rainfall is less than the min^m requirement for the satisfactory growth of crops, the irrigation system is essential.

(b) Uneven distribution of Rainfall:—
When the rainfall is not evenly distributed during the crop period or throughout the culturable area, the irrigation is extremely necessary.

(c) Improvement of (Perennial) crops:—
Some perennial crops like sugar-cane, cotton, etc. require water throughout the major part of the year.
→ But the rainfall may fulfil the water requirement in rainy season only. So for the remaining part of the year, irrigation becomes necessary.

(d) Development of Agriculture in Desert Area:—
In desert area where the rainfall is very scanty, irrigation is required for the development of agriculture.

BENEFITS OF IRRIGATION:—

(a) Yield of crops:— In the period of low rainfall or drought The yield of crop may be increased by the irrigation system.

(b) Protection from famine:—
The food production of a country can be improved by ensuring the growth of crops by availing the irrigation facilities. This helps a country to prevent famine situation.

(c) Improvement of cash crops:—
Irrigation helps to improve the cultivation of cash crops like, vegetables, fruits, tobacco, etc.

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(d) Prosperity of Farmers:-

→ When the supply of irrigation water is assured, the farmers can grow two or more crops in a year on the same land. Thus the farmers may earn more money and improve their living standard.

(e) Hydroelectric power Generation:-

→ ~~It~~ In some river valley project, multipurpose reservoirs are formed by constructing high dams where hydroelectric power may be generated along with the irrigation system.

(f) General Communication:-

→ The inspection road along the canal banks may serve as a communication link with the otherwise remote villages.

(g) Water Supply:- The irrigation canals may be the source of water supply for domestic and industrial purposes.

(h) Development of fishery:-

→ The reservoir and the canals can be utilised for the development of fisher projects.

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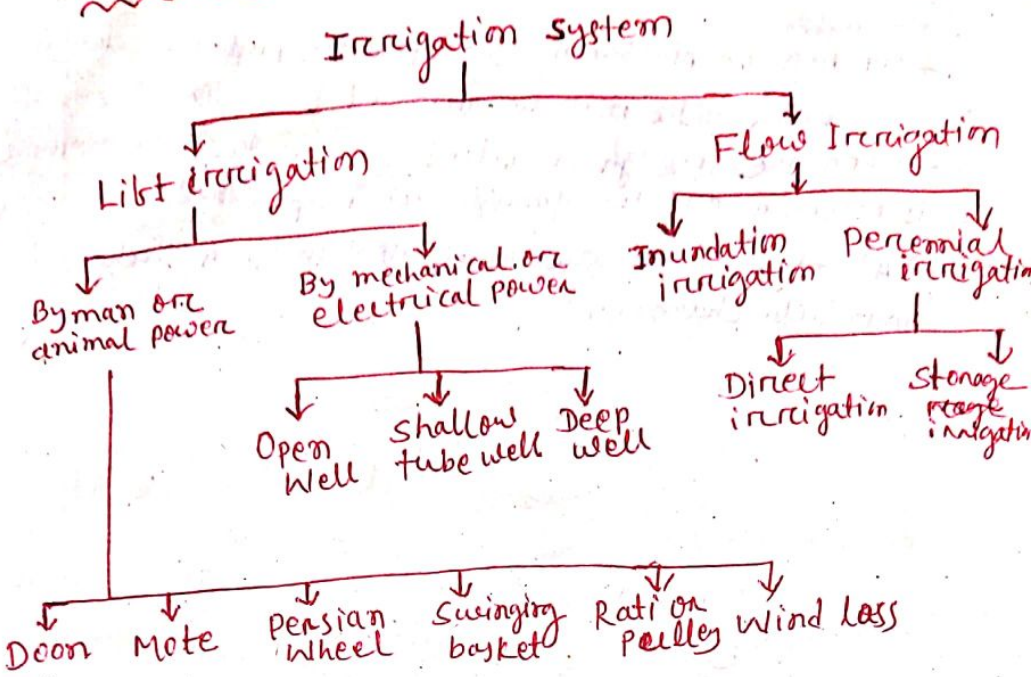
TYPES OF IRRIGATION



Lift Irrigation :-

When water is lifted from surface sources or underground sources by man or animal power, mechanical or electrical power and directly supplied to the agricultural land, then it is known as lift irrigation.

(a) Lifting water by man or animal power :-

When mechanical or electrical powers are not available in villages or the economic condition of the farmers is not good enough to afford this expensive method, then the water is lifted by following methods :-

(i) Doon
(ii) Mote
(iii) Persian wheel
(iv) Swinging Bucket
(v) Dhenkli
(vi) Rati or Pulley
(vii) Wind Lass.

* DOON :-

Channel/drain

→ It consists of a trough made of wooden or galvanised tin sheet. The trough is open at one end and closed at other.
→ It is anchored over a fulcrum.
→ The closed end is tied to a Lever with a rope.
→ The other end of the lever carries a counter weight.

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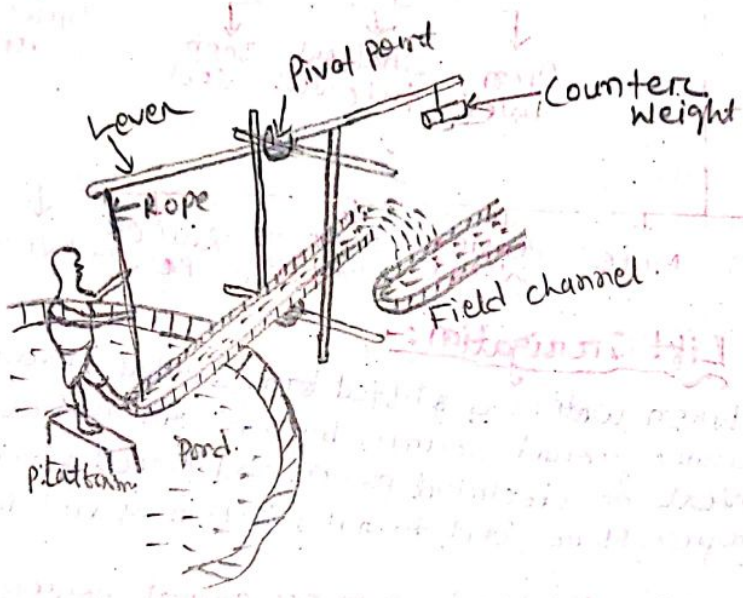
→ The lever is pivoted at middle point. ~~and a weight is suspended on other~~

→ standing on platform, the operator pushes the trough downwards until it is ~~sa~~ immerged in water.

→ when the trough is filled with water, the pressure is released and the trough rises up-wards.

→ At this position water of ~~t~~ trough is discharged into field channel.



* MOTE :-

→ This method is suitable for lifting water from deep open well.

(i) It consists of two pulleys.

(ii) A lather container known as (mote) is tied with the ropes at both ends.

(iii) One rope ~~passes~~ tied with large pulley and other with small one.

(iv) The ropes passes over pulleys and are tied with bullocks & who pulls it.

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(v) Bullocks move on sloping platform, known as ramp.
(vi) When bullocks are near the well (at A), the mote gets immersed in water and when bullock moves away from well, than mote is raised upward.
(vii) When bullock reaches B, the mote becomes inclined and water is discharged
(viii) Thus due to forward & backward motion of bullocks, water is lifted and discharge into field channel.

③ PERSIAN WHEEL:-

→ It consists of vertical axle which carries a toothed wheel.
→ This wheel is geared with a horizontal shaft which carries a drum.
→ An endless chain passes over the drum and a no. of G.I. buckets are properly fixed on the chain.
→ The vertical axle carries a yoke with which, two bullocks are tied. The bullocks move in circular path and thus the tooth wheel rotates.

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→ With this the drum also rotates & buckets move upward and downward with the chain.

→ In downward motion, bucket becomes inverted position and at the end point it gets immersed in water. Then, in upward motion bucket becomes erected and raised full of water.

→ During the movement of the chain, the buckets discharge water in a trough near the top of drum.

(4) Swinging Bayket/Bucket :-

→ In this method water is lifted by a swinging bucket with the help of pair of ropes holded by two persons standing on opposite sides.

→ In downward motion, bucket is immerged in water by loosening the ropes & in upward motion, the two person exert maximum pressure on rope to raise the bucket & to discharge water in field channel.

(5) DHENKLI :-

→ It is used for lifting water from the dug well.

→ It consists of a lever pivoted on a vertical post. One end of the lever carries a weight and the other carries a bucket suspended by rope.

→ When operator exerts downward thrust, the bucket goes inside and submerged &

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when operater releases the pressure, the bucket comes up automatically because of the counter weight place on opposite side of lever.

→ When the bucket comes above the ground level, the operator delivers the water in the field channel.

(6) Rati or Pulley :-

→ It consists of a inclined post carrying a pulley. Over the pulley rope passes which carries a bucket at one end.

→ The other end of rope is held by the operator who pulls the bucket up & down.

→ When operator releases the rope, bucket goes down & when operater pulls it, bucket comes up.

→ The water is then discharged to the field channel.

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(7) Wind Lift :-

→ It is suitable for lifting water from deep open wells.

→ In this method wooded frame is fitted with a wind lift.

→ The wheel consist of four projecting arm with which the operator can rotate the wheel clockwise & A.C.W.

→ A rope is ~~coiled~~ coiled with cylinder. The free end of rope carries a bucket.

→ In clockwise rotation of wheel, the rope is coiled with cylinder & bucket is raised.

→ In clockwise rotation, rope is uncoiled & bucket goes down inside the well & immersed in water.

→ In A.C.W. rotation the rope is ~~re~~ uncoiled & the bucket gets lowered & gets immersed in water.

→ In C.W. rotation of the wheel, the rope is coiled with cylinder & the bucket get raised.

→ When the bucket comes to the top of the well, the operator discharges the water into the field ~~that~~ channel.

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(b) Lifting water by electrical or mechanical method:-

→ When mechanical or electrical power is available in villages or the farmer can afford expenditure for the same, then, the underground water is lifted by pumps and directly supplied to agricultural land.

→ I can be done by :-

(i) Open well
(ii) Shallow tube well
(iii) Deep tube well

Advantages of lift irrigation:-

→ The farmers can supply water to their fields according to the requirement.

→ The water table is lowered when water is lifted from the wells thereby reducing chances of water logging in the area.

→ As water is supplied directly to the field, there is no water loss due to conveyance.

→ Initial cost is low as there is no necessity of constructing hydraulic structures.

→ As loss of water is low, duty of water is high.

→ Maintenance cost is low.

→ More than one crop can be grown in a year on the same land.

Disadvantages of lift irrigation:-

→ In summer the surface water may be dried up & the water table may go down below the suction head. Hence lift irrigation from the surface source & from the shallow tube well may fail in summer.

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(ii) If the lifting mechanism fails due to mechanical or electrical failure, then water cannot be supplied until the mechanism is restored.

(iii) The lift irrigation with the yield of crop is not cost effective.

(iv) The well water has no silt content. The yield of crop therefore depends on chemical fertilisers, which is costly.

2. FLOW IRRIGATION

⇒ When water flows under gravitional pull through the artificial canal towards the agricultural land, then it is termed as flow irrigation.

⇒ In this irrigation system, the ~~source~~ head of the canal should always be at higher elevation than the land to be irrigated.

⇒ Flow irrigation system is of following types :-

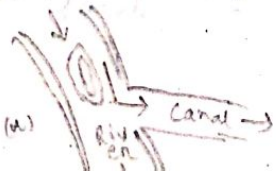
a) Inundation irrigation system
b) Perennial system of irrigation
   (i) Direct irrigation system
   (ii) storage irrigation system.

(a) Inundation Irrigation System :-

→ The irrigation system in which the canal is excavated from the bank of inundation river is known as inundation irrigation system.

(Inundation river is the river which over flows in rainy season but nearly dries up in summer)

→ In this system, water flows to the agricultural land in rainy season only.



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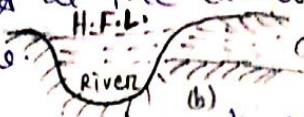
→ No regulator is needed at the canal head to control the water flow.



→ This irrigation system depends completely on water level of river.

→ Over irrigation may happen as this irrigation system donot have regulator & it is possible resulting in damaging the crops.



(b) Perennial System of Irrigation:-

→ In this system, a weir or a barrage is constructed across the perennial river (i.e, the river which flows throughout the year in its full capacity) to raise the water level on the upstream side

or a

→ The irrigation system in which the canal is excavated from the bank of perenial river is known as perennial irrigation system.
(perennial river is the river which flows through out the year in it's full capacity)

→ The main canal is constructed on either or both the banks of the river.

→ In this system, water flows to the agricultural land throughout the year.

→ A weir or a barrage is constructed across the river to raise the water level on upstream side or a dam is constructed across the river to form storage reservoir.

Regulator is also constructed at the head of canal to control the flow of water through the canal of the agricultural land.

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→ This irrigation system depends on control of regulation.

→ Due to regulation of water flow, there is no chance of over-irrigation.

→ Perennial irrigation system is of two type
(i) Direct irrigation system
(ii) Storage irrigation system.

(i) Direct irrigation system:—

→ In this system, a weir is generally constructed across a perennial river to raise the water level on upstream side.

→ A weir is an impervious barrier which constructed across a river to raise the water level on the upstream side.

→ Here, water level is raised up to the required height and the surplus water is allowed to flow over the weir.

→ It is also constructed across inundation river.

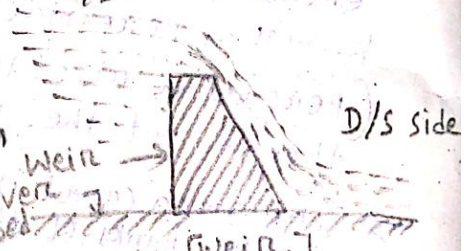

fig. [Weir]

D/s → Down stream
U/s → Up stream

⇒ Sometimes a barrage is constructed, in place of weir, to regulate the water level on upstream side.

→ When adjustable gates are installed over a weir to maintain the water surface at different levels at different times, then it is known as barrage.

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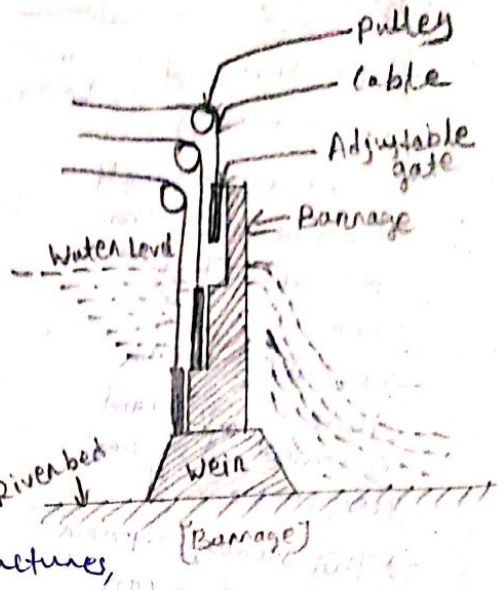
→ The water level is adjusted by operating the adjustable gates or shutters.

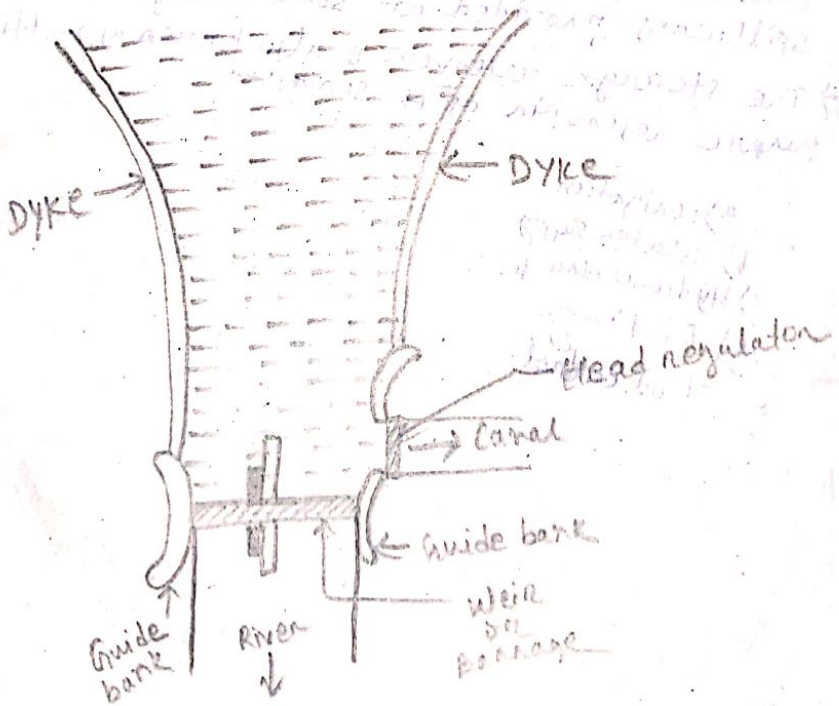
→ The gates are placed at different tiers and these are operated by cables from cabin.

→ The gates are supported on piers at both ends.

→ These hydraulic structures, when constructed across a river, then it is called as **Diversion Head work.**



[Fig. Barrage]



[Fig. Diversion Head Work]

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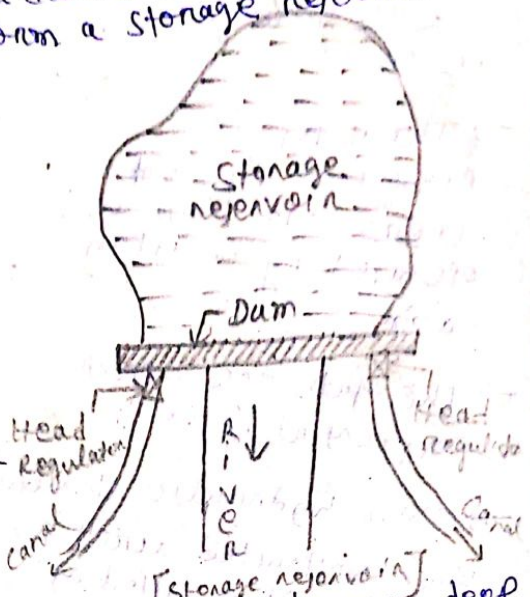
(ii) Storage Irrigation System:-

→ In this system a dam is constructed across a river valley to form a storage reservoir.

→ The main canals may be taken from both sides of the dam.

→ Flow of water through the canal is controlled by head regulator.

→ This storage reservoir on the dam can be defined as high impervious barrier constructed across a river valley to form a deep storage reservoir.

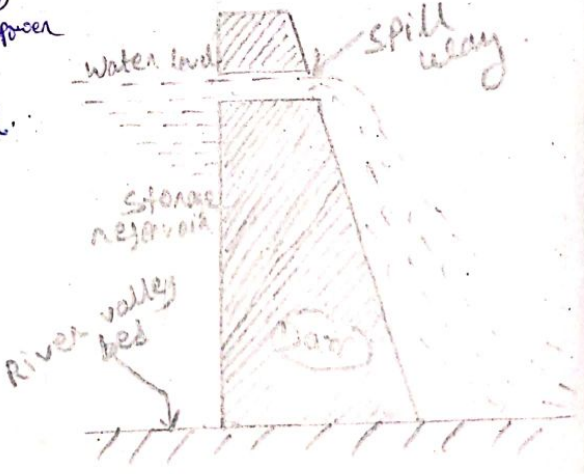


[Storage reservoir]

→ The supply water is not allowed to flow over the dam, but it flows through the spillways provided at some designed level.

→ The storage reservoir is also known as multi-purpose reservoir as it serves:-

a) Irrigation.
b) Water supply
c) Hydro-electric power
d) Fishery
e) Flood control.



[Fig. Dam]

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2.2 CROP SEASON :-

The period during which some particular types of crops can be grown every year on the same land is known as crop seasons.

(a) Kharif Season :- This season ranges from June to October. The crops are sown in the very beginning of monsoon and harvested at the end of autumn.

→ The major kharif crops are- Rice, Millet, Maize, Jute, Groundnut, etc.

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(b) **Rabi Season:-** → This season ranges from october to March. The crops are sown in the very beginning of ~~winter~~ winter and harvested at the end of spring.

→ The major Rabi crops are - Wheat, Gram, Mustard, Rapeseed, Linseed, Pulses, Onion, etc.

→ Again there are several crops ~~which~~ which are not included in Kharif & Rabi as they require more time and they ~~cover~~ cover both the main seasons.

As for eg, Cotton requires eight months to mature and sugarcane requires about whole year to mature.

(i) Cotton - eight month's crop
(ii) Sugarcane - perennial crop.

Crop Rotation:-

→ The process of changing the type of crop for the cultivation on the same land is known as crop rotation.

Eg:- (i) Rice - gram
(ii) Wheat - Millet - Gram.
(iii) Rice - Gram - Wheat

→ If same crop is cultivated on the same land every year, the fertility of the land gets reduced and the yield of crop also gradually reduces. But if different crops are cultivated on the same land every year then the fertility of the land is enhanced. Hence crop rotation is very essential.

Crop Ratio:-

→ It is defined as the ratio of the areas of the two main crop seasons, e.g. Kharif & Rabi.

Eg:- If the area of the kharif crop = 2500 hectares and area of Rabi crop = 5000 hectare

Then, crop ~~rotation~~ ratio = $\frac{2500}{5000} = \frac{1}{2} = 1:2$

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Crop Period:-

⇒ It is defined as the total time period that is from the time of sowing a crop to the time of harvesting it.

No. of Watering:-

⇒ The total depth of water required by a crop is not supplied at one time; But rather it is supplied stage by stage at some specific interval.

⇒ The initial watering which is done on the land to provide moisture to the soil just before sowing any crop is known as paleo or paleva.

⇒ After the crop grows to about 3cm the first watering is done done known as Kor watering and that period as Kor period.

⇒ Subsequently, watering is done as per the requirement till the maturing of the crop

Cash Crops:-

The crops which are cultivated by the farmers to sell in the market to meet their current financial requirements are known as Cash crops.

Ex:- fruits, vegetables etc.

Cumec Day:-

Cubic metre per sec (m^3/sec) for 1 day.

⇒ The quantity of water flowing continuously for one day at the rate of one m^3/sec is known as cumec day.

$1\ m^3/sec$ — for 1 day

$= \frac{1\ m^3}{1\ sec} \times 1\ day$

$= \frac{1\ m^3}{1\ sec} \times 24 \times 60 \times 60\ sec$

$= 24 \times 60 \times 60\ m^3$

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1 cumec-day = $86400 m^3$

$= 86400 (m^2 \times m)$

$= \frac{86400}{10,000} \times 10,000 m^2 \times m$

$1 m^3 = 1 m^2 \times 1 m$

$[10,000 m^2 = 1 hec]$

$\Rightarrow [1 m^2 = \frac{1}{10,000} hec]$

$\Rightarrow$ 1 cumec day $= 8.64 \times (10,000 m^2) \times 1 m$

$= 8.64$ hec-m (area; depth 1 m)

[1 acre = ~~4046~~ $4046.856 m^2$]

$= 86400 m^2 \times m$

$= \frac{86400}{4046.856} \times 4046.856 m^2 \times m$

$= 21.34 \times$ acre-m

$10,000 m^2 = 1$ hec

$\Rightarrow 1 m^2 = \frac{1}{10,000}$ hec

$4046.856 m^2 = 1$ acre

$\Rightarrow 1 m^2 = \frac{1}{4046.856}$ acre.

$\frac{1}{10,000}$ hec $= \frac{1}{4046.856}$ acre

$\Rightarrow 1$ hec $= \frac{10,000}{4046.856}$ acre

$= 2.471$ acre

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2.3 DUTY, DELTA & BASE PERIOD

(i) Base Period :- (Base)

It is defined as time period from the first to the last watering of the crop just before its maturity. It is also known as Base.

⇒ It is denoted by 'B'.

Unit:- No of days

Eg:- Base period of Rice - 120 days

(ii) Delta (Depth of water) :-

⇒ Each crops requires certain amount of water per hectare for its maturity.

⇒ If the total amount of water supplied to the crop from the first watering to the last watering is stored in the land without any loss then there will be a thick layer of water standing on the land. This depth of water layer is known as delta of the crop.

⇒ It is denoted by 'Δ'.

Unit - cm

Eg:- for rice $\Delta = 125$ cm

(iii) Duty :-

⇒ It is defined as no. of hectares that can be irrigated by constant supply of water at the rate of 1 m^3/sec through out the base period.

⇒ It is denoted by 'D' & expressed in hectares/cumec.

Eg:- ~~duty of rice if water will be~~ = 900 hectare/cumec

"that means while cultivating rice, if water will be supplied at the rate of 1 cumec then a total of 900 hectares can be irrigated throughout the base period.

* The duty of water is not const., but it varies with various factors like soil cond^n, method of ploughing, method of appl^n of water, etc. The duties of some common crops are given in table:-

crop	Duty in hectares/cumec
Rice	900
wheat	1800
cotton	1400
sugarcane	800

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Factors affecting Duty:-

The factors that affect the duty are :-

(1) Soil characteristics:-

⇒ If it is the soil of the canal bed is porous & coarse grained, it leads to more seepage loss and consequently low duty.

⇒ If the soil is compact & closed grained, the seepage loss will be less and the duty will be high.

⇒ If the agricultural land consists of sandy soil, the percolation loss will be high causing the duty to be low.

⇒ If it consist of alluvial soil, the percolation loss will be less & the soil retains the moisture for longer period & consequently the duty will be high.

(2) Climatic condition:-

When the atmospheric temp^R of the command area becomes high, the evaporation loss is more & the duty becomes low & vice versa.

(3) Rainfall:-

If the rainfall is sufficient during the crop period, less quantity of irrigation water shall be required & therefore the duty will be more & vice versa.

(4) Base period:- When the base period is longer, the water requirement will be more & the duty will low & vice-versa.

(5) Type of crop:- The water requirement of various crops are different. So, the duty varies from crop to crop.

(6) Topography of Agricultural land:-

If the agricultural land is un-even. The water requirement will be more & hence the duty will be low.

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→ If the land has slight slope, the duty will be high as water requirements is optimum. As the ground slope increases the duty decreases because there is wastage of water.

(7) Method of Ploughing :—

Proper deep ploughing which is done by tractors requires overall less quantity of water & hence the duty is high.

⇒ But, shallow ploughing will bullocks requires overall more quantity of water, & hence the duty is low.

(8) Methods of irrigation :—

The duty of water is high in case of perennial irrigation system as compared to that in inundation irrigation system.

⇒ It is so because in perennial system head regulator is used whereas in undation system there is no regulator.

(9) Water Tax :—

If some tax is imposed on the basis of the volume of water consumption the farmer will use the water economically, & thus the duty will be high.

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Relation between Base, Delta and Duty :-

Let, D = Duty of water in hectares/cumec

B = Base in days

Δ = Delta in m

From, defⁿ, one cumec of water flowing continuously for 'B' days gives a depth of water Δ over an area 'D' hectares. That is,

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1 cumec for B days gives Δ over D hectares

or 1 cumec for 1 days gives Δ over $\frac{D}{B}$ hectares

or 1 cumec for 1 day = $\frac{D}{B} \times \Delta$ hectare-metre

So, 1 cumec-day = $\frac{D}{B} \times \Delta$ - hectare-metre - - (i)

Again, 1 cumec-day = $1 \times 24 \times 60 \times 60 = 86400 m^3$

$= 8.64$ hectare-metre - - - - (ii)

(1 hectare = $10,000 m^2$)

From eqn (i) & (ii)

$$\frac{D}{B} \times \Delta = 8.64$$

$$\therefore \Rightarrow \Delta = \frac{8.64 \times B}{D} = \text{in m}$$

Numerical Problem

Prob-1:- A Channel is to be designed for irrigating 5000 hectares in Kharif Crop & 4000 hectares in Rabi crop. The water requirement in Kharif & Rabi are 60 cm & 25 cm, respectively. The for Kharif is 3 weeks and for Rabi is 4 weeks. Determine the discharge of the channel for which it is to be designed.

Sol:- We know that

$$\Delta = \frac{8.64 \times B}{D}$$

Discharge for Kharif Crop

Here, $\Delta = 60 cm = 0.6 m$

$B = 3 weeks = 21 days$

$$\therefore \text{Duty } (D) = \frac{8.64 \times B}{\Delta}$$

$$= \frac{8.64 \times 21}{0.6} = 302.4 \text{ hectares/cumec}$$

Area to be irrigated = 5000 hectares

Required discharge of channel = $\frac{5000}{302.4} = 16.53$ cumec

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Discharge for Rabi Crop :-

Here, $\Delta = 25\text{cm} = 0.25\text{m}$

$B = 4 \text{ weeks} = 28 \text{ days}$

$\therefore \text{Duty} = \frac{8.64 \times 28}{0.25} = 967.68$ hectares/cumec.

Area to be Irrigated = 4000 hectares.

Required discharge of Channel = $\frac{4000}{967.68} = 4.13$ cumec.

So, Channel is to be ~~irrigated~~ designed for the max^m discharge of 16.53 cumec, because this discharge capacity of the channel will be able to supply water to both the seasons

2.4 Gross Command Area (G.C.A)

⇒ The whole area enclosed b/w an imaginary boundary line which can be included in an irrigation Project for supplying water to agricultural land by the network of canals is known as Gross Command Area (G.C.A).

⇒ It includes both the culturable & unculturable areas.

Unculturable Area :-

⇒ The area where the agriculture cannot be done & crops cannot be grown is known as unculturable area.

⇒ The marshy lands, barren lands, lakes, ponds, forests, villages, etc. are considered as unculturable area.

Culturable Area

⇒ The area where the agriculture can be done satisfactorily is known as culturable area.

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(4) Culturable Command Area:- (C.C.A)

⇒ The total area within an irrigation project where the cultivation can be done & crops can be grown is known as culturable Command Area (C.C.A). Again C.C.A may be of two types. Categories.

(a) Culturable Cultivated Area:-

It is the area within C.C.A where the cultivation has been actually done at present.

(b) Culturable 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 some reasons.

Intensity of Irrigation:-

The total culturable command area may not be cultivated at the same time in a year due to various reasons. Some area may remain vacant every year.

⇒ Again, various crops may be cultivated in the culturable command area.

⇒ So, the intensity of irrigation may be defined as a ratio of cultivated land for a particular crop to the total culturable command area, it is expressed as a percentage of C.C.A. ~~For example~~

For Example:- If total culturable command area is 1000 hectares where wheat is cultivated in 250 hectares, then,

Intensity of irrigation for wheat $= \frac{250}{1000} \times 100$

$= 25\%$

So, area to be ~~covered~~ irrigated = C.C.A × Intensity of irrigation / 100

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Field capacity:—

⇒ The field capacity is defined as the amount of maxm moisture that can be held by the soil against gravity.

⇒ It is expressed in %age.

[B] permanent Wilting point:—

⇒ It is defined as the amount of moisture held by soil which cannot be extracted by plant roots for transpiration.

⇒ At this point the wilting of plant occurs.

⇒ It is also expressed in percentage.

(C) Consumptive use of water:—

⇒ The quantity of water used by the plant for its transpiration and also the water lost during evaporation is known as consumptive use of water.

⇒ Evaporation & transpiration is combinedly called as evapo-transpiration.

⇒ So, consumptive use = water required for evapo-transpiration.

⇒ It is expressed as depth of water (in m or in cm.)

⇒ Consumptive use of water is found to know the water requirement for each crop.

Again ~~Frequency of irrigation:—~~

(C) Consumptive use of water:—

⇒ The Consumptive use of water is defined as the total quantity of water used for the growth of the plants by transpiration and the amount lost by evaporation (which is known as evapotranspiration)

⇒ It is expressed in hectare-metre per hectare or as depth of water (in m) for a specified period.

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→ The consumptive use of water is ascertained (learn) to know the water requirement for each crop.
→ The value of consumptive use of water varies from crop to crop, time to time & even from place to place.

Frequency of Irrigation:—

→ The irrigation water is supplied to the field to raise the moisture content of the soil up to its field capacity.
⇒ The application of water is then stopped.
⇒ The water content also reduces gradually due to transpiration & evaporation.
⇒ If the moisture content is dropped below the requisite amount, the growth of the plants gets disturbed.
⇒ So to maintain moisture content irrigation is immediately done.
⇒ This is known as frequency of irrigation.

Mathematically it is expressed as:—

(a) $D_w = \frac{W_s \times d}{W_w} [F_c - M_o]$.

Notation:—

f_w = frequency of irrigation (watering)
C_u = consumptive use of water or daily consumptive use
D_w = ~~Unit weight of soil~~ Depth of water to be applied in each watering
W_w = Unit weight of water = 1000 kg/m³
W_s = Unit weight of soil
d = depth of root zone
F_c = field capacity.
M_o = optimum moisture content.

(b) $f_w = \frac{D_w}{C_u}$:

where, f_w = frequency of watering, D_w = Depth of water to be applied in each watering.
C_u = Daily consumptive use of water

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Overlap Allowance :-

In certain cases a crop season may overlap the next crop season by few days. During this period of overlapping the irrigation water is to be supplied for the crops of both the seasons. Due to this extra demand during this period, discharge of canal has to be increased. So, for canal designing a provision should be made for the extra demand which is known as overlap Allowance.

Time Factor :-

It is the ratio between the no. of days the canal has actually been kept open to the no. of days the canal was designed to remain open during the base period is known as time factor.

$$\text{Time Factor} = \frac{\text{No. of days the canal practically open}}{\text{No. of days the canal was designed to keep open}}$$

$$= \frac{\text{Actual Discharge}}{\text{Designed Discharge}}$$

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3.0 FLOW IRRIGATION

Introd^{n}:- The irrigation system in which water flows under gravity from the source to the agricultural land is known as flow irrigation.

→ It involves,

(a) the construction of weir or barrage across a river (known as diverting head works)

(b) the construction of dam across a river valley (to form a storage reservoir).

(c) The excavation of canal system

→ This type of irrigation is popular nowa-days because a vast area can be irrigated under this system.

Live examples:-

(a) Bhakra Nangal Project

(b) Ukai Project

(c) Damodar Valley Project.

(d) Hirakud Dam Project etc.

→ The flow irrigation may be of two types.

(i) Inundation irrigation &

(ii) Perennial irrigation.

(i) Inundation Irrigation system:-

→ In this system the canals are excavated from the banks of the inundation river.

→ The bed level of canal is such that the water can flow in rainy season only, when the water level in the river rises above the canal bed.

→ The construction of hydraulic structure is not necessary in this system.

→ There is no head regulator to control the flow of water through the canal.

→ In this system water is not available throughout the year.

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(ii) Perennial Irrigation System:—

→ In this system either a weir or a barrage is constructed across the perennial river to raise the water level or a dam is constructed to form a storage reservoir.

→ Then the network of canals (i.e, main, branch, distributary) is constructed from the source to the agricultural land.

→ Here head regulator is constructed to control the flow of water through the canal.

→ In this system, the water is available throughout the year.

Types of canals:—

1) Based on purpose:—

(a) Irrigation canal:—

→ The canal which is constructed to carry water from the source to the agricultural land for the purpose of irrigation is known as irrigation canal such as Bhakra canal, Rajasthan canal, etc.

(b) Navigation Canal:—

→ The canal which is constructed for the purpose of inland navigation is known as navigation canal.

→ This type of canal is also utilised for irrigation such as Ganga-Brahmaputra navigation cum irrigation canal.

(c) Power canal:—

→ The canal which is constructed to supply water with very high force to the hydroelectric power station for the purpose of moving turbine to generate electric power is known as power canal or hydel canal such as Nangal Hydel Canal.

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(d) Feeder Canal:-

→ The canal which is constructed to feed another canal or river for the purpose of irrigation or navigation is known as feeder canal such as Farakka barrage feeder canal.

2. Based on Nature of Supply:-

Based on the nature of supply the canals are designed as (a) Inundation canal
(b) Perennial canal.

(a) Inundation Canal:-

→ The canal which is encavated from the banks of the inundation river to carry water to the agricultural land in rainy season only when the river flows to its full capacity is known as inundation canal.

→ No regulator is provided at the head of such canal.

→ The flow of water through the canal depend on the fluctuation of water level in the river.

→ When the water level rises above the bed level of the canal the water starts flowing through the canal.

→ When the water level fall below the bed level of the canal the flow of water through the canal stops.

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(b) Perennial canal :-

→ The canal which can supply water to the agricultural land throughout the year is known as perennial canal.

→ This type of canal is taken from the UP stream side of the diversion head works (weir or barrage) or from the storage reservoir with regulator at the head of the canal.

(3) Based on Discharge :-

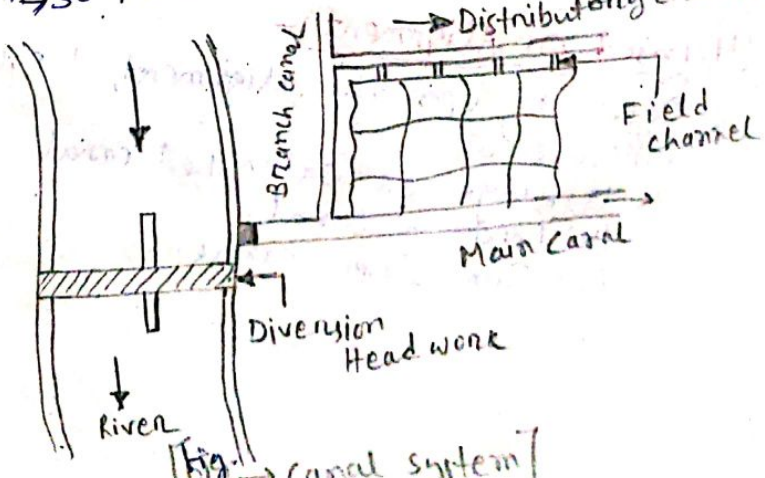
→ Acc. to the discharge capacity, the canals are designated as

(a) Main canal
(b) Branch "
(c) Distributary channel
(d) Field channel.

(a) Main canal :-

⇒ The large canal which is taken directly from the diversion head work or from storage reservoir to supply water to the network of other small canals is known as main canal.

→ The water is taken to the field through the branch canal, distributary channel & field channel → so main canal is backbone of the canal system.



[Fig → Canal system]

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(b) Branch Canals :—

→ The branch canals are taken from either side of the main canal at suitable points so that whole command area can be covered by the network.

→ The discharge capacity of the branch canal is smaller than that of the main canal.

→ The discharge varies from 5 to 10 cumec.

(c) Distributory Channels :— (Above fig)

→ The distributory channels are taken from the branch canals to supply water to different sectors.

→ The discharge capacity of these channels varies from 0.25 to 3 cumec.

→ Again, these are designated as major distributory and minor distributory according to their function in the total network. (Above fig)

(d) Field Channels :—

→ These channels are taken from the outlets of the distributory channels by the cultivators to supply water to their own lands.

→ These channels are maintained by the cultivators (Above fig)

(4) Based on Alignment :—

Depending upon the alignment, the canals are designated as

(a) Ridge or watershed canal
(b) Contour canal
(c) Side slope canal.

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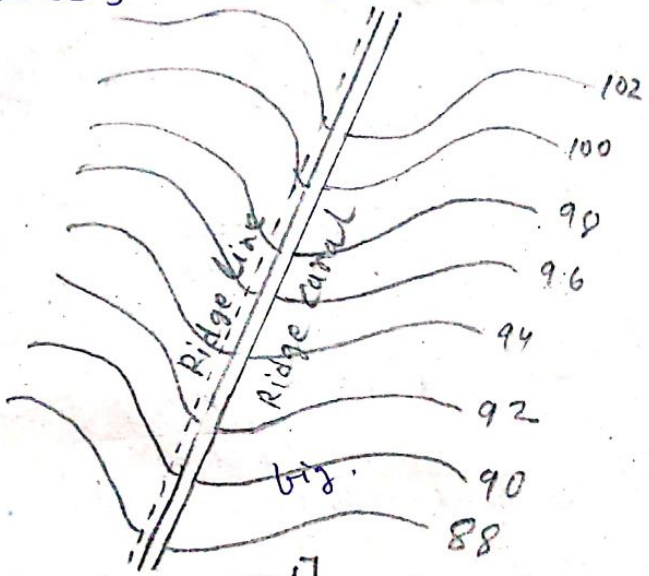
(a) Ridge or watershed canal :-

⇒ The canal which is aligned along the ridge line (watershed line) is known as ridge canal or watershed canal.

⇒ The advantage 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 & hence no cross-drainage work is necessary.



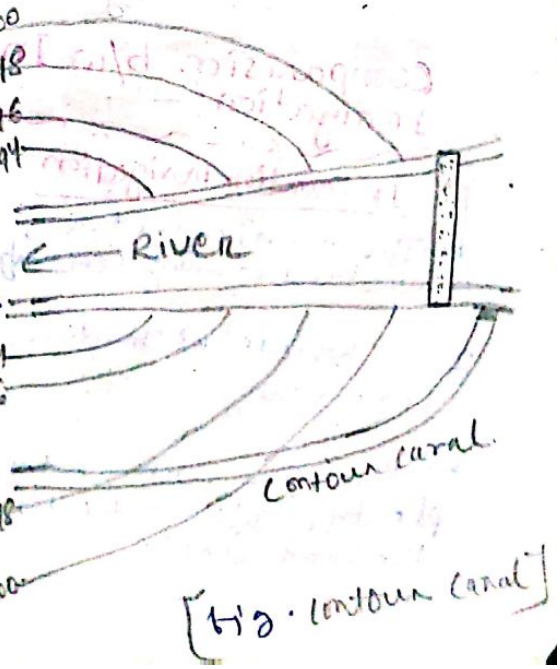
[Ridge canal]

(b) Contour canal :-

⇒ The canal which is aligned approximately parallel to the contour lines is known as contour canal.

⇒ This canal can irrigate the areas on one side only.

⇒ This canal may cross natural drainage & hence cross-drainage works are necessary.



[Fig. contour canal]

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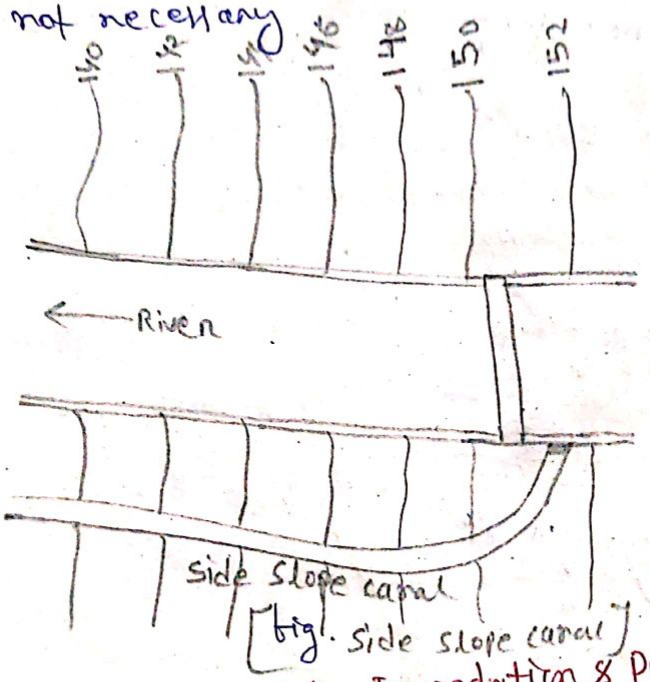
(c) Side slope canal

⇒ The canal which is aligned approximately at right angle to the contour lines is known as side slope canal.

⇒ It can irrigate the areas on one side only.

⇒ Again, it does not cross any natural drainage & hence the cross drainage works are not necessary



[fig. Side slope canal]

Comparision b/w Inundation & Perennial Irrigation:-

Inundation irrigation	Perennial irrigation
1. The irrigation water is available in rainy season only.	1. The irrigation water is available throughout the year.
2. No hydraulic structure is necessary.	2. Hydraulic structure are necessary, (such as diversion-head works, cross-drainage works etc)
3. The canal water contains plenty of silt which makes the land fertile.	3. The canal water contains Practically no silt & hence chemical manure is essential.

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→ Large area cannot be included under this system.	→ Large area can be included under this system.
→ The silting of the canal bed is a major problem.	→ Negligible silting takes place in the canal bed.
→ Water tax cannot be imposed.	→ Water tax can be imposed.
→ Initial cost is low	→ Initial cost is high.
→ No technical persons are required for the operation of the irrigation system.	→ Technical persons are always required for the operation of the irrigation system.
→ The main canal is not provided with regulator & hence there is a possibility of over irrigation.	→ The main canal is provided with head regulator & hence there is no possibility of over irrigation.

3.3 Canal section :-

Different components of irrigation canals & their function :-

The different terms related to the canal are

(a) Canal bank
(b) Berm
(c) Hydraulic gradient
(d) Contour Berm
(5) Free board
(6) side slope
(7) Service road or inspection road
(8) Dowel or Dowla
(9) Borrow pit
(10) Spoil Bank
(11) Land width.

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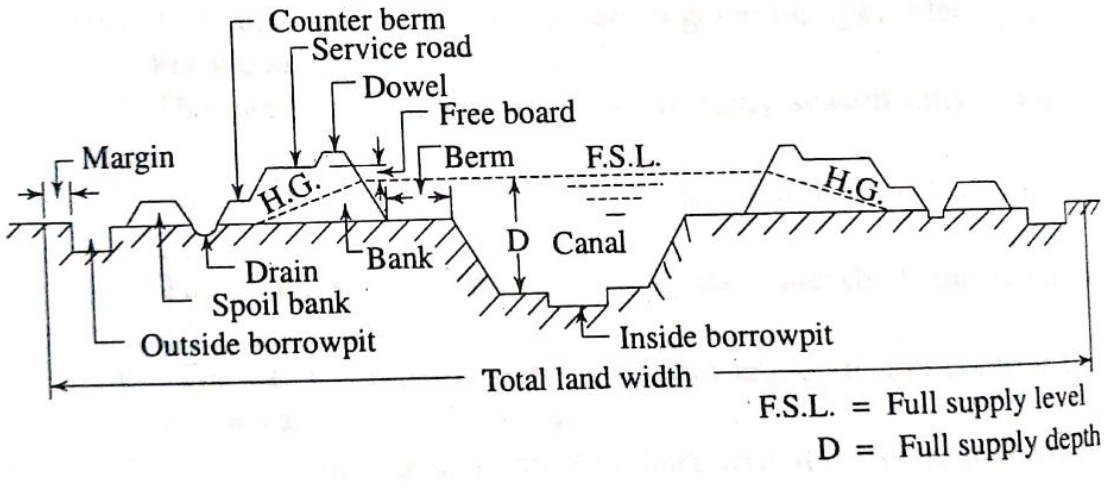

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(1) Canal Bank :-

The canal bank is necessary to retain water in the canal to the full supply level. But the section of the canal bank is different for different site condition.

→ The following are the different forms for different site condition.

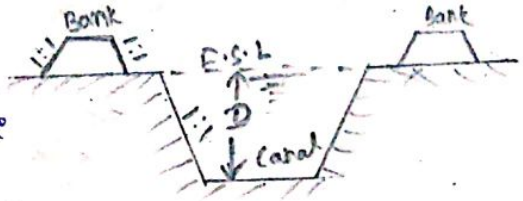
(a) When the canal fully in Cutting :-

→ 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 & the top width will be min^m just to provide the road way.

→ The side slope will be $1\frac{1}{2}:1$ or $2:1$ according to the nature of the soil.



[fig: canal in full cutting]

(b) When the canal in Partial cutting and Banking :-

→ In this case, the banks are constructed on sides of the canal to retain water.

→ The height of the banks depend on the fully supply level of the canal.

* → Again, the section of the canal depends on the hydraulic gradient.

→ The top width & the side slope of the bank should be such that the hydraulic gradient should have a min^m cover of 0.5 m

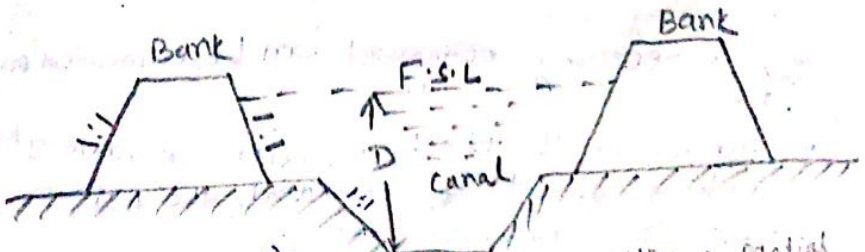


fig: canal in partial cutting & partial banking

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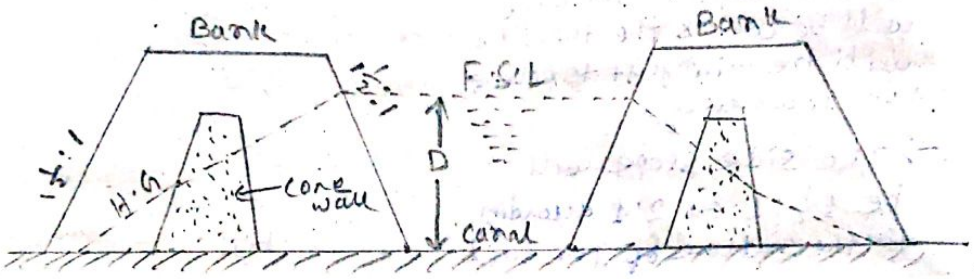
(c) When the canal in full Banking:-

→ In this case, the canal & both the canal 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 of puddle clay is provided which deflects the hydraulic gradient downward.



[Canal in full banking]

(2) BERM :-

→ The distance b/w the toe of the bank & the top edge of cutting is termed as berm.
→ The berm is provided for the following reasons,
(a) To protect the bank from erosion.
(b) To provide a space for widening the canal section in future if necessary.
(c) To protect the bank from sliding down towards the canal section.
(d) The silt deposition on the berm makes an improving lining.
(e) If necessary borrowpit can be excavated on the berm.

* The width of the berm depends on various factors such as capacity of the canal, the nature of the soil, the site condition, etc.

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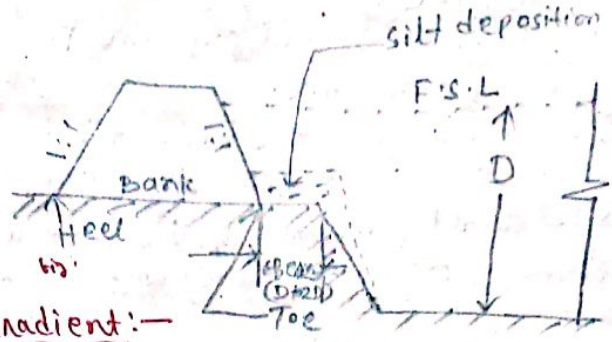
→ However, the width of the berm varies from D to 2D, where, D is the full supply depth of the canal.



(3) Hydraulic Gradient:—

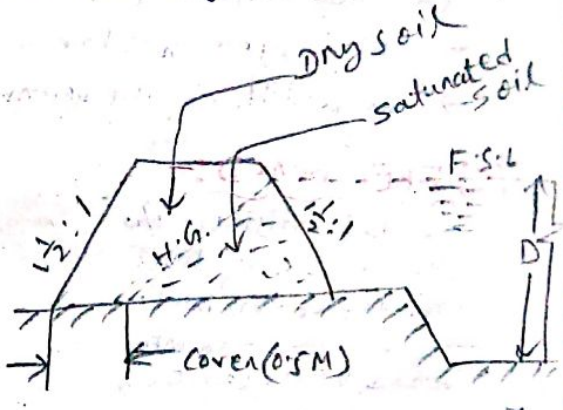
→ When the water is retained by the canal bank, the seepage occurs through the body of the bank.

→ Due to the resistance of the soil, the saturation line forms a sloping line which may pass through contryside of the bank. This sloping line is known as the hydraulic gradient or saturation gradient.

→ The soil below this line is saturated, but the soil above this line is dry.

→ The hydraulic gradient depends on the permeability of the soil.

→ So while constructing the bank, the soil should be tested in soil testing laboratory and the nature of the hydraulic gradient should be ascertained.



[fig. Hydraulic Gradient]

→ This will help in fixing the height, top width and side slope of the bank.

→ The following are the approximate values of hydraulic gradient for different soil.

Soil	H.G.
Clayey soil	1:4
Alluvial soil	1:5
Sandy soil	1:6

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(4) Counter Berm:-

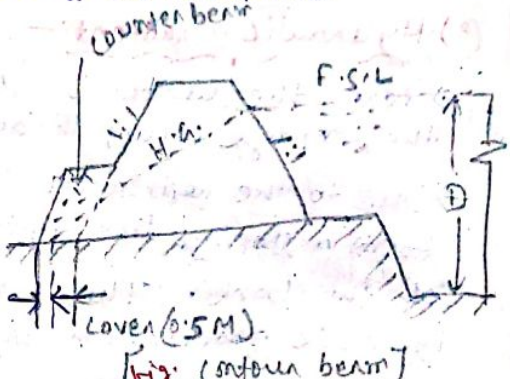
→ When the water is retained by a canal bank the hydraulic gradient line passes through the body of the bank.

→ For stability of the bank, this gradient should not intersect the outer side of the bank.

→ It should pass through the base and a minimum cover of 0.5m should always be maintained.

→ Sometimes, it may occur that the hydraulic gradient line intersects the outer side of the bank.

→ In that case, a projection is provided on the bank to obtain min^m cover.



[fig. (outer berm]

→ This projection is known as ~~outer~~ counter berm.

→ The width of this berm depends on the site cond.^n

(5) FREE BOARD:-

→ It is the distance b/w the full supply level and top of the bank.

→ The amount of free board varies from 0.6m to 0.75m.

→ It is provided for the following reasons.



[Free board]

(a) To keep a sufficient margin so that the canal water does not overtop the bank in case of heavy rainfall or fluctuation in water supply.

(b) To keep the saturation gradient much below the top of the bank.

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(6) SIDE SLOPE :-

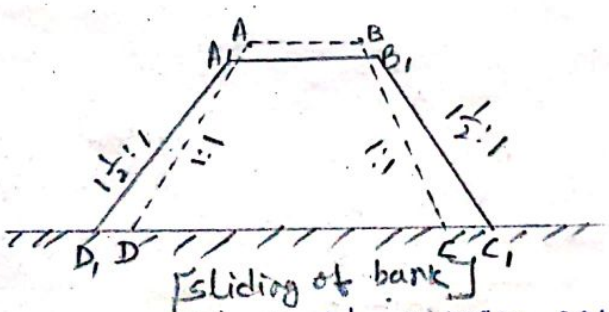
→ The side slopes of the canal bank and canal section depend on the angle of repose of the soil existing on the site.

→ So to determine the side slopes of different sections, the soil samples should be collected from the site & should be tested in the soil testing laboratory.

→ The necessity of such test is that if the permissible slope 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 $1\frac{1}{2}:1$.

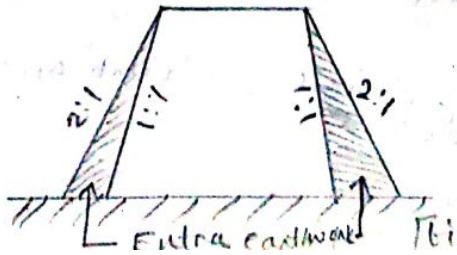
→ Then the initial shape ABCD will automatically take the final shape A_1, B_1, C_1, D_1 after slide in the due course.



[Sliding of bank]

→ Again, an opposite incident may occur, suppose, an embankment was constructed with side slope 2:1, but latter 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.



[Fig. Extra earth filling]

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(7) 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.

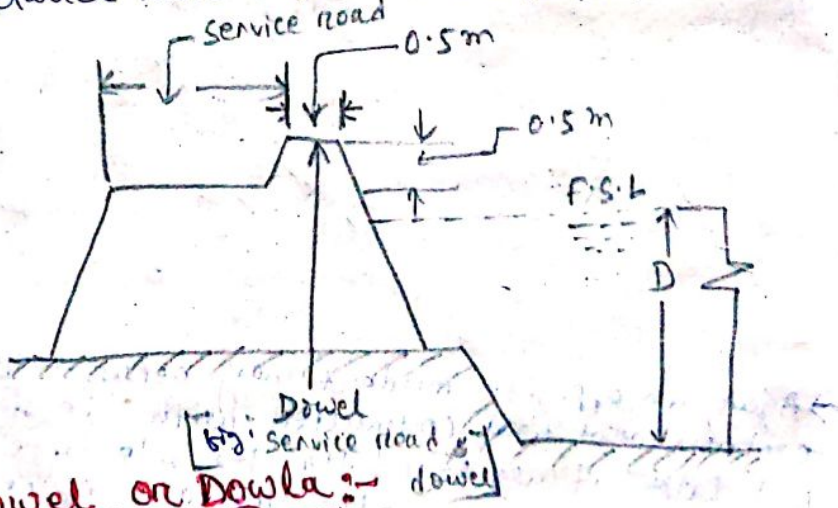
→ But for branch canal, the road is provided on one bank only.

→ The width of the service roads for main canal varies from 4 to 6m. The width of the road for the branch canal varies from 3 to 4m.

→ The initial purpose of the service road is to conduct inspection and maintenance works.

→ But finally these roads serve the purpose of communication b/w the different villages & for transporting agricultural goods.

→ Therefore it becomes necessary to construct metalled road to serve these purposes.



(8) Dowel or Dowla:-

→ The protective small embankment which is provided on the canal side of the service road for the safety of the vehicles plying on it is known as dowel or dowla.

→ Practically it acts as a curb on the canal side of the road.

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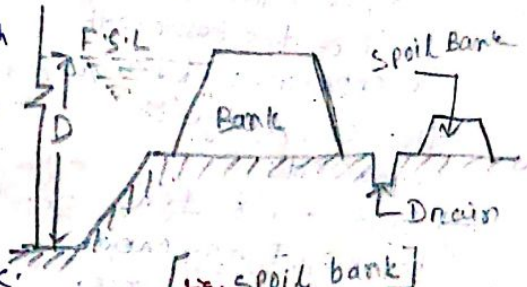
→ It is provided above F.S.L with the provision of freeboard.
→ The top width is generally 0.5m and the height above the road level is about 0.5m.
→ The side slope is similar to the side slope of the bank.

[Above fig.]

(9) SPOIL BANK :-

→ When the canal is constructed in full cutting, the excavated earth may not be completely required for forming the bank.
→ In such a case, the extra 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 quantity of excess earth and the available space.
→ The spoil banks run parallel to the main bank.



[fig. spoil bank]

→ But are not continuous, sufficient spaces are left between the adjacent spoil banks for proper drainage.

(10) 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 borrow pits. The borrow pits may be inside or outside, the canal.

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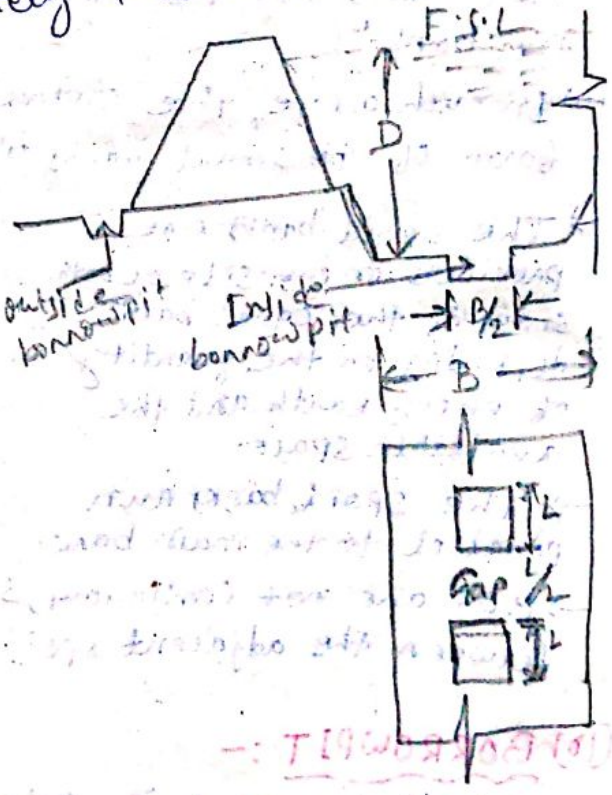
→ The inside borrowpit may be located at the centre of the canal.
→ The width of the borrowpit should be half of the base width of canal.
→ The maximum depth should be 1m.
→ The excavation is done in a no. of borrowpits leaving a gap b/w them.
→ The gap is generally half of the length of each borrowpit.
→ The idea behind this is that the borrowpits will act as water pockets where the silt will be deposited and ultimately the canal bed will get levelled up

→ The outer borrowpit may be adjacent to the heal of the bank with a clearance of 1m b/w the heel & edge of borrowpit.
→ But the outer borrowpit may create some inconvenience.
→ So, it is better to borrow earth from the barren lands far away from the canal.



[Borrowpits]

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(11) 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 to the 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 dimension should be added.

(i) Top width of the canal
(ii) Twice the berm width
(iii) Twice the bottom width of banks.
(iv) A margin of one metre from the heel of the bank on both sides.
(v) Width of external borrowpit if any.
(vi) A margin of 0.5 m from the outer edge of borrowpit on both sides, if external borrowpit becomes necessary.

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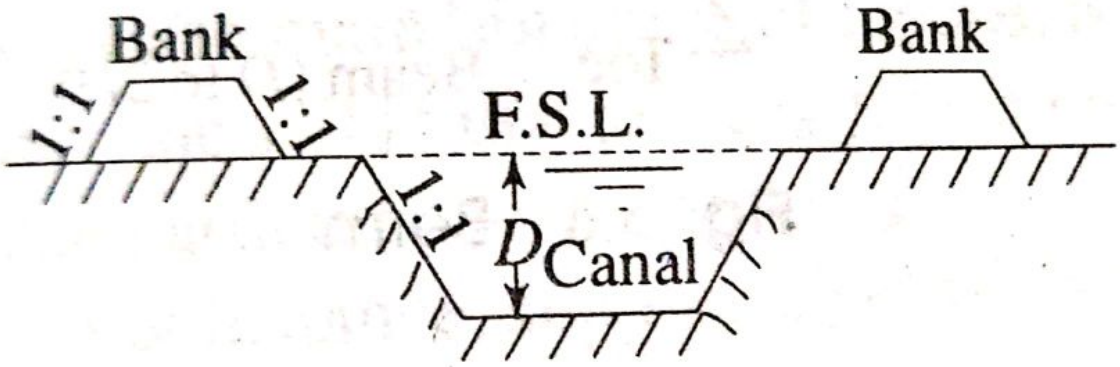

Fig. 6.2 Canal in full cutting

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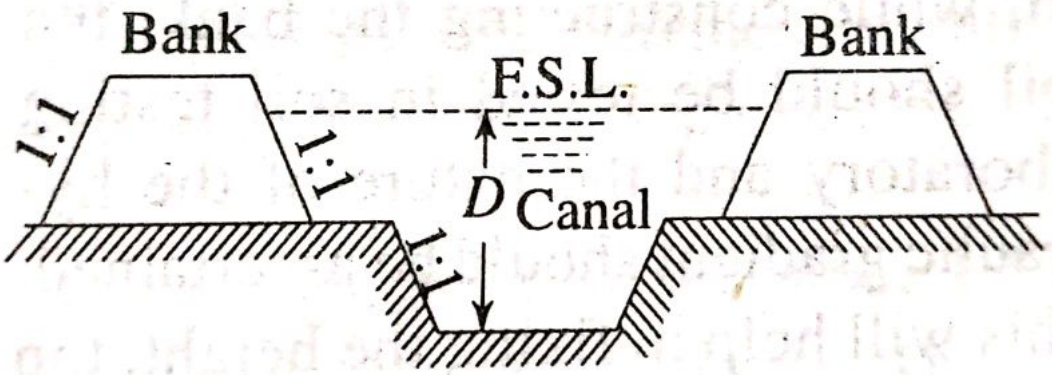

Fig. 6.3 Canal in partial cutting and partial banking

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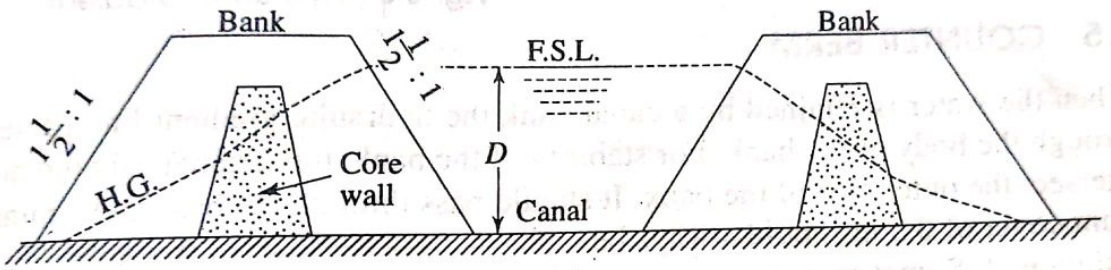

Fig. 6.4 Canal in full banking

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CANAL LINING

Def^{n} :- Canal lining is the process of reducing seepage loss of irrigation water by adding an impermeable layer to the edges of the trench.

Objects of Canal Lining :-

a) To control seepage
b) To prevent water-logging
(c) To increase the capacity of canal
(d) To increase the command area.
(e) To protect the canal from the damage by flood.
(f) To control the growth of weeds.

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Types of canal lining:-

Various lining are recommended acc. to site conditions.

(1) 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 resist the effect of scouring and erosion very efficiently.

→ The velocity of flow may be kept above 2.5 m/sec (clean water) (rubbing)

→ It can eliminate completely growth of weeds.

(a) Procedure of Cement Concrete lining:-

(a) Preparation of sub-grade:-

→ The sub-grade is prepared by ramming the surface properly with a layer of sand (about 15cm)

→ Then, a slurry of cement & sand (1:3) is spread uniformly over the prepared bed.

(b) Laying of concrete:-

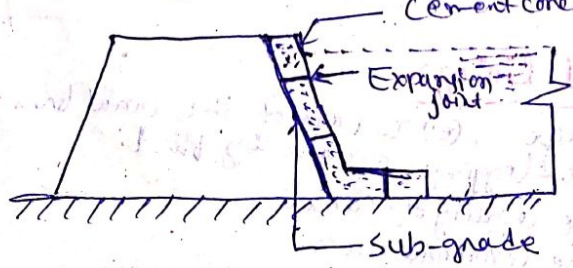
→ The cement concrete of grade M_{15} is spread uniformly according to the desired thickness (100mm - 150mm)

→ After laying, the concrete is tapped gently until the slurry comes on the top.

→ Top curing is done for two weeks. As the concrete is liable to get damaged by the change of temp^{n}, the expansion joints are provided at appropriate places.

→ Normally no re-inforcement is required for this cement concrete.

→ But in special cases, a network of 6mm dia rods may be provided with spacing 10 cm C-C.



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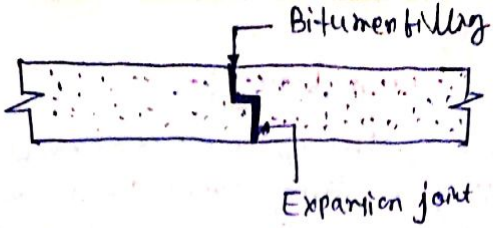


[Cement concrete lining]

(2) Pre-cast Concrete Lining :-

→ This lining is recommended for the canal in full banking.

→ It consists of pre-cast concrete slabs of size 60 cm × 60 cm × 5 cm which are set along the canal bank / bed with cement mortar (1:6).

→ A network of 6mm dia rod is provided in the slab with spacing 10cm C to C.

⇒ 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.

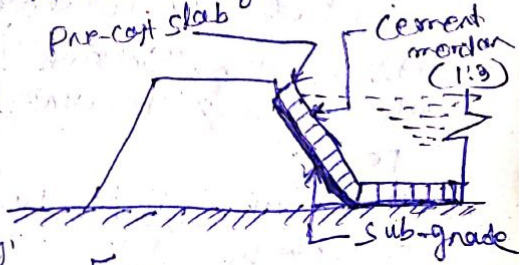
The slabs are set in the following sequence

(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 as per estimate along the course of the canal.

→ The slabs are placed with cement mortar (1:6) by setting the rebates properly.

→ The joints are finished with cement mortar (1:3)



[Pre-cast concrete lining]

(c) The curing is done for a week

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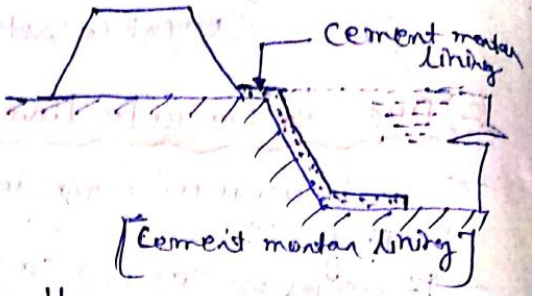
(5) Cement Mortar Lining:-

→ This type of lining is recommended for the canal belly in cutting where hard soil or clayey soil is available.



→ The thickness of the cement mortar (1:4) is generally 2.5 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 ~~the~~ neat ~~sketch~~ cement polish.

→ This lining is impervious, but is not durable.

→ The curing should be done properly

Lime concrete Lining:-

→ When hydraulic lime, sunki & brick ballast are available in plenty along the course of the canal or in the vicinity of the irrigation project, then the lining of the canal may be made by the lime concrete of proportion 1:1:6.

→ The procedure of laying this concrete is same as that of the cement concrete lining.

→ Here, the thickness of concrete varies from 150 mm to 225 mm & the curing should be done for long period.

→ This lining is less durable than the cement concrete lining.

→ However, it is recommended because of the availability of the materials & also because of the economics.

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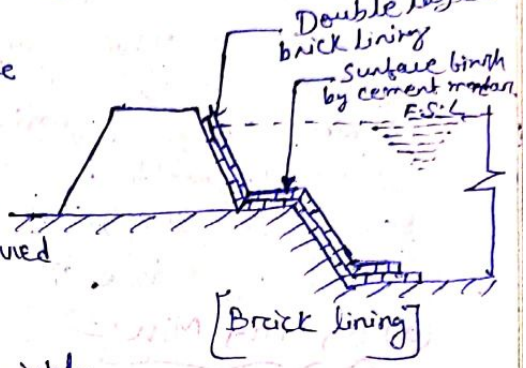
Brick lining:-

→ This lining is prepared by the double layer brick flat soling laid with cement mortar (1:6) over the compacted sub-grade.

→ The 1st class bricks should be recommended for the work.

→ The surface of the lining is finished with cement plaster (1:3)

→ The curing should be done perfectly.



Brick lining

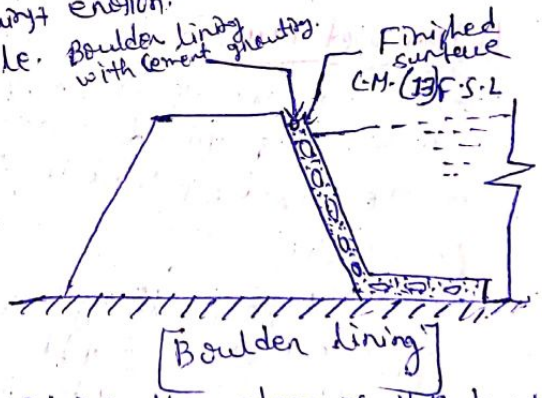
→ This lining is always preferred for the following reasons.

(a) This lining is economical.
(b) Work can be done very quickly.
(c) Expansion joints are not required.
(d) Repairing works can be done easily.
(e) Bricks can be manufactured from the excavated earth near the site.

However this lining has certain disadvantages

(a) It is not completely impervious.
(b) It has low resistance against erosion.
(c) It is not so much durable.

Boulder lining:-



Boulder lining

→ In hilly area where the boulders are available in plenty, this type of lining is generally recommended.

→ The boulders are laid in single or double layer maintaining the slope of the bank and the bed level of the canal.

→ The joints of the boulders are grouted with cement mortar (1:6).

→ The surface is finished with cement mortar (1:3).

→ Curing is necessary in this lining too.

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→ This lining is very durable & impervious.
→ But the transporting 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 known as gunitting, 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 in 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 & forced on the sub-grade.
→ This type of lining is very costly & it is not durable. It is suitable for resurfacing the old cement concrete lining.

ASPHALT LINING:—

→ This lining is prepared by spraying asphalt (i.e. bitumen) at a very high temp (about 150°C) on the subgrade to a thickness varies from 3 mm to 6 mm.

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→ The hot asphalt when becomes cold to form a water proof membrane over the sub-grade.
→ This membrane is covered with a layer of earth & gravel. → The lining is very cheap & 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 mass.
→ This mass 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~~ recommended for small channels.

SOIL-CEMENT LINING

→ This lining is prepared with a mixture of soil & 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 texture.
→ The mixture is laid on the sub-grade and it is made thoroughly compact.
→ The lining is efficient to control the seepage of water, but it cannot control the growth of weeds. So, this is recommended for small channels only.

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ADVANTAGES OF CANAL LINING

→ It reduces the loss of water due to seepage and hence the duty is enhanced.
→ It controls the water logging & hence the bad effects of water-logging are eliminated.
→ It provides smooth surface & hence the velocity of flow can be increased.
→ Due to the increased velocity the discharge capacity of a canal is also increased.
→ Due to the increased velocity, the evaporation loss also be reduced
→ It eliminates the effect of scouring in the canal bed.
→ The increased velocity eliminates the possibility of silting in the canal bed.
→ It controls the growth the weeds along the canal sides and bed

Disadvantages

→ The initial cost of the canal lining is very high. So, it makes the project very expensive w.r.t. to the output.
→ It involves much difficulties for repairing the damaged section of lining.
→ It makes too much time to complete the project works.
→ 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.

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4.0 WATER LOGGING AND DRAINAGE

Introduction :-

In agricultural land, when the soil pores within the root zone of the crops get saturated with the subsoil water, the air circulation within the soil pores gets totally stopped. This phenomenon is termed as water logging.

→ The water logging makes the soil alkaline in character and the fertility of the land is totally destroyed and the yield of crops is reduced.

Causes of Water logging :-

(1) Over Irrigation :-

→ In inundation irrigation since there is no controlling system of water supply it may cause over irrigation.

⇒ The excess water percolates & remains stored within the root zone of the crops.

⇒ Again in perennial irrigation system if water is supplied more than what is required.

⇒ This excess water is responsible for the water logging.

(2) Seepage from canals :-

→ In unlined canal system, the water percolates through the bank of the canal and gets collected in the low lying areas along the course of the canal & thus the water table gets raised.

→ This seepage is more in case of canal in banking.

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(3) Inadequate surface drainage:—

⇒ When the rainfall is heavy & there is no proper provision for surface drainage the water gets collected & submerges vast area.

⇒ When this condition continues for a long period, the water table is raised.

(4) 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 & this causes water logging.

(5) Obstruction in sub-soil Drainage:—

⇒ If some impermeable stratum exists at a lower depth below the ground surface, then the movement of the sub soil water gets obstructed and causes water logging in the area.

(6) Nature of soil:—

⇒ The soil having low permeability, like black cotton soil, does not allow the water to percolate through it.

⇒ So, in case of over irrigation or flood, the water retains in this type of land and causes water logging.

(7) Incorrect Method of cultivation:—

⇒ If the agricultural land is not levelled properly and there is no arrangement for the surplus water to flow out, then it will create pools of stagnent water leading to water logging.

(8) Seepage from Reservoir:—

⇒ If the reservoir basin consists of permeable zones, cracks & fissures which were not detected during the construction of dam. these

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may cause seepage of water.
→ This sub-soil water will move towards the low-lying areas and cause water logging.

(9) Poor irrigation Management:—
→ If the main canal is kept open for a long period un-necessarily without computing the total water requirement of the crops, then this leads to over irrigation which shall result in water logging.

(10) Excessive Rainfall:—
→ If the rainfall is excessive and the water gets no time to get drained off completely, then a pool of stagnant water is formed which might lead to water logging.

(11) Topography of the land:—
→ If the agricultural land is flat, i.e. with no country slope & consists of depressions or undulations, then this leads to water logging

(12) Occasional flood:—
→ If an area gets affected by flood every year and there is no proper drainage system the water table gets raised & this causes water logging.

EFFECTS OF WATER LOGGING

→ The effects of water logging are

(1) Salinization of soil:—
→ Due to water logging the dissolved salts like Sodium Carbonate, Sodium Chloride & Sodium Sulphate come to the surface of the soil.

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⇒ When the water evaporates from the surface, the salts are deposited there.
⇒ This process is known as salinization of soil.
⇒ Excessive concentration of salt makes the land alkaline.
⇒ It does not allow the plants to thrive (grow/develop) and thus the yield of crop is reduced. This process is also known as salt efflorescence.

(2) Lack of Aeration :—
⇒ The crops require some nutrients for their growth which are supplied by some bacteria or micro-organisms by breaking the complex nitrogenous compounds into simple compounds which are consumed by the plants for their growth.
⇒ But the bacteria requires oxygen for their life & activity. When the aeration in the soil is stopped by water logging, these bacteria cannot survive without oxygen & the fertility of the land is lost which results in reduction of yield.

(3) Fall of soil temperature :—
⇒ Due to water logging the soil temperature is lowered.
⇒ At low temperature of the soil, the activity of the bacteria becomes very slow & consequently the plants do not get the requisite amount of food in time.
⇒ Thus, growth of the plants is hampered (difficult) & the yield is also reduced.

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(4) Growth of weedy and Aquatic plants :—

⇒ Due to water logging, the agricultural land is converted to marshy land & the weedy & aquatic plants are grown in plenty.

⇒ These plants consume the soil food in advance and thus the crops are destroyed.

(5) Diseases of crops :—

⇒ Due to low temperature & poor aeration, the crops get some diseases which may destroy the crops or reduced 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.

(7) Restriction of Root Growth :—

⇒ When the water table rises near to root zone soil gets saturated.

⇒ The growth of roots is confined (very small) only to the top layer of the soil.

⇒ So, the crops cannot be matured properly and the yield is reduced.

* Control of water-logging :—

Prevention of percolation from canals :—

→ The irrigation canals should be lined with impervious lining to prevent the percolation of H_2O through the bed and banks of the canals. Thus the water logging may be prevented.

→ Interception drains may be provided along the course of the irrigation canals in places where the percolation of water is detected.

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→ The percolating water is intercepted by the drain and the water is carried to other natural water course.

(2) Prevention of percolation from Reservoir:—

→ During the construction of dam, the geological survey should be conducted on the reservoir basin to detect the zone of permeable formation through which water may percolate.

⇒ These zones should be treated properly to prevent the 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 so that there is no possibility of water logging in a particular area.

(4) Economical use of water:—

⇒ If the water is used economically, then it may control the water logging & the yield of crops may be high. ⇒ So special training is required to be given to the cultivation to realise the benefit of economical use of water.

→ It helps them to get more crops by eliminating the possibility of water logging.

(5) Fixing of crop pattern:—

⇒ Soil survey should be conducted to fix the crop pattern. ⇒ The crops having high rate of evaporation should be recommended for the area susceptible to water logging.

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(6) Providing drainage system:-

⇒ Suitable drainage system should be provided in the low lying areas so that the rain water does not stand for long days.

⇒ A network 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.

(7) Improvement of Natural drainage:-

⇒ Some times the natural drainage may be completely silted up or obstructed by weeds, aquatic plants, etc. The affected section of the drainage should be improved by excavating and clearing the obstructions.

(8) Pumping of ground water:-

⇒ A no. 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.

⇒ The lifted ground water may be utilised for irrigation or may be discharged to the river or any other water course.

(9) Construction of Sump Well:-

⇒ Sump wells may be constructed within the water logged area and may be pumped to the irrigable land or may be discharged to any river.

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Detection of Water-logging :-

→ When ground water reservoir goes on building up continuously it indicates water-logging.

→ When storage starts building up in intial stages the crop growth is actually increased because more water is available for crop growth.

→ After some time the water table rises very high & the land gets water logged. Finally land is rendered unproductive & infertile.

→ It develop in its full form slowly. So its early detection is possible by keeping a close water over the yield & also the variations in ground water level.

→ A comparative reduction in crop yield inspite of irrigation & fertilisation & early maturity of crop indicates the symptoms of water logging.

→ When harmful salts start appearing on the fields as white deposit it indicates that water logging is likely to follow.

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Remedies for Water-logging :-

a) Installation of lift irrigation System :

When lift Irrigation project in the form of tube well irrigation system is introduced in the water logged area the water table gets lowered sufficiently. It is found to be very successful method of reclaiming water logged land. Thus a combination of a canal system & a supplementary tube well irrigation system may be considered to be most successful & efficient irrigation scheme.

b) Implementation of Drainage Schemes :

The water logged area may be reclaimed by introducing overland & underground drainage schemes.

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5.0 DIVERSION HEADWORKS & REGULATORY STRUCTURE

Canal headworks:-

Any hydraulic structure which is constructed to form a storage reservoir or to raise the water level at the head of the canal, are known as Canal head works.

⇒ It may be of 2 types/forms.
(a) Storage head works
(b) Diversion head works.

(a) Storage head works:-

⇒ When a dam is constructed across a river valley to form a storage reservoir, it is known as storage reservoir.
⇒ The water is supplied to the canal from this reservoir through the canal head regulator.

(b) Diversion Head works:-

⇒ When a weir or barrage is constructed across a perennial river to raise the water level & to divert the water to the canal, then it is known as diversion head works.
⇒ The flow of water in the canal is controlled by canal head regulator.

5.1 Necessity of diversion Headworks:-

(a) To raise the water level at the head of the canal
(b) To form a storage by constructing dykes on both the banks of the river so that water is available throughout the year.
(c) To control the entry of silt into the canal.
(d) To control the fluctuation of water level in the river during different seasons.

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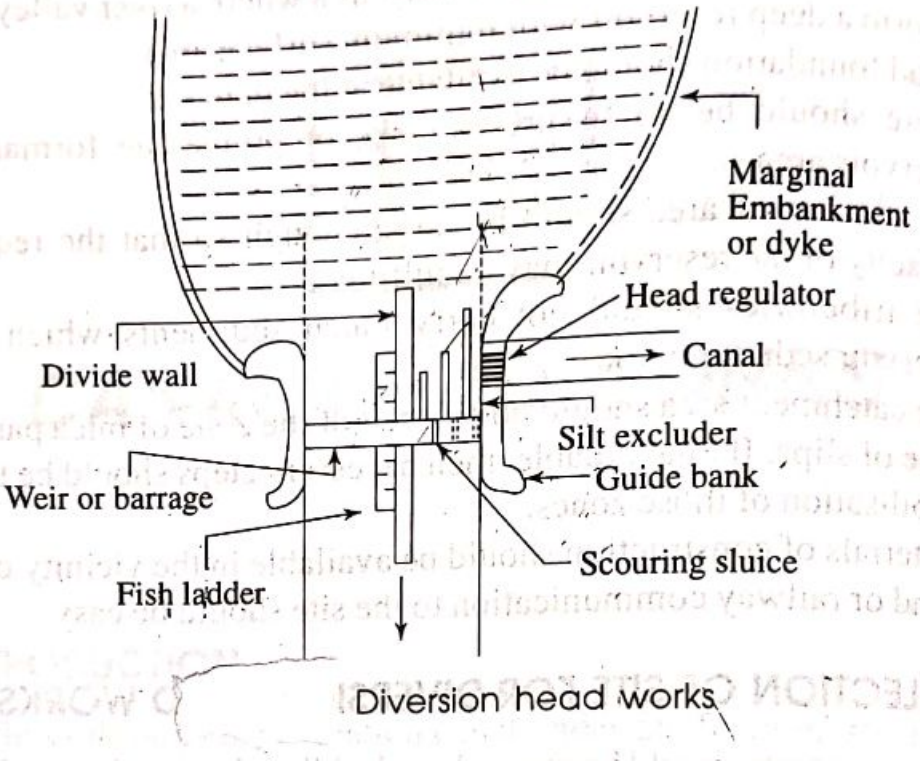

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Components Parts of Diversion Head Works

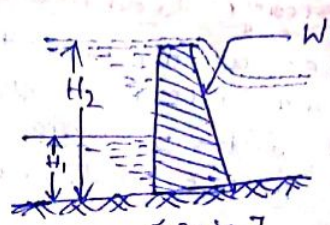
(a) Wein or Barrage

(i) **Wein:-** Normally, the water level of any perennial river is such that it cannot be diverted to the irrigation canal.

→ The bed level of the canal may be higher than the existing water level of the river.

→ In such case, a wein is constructed across the river to raise the water level from H_1 to H_2. in fig.

→ Then, the water can be easily diverted to the canal.



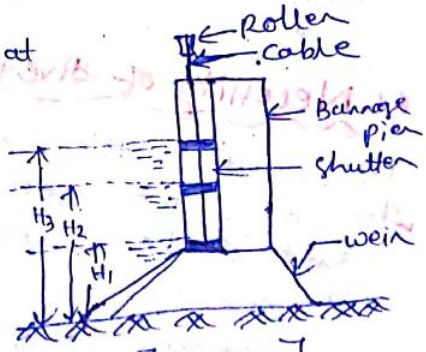
[Wein]

(ii) **Barrage:-**

→ When the water level on the up stream side of the weir is required to be raised to different levels at different time, then the barrage is constructed.

→ Practically a barrage is an arrangement of adjustable gates or shutters at different tiers over the wein.

→ The water level can be adjusted at H_1, H_2, H_3 --- etc. by operating the adjustable gates.



[Barrage]

(b) Divide Wall:-

→ The divide wall is a long wall constructed at right angles to the weir or barrage, it may be constructed with stone masonry or cement concrete.

→ On the up-stream side, the wall is extended just to cover the canal head regulators & on the down-stream side, it extended up to the launchy apron.

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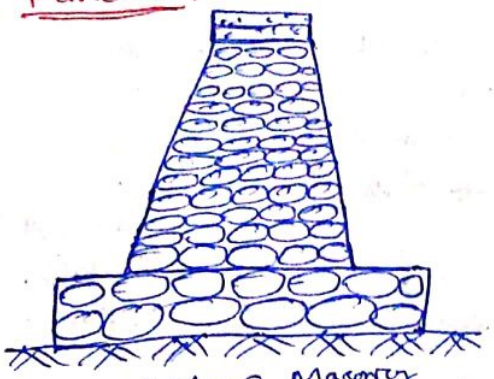
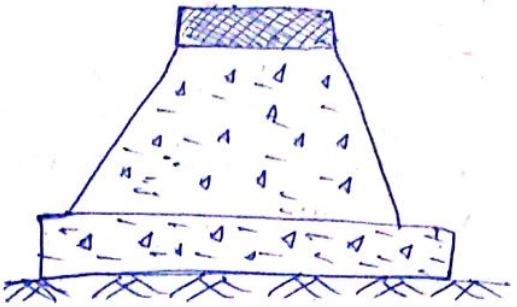
Function :-



(a) Stone Masonry



(b) Cement Concrete

(a) To form a still water pocket in front of the canal of the canal head so that the suspended silt can be settled down which then later be cleaned through the scouring sluices from time to time.

(b) It controls the eddy current or cross current in front of the canal head.

(c) It provides a straight approach in front of the canal head.

(d) It resists the overturning effect on the weir or barrage caused by the pressure of the impounding water.

(3) Scouring sluices or Under sluices :-

→ The scouring sluices are the openings provided at the base of the weir or barrage.

→ These openings are provided with adjustable gate.

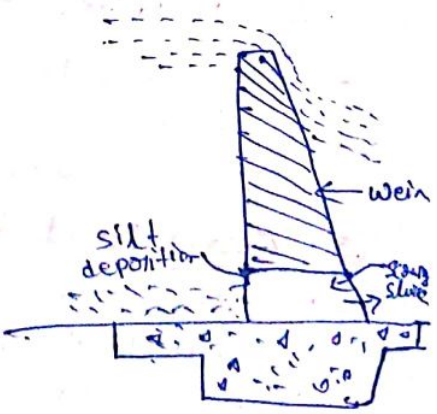
→ Normally, the gates are kept closed.

→ The suspended silt goes on depositing in front of the canal head regulator.

→ When the silt deposition becomes appreciable the gates are opened and the deposited silt is loosened with an agitator mounted on a boat.

→ The muddy water flows towards the downstream through the scouring sluices. The gates are then closed.

→ But, at the period of flood, the gates are kept opened.



Scouring sluice

fig.

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(4) Fish Ladder :-

⇒ The fish ladder is provided just by the side of the divide wall for the free movement of fishes.

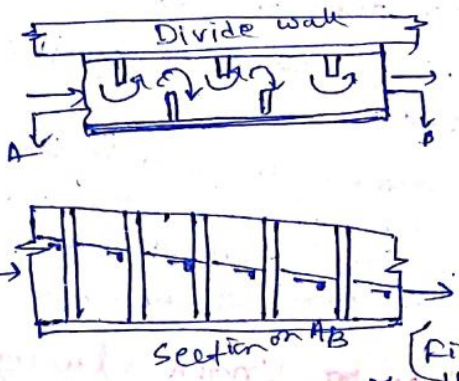
→ Rivers are the important source of fishes.

→ There are various types of fish in the river.

⇒ The nature of the fish varies from type to type. But in general, the tendency of fish is to move from upstream to downstream in winters & from downstream to upstream in monsoons.

⇒ This movement is essential for their survival.

→ Due to construction of weir or barrage, this movement gets obstructed, & is detrimental to the fishes.



(Fish ladder)

⇒ For the free movement of the fishes along the course of the river, the fish ladder is essential.

⇒ In the fish ladder, the baffle walls are constructed in a zig-zag manner so that the velocity of flow within the ladder does not exceed 3m/sec.

→ The width, length & height of the fish ladder depends on the nature of the river & the type of weir or barrage.

(5) Canal Head Regulator :-

→ A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator.

⇒ It consists of a no. of piers which divide the total width of the canal into a no. of spans which are known as bays.

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→ The piers consist of a no. of piers on which the adjustable gates are placed.

→ The gates are operated from the top by suitable mechanical device.

→ A platform is provided on the top of the piers for the facility of operating the gates.

→ Again some piers are constructed on the down stream side of the canal head to support the road-way.

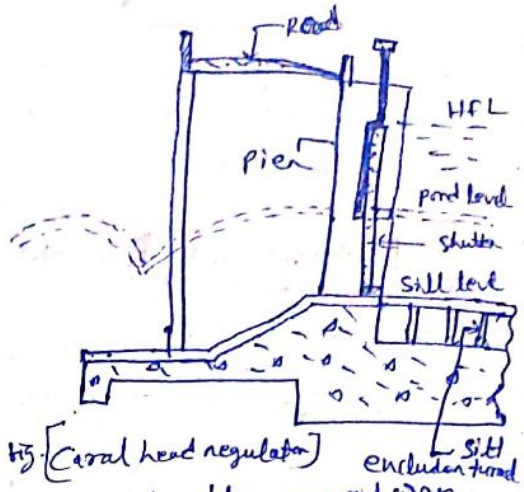

Fig: [Canal head regulator]

(6) SILT EXCLUDER:-

→ When silt water pocket is formed in front of the canal head by constructing the divide wall, then it is found that the lower layer of water contains heavy silt & the upper layer ~~consist~~ contains very fine silt.

→ The fine silt is very fertile & it may be allowed to enter the canal. But the heavy silt causes sedimentation in the pocket.

→ To eliminate the suspended heavy silt, the silt excluder is provided. It consists of a series of tunnels starting from the side of the head regulator up to the divide wall.

→ The tunnel nearest to the head regulator is longest & the successive tunnels decrease in length, the tunnel nearest to the divide wall is shortest.

→ The tunnels are covered by R.C.C. slab.

→ The top level of the slab is kept below the sill level of the head regulator.

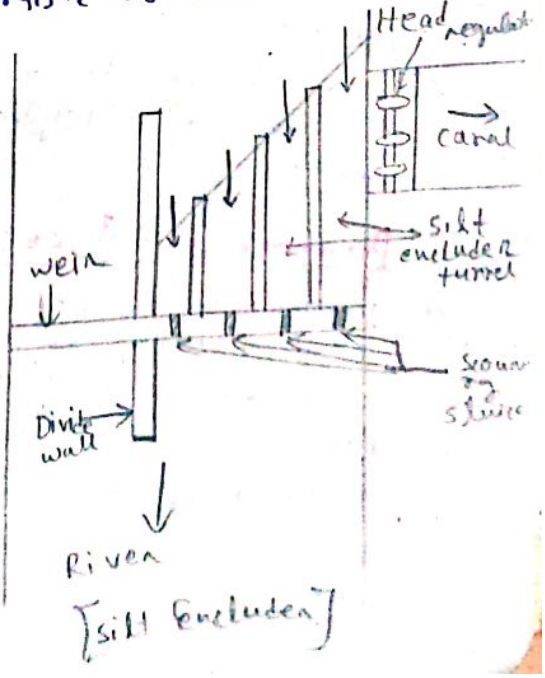

Fig: [Silt Excluder]

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→ So, the comparatively clean water is allowed to flow in the canal through the head regulator.

→ The suspended heavy silt carried by the water enters the silt excluder tunnels & passes out through the scouring sluice.

(7) Marginal Embankment or Dyke:-

→ The marginal embankment or dykes are earthen embankments which are constructed ||le to the river bank on one or both the banks according to the condition.

⇒ The top width is generally 3m to 4m. The side slope of the river side is generally $1\frac{1}{2}:1$ & that on the country side is 2:1.

⇒ The height of the embankment depends on the highest flood level.

⇒ A suitable margin is provided b/w toe of the embankment & the bank of the river.

⇒ To resist the effect of erosion on the embankment, wooden piles are driven along the river banks throughout the length of ~~the~~ dyke.

⇒ The length of the dyke depends on the area to be protected.

⇒ The river side slope of the dyke is protected by boulder pitching with cement grouting & the country side slope is protected by turfing.

Purpose of dyke:-

(a) It prevents the flood water or storage water from entering the surrounding area which may be submerged or may be water logged.

(b) It retains the flood water or storage water within a specified section.

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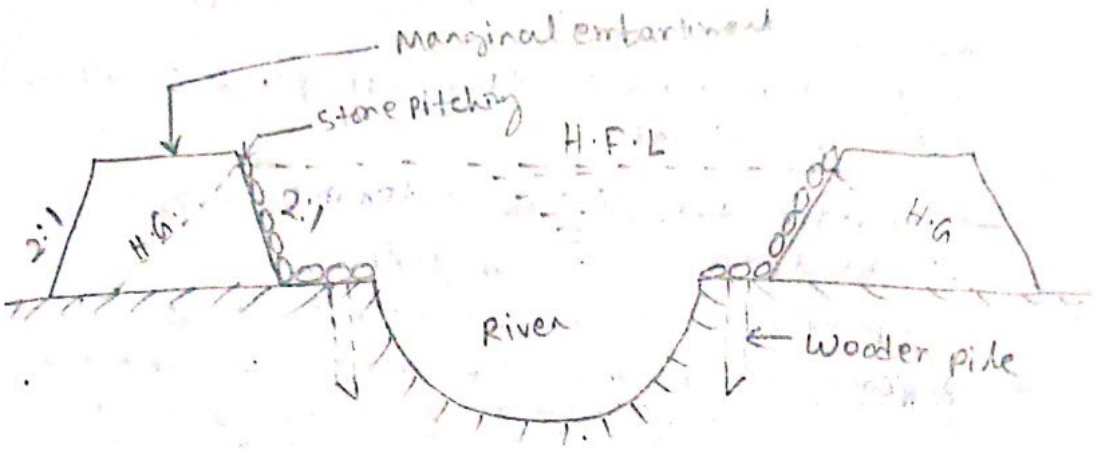
(c) It protects the towns & villages from devastation (destruction) during the heavy flood.

(d) It protects valuable agricultural lands.



[fig. Marginal embankment]

(8) Guide Bank :—

These are provided on either side of the diversion head work for a smooth approach (motion) (move near) & to protect the structure from erosion.

Barrage	Weir
→ ~~Low set~~ crest → high cost	→ ~~High set crest~~ → low cost
→ ~~Gates are of greater height~~	→ ~~Shutter in~~ part length
→ Gate are raised to clear off the high flood	→ Shutters are dropped to pass flood
→ Gate are of greater height	→ Shutters are of smaller height
→ Perfect control on river flow	→ No control of river in low flood.
→ Longer construction period	→ Shorter construction period.
→ ~~to~~ costly structure	→ Relatively cheaper structure.
→ Silt removal is done through under sluices	→ No means for silt disposal.
→ Gated over entire length	→ Shutters in part length.
→ Road & rail bridge can be constructed at low cost.	→ Not possible to provide road/rail bridge.

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Cross regulator:-

→ It is a structure constructed across a canal to regulate the water level in the canal upstream of itself & the discharge passing downstream of it for one or more of

Purpose:=> To feed off taking canals located upstream of cross regulator.

→ To control the flow of water in canal system

→ To feed the off taking canals.

→ To provide roadway for vehicular traffic

* A Regulator constructed in the main canal or parent canal d/s of an off take canal is called cross drainage regulator.

→ It is generally constructed at a dist of 9 to 12 km along the main canal & 6 to 10 km along the branch canal

Head regulator:-

→ It is a structure at the head of canal taking off from a reservoir may consist of no. of spans separated by piers & operated by gates.

→ When it is constructed at the head of main canal it is known as canal head regulator. And when it is constructed at the head distributary channel, it is called distributary head regulator.

* => Regulators are normally aligned at 90° to the weir.

Function:-

=> To control the entry of water either from the reservoir or from the main canal.

=> To control the entry of silt into off taking or main canal.

=> To serve as a meter for measuring discharge of water.

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CANAL FALLS

Introduction:-

→ Irrigation canals are constructed with some permissible bed slopes so that there is no silting or scouring in the canal bed. But it is not always possible to run the canal at the desired bed slope throughout the alignment due to the fluctuating nature of the country slope.

→ Generally, the slope of the natural ground surface (i.e. country slope) is not uniform throughout the alignment.

→ Some times, the ground surface may be steep & sometimes it may be very irregular with abrupt (rapid) change of grade.

→ In such cases, a vertical drop is provided to step down the canal bed and then it is continued with permissible slope until another step down is necessary.

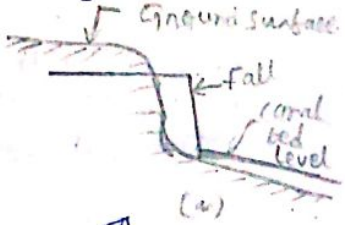
→ This is done to avoid unnecessary huge earth work in filling. Such vertical drops are known as canal falls or simply falls.

Necessity of Canal falls:-

→ The canal falls are necessary in case the following conditions occur:-

(a) When the slope of the ground suddenly changes to steeper slope, the permissible bed slope cannot be maintained. It requires excessive earthwork in filling to maintain the slope.

→ In such case canal falls are provided to avoid excessive earth work in filling.



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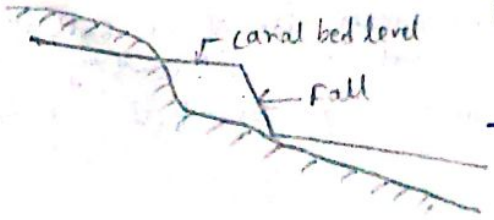

→ When the slope of the ground is more or less uniform & the slope is greater than the permissible bed slope of canal.

→ In that case also the canal falls are necessary.

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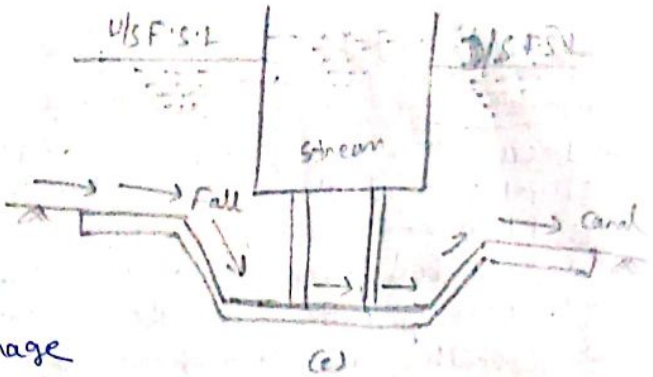
(c) In cross-drainage works when the difference b/w bed level of canal & that of drainage is small or when the F.S.L. of the canal is above the bed level of drainage then the canal fall is necessary to carry the canal water below the stream or drainage (i.e in case of siphon super passage)



(c)

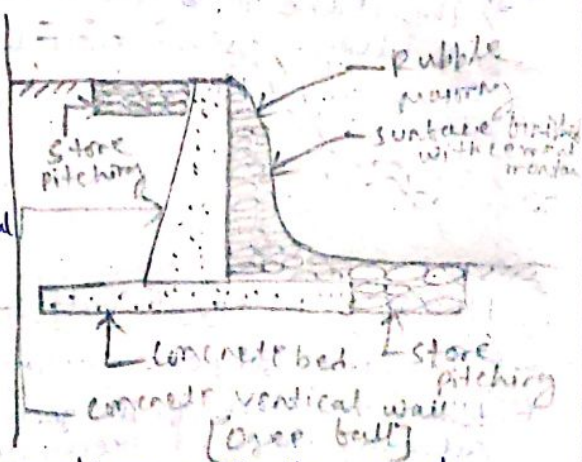
TYPES OF CANAL FALLS

→ The types of canal falls that may be adopted according to the site condition.

Ogee Fall:-

→ In this type of fall, an ogee curve (a combination of convex curve & concave curve) is provided for carrying the canal water from higher level to lower level.

→ This fall is recommended when the natural ground surface suddenly changes to a steeper slope along the aligment of the canal.



→ The fall consists of a concrete vertical wall & concrete bed.

→ Over the concrete bed the rubber masonry is provided in the shape of ogee curve (inverted oblong 'S').

→ The surface of the masonry is finished with rich cement mortar (1:3).

→ The upstream and down stream side of the fall is protected by stone pitching with cement grouting.

→ The design consideration of the ogee fall depends on the site condition.

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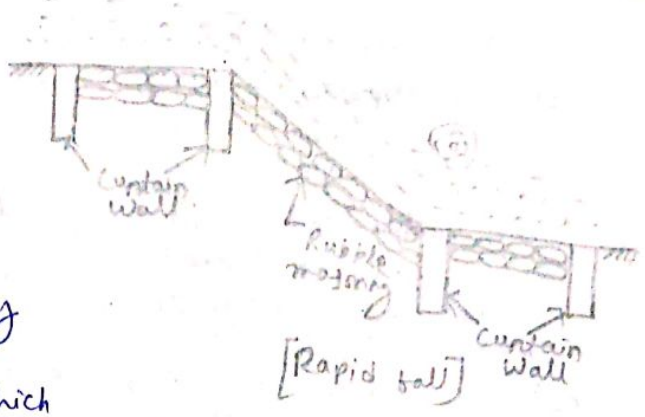
Rapid fall :-



[Rapid fall]

⇒ The rapid fall is suitable when the slope of the natural ground surface is even and long.

⇒ It consists of a long sloping glacis with longitudinal slope which varies from 1 in 10 to 1 in 20.

⇒ Curtain walls are provided on the upstream and downstream side of the sloping glacis.

⇒ The sloping bed is provided with rubber masonry.

⇒ The upstream and downstream side of the fall is also protected by rubble masonry.

⇒ The masonry surface is finished with rich cement mortar (1:3).

Stepped Fall :-

⇒ Stepped fall consists of a series of vertical drops in the form of steps.

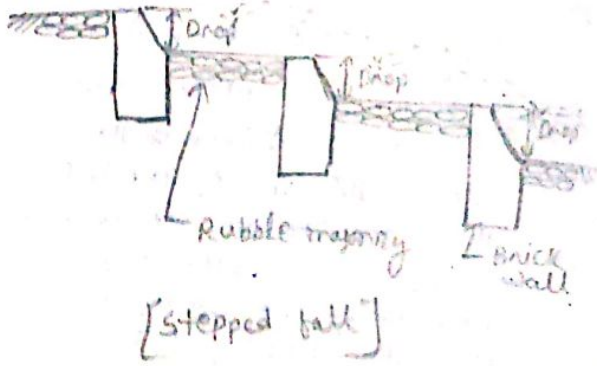
⇒ This fall is suitable in places where the sloping ground is very long and requires long glacis to connect the higher bed level with lower bed level.

⇒ This fall is practically a modification of the rapid fall.

⇒ Here, the sloping glacis is divided into a no. of drops so that the flowing water may not cause any damage to the canal bed.

⇒ Brick walls are provided at each of the drops.

⇒ The bed of the canal within the fall is protected by rubble masonry with surface finishing by rich cement mortar (1:3).



[Stepped fall]

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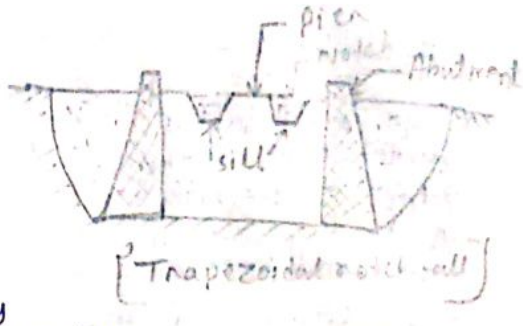
Trapezoidal Notch fall



[Trapezoidal notch fall]

→ In this type of fall a body wall is constructed across the canal.

→ The body wall consists of several trapezoidal notches b/w the side piers & the intermediate pier or piers.

→ The sills of the notches are kept at the upstream bed level of the canal.

→ The body wall is constructed with masonry or concrete.

→ An impervious floor is provided to resist the scouring effect of the falling water.

→ The upstream & downstream side of the fall is protected by stone pitching finished by cement grouting.

→ The size & no. of notches depend upon the full supply discharge of the canal.

Vertical Drop fall or Sarda Fall

→ It consists of a vertical drop wall which is constructed with masonry work.

→ The water flows over the crest of the wall.

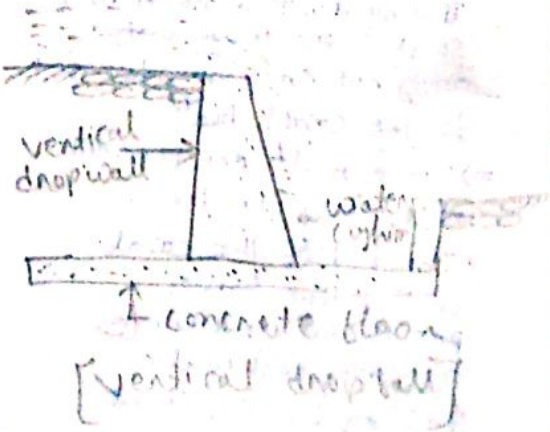
→ A water cistern is provided on the down-stream side which acts as a water cushion to dissipate the energy of falling water.

→ A concrete floor is provided on the down-stream side to control the scouring effect of the flowing water.

→ Cutoff walls are provided on the upstream & D/s side.

→ Stone pitching with cement grouting is provided on the U/s & D/s side of the fall to protect it from scouring.

→ This type of fall were provided on the Sarda Canal in UP. Hence, it is sometimes known as Sarda fall.



[Vertical drop fall]

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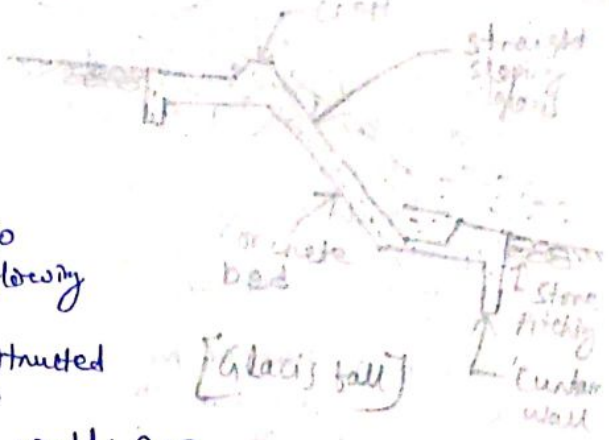
Glacis fall :-

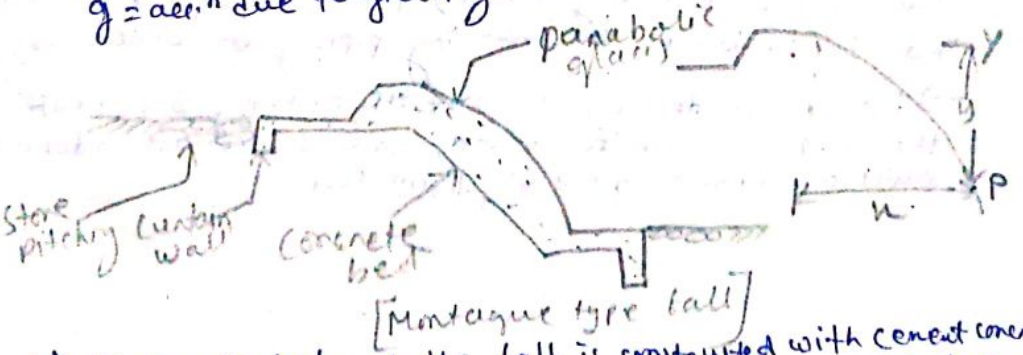


→ It consists of a straight sloping glacis provided with a crest.
→ A water cushion is provided on the d/s side to dissipate the energy of flowing water.
→ The sloping glacis is constructed with cement concrete.
⇒ Curtain walls & toe walls are provided on the upstream & downstream side.
⇒ The space b/w the toe walls & curtain walls is protected by stone pitching.
⇒ This type of fall is suitable for drops upto 1.5 m.
⇒ For the improvement in energy dissipation, the glacis falls have been modified as follows.

(a) Montague type fall :-

→ In this type of fall, the straight sloping glacis is modified by giving parabolic shape which is known as Montague profile.
→ Taking 'O' as the origin, the Montague profile is given by the eqn $x = V\sqrt{\frac{4y}{g}} + y$

where, x = distance of pt. P from OX axis, y = distance of point P from OY axis, V = velocity of water at the crest, g = acc.n due to gravity.



⇒ The main body of the fall is constructed with cement concrete.
→ Toe walls are same as in the case of straight sloping glacis.
→ The bed protection by stone pitching is also same.

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(b) Inglis type fall :-

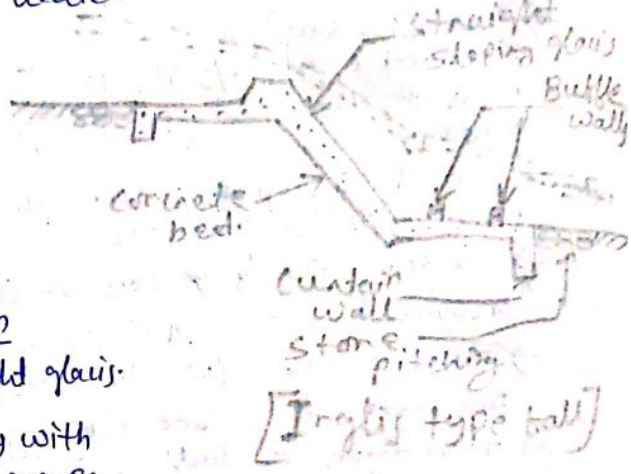
→ In this type of fall, the glacis is straight & sloping but baffle walls are provided on the d/s floor to dissipate the energy of flowing water.

→ The height of baffle depends on the head of water on the U/s side.

→ The main body of the fall is constructed with cement concrete.

→ The toe walls & curtain wall are same as straight glacis.

→ The protection works with stone pitching are also same.



[Inglis type fall]

⇒ Some times, this fall is known as baffle fall.

ENERGY DISSIPATION :-

Def? :- When water spills and flows over the spillways, then it acquires a very high velocity, as the whole potential energy is transformed into KE. The process of destruction of this KE is known as energy dissipation.

Purpose of Energy Dissipation :-

→ The high velocity of flow over the spillway has a tendency to erode the bed of the river at the toe of the spillway & thus the stability of the spillway may be in danger by forming piping or undermining.

→ Hence to destroy the Kinetic Energy & to resist the erosion of the river bed, some devices are adopted which are known as energy dissipators.

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6.0 CROSS-DRAINAGE WORKS

Introduction:- When the network of main canals, branch canals, distributories, etc are provided, then these canals may have to cross the natural drainage like rivers, streams, nallahs, etc at different points within the command area of the project.

→ The crossing of the canals with such obstracles cannot be avoided. So, suitable structures must be constructed at the crossing point for the easy flow of water of the canal & drainage in the respective directions. These structures are known as cross drainage works.

Necessity of cross-drainage works:-

→ The water-shed canal donot cross natural drainages But in actual orientation of the canal network, this ideal condition may not be available & the obstacles like natural drainages may be present across the canal. So, the cross drainage work must be provided for running the irrigation system.

(b) At the crossing point, the water of the canal & the drainage get intermixed. So, for the smooth running of the canal with its design discharge the cross drainage works are required.

(c) The site condition of the crossing point may be such that without any suitable structure, the water of the canal & drainage cannot be diverted to their natural directions. → So, the cross drainage works must be provided to maintain their natural dirn of flow.

Types of cross-drainage works:-

According to the relative bed levels, maximum water levels & relative discharges of the canals & drainages the cross drainage works may be of the following types:-

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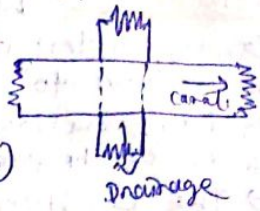
Type-I :- Irrigation canal passes over the drainage :-

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 clearly below the canal.

(b) Siphon Aqueduct :-

→ It a hydraulic structure where the canal is taken over the drainage, but the drainage water cannot pass clearly below the canal.

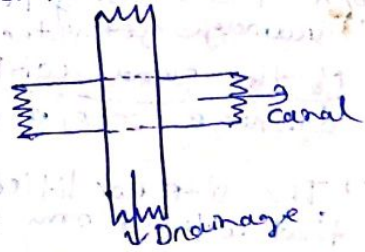
→ It flows under siphonic action. So, it is known as siphon aqueduct.

→ This structure is suitable when the bed 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 drainage is above the full supply level of the canal.

→ The water of the canal passes clearly 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.

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Type-III :- Drainage & Canal Intersection Each other at the same level :-

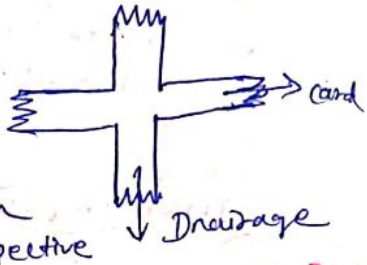
This cond.n involves the constr.n of the following :-

(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 level crossing.

→ This is suitable for the crossing of large ~~discharge~~ drainage with main canal.

(b) Inlet and Outlet :-



→ In ~~case~~ 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 direction.

→ This arrangement is known as inlet & outlet :

AQUEDUCT :-

→ An aqueduct is just like a bridge where a canal is taken over the deck supported by piers instead of a road or railway.

→ Generally, the canal is in the shape of a rectangular though 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 ~~of~~ & 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 & section of piers are designed acc. to the highest flood level & velocity of flow of the drainage.

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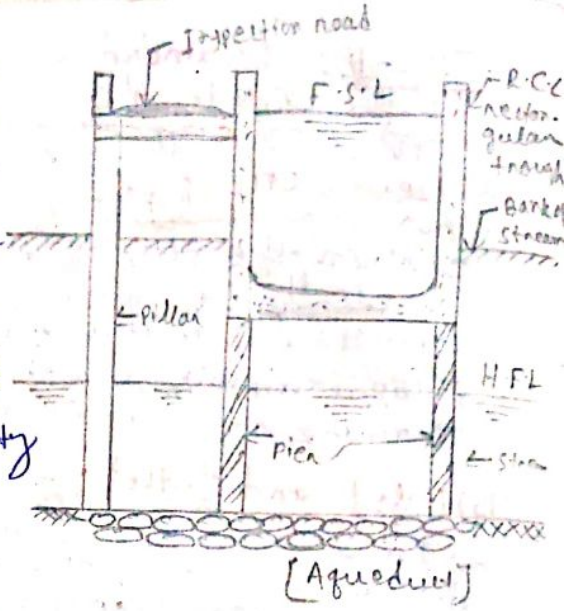
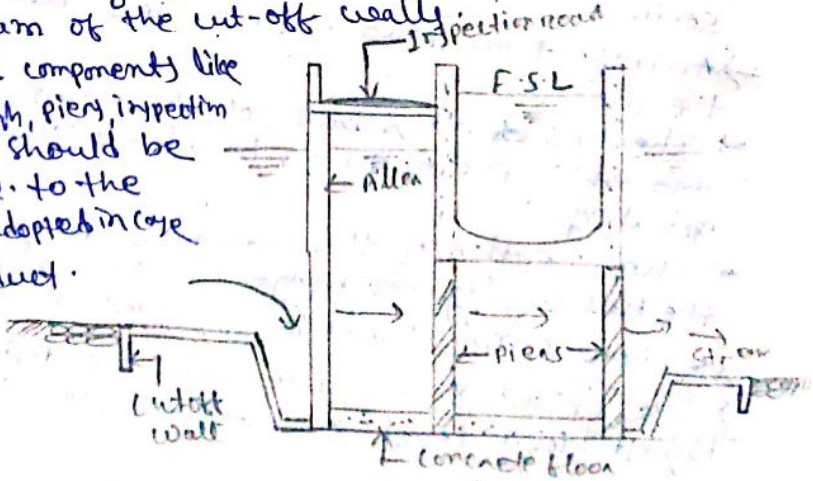
⇒ The piers may be of brick masonry or reinforced cement concrete.
⇒ Here, deep foundation is not necessary for the piers.
⇒ The concrete foundation may be done by providing the depth of foundation according to the availability of hard soil.



SIPHON AQUEDUCT :-

⇒ The Siphon aqueduct, the bed of the drainage is depressed below the bottom level of the canal through by providing sloping apron on both sides of the crossing.
⇒ The sloping apron may be constructed by stone pitching or cement concrete.
⇒ The section of the drainage below the canal through 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 & downstream of the cut-off walls.
⇒ The other components like canal trough, piers, inspection road, etc. should be designed acc. to the methods adopted in case of aqueduct.



[fig. Siphon aqueduct]

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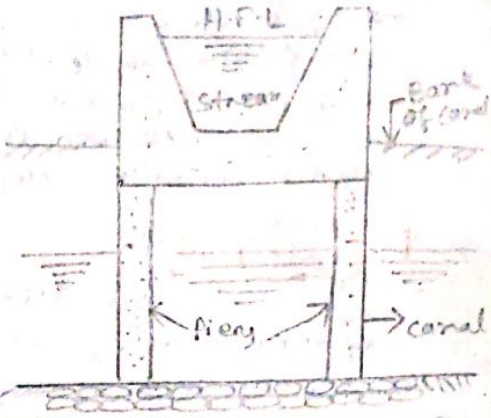
SUPER PASSAGE :-

⇒ 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 or trapezoidal trough of channel which is constructed on the deck supported by piers.



[Super passage]

⇒ A free board of about 1.5m should be provided for safety.

⇒ The trough should be ~~provided~~ constructed of reinforced cement concrete.

⇒ The bed & banks of the canal below the drainage trough should be protected by boulder pitching or lining with concrete slabs.

⇒ The foundation of the piers will be same as in the case of aqueduct.

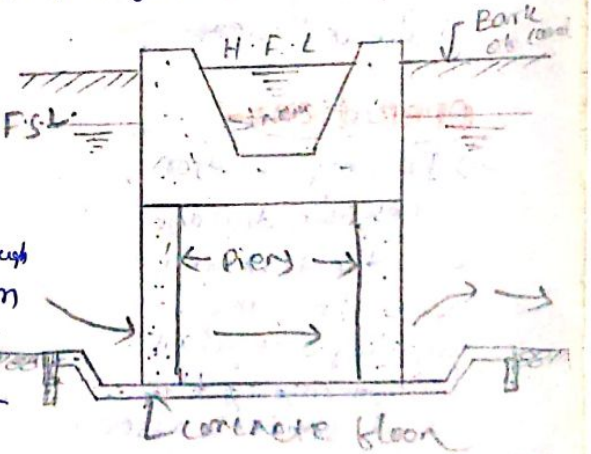
Siphon Super passage :-

⇒ In this case, the canal passage below the drainage ~~thaou~~ trough.

⇒ The section of the trough is designed acc. to high flood discharge.

⇒ The bed of the canal is depressed below the bottom level of the drainage trough by providing sloping apron on both sides of the crossing.

⇒ The sloping apron may be constructed with cement concrete in the form of tunnel which acts as siphon.



[Siphon Super passage]

⇒ Cut-off walls are provided on upstream & downstream side of sloping apron.

⇒ Other components are same as in the case of siphon aqueduct.
↳ like pier, inspection road etc.

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LEVEL CROSSING :-

⇒ The 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.

(1) 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 the full supply level of the canal..

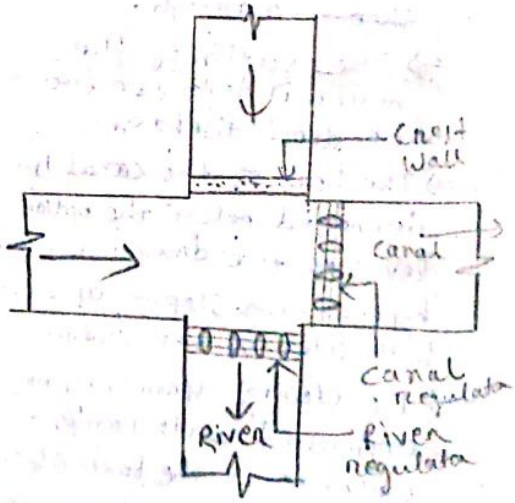
(2) Drainage Regulator :-

⇒ It is provided across the drainage just at the down-stream side of the crossing point. The regulator consist of adjustable shutters at different tiers.

(3) Canal regulator :-

⇒ It is provided across the canal just at the down-stream side of the crossing point.

⇒ This regulator also consists of adjustable shutters at different tiers.



[Level crossing]

Operation :-

⇒ In dry season, when the discharge of the drainage is very low, the drainage regulator is kept closed & the canal water is allowed to flow as usual.

⇒ In rainy season, when the discharge of the drainage is very high, the drainage regulator is kept completely open & the canal regulator is adjusted according to requirement. The level crossing is recommended for the crossing of main canal with large drainage.

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INLET AND OUTLET :-

→ In case of crossing of a ~~canal~~ small irrigation ~~canal~~ channel with a small drainage, no hydraulic structure is constructed.

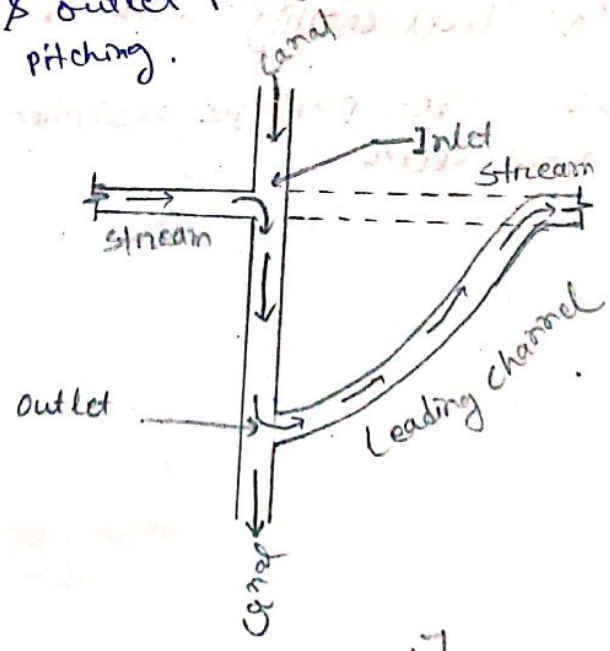
⇒ Because, the discharges of the drainage & the channel are practically low and these can be easily tackled by easy system like inlet & outlet arrangement.

⇒ In this system an inlet is provided in the channel bank simply by open cut and the drainage water is allow to join the channel.

⇒ Then at a suitable point on the downstream side of the channel an outlet is provided by open cut & the water from the irrigation channel is allowed to flow through a leading channel towards the original ~~channel~~ course of the drainage.

→ At the points of inlet & outlet the bed & banks of the drainage are protected by stone pitching.

⇒ The bed and banks of the irrigation channel b/w inlet & outlet points should also be protected by stone pitching.



[Inlet and Outlet]

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7. DAM

Def^n:- It is a solid barrier which is constructed across a river valley to form a deep storage reservoir is known as dam.

Need & Necessity

→ It is meant for multipurpose function.
→ Irrigation
→ water for domestic consumption (water supply)
→ Drought & flood control.
→ For navigational facilities.
→ Hydro electric power generation
→ Recreation
→ Fishery

Types of Dam :-

Dams may be classified on the following basis

(a) Based on material 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 material, etc.

→ It is designated as (a) earthen dam, (b) rock-fill dam, (c) Composite dam.

(b) 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.

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(2) Arch dam:-

⇒ It is curved masonry or concrete dam with 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 no. of buttresses in the form of triangular concrete wall.
⇒ It resists the forces acting on it by the buttress.

(4) Embankment dam:-

⇒ It is non rigid dam constructed simply by earth work in trapezoidal section.
⇒ Sometimes, it may be of earthwork with clay core wall or rock fill.
⇒ It resists forces acting on it by its shear strength.

(C) Based on function:-

(1) Storage dam:-

⇒ It is constructed to form a reservoir in which the water is stored during the period of rainy season or flood & ~~utilised~~ utilised for the irrigation in the period of draught.
⇒ The water is also utilised for the generation of hydroelectric power, water supply, etc.

(2) Detention Dam:-

⇒ It is mainly constructed to detain (keep), slow down the flood water temporarily in a reservoir & then release gradually so that the downstream area may not be damaged due to sudden flood water.

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(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 piling for excluding water for the sake of construction of well foundation in (pier foundation) then it is known as coffer dam.

[D] Based on Hydraulic Behaviour

(1) Over flow dam :- The dam which consists of crest shutter or 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 spillways are provided to discharge the surplus water & the water is not allowed to flow over the crest is known as non-overflow dam.

EARTHEN DAM

→ Earthen dams are constructed purely by earth work in trapezoidal section.

→ These are most economical & suitable for weak foundation.

⇒ Earthen dams are classified as following :-

Based on Method of construction :-

Rolled fill Dam :-

→ In this method, the dam is constructed in successive layers of earth by mechanical compaction.

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→ The selected soil is transported from borrow pits and laid on the dam section to layers of about 45cm. The layers are thoroughly compacted by rollers of recommended weight & type.

⇒ When the compaction of one layer is fully achieved, the next layer is laid and compacted in the usual way.

⇒ The designed dam section hence is completed layer by layer.

Hydraulic fill Dam:-

→ In this method, the dam section is constructed with the help of water.

→ Sufficient water is poured in the borrowpit & by pugging thoroughly, slurry (semi liquid mixture → cement) is formed.

→ This slurry is transported to the dam side by pipe line & discharged near the upstream & downstream faces of the dam.

→ The coarser material gets deposited near the face and the finer material move towards the centre to get deposited there.

→ Thus the dam section is formed with faces of coarse material and central core is of impervious material like clay and silt. In this case, compaction is not necessary

Semi-Hydraulic fill Dam :-

⇒ In this method the selected earth is transported from the borrowpit and dumped within of the dam, as done in the case of rolled fill dam.

↔ While dumping no water is used. But after dumping the water jet is forced on the dumped earth.

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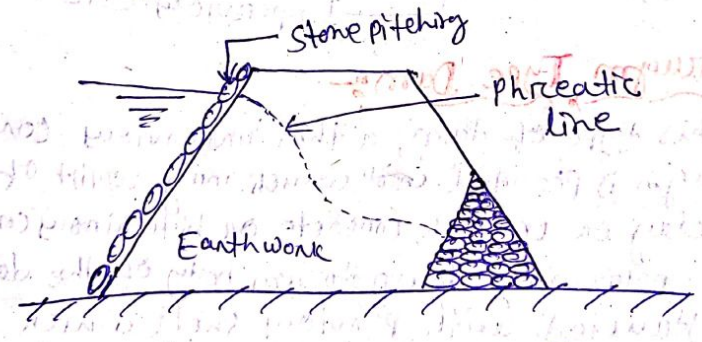
→ Due to the action of water the finer materials move towards the centre of the dam and an impervious core is formed with fine materials like clay.
→ The outside body is formed by coarse material, In this case also compaction is not necessary.

Homogeneous Type Dam :-

→ This type of dam is constructed purely with 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 gradient of the seepage line.
→ The phreatic line (top level of seepage line) should pass well within the body of the dam.
→ This type of dam is completely pervious. The upstream face of the dam is protected by ~~ser~~ stone pitching.
→ Now-a-days, the earthen dam is modified by providing horizontal drainage blanket or rock toe.



[Homogeneous type dam]

Zoned type Dam :-

→ This type of dam consists of several materials, the impervious core is made of puddle clay (by sprinkler sand to make plastic state) and the outer ~~pa~~ pervious shell is constructed with the mixture of earth, sand, gravel, etc.

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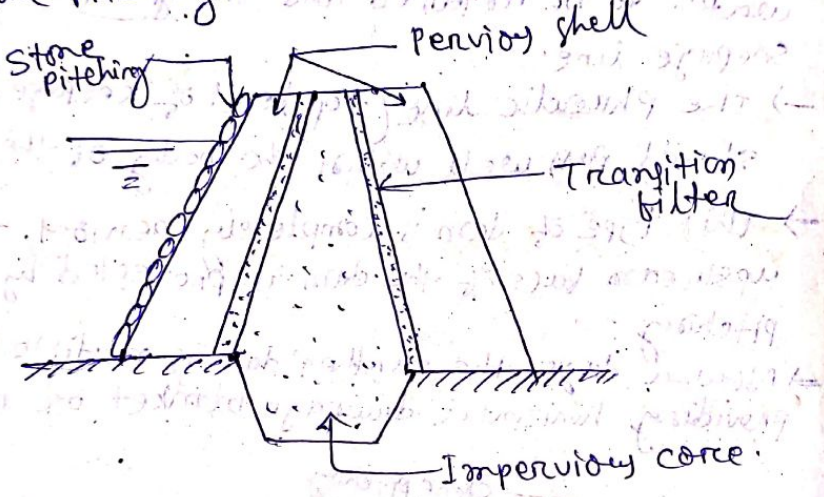
→ The core (central) is trapezoidal in section and its width depends on the seepage characteristics of the soil mixture on the upstream side.
→ The core is extended below the base of the dam to control the sub-soil seepage.
→ Transition filters are provided on both sides of the impervious core to control the seepage.
→ The transition filter is made of gravel and coarse sand.
→ The upstream face of the dam is protected by stone pitching.



Diaphragm Type Dam:-

→ In this type of dam, a thin impervious core or diaphragm is provided which may consist of puddle clay or cement concrete or bituminous concrete.
→ The upstream & downstream body of the dam is constructed with pervious shell which consists of the mixture of soil, sand, gravel, etc.
→ The thickness of the core is generally less than 3m.
→ A blanket of stone is provided on the toe of the dam for the drainage of the seepage water without damaging the base of the dam.

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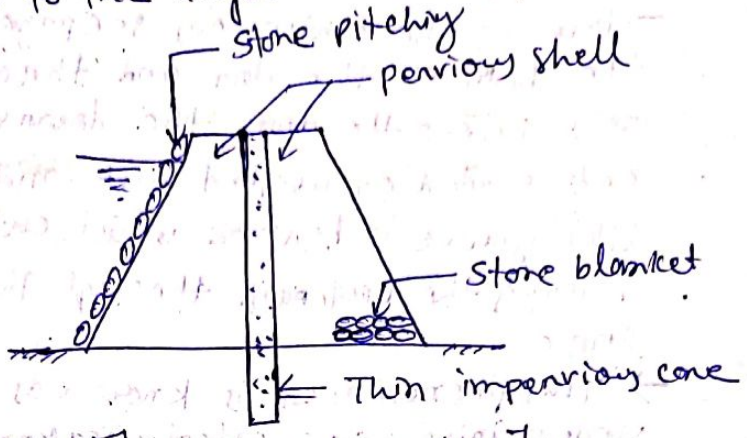
⇒ The up-stream face is protected by stone pitching.

⇒ The side slope of the dam should be decided according to the angle of repose of the soil mixture.



[Diaphragm type dam]

Causes of Failure of Earthen Dam

The failure of the earthen dam may be caused due to the reasons;

(1) Hydraulic Failure:- This type of failure may be caused by:

(a) Overtopping:- If the actual flood discharge is much more than the estimated flood discharge or the free board is kept insufficient or there is settlement of the dam or the capacity of spill way is insufficient then it results in the overtopping of the dam.

⇒ During the overtopping the crest of the dam may be washed out and the dam may collapse.

(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.

⇒ The downstream side also may be damaged by ~~the~~ tail water, rain water, etc. ⇒ The toe of the dam may also get damaged by the water flowing through the spill ways.

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(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.

→ This phenomenon is known as piping or undermining. This effect weakens the dam & ultimately causes the failure of the dam.

(b) Sloughing :-

⇒ The crumbling (breaking up into small fragments) of the toe of the dam is known as sloughing.

→ When the reservoir runs full, for a longer time, the downstream base of the dam remains saturated.

→ Due to the force of the seepage water the toe of the dam goes on crumbling gradually. Ultimately the base of the dam collapse.

(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.

→ The dam goes on depressing gradually and then overtopping occurs which leads to the failure of the dam.

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(b) Damage by burrowing animals:—

→ Some borrowing animals like crawfish, snakes, squirrel, rats, etc cause 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 & the dam may eventually collapse.

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Protection Measures of Earthen dams :—

The dam safety can be ensured if the following aspects are taken care of :-

1) Hydraulic Failure :
   → Such type of failure can be averted by providing
   a) Adequate spillway Capacity.
   b) Adequate freeboard so that dam safety is not endangered by overtopping during high floods.
   c) Proper maintenance of gates so that they are always operative & do not get clogged.

2) Seepage Failure :
   → Seepage failure can be taken care by
   a) Providing filter at the toe to minimize movement of the material.
   b) Seepage line is well within the body of dam.
   c) Provision of settlement, after the composition of the dam, be made from a normal 1% of ht. to a max. of 6%.

3) Structural Failure :—
   → In spite of best geological & foundation investigations done prior to dam construction, geological problems may arise such as induced seepage,

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earth tremors, slides, gougy seams & sloughing in the vicinity of dam & reservoir area surface during the construction or several years after the reservoir filling.

→ Periodic geotechnical inspection is essential for early detection & resolution of potential problems, besides provision of adequate rip-rap & its maintenance & draw down within permissible limits.

Earthquake Failure :-

→ Necessary provisions shall be made in the design of a dam to account for additional forces due to earthquakes.

Gravity Dams :-

It is a dam which is constructed from concrete or stone masonry & designed to hold back water by primarily using the weight of the material alone to resist the horizontal pressure of water pushing against it.

Gravity dams are designed so that each section of dam is stable & independent of any other dam section.

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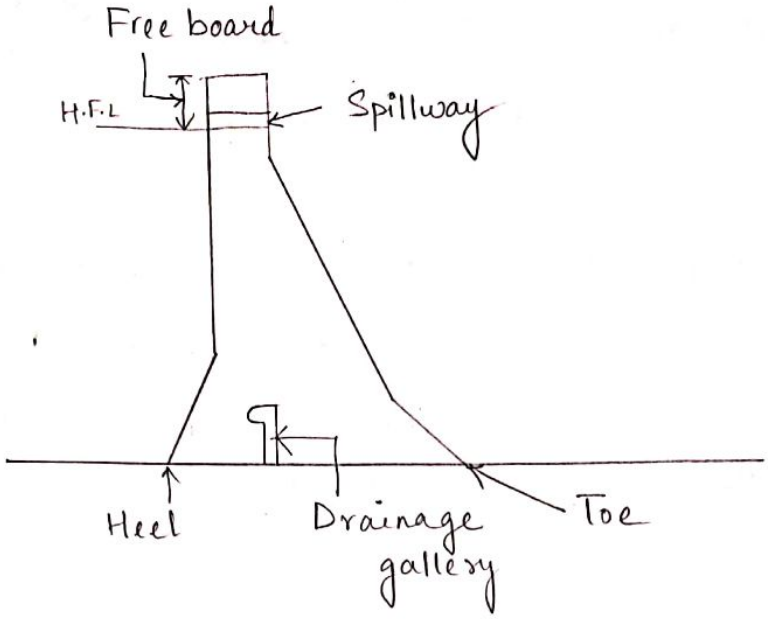

Causes of failure of Gravity Dam :-

i) By Over Turning :-

The 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 case, the resultant force passes through a point outside the middle-third of the base of dam. Over turning may be caused at the downstream edge of any horizontal section.

ii) By sliding :-

The total horizontal forces acting on the dam tend to slide the entire dam at its base or along any horizontal section of dam. The sliding may occur when total horizontal forces acting on dam are greater than the combined resistance offered by

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shearing resistance of the joint & the static friction.

iii) By Over Stressing :-

If the permissible working compressive stress of concrete or masonry exceeds due to some adverse conditions, then the dam may fail by crushing due to over stressing of concrete or masonry.

iv) By cracking :-

The tensile stresses should not be allowed to develop on the upstream face of dam. If due to some reasons, the tension is developed in the dam section, crack will form in the body of dam & ultimately this will cause the failure of dam.

Protection Measures for Gravity Dams :-

→ To avoid overturning, the resultant of all forces acting on the dam should remain within the middle third of base width of dam. This condition should be achieved in both the cases, when the reservoir is full & also when it is empty.

→ In the 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 :-

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$$\tan\theta = \frac{\Sigma P}{\Sigma W}$$

& $\tan\theta < \mu$

where,

ΣP = sum of horizontal forces

ΣW = sum of vertical forces

μ = coefficient of friction of material.

→ In dam section, the comp. stresses of concrete or masonry should not exceed the permissible working stresses to avoid failure due to crushing.

→ There should be no tension in dam section to avoid the formation of cracks. This condition may be achieved by maintaining the middle third rule.

→ The factor of safety should be taken 4 to 5.

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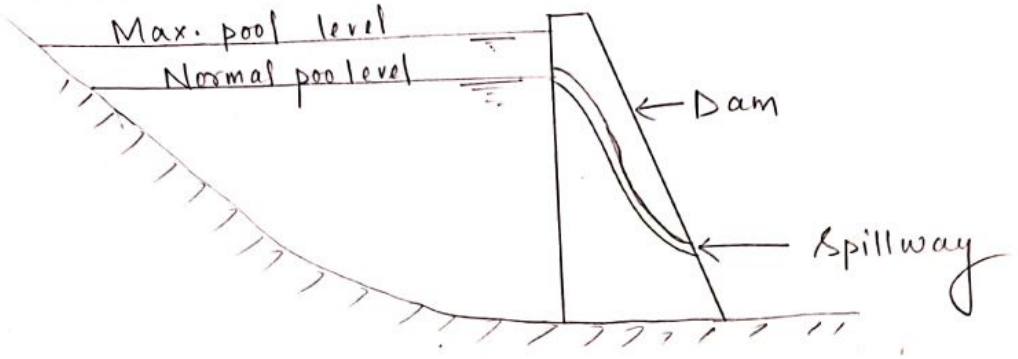
Spillway :-

These are opening provided at the body of dam to discharge safely the excess water or flood water when the water level rises above the normal pool level.



Necessity of Spillway :-

i) The ht. of dam is always fixed according to the max. reservoir capacity. The normal pool level indicates the max. capacity of reservoir. The water is never stored in the reservoir above this level. The dam may fail by over turning so, for the safety of dam the spillways are essential.

ii) The top of dam is generally utilised by making road. The surplus water is not be allowed to over top the dam, so to stop the over topping by the surplus water, the spillways become extremely essential.

iii) To protect the downstream base & floor of dam from the effect of scouring & erosion, the

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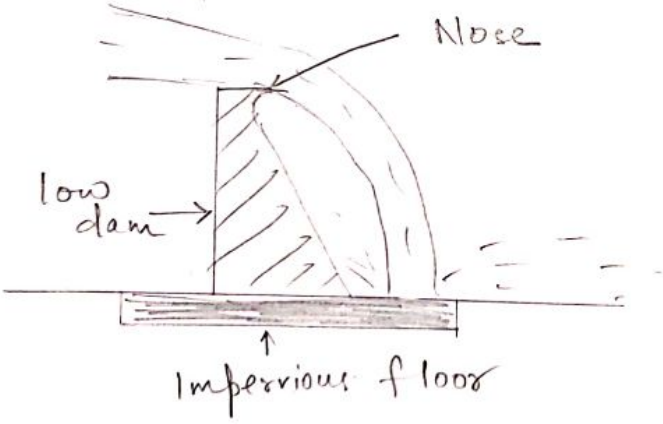
spillways are provided so that the excess water flows smoothly.

Types of Spillways :-

i) Drop spillway

In this spillway, the over flowing water falls freely & almost vertically on downstream side of structure. It is suitable for weirs or low dams. The spillway crest is provided with nose so that the water jet may not strike the downstream base of structure. Imperious apron is provided in downstream side to protect from scouring.

It is not suitable for high dam, because the water will have high impact force for which massive protection works will be required.



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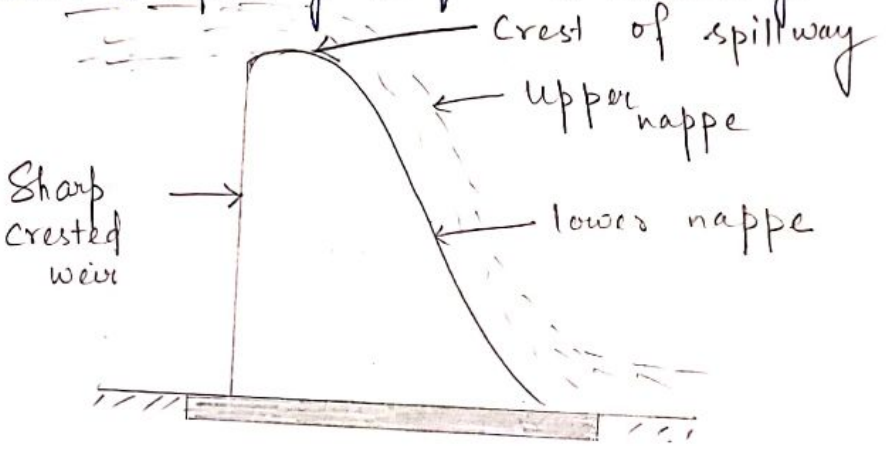
ii) Ogee Spill Way

It is modified form of drop spillway. The down stream profile of spillway is made to coincide with the shape of lower nappe of free falling waterjet from sharp crested weir. If the spillway runs with max. head, the overflowing water just follows the curved profile of spillway & there is no gap between the water & spillway surface & discharge is max.



iii) Chute or Trough Spill Way

It is a rectangular open channel or trough provided on the dam to discharge the surplus water from the reservoir to the same river on downstream side. It is constructed by joining pre-cast R.C.C channels in a longitudinal slope of 1 in 4 or 1 in 6. The channel is supported on pillars. The section of channel is designed according to volume of surplus water or flood discharge.

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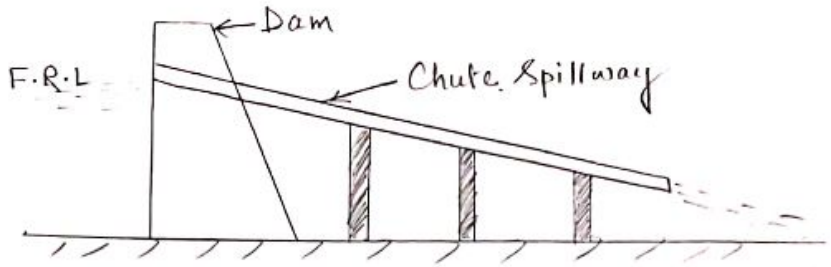
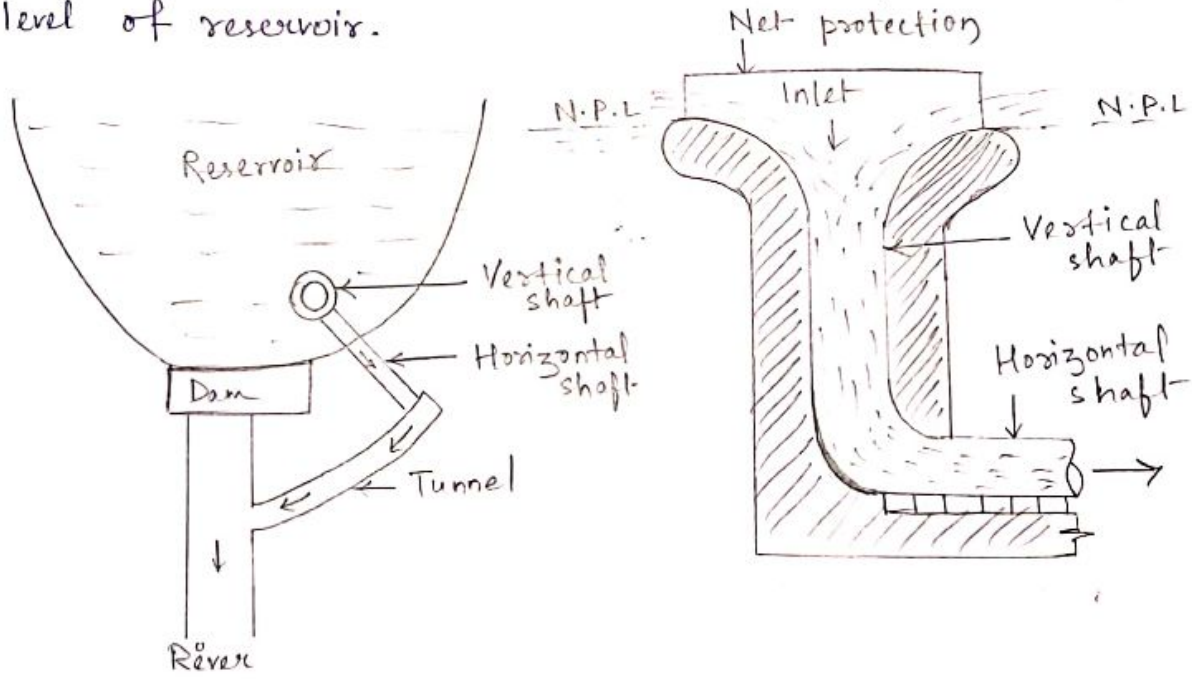


iv) Shaft Spillway :-

It consist of vertical shaft constructed with masonry work or plain cement concrete or R.C.C on the bed of reservoir just at upstream side of dam. The inlet mouth is conical. The vertical shaft is connected with horizontal shaft. This horizontal shaft may be taken through the body of dam or through base of dam or may be connected to a tunnel outside the dam. The inlet mouth is kept at normal pool level of reservoir.



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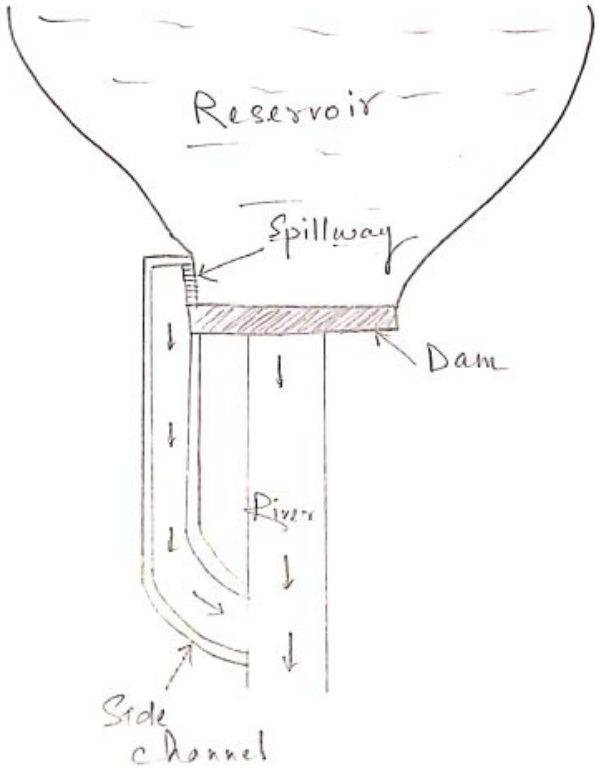
v/ Side Channel Spill Way

It is completely separate from the main body of dam. It is constructed at right angle to dam & at any side according to site condition. The crest of spillway is kept normal to pool level of reservoir. When water rises above N.P.L it spills over crest & flow through side channel & ultimately meets the same river on downstream side.



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1.0 Introduction To Surveying, Linear Measurements

1.1.1 Definition :-

Surveying is the art of determining the relative positions of distinctive features on the surface of earth or beneath the surface of earth, by means of measurements of distances, directions & elevations.

The branch of surveying which deals with the measurements of relative heights of different points on the surface of the earth is known as levelling.

1.1.2 Aims & Objectives :-

The aim of surveying is to prepare a map to show the relative positions of the objects on the surface of the earth. The map is drawn to some suitable scale.

It shows the natural features of a country such as towns, villages, roads, railways, rivers etc. Maps also include details of different engg works such as irrigation canals.

Surveying is the first step for the execution of any project.

1.2 Classification of Survey :-

1.2.1 Plane Surveying :-

We know, that the shape of earth is spheroidal. Thus the surface is obviously curved. But in

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plane surveying, the curvature of earth is not taken into consideration.

This is because, plane surveying is carried out over a small area. So, the surface of earth is considered plane.

In such surveying, a line joining any two points is considered straight. The triangle formed by any three points is considered a plane triangle & the angles of the triangle are assumed to be plane angles.

Plane surveying is conducted by state agencies like irrigation department, Railway dept. etc.

Plane surveying is done on an area of less than 250 km^2.

1.2.2 Geodetic Surveying :- In geodetic surveying, the curvature of earth is taken into consideration. It is extended over a large area. The line joining any two points is considered a curved line.

The triangle formed by any 3 points is considered spherical & the angles of triangle are assumed to be spherical angles.

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Geodetic surveying is conducted by the survey of India department, & is carried out over an area exceeding 250 km^2.

1.2.3 Instrumental Surveying :- Based on instruments, surveying is of following types :—

(a) chain surveying
(b) Compass "
(c) Plane table "
(d) Theodolite "
(e) Tacheometric "
(f) Photographic "

1.2 Principle of Surveying

The fundamental principles of upon which diff. methods of surveying are based, are as follows :—

1) Working from whole to part :- According to this first principle, the whole area is first enclosed by main stations (i.e. controlling stations) and main survey lines i.e. (controlling lines).

The area is then divided into a no. of parts by forming well-conditioned triangles. A nearly equilateral triangle is considered the best well-conditioned triangle.

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↳ The main survey lines & sides of triangles are measured very accurately with a standard chain.

↳ During this ~~proud~~ procedure, if there is any errror on the measurement of any side of a triangle, then it will not affect the whole work & the errror can be always detected & eliminated.

↳ So, the purpose of this principle is to prevent accumulation of errrors.

2) Location of a point by measurement from two control points:- The control points are selected in the area and the distance between them, is measured accurately. The lines then plotted to a convenient scale on the drawing sheet.

↳ So, the new stations should always be fixed by at least two measurements i.e. linear or angular from fixed reference points.

↳ Let P & Q be two given control points. Any other point say R can be located & with reference to these points, by any

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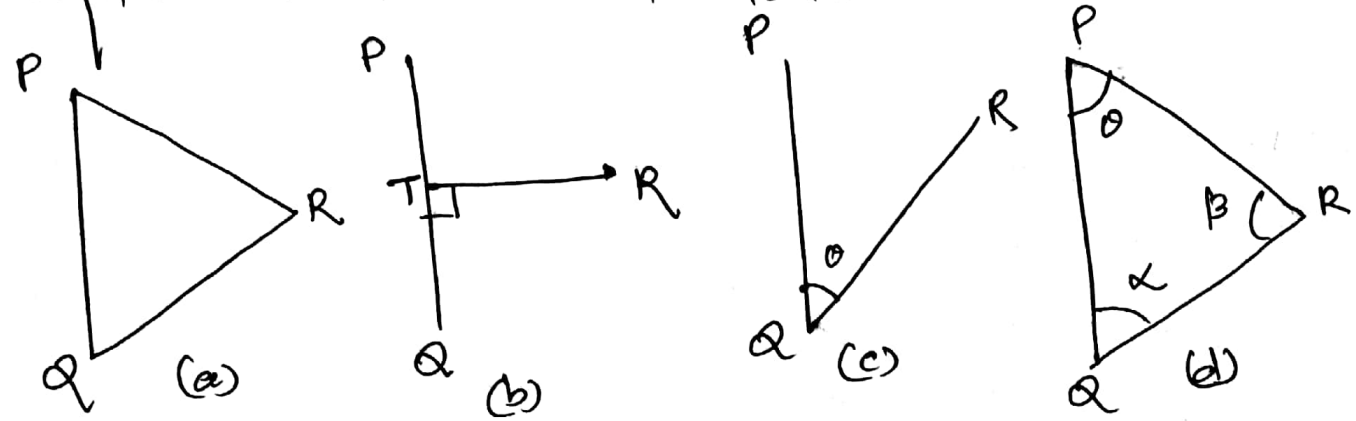
of the following methods :—

(a) By measuring the distances PR & QR.

(b) By dropping perpendicular from R on PQ. Distance PT, PQ & RT are measured & the location of R may be plotted by drawing perpendicular RT in the drawing sheet.

(c) By measuring the distance QR & the angle ∠PQR.

(d) By measuring internal angles of the triangle PQR by using angle measuring instrument such as theodolites etc.



1.3.1 Precision & Accuracy of measurements :—

The degree of precision required in surveying mainly depends upon the purpose and scale of the map. Larger the scale, better the precision & accuracy required & vice versa.

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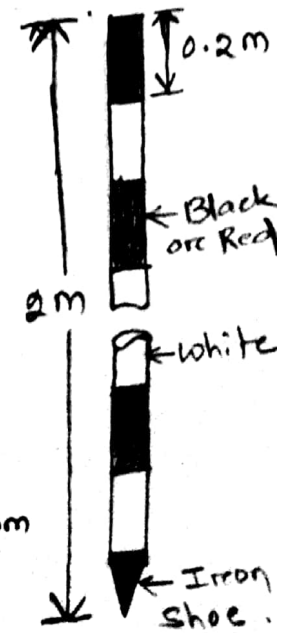
If a map is required to be on scale 1cm = 1km then plotted permissible error on map is 0.25mm. It is therefore necessary to have precision in linear measurements to 25 meters. That means an error of 25 meters in linear measurements of a line do not affect the accuracy of the map.

On the other hand, if the scale is 1cm = 5m, the plotted permissible error is 0.5 × 0.25 = 0.125m.

1.3.2 Instruments Used for measurement of distance :-

1) Ranging Rods :- Rods which are used for ranging (i.e. the process of making a line straight) a line are known as ranging rods.



- Such rods are made of seasoned timber or seasoned bamboo. Sometimes GI pipes of 25mm diameter are also used as ranging rods.
- They are generally circular in section. The rod is divided into equal parts of

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20 cm each and divisions are painted black and white and red & white alternatively, so that the rod is visible from a long distance. The lower end of the rod is pointed or provided with an iron shoe.

2) Chains :- The chains used in surveying are generally of different types described below. Chains are prepared with galvanised mild steel wires of 4 mm diameter. The ends are bent to form loops. The wire pieces are connected together with the help of 3 oval rings, which make the chain flexible. Two brass handles are provided at the two ends of chain. Tallies are provided at every 10 or 25 links for facilitating the counting.

One link means the distance between the centres of adjacent middle rings.

Types of chains :—

(a) Metric chain :- Metric chains are available in lengths of 20m and 30m. The 20m chain is divided into 100 links, each of 0.2m.

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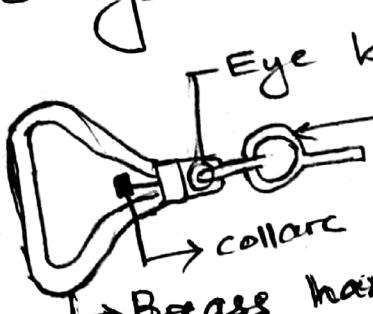
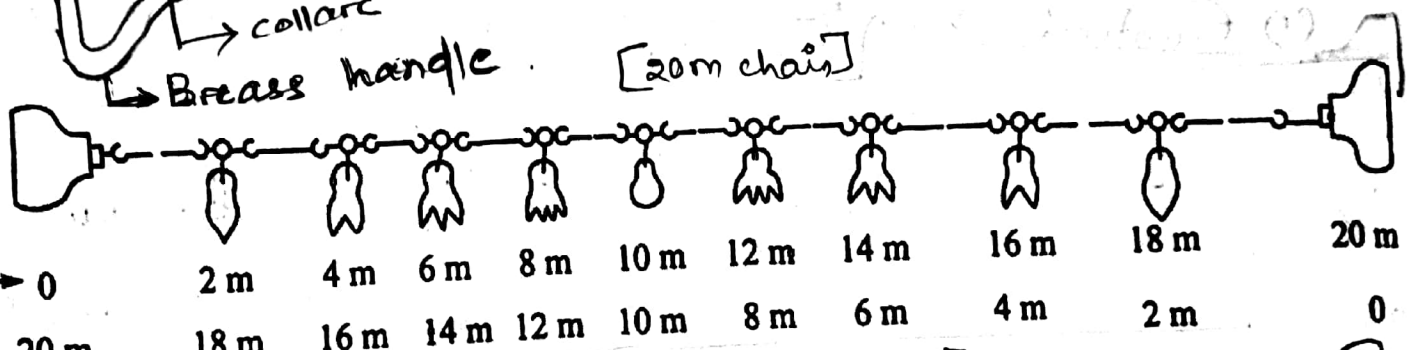
Tallies are provided at every 10 links (2m). This chain is suitable for measuring distances along ~~flat~~ fairly level ground.



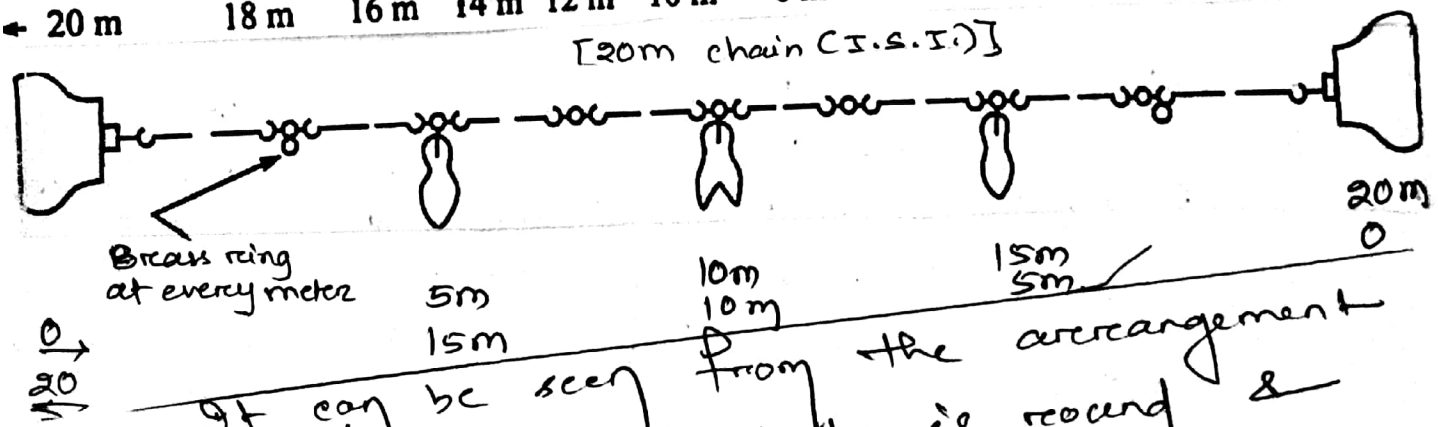
[Brass ring] [Tallies]



[20m chain]



[20m chain (I.S.I.)]



It can be seen from the arrangement that, the central tally is round & other tallies have one, two, 3 or 4 teeth. So, each tooth may correspond to 2 diff. readings when considered from opposite ends.

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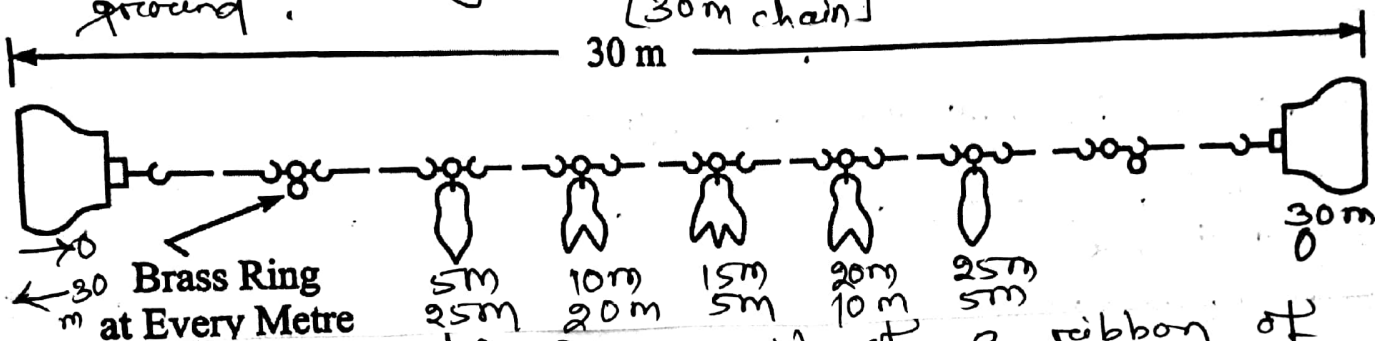
As per ISI, recommendations, tallies should be provided after every 5m & brass rings after every 1m.

The 30m chain is divided into 150 links. So, each link is of 0.2m. The tallies are provided at every 25 links (5m). A round brass ring is fixed after every one meter.

This chain is heavy & is also suitable for measuring distances along fairly level ground.

[30m chain]



(b) Steel Band :- It consists of a ribbon of steel of 16mm width & of 20m or 30m length. It has a brass handle at each end. It is graduated in metres, decametres & centimeters on one side and has 0.2m links on other. The steel band is used in projects where more accuracy is required.

(c) Engineer's chain :- It is 100 ft long and is divided into 100 links. So each link is of 1 ft. Tallies are provided at every 10 links (10 ft); the central tally being round. These are used for all engg. works.

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(d) Gunter's chain :- It is 66 ft. long and divided into 100 links. So, each link is of 0.66 ft. It was previously used for measuring distances in miles and furlongs. (yards & acres).

(e) Revenue chain :- It is 33 ft. long and divided into 16 links. It is mainly used in cadastral survey.

Advantages :-

(a) chains can be read easily & quickly

(b) They can withstand wear & tear.

(c) They can be easily repaired or rectified in the field.

Disadvantages :-

(a) They are heavy & take too much time to open or fold.

(b) ~~They maintain their standard length in suspension, they sag slightly.~~

(b) They become longer or shorter due continuous use.

(c) When the measurement is taken in suspension, the chain sags excessively.

3) Tapes :- Various types of tapes available are as follows :-

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(a) Cloth or Linen tape :- Such a tape is made of closely woven linen and is varnished to resist moisture. It is 15mm wide and available in length of 10 and 15 m. This tape is generally used for measuring offsets & for ordinary works.

(b) Metallic tape :- When linen tape is reinforced with brass or copper wires to make it durable then it is called a metallic tape. This tape is available in lengths of 15, 20 and 30m. It is wound on a leather case with a brass handle at the end. It is commonly used for all survey works.

(c) Steel tape :- The steel tape is made of steel ribbon of width varying from 6 to 16mm. The commonly available lengths are 10, 15, 20, 30 & 50m. It is graduated in meters; decimeters and centimeters. It is not used in the field, but chiefly for standardising chains and for measurements in constructional works.

(d) Invar tape :- It is made of an alloy of steel (64%) and nickel (36%). It's thermal coefficient is very low. Therefore

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It is not affected by change of temperature. It is made in the form of a ribbon of 6mm width & is available in length of 30, 50 & 100m. It is used at places where maximum precision is required.

1.4 Errors & Mistakes in linear measurements

Errors in chaining may be caused due to variation in temperature (and pull), defects in instruments etc. They may be either :-

(i) Compensating or

(ii) Cumulative.

(i) Compensating errors :- Errors which may occure in both directions (i.e both positive and negative) and which finally tends to compensate are known as compensating errors. These errors do not affect survey work seriously. They are proportional to $\sqrt{L}$, where L is the length of line. Such errors may be caused by :—

(a) Incorrect holding of chain.

(b) Horizontality & Verticality of steps not being properly maintained during the stepping operation.

(c) Fractional parts of chain or tape not being uniform throughout it's length.

(d) Inaccurate measurement of right angle & with chain & tape

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(ii) Cumulative errors :- Errors which may occure in the same direction and which finally tends to accumulate are said to be cumulative. They seriously affect the accuracy of work & are proportional to L. They errors may be positive or negative.

Positive errors :- when the measured length is more than the actual length (i.e. when the chain is too short); the error is said to be positive. Such errors occure due to
(a) The length of chain or tape being shorter than the standard length.
(b) Slope correction not being applied.
(c) Correction for sag not being made.
(d) Measurement being taken with faulty alignment &
(e) Measurement being taken in high winds with tape in suspension.

Negative errors :- when the measured length of the line is less than the actual length (i.e. when the chain is too long); the error is said to be negative. These errors occure when the length of the chain or tape is greater than the standard length due to following reasons :-
(a) The opening of ring joints.
(b) The applied pull being much greater than the standard pull.

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(c) The temperature during the measurement being much higher than the standard temperature.

(d) Wearing of connecting rings &

(e) Elongation of the links due to heavy pulls.

Mistakes :-

Errors occurring due to the carelessness of the chainman are called mistakes. The following are a few common mistakes :-

(a) Displacement of arrows :- Once an arrow is withdrawn from the ground during chaining, it may not be replaced in proper position, if required due to some reasons.

(b) A full chain length may be ommitted or added. This happens when arrows are lost or wrongly counted.

(c) A reading may be taken from the wrong end of the chain. This happens when the tooth of the tally is noted without observing the central tally.

(d) The numbers may be read from wrong direction ; for example a '6' may be read as '9'.

(e) Some numbers may be called wrongly.

(f) while entering in field book, the figures may be interchanged (245 → 254).

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Precautions or Remedies

1) The point where the arrow is fixed on the ground should be ~~mark~~ marked with a cross (x).
2) The zero end of the chain or tape be properly held.
3) During chaining, the number of arrows carried by the leader & follower should always tally with total no. of arrows taken.
4) while noting the measurements from the chain, the teeth of the tally should be verified w.r.t. the correct end.
5) Chainman should call the measurements loudly & distinctly and the surveyors should repeat them while booking.
6) Measurements should not be taken with the tape in suspension during high winds.
7) In stepping operations, horizontality & verticality should be properly maintained (Stepping means counting no. of steps by walking on the distance to be measured)
8) Ranging should be done accurately.
9) No measurements should be taken with the chain in suspension.
10) ~~Ooo~~ Care should be taken so that the chain is properly extended.

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1.5 Corrections to measured lengths due to incorrect length.

~~There are~~

(A) Tape Correction:

1) Temperature Correction (C_t) :- This correction is necessary because the length of the tape or chain may increase or decrease due to rise or fall of temperature during measurement. The correction is given by:-

$$C_t = \alpha (T_m - T_0) L$$

C_t = correction for temp. in metres.

α = coefficient of thermal expansion.

T_m = temp. during measurement (°C).

T_0 = " at which tape was standardised (°C)

L = length of tape in m.

The sign of correction may be +ve or -ve according to the value of T_m & T_0.

When α is not given, it may be assumed to be 11×10^{-6} per °C.

2) Pull Correction (C_p) :- During measurement, the applied pull may be either more or less than the pull at which the chain or tape was standardised. Due to elastic property of materials, the strain

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will vary according to the variation of applied pull, & hence necessary correction should be applied. The expression is given by :-

$$C_p = \frac{(P_m - P_o)L}{AE}$$ (may be +ve or -ve)

C_p = pull correction in (m)

P_m = pull applied during measurement (kg)

P_o = pull at which tape was standardised (kg).

L = length of tape (m).

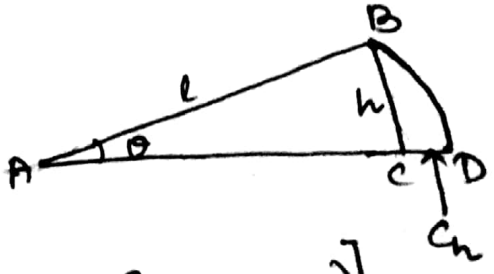
A = cross-sectional area of tape (cm^2).

E = modulus of elasticity (young's modulus).

$= 2.1 \times 10^6$ kg/cm^2 (if not given).

3) Slope Correction (C_h).

It is calculated as follows :-



$$C_h = l - \sqrt{l^2 - h^2}$$

$$= L(1 - \cos\theta) \cdot \left[\because C_h = AD - AC = (l - l\cos\theta)\right].$$

$$C_h = \frac{h^2}{2L}.$$

This correction is always negative.

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4) **Sag Correction (C_s)**:- This correction is necessary when the measurement is taken with the tape in suspension. G. It is given by:-

$$C_s = \frac{L(\omega L)^2}{24 n^2 P_m^2}$$ (when unit weight (ω) is given).

&

$$C_s = \frac{L W^2}{24 n^2 P_m^2}$$ (when total weight (W) is given).

C_s = Sag correction in m.
L = length of tape or chain in (m).
ω = weight of tape per unit length (kg/m)
W = total weight of tape (in kg).
n = no. of spans.
P_m = pull applied during measurement (kg).

This correction is always negative.

5) **Normal tension (P_n)**:- the tension at which the effect of pull is neutralised by the effect of sag is known as normal tension. At this tension, the elongation due to pull is balanced by the shortening due to sag. So expressions for correction for pull & sag can be equated.

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Hence, $\left(\frac{P_n - P_0}{AE}\right) L = \frac{L (wL)^2}{24 P_n^2}$ (considering no. of span, n=1)

· P_n = normal pull or normal tension.

Hence, the value of P_n may be determined by trial, by forming an equation :-

$$\frac{(P_n - P_0) L}{AE} = \frac{L w^2}{24 P_n^2}$$

$$\Rightarrow \frac{(P_n - P_0)}{AE} = \frac{w^2}{24 P_n^2} \Rightarrow (P_n - P_0) P_n^2 = \frac{w^2 AE}{24}.$$

By substituting the values of P_0, w, A & E, an equation can be obtained as below:-

$$\boxed{x P_n^3 \pm y P_n^2 \pm c = 0}$$

(B) Applying Corrections to measurements.

1) Correction applied to incorrect length.

True length of line, $(TL) = \left(\frac{L'}{L}\right) \times$ Measured length (ML).

L = standard or true length of chain or tape

L' = true length of chain ± error

$= L \pm e$.

e = error in chain or tape when it is too long or too short.
(+) (-)

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2) Correction of incorrect area

It is given by :-

$$\text{True area} = \left(\frac{L'}{L}\right)^2 \times \text{measured area.}$$

3) Hypotensual Allowance

Hypotensual allowance per tape $= L(\sec\theta - 1)$

L = length of tape

θ = slope of ground.

This ~~alw~~ allowance is always added to the tape length.

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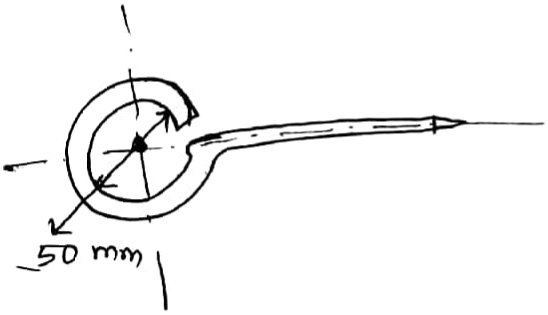
2.0 CHAINING & CHAIN SURVEYING

2.1 Instruments & Accessories for chaining

The list of equipments required for chain surveying should include the following :—

1) A 20m or 30m chain
2) A 20m metallic tape
3) Arrows (10 nos.)
4) Ranging rods
5) Offset rods.
6) Optical square
7) Cross staff.
8) Plumb bob
9) Wooden pegs
10) A Hammer
11) A field book
12) Good quality pencils

1) Chain → discussed earlier
2) Tape → do
3) Arrows :— An arrow is made of steel wire 4mm in diameter. The length of arrows may vary from 25cm to 50 cm but commonly 40cm length is used. One end of the arrow is bent into a loop of a circle of 50m dia & other end is made pointed sharp.



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4) Ranging rods - discussed earlier.

5) Offset rods :- It is similare to ranging rod is 3m long. It is divided into equal p of 0.2m. The top is provided with an open hook for pulling or pushing chain through obstructions (eg:- bushes). Two narrow vertical slots passing throu the centre of setting at right angle to one another are provided at the eye level. It is used for aligning the offset line & measuring short offsets.

6) Optical Square :-
7) Cross staffs :- } These are used for settin out right angles in chain surveying & are discussed later.

8) Plumbob :- It is used to transfer the end points of the chain onto the ground while measuring distances in hilly terrain. It is also used for testing the verticality of ranging poles, ranging rods or levelling staves.



9) Wooden Pegs :- Wooden pegs usually square & 15cm deep are used to mar the position of survey stations.

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1) Field book :- ~~It has~~ The notebook in which, chainages, offset measurements & sketches of detail points are recorded, is generally called a field book.

·2 Ranging

1) Purpose :— The process of marking a number of intermediate points on a survey line joining two stations in the field so that the length between may be measured correctly is called ranging.

Ranging must be done before a survey line is chained, with the help of line ranger or by theodolite or by direct observation through naked eye.

Generally ranging is done by naked eye with the help of 3 ranging rods.

2) Signalling :— Following codes of signals are used to direct the assistant while ranging :—

1) Rapid sweep with right hand → Move considerably to the right.

2) Rapid sweep with left hand → Move considerable to the left.

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3) Slowo sweep with right hand → Move slowly to right
4) " " " left " → " " " left
5) Right arm extended → Continue to move to the right.
6) Left " " → Continue to move to the left.
7) Right arm up & moved to the right → Plumb the rod to the right.
8) Left arm up & moved to the left → Plumb the rod to the left.
9) Both arms above head & then brought down → Correct.
10) Both arms extended forward & depressed → Fix.

3) Classification of ranging

It is of two types:-

(a) Direct ranging (b) Indirect ranging.

(a) Direct ranging :- When intermediate ranging rods are fixed on a straight line by direct observation from end stations the process is known as direct ranging.

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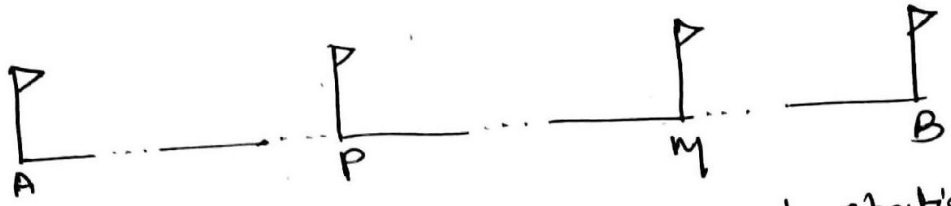
Direct ranging is possible when end stations are inter-visible. Following procedure is adopted for direct ranging:-



Assume A & B are two end stations of a chain line, where 2 ranging rods are already fixed. Suppose it is required to fix a ranging rod at intermediate point P on the chain line in such a way that points P & B are in the same straight line.

(i) The surveyor stands about 2m behind ranging rod A by looking towards the line AB.

(ii) The assistant holds a ranging rod at P vertically at arms length. The rod should be held lightly by thumb & forefinger.

(iii) Then the surveyor directs the assistant to move the ranging rod to left or right, until the 3 ranging rods come exactly in the same straight line.

(iv) To check the non-verticality of the rods, the surveyor bends down & looks through the bottom of rods.

(v) The ranging will be perfect when the

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three ranging rods coincide & appear as a single rod. When the surveyor is satisfied that the ranging is perfect, he signals the assistant to fix the ranging rod on the ground by waving both his hands up & down.

(vi) Following the same procedure, the other ranging rods may be fixed on the line.

(b) Indirect or Reciprocal Ranging :-

When the end stations are not intervisible, due to some obstacles like high ground between them, intermediate ranging rods are fixed on the line in an indirect way. This method is known as indirect ranging or reciprocal ranging. Following procedure is adopted here :—

Suppose A & B are 2 end stations which are not intervisible due to high ground existing between them.

(i) Two chain men take up position at R_1 & S_1 with ranging rods in their hands.

(ii) The chain man at R_1 stands with his face towards B, so that he can see the ranging rod at S_1 & B.

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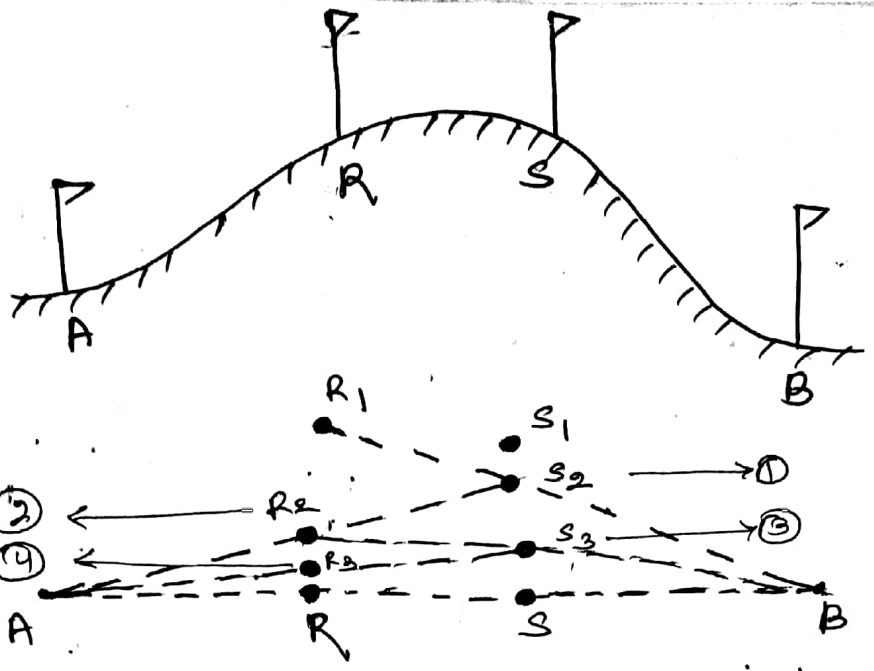

(iii) Again the chain man at S_1 stands with his face towards A, so that he can see ranging rods at R_1 & A.

(iv) Then the two chainmen proceed to range the line by directing each other alternatively

(v) The chainman at R_1 directs the chainman at S_1 to come to the position S_2 so that R_1, S_2 & B are in same straight line.

(vi) Again chain man at S_2 directs the chainman at R_1 to move to position R_2, so that S_2, R_2 & A are in same straight line.

(vii) By directing each other alternatively in this manner, they change their positions every time until they finally come to position R_1 & S, which are in the straight line AB. That means, now, the points A, R, S & B are in the same straight line.

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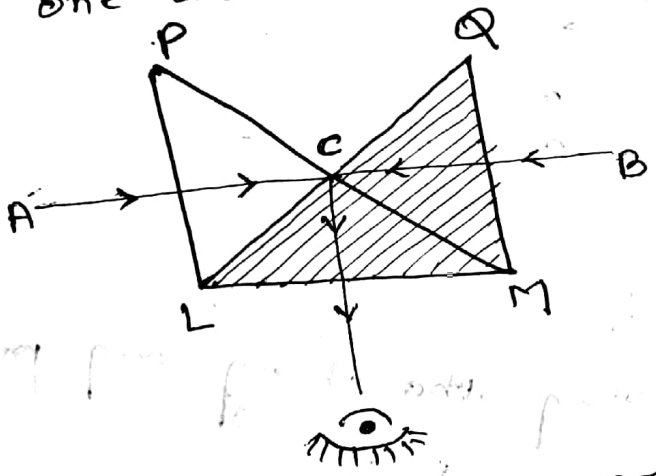
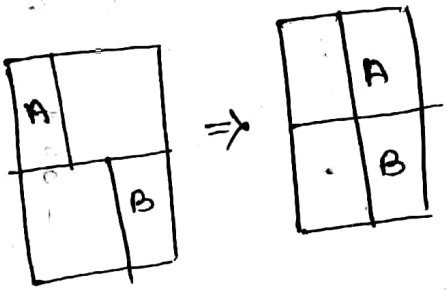
4) Line Ranger :- It is a small reflecting instrument used for fixing intermediate points on a chain line. It consists of two right angled isosceles triangular prisms, placed one above the other.





Suppose A & B are the ends of a line & C is an intermediate point to be fixed on this line. Following steps are followed for locating the intermediate point C.

(i) Stand approximately in line near C & hold the line ranger at eye level.

(ii) Observe the ray of light from A, which enters the upper prism, gets reflected from the hypotenuse LQ.

(iii) Similarly, observe the ray of light from B, which enters the lower prism, gets reflected from hypotenuse PM.

(iv) Observe the images of the ranging rods A & B in upper & lower prism simultaneously.

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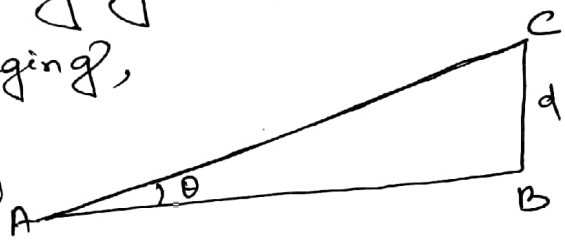
(v) If the point C is not in line with AB, two images will appear to be separated.

(vi) Move the instrument backward & forward at right angles to the line - until two images appear one above the other exactly in same vertical line.

(vii) The centre of the instrument defines the location of C on line AB.

5) Error due to incorrect ranging



Due to incorrect ranging, the distance between two stations is obviously increased & the error due to incorrect ranging may be deduced as under :-

Let the length AB between two stations A & B is required to be measured. Due to incorrect ranging, the chain is laid along AC where perpendicular distance BC is equal to d.

Error in distance is $\delta l = AC - AB$.

$$AB = \sqrt{AC^2 - d^2}$$

$$\& \quad \delta l = AC - AB = AC - \sqrt{AC^2 - d^2}$$

$$= AC - AC\sqrt{1 - \left(\frac{d}{AC}\right)^2}$$

$$\Rightarrow \boxed{\delta l = AC\left[1 - \sqrt{1 - (d/AC)^2}\right]}$$

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For eg:- if AC = 20m = 1 chain length.

& BC = d

then, $\delta l = AC\left[1 - \sqrt{1 - (d/AC)^2}\right]$.

$= 20 - 20\sqrt{1 - (d/20)^2}$

where $AB = 20\sqrt{1 - d^2/400}$.

After bionomical expansion, ignoring higher power of d, $AB = 20\left(1 - \frac{d}{800}\right)$

so $\delta l = 20 - 20\left(1 - \frac{d}{800}\right)$

$= 20 - 20 + 20 \times \frac{d}{800}$.

$\delta l = d/40$ m.

If total length of line is 100 m & displacement per 20 m chain is 1 m, then

no. of chaining $= \frac{100}{20} = 5$.

& total displacement, $d = 5 \times 1 = 5$ m

hence $\delta l = \frac{5m}{40} = 12.5$ cm.

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2.3 Methods of chaining

1) Chaining on flat ground

1) Before starting chaining operation, two ranging rods should be fixed on the chain line, at the end stations. The other ranging rods should be fixed near the end of each chain length, during ranging operation.

2) To chain the line, the leader moves forward by dragging the chain & by taking with him a ranging rod & 10 arrows.

3) The follower stands at the starting station by holding the other end of the chain. When the chain is fully extended, the leader holds the ranging rod vertically at arms length.

4) The follower directs the leader to move his rod to the left or right until the ranging rod is exactly in line.

5) Then the follower holds the zero end of the chain by touching the station peg.

6) The leader stretches the chain by moving it up & down with both hands & finally places it on the line. He then inserts an arrow on the ground at

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the end of the chain and marks with a cross (X).

7) Again the leader moves forward by dragging the chain with nine arrows & the ranging rod. At the end of the chain, he fixes another arrow as before. As the leader moves further, the followers pick up the arrows which were inserted by the leader.

8) During chaining, the surveyor or an assistant should conduct the ranging operation.

9) In this way, chaining is conducted. When all the arrows have been inserted and the leader has none left with him, the follower hands them over to the leader. This is noted by the surveyor.

10) To measure the remaining fractional length, the leader should drag the chain beyond the station & the follower should hold the zero end of the chain at the last row. Then the odd links should be counted.

P.T.O.

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2) Chaining on sloping ground

(1) Horizontal distances are required in surveying. So, in chaining along a sloping ground, the horizontal distance between two stations are measured carefully by applying some convenient methods :-

1) Direct method or stepping method
2) Indirect method.

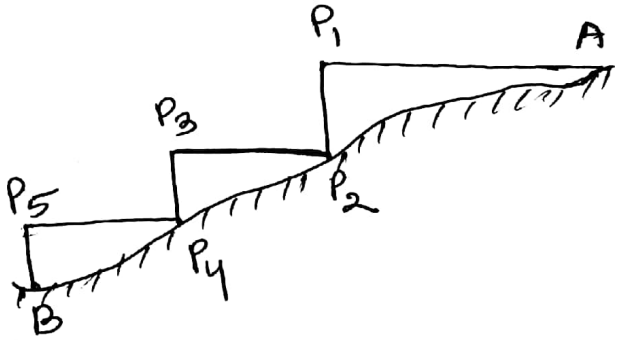
1) Direct Method :-

This method is applied when the slope of the ground is very steep. In this method, the sloping ground is divided into a number of horizontal & vertical strips, like steps. So this method is also known as stepping method.

PROCEDURE

(i) Suppose the horizontal distance between points A & B is to be measured. The line AB is first ranged properly. Then, the follower holds the zero end of the tape at A.

(ii) The leader selects a suitable length AP_1, so that P_1 is at chest height & AP_1 is just horizontal.



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(iii) The horizontality is maintained by eye estimation, by wooden set-square or by trisquare.

(iv) The point P_2 is marked on the ground by plumb-bob so that P_1 is just over P_2.

(v) The horizontal length AP_1 is noted. Then the follower moves to P_2 & holds the zero end of the tape at that point.

(vi) Again the leader selects a suitable length P_2P_3 in such a way that P_2P_3 is horizontal & P_4P_5 is vertical.

(vii) Then the horizontal lengths P_2P_3 and P_4P_5 are measured.

So, horizontal length, $AB = AP_1 + P_2P_3 + P_4P_5$.

2) <u>Indirect Method</u>

When, the slope of the ground surface is long and gentle, the stepping method is not suitable. In such a case, the horizontal distance, may be obtained by following process:

(i) By measuring the slope with clinometer
(ii) By applying hypotenusal allowance.
(iii) By knowing the difference of level between the points.

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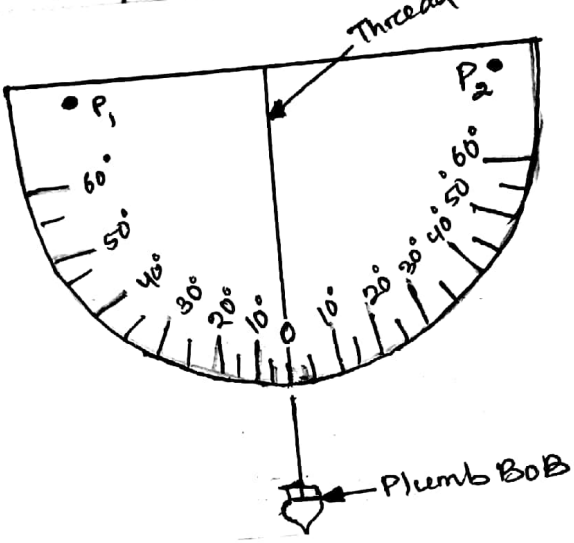
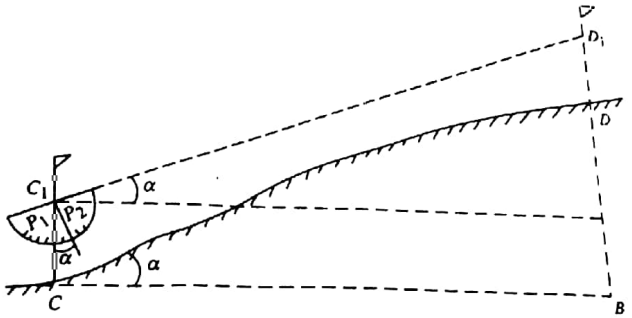
Measuring slope with a clinometer :—

A clinometer is a graduated semicircular protractor. It consists of 2 points P_1 & P_2 for sighting the object. A plumb bob is suspended from point O with a thread. When, the straight edge is just horizontal, the thread passes through 0°. When the straight edge is tilted, the thread ~~moves~~ remains vertical, but passes through a graduation on the arc, which shows angle of slope.



PROCEDURE :—

- Suppose C & D are two points on the sloping ground. Two ranging rods are fixed at these points.
- Then two other points C_1 & D_1 are marked on the ranging rods such that $CC_1 = DD_1$.
- The clinometer is then placed in such a way that its centre just touches the mark C_1.
- The clinometer is then inclined gradually so that P_1, P_2 & D_1 are in same line.



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↳ At this position, the thread of clinometer will show an angle which is the angle of slope of the ground.

↳ Suppose this angle is α & sloping distance is l.

The required horizontal is $CB = l\cos\alpha$.

ii) By applying hypotenseal allowance

In this method, the slope of the ground is first found out by using the clinometer or Abney level.

Let θ = angle of slope.

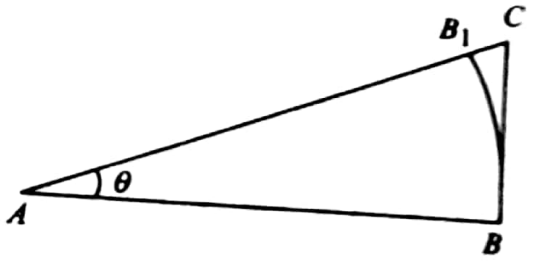
$AB = AB_1 = 20m = 100$ links

$AC = AB \sec\theta = 100 \sec\theta$

$B_1C = AC - AB_1$

$= 100\sec\theta - 100$

$= 100(\sec\theta - 1)$.



Hypotenseal allowance $= 100(\sec\theta - 1)$.

while chaining along the slope, one chain would be actually located at B_1. But the arrow should be placed at C after making hypotenseal allowance.

The next chain length will start from C.

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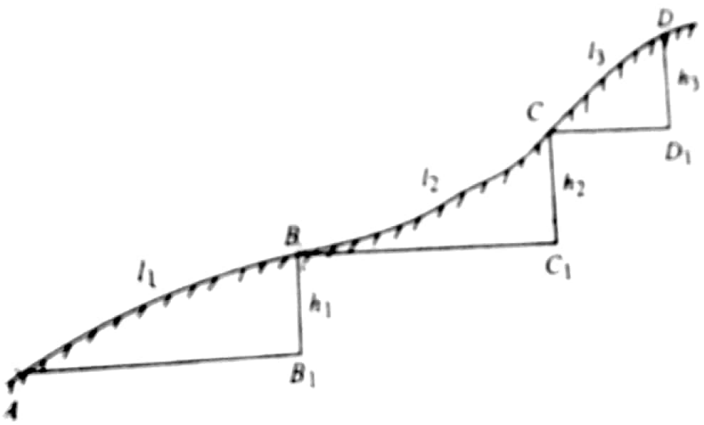
iii) Knowing the difference of level

Suppose A, B, C and D are different points on sloping ground. The difference of level between the points is determined by a levelling instrument. Let respective differences be h_1, h_2 & h_3. The sloping distances AB, BC & CD are measured as l_1, l_2 & l_3.

The required horizontal distances are given by, $AB_1 = \sqrt{l_1^2 - h_1^2}$, $BC_1 = \sqrt{l_2^2 - h_2^2}$, $CD_1 = \sqrt{l_3^2 - h_3^2}$

Total horizontal distance = $AB_1 + BC_1 + CD_1$.



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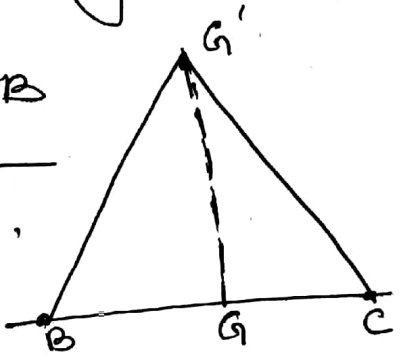
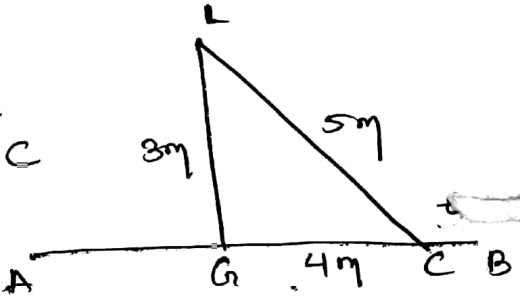
2.4 Setting Perpendiculars with chain & tape

Prob-1 For erecting a perpendicular to a chain line, one of the following methods may be adopted.

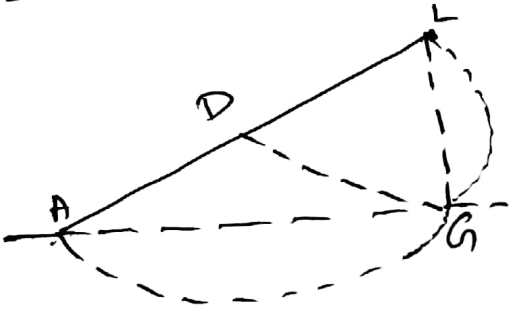


Method-1 :- Select two points B and C equidistant from the given point G on the chain line. Pin/hook the ends of a 30m tape i.e. zero end & 30m mark at B & C respectively. Hold the 15m mark & move it away from the line till two halves are fully stretched. Then the point G' at 15m mark is on the required perpendicular G'G.



Method-2 :- On the chain line AB, select a point C, 4m away from G. With C & G as centres and 5m & 3m as radii, sweep 2 arcs to intersect at L to get the required perpendicular GL.

Method-3 :- Select any convenient point D, so that GD is less than length of tape. With



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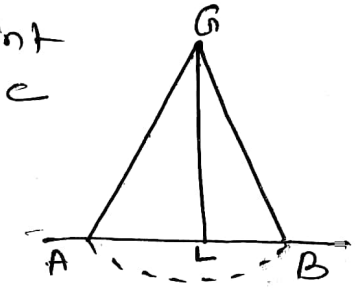
D as centre & DG as radius, swing an arc cutting the chain line at A & also AD produced at L. GL is required perpendicular.

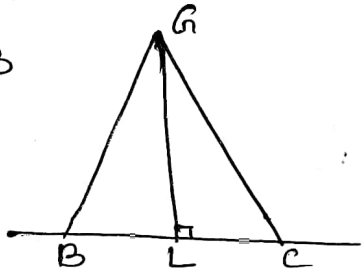
PROB-2 To drop perpendiculars to a chain line from outside points, two cases may arise:—

(a) When point is accessible :—

Method-1 :- With the given point G as centre, swing an arc to cut the chain line at A & B. Find the mid-point L of AB which is the foot of the required perpendicular GL.

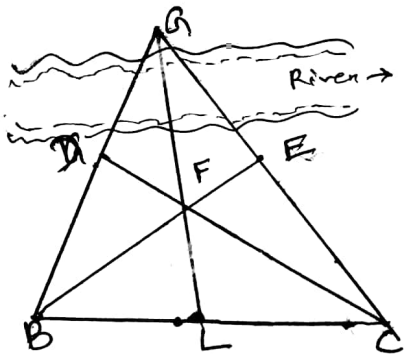


Method-2 :- Select any point B on the given chain line & measure a convenient length BG. Set off BC on the chain line equal to BG. Measure CG. Obtain the point L on BC measuring $CL = \frac{(CG)^2}{2BC}$. LG is required perpendicular.



(b) When the point is inaccessible :—

Select two points B & C on the chain line. Drop perpendiculars BE to CG & CD to BG. Locate their point of inter-



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sections F. produce GF to L which is the required perpendicular to BC.

Their solution is based on geometrical property of a triangle i.e. perpendiculars drawn from verticals on opposite sides, intersect at a point.

Chaining across different types of Obstacles

A chain line may be interrupted in following situations.

1) when chaining is free, but vision is ~~obstru~~ obstructed, or ranging is obstructed.
2) when chaining is obstructed but vision is free, or ranging is free.
3) when chaining & vision are both obstructed

1) When chaining is free, but vision is obstructed

Such a problem arises when a rising ground or jungle area interrupts the chain line. Here, the end stations are not intervisible. There may be 2 cases :—

Case-1 :- The end stations may be visible from some intermediate points on the rising ground. In this case, reciprocal ranging is resorted to, & the chaining is done by stepping method.

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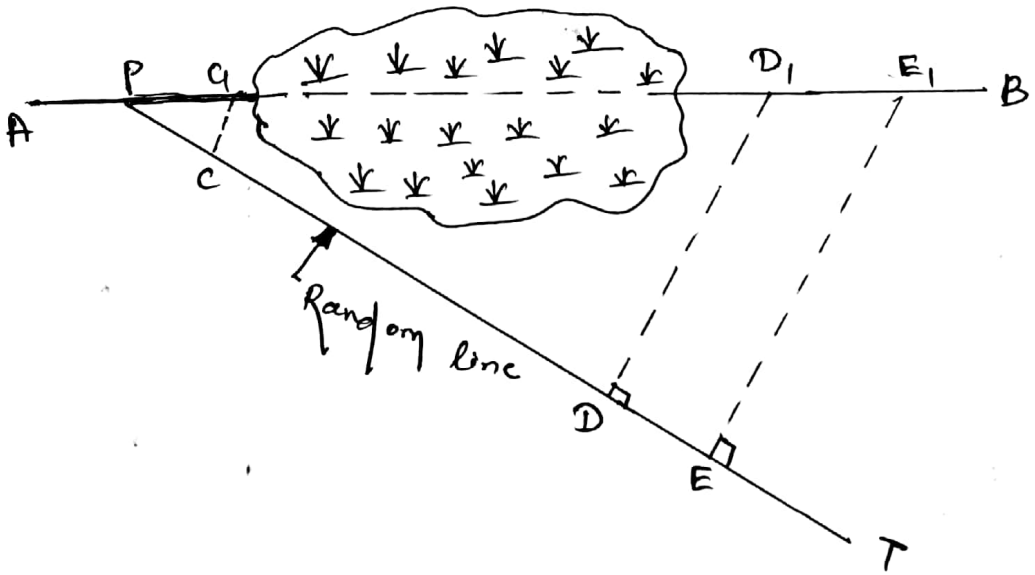
Case-II :- The end stations are not visible from intermediate points when a jungle area comes across the chain line. In this case the obstacle may be crossed over using a random line as explained below :-

Let AB be the actual chain line which cannot be ranged & extended because of intercepted by a jungle. Let the chain line be extended upto R. A point P is selected on the chain line & a random line PT is taken in a suitable direction. Points C, D & E are selected on the random line, ~~PT &~~ perpendiculars are projected from them.

The perpendiculars at C meets the chain line at Q. Theoritically the perpendiculars at D & E will meet the chain line at D_1 & E_1.



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Now, the distances PC, PD, PE & CC_1 are measured.

From the triangles PDD, & PCC_1,

$$\frac{DD_1}{PD} = \frac{CC_1}{PC}$$

$$\Rightarrow DD_1 = \frac{CC_1}{PC} \times PD.$$

Again, from triangles PEE_1 & PCC_1,

$$\frac{EE_1}{PE} = \frac{CC_1}{PC}$$

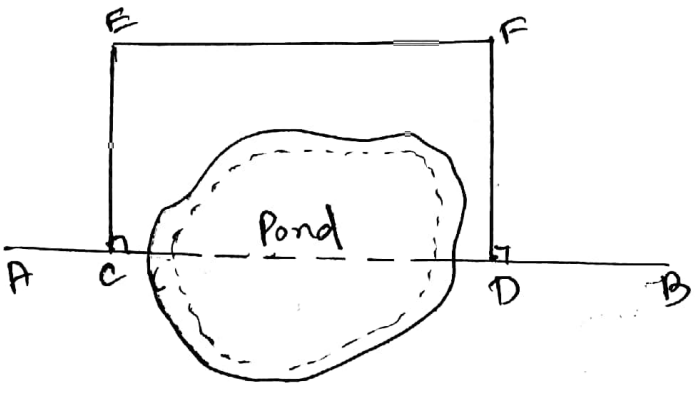
$$\Rightarrow EE_1 = \frac{CC_1}{PC} \times PE.$$

Now, distance $PE = \sqrt{PE^2 + EE_1^2}$.

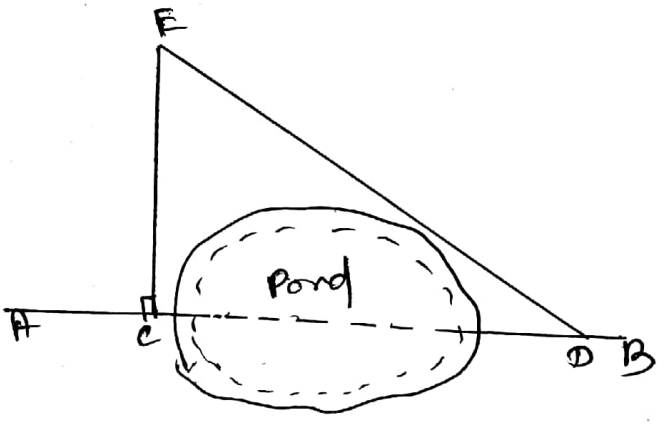
2) Chaining obstructed but vision free

Such a problem arises when a pond or a river come across the chain line. It may be tackled in following ways:→

Case-1:- When a pond intercepts the chain line, it is possible to go around the obstruction.



(a)



(b)

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Suppose AB is the chain line. 2 points C & D are selected on it on opposite banks of the pond. Equal perpendiculars CE and DF are erected at C and D. The distance EF is measured.

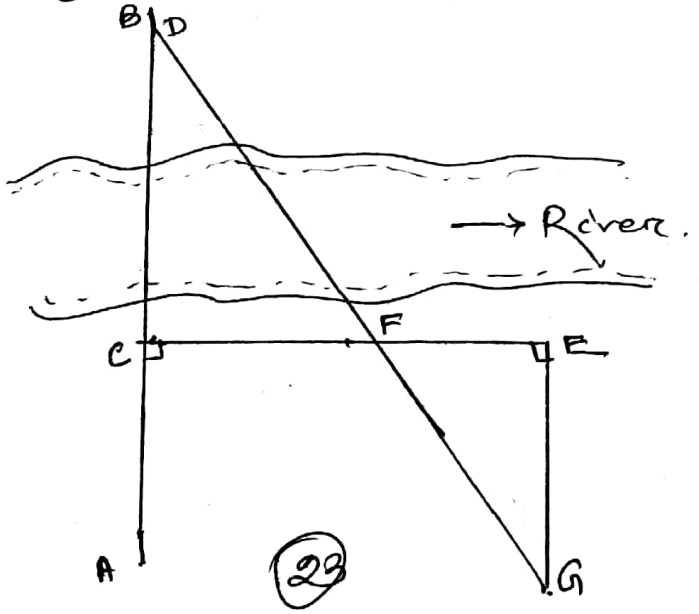
$CD = EF$.

Here, the pond may also be crossed by forming a triangle as shown in fig. A point C is selected on the chain line. The perpendicular CE is set out at C, & a line ED is suitably taken. The distances CE & ED are measured.

$$CD = \sqrt{ED^2 - CE^2}$$

Case-II :- Some times it is not possible to go around the obstruction.

(a) Imagine a small river comes across the chain line. Suppose AB is the chain line. 2 points C & D are selected on this line on opposite banks of the river. At C, a perpendicular CE is erected & bisected at F.



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A perpendicular is set out at E & at a point G is so selected on it that D, F and G are in same straight line.

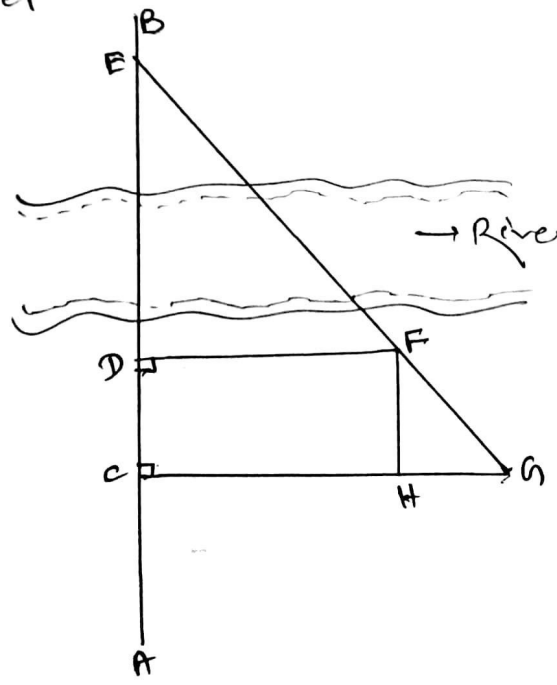
From triangles DCF and GEF,

$GE = CD$.

This distance GE is measured, & thus the distance CD is obtained indirectly.

(b) Consider the case when a large river intercepts the chain line.

Let AB be the chain line. Points C, D & E are selected on this line such that D & E are on opposite banks of the river. The perpendiculars DF & CG are erected on the chain line in such a way that E, F & G are on the same straight line. The line FH is taken parallel to CD.



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Now, from triangles DEF & HFG,

$\frac{ED}{DF} = \frac{FH}{HG}$ where, $FH = CD$.

$\rightarrow ED = \frac{FH}{HG} \times DF$. $CH = DF$.

$= \frac{CD}{CG - DF} \times DF$. $HG = CG - CH$.
& $HG = CG - DF$

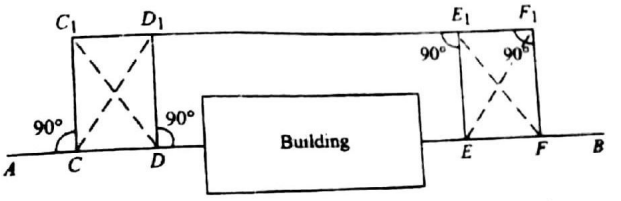
The distances CD, DF & CG are measured. & required distance is found.

8) Chaining & Vision (Ranging) both obstructed

Such a problem arises, when a building comes across the chain line. It is solved in following manner.

Suppose AB is the chain line. Two points C & D are selected on it at one side of the building. Equal perpendiculars CC$_1$ & DD$_1$ are erected. The line C$_1$D$_1$ is extended until the building is crossed. On extended line, 2 points E$_1$ & F$_1$ are selected. Then perpendiculars E$_1$E & F$_1$F are so erected that, E$_1$E = F$_1$F = D$_1$D = C$_1$C. Thus points C, D, E & F will lie on same straight line AB.

Here, DE = D$_1$E$_1$.



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Q:- A survey line CD intersects a building. To overcome the obstacle, a perpendicular DE, 87m long, is set out at D. From E, two lines EF and EG are set out at angles of 50° and 65° respectively with ED. Find the lengths EF and EG such that points F and G fall on the prolongation of CD. Also find the obstructed distance DF.

Q:- A chain line PQ intersects a pond. Two points A and B are taken on the chain line on opposite sides of the pond. A line AC, 250m long, is set on the left of AB and another line AD, 300m long, is set out on the right of AB. Points C, B, and D are in the same straight line. CB and BD are 100 and 150m long respectively. Calculate the length of AB.

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1) To secure necessary data for exact description of the boundaries of a plot of land
2) To determine the area of a plot of land.
3) To prepare an accurate plan of a plot of land.
4) To demarcate the boundaries of a plot of land in a previously surveyed land.
5) To divide a plot of land into a number of smaller units.
6) To secure data for executing engineering projects i.e. alignment of roads, railway lines, canals etc.

Principle of chain surveying

The principle of chain surveying is triangulation. This means that the area to be surveyed is divided into a number of small triangles which should be well conditioned.

Since a triangle is the only simple plane geometrical figure which can be plotted with it's sides alone, a network of triangles is preferred to in chain surveying.

Well-Conditioned & Ill-Conditioned Triangles

A triangle is said to be well conditioned when no angle in it is less than 30° or greater than 120°. An equilateral triangle is considered to be the best-conditioned or ideal triangle.

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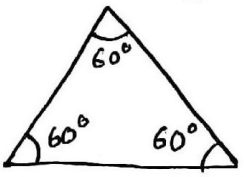
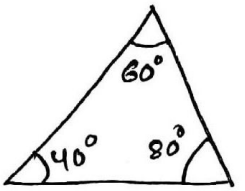
Well conditioned triangles are preferred because their apex points are very sharp and can be plotted or located by a single dot. In such a case there is no possibility of relative displacement of plotted point.

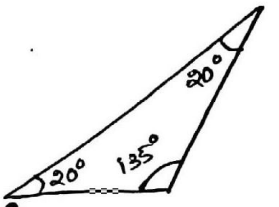
A triangle in which an angle is less than 30° or more than 120° is said to be ill-conditioned. These are not used in surveying as their apex points are not sharp and well-defined, which is why a slight displacement of these points may cause considerable error in plotting.



(Ideal triangle)



(well-conditioned triangle)



(Ill-conditioned triangle)

Concept of field-book

The note book in which field measurements are noted is known as field book. The size of field book is 20cm × 12cm & it opens length wise. Field books may be of two types:—

1) Single-line field book
2) Double-line " "

1) **Single-line field book** :- Here, a single red-line is drawn through the middle of each page. This line represents the chain line, & the

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chainages are written on it. The offsets are recorded, with sketches, to the left or right of the chain line.

The recording of the field book is started from the last page and continued towards the first page. The main stations are marked by 'Δ' and subsidiary stations or tie stations are marked by '⊙'.

2) Double-line field book :—

In this type of field book, two red lines, 1.5 cm apart are drawn through the middle of each page. This column represents the chain line & the chainages are written in it.

The offsets are recorded, with sketches, to the left or right of this column. The recording is begun from the last page and continued towards the first. Main stations are marked as 'Δ' and subsidiary or tie stations by '⊙'. This type of field book is commonly used.

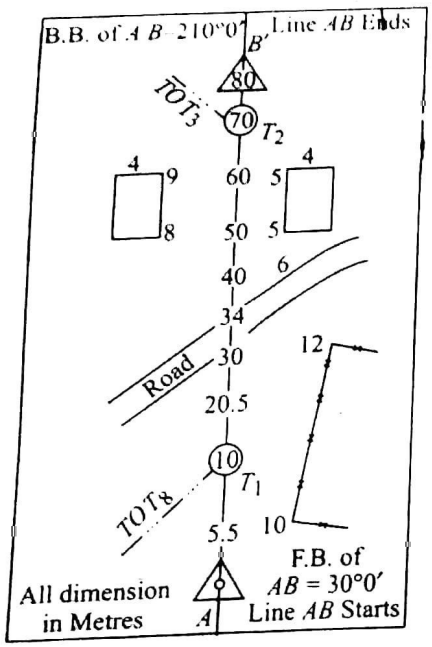

Fig. 2.16 Single-Line Field Book

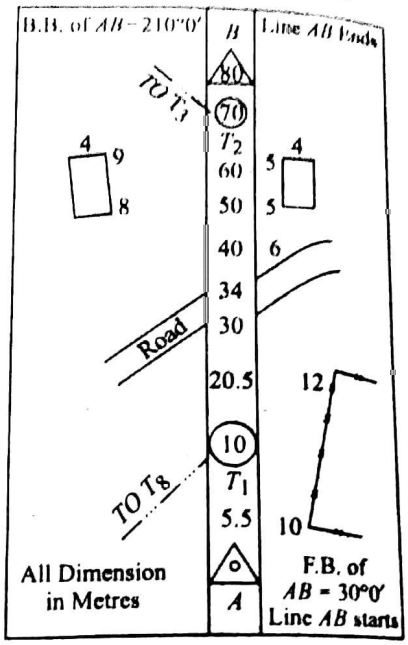

Fig. 2.17 Double-Line Field Book

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Selection of Survey stations

Survey stations are the points at the begining and the end of a chain line. They may also occures at any convenient points on the chain line. Such stations may be

1) Main stations.
2) Subsidiary stations.
3) Tie stations.

1) **Main stations**:- Stations taken along the boundary of an area as controlling points are known as main stations. The lines joining the main stations are called main survey lines. The main survey lines should cover the whole area to be surveyed. The main stations are denoted by 'Δ' with letters A, B, C, D etc. The chain lines are denoted by "_____"

2) **Subsidiary stations**:- Stations which are on the main survey lines or any other survey lines are known as subsidiary stations. These stations are taken to run subsidiary lines for dividing the area into triangles, for checking the accuracy of triangles and for locating interior details. These stations are denoted by 'o' with letters S_1, S_2, S_3 etc.

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3) Tie stations :- These are also subsidiary stations taken on the main survey lines. Lines joining the tie stations are known as tie lines. Tie lines are mainly taken to fix the directions of adjacent sides of the chain survey map. These are also taken to form chain angles in chain traversing, when triangulation is not possible. Sometimes tie lines are taken to locate interior details. Tie stations are denoted by '⊙' with letters T_1, T_2, T_3 etc.

Base line :- The line on which the framework of the survey is built is known as the base line. It is the most important line of the survey. Generally the longest of the main survey lines is considered the base line. This line should be taken through fairly level ground, & should be measured very carefully and accurately. The magnetic bearings of the base line are taken to fix the north line of map.

Check line :- The line joining the apex point of a triangle to some fixed point on its base is known as check line. It is taken to check accuracy of triangle. & also to locate interior details.

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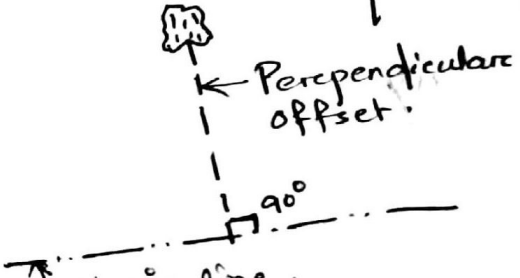
2.7 Offsets

The lateral measurement taken from any object to the chain line is known as offset. It is necessary to locate objects with reference to the chain line. Offsets may be of two kinds :—

(a) Perpendicular offset.

(b) Oblique offset.

1) **Perpendicular offset** :- when the lateral measurements are taken perpendicular to the chain line, they are known as perpendicular offsets.



Perpendicular offsets may be taken in following ways :—

(a) By setting a perpendicular by swinging a tape from the object to the chain line. The point of minimum reading on the tape will be the base of the perpendicular.

(b) By setting a right angle in the ratio of 3:4:5.

(b) By setting a right angle with the help of builder's square or tri-square.

(c) By setting a right angle by cross-staff or optical square.

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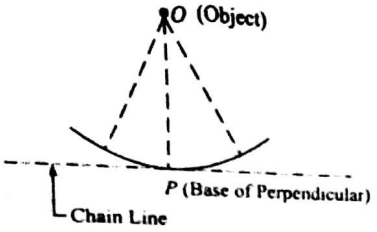


Fig. 2.4 Setting a Perpendicular in the ratio 3:4:5

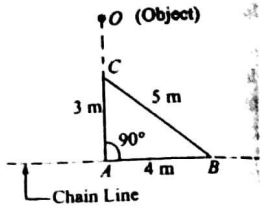


Fig. 2.5 Setting a Right

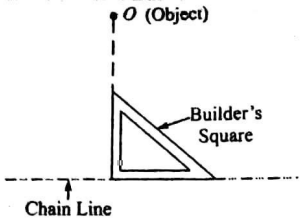
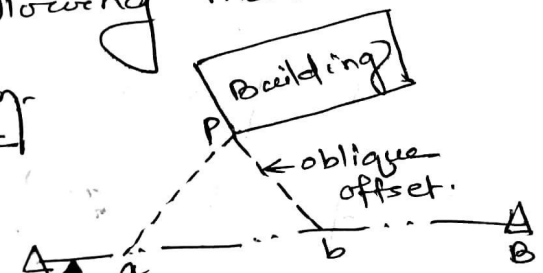


Fig. 2.6 Setting a Right Angle by a Builder's Square

2) <u>Oblique offset</u> :- Any offset not perpendicular to the chain line is said to be oblique. These are taken when the objects are at a long distance from the chain line or when it is not possible to set up a right angle due to some difficulties. Such offsets are taken in following manner :→

Suppose AB is a chain line and P is the corner of a building. Two points a and b are taken & their chainages are noted.



Their distances ap & bp are measured & noted in the field book. Then ap & bp are the oblique offsets.

Perpendicular offsets are preferred for the following reasons :→

(a) They can be taken very quickly
(b) The progress of survey is not hampered
(c) The entry in field book is easy.
(d) Plotting of offsets also becomes easy.

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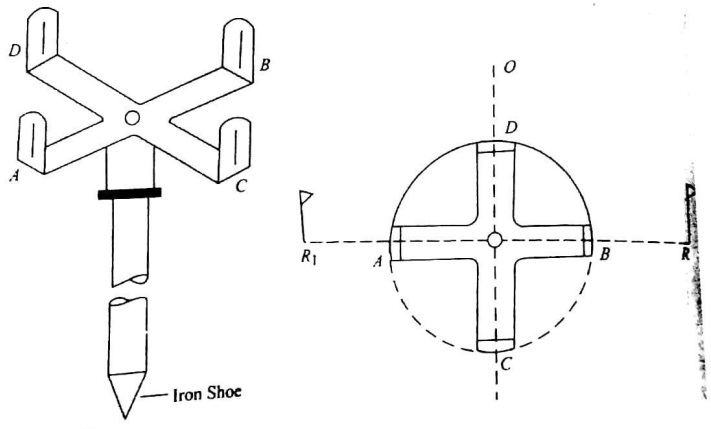
Instruments for setting offsets

(A) **CROSS-STAFF** :- The cross-staff is a simple instrument for setting out right angles. There are 3 types of cross-staffs.

1) Open cross-staff.
2) Fresh cross staff.
3) Adjustable cross-staff.

1) **Open cross-staff** :- It is the simplest type of cross-staff which is most commonly used.
↳ It consists of a head and a leg. The two arms AB and BC are at right angles to each other. The vertical slits are meant for sighting the object and the ranging rods. The cross-staff is mounted on a wooden pole of 1.5m length and 2.5 cm diameter. The pole is fitted with an iron shoe.



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For setting out a perpendicular on a chain line, the cross staff is held vertically at the approximate position. Suppose slits A and B are directed to the ranging rods (R, R1) fixed at the end stations. Slits C and D are directed to the object (O).

Looking through slits A and B, the ranging rods are bisected. At the same time, looking through slits C and D, the object O is also bisected. To biseet the object and the ranging rods simultaneously, the cross-staff may be moved forward or backward along the chain line.

2) French cross-staff:- It consists of an octagonal brass tube with slits on all the eight faces. It has alternate vertical slits and on opposite vertical faces, windows are provided. Vertical fine wires are stretched on each of the four sides. These are used for setting out right angles. Vertical right slits at the centres of opposite slits, make angles of 45° with each other. With this arrangement, it is some time, possible to set out 45° angle also.

The base of french cross staff is fitted on a pointed staff with socket. It

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a socket. It is inferior to the open cross staff because sights are too close i.e. 8cm apart.

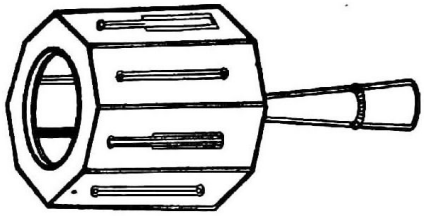

Fig. 3.21. A French cross staff.

3) Adjustable cross staff :— It consists of a brass cylindrical tube about 8cm in dia & 10cm deep, divided at the middle. The lower portion remains fixed & upper portion can be rotated relative to the lower one, by a circular rack & pinion arrangement. Sighting slits are provided in both the parts. The lower part is graduated into degrees & their sub-divisions, while upper one carries a vernier. On the top of instrument, a magnetic compass is provided to take bearings of lines. It is also an inferior type & used for approximate work.

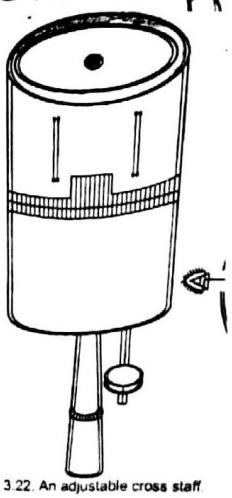

Fig. 3.22. An adjustable cross staff.

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(B) OPTICAL SQUARE :-

An optical square is also used for setting right angles. It consists of a small circular metal box of 5cm diameter and 1.25cm depth. It has a metal cover which slides round the box to cover the slits. The following are internal arrangements of optical square.

1) A horizon glass H is fixed at the bottom of the box. The lower half of the glass is unsilvered and upper half is silvered.

2) An index glass I is also fixed at the bottom of the box which is completely silvered

3) The angle between index glass and horizon glass is maintained at 45°.

4) The opening 'e' is a pinhole for eye E, 'b' is a small rectangular hole for ranging rod B, 'p' is a large rectangular hole for object P.

5) The line EB is known as horizon sight & IP as index sight.

6) The horizon glass is placed at 120° with the horizon sight. The index glass is placed at an angle of 105° with index sight.

7) The ray of light from P is first reflected from I, then it is further reflected from H, after which it ultimately reaches the eye E.

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PRINCIPLE

According to the principle of reflecting surfaces, the angle between the first incident ray & last reflected ray is twice the angle between the mirrors. In this case the angle between the mirrors is fixed at 45°, So, the angle between horizon sight and index sight will be 90°.

Setting up perpendicular by optical square :—

(a) The observer should stand on the chain line & approximately at the position where perpendicular has to be set up.

(b) The optical square is held by arm at the eye level. The ranging rod at forward station B is observed through unsilvered portion on the lower part of horizon glass.

(c) Then the observer looks through the upper silvered portion of horizon glass to see the image of object P.

(d) Suppose, the observer finds that the ranging rod B & image of object P donot coincide. Then, he should move forward or backward along chain line until the same exactly coincide.

(e) At this position, observer mark, a point on the ground to locate foot of the perpendicular.

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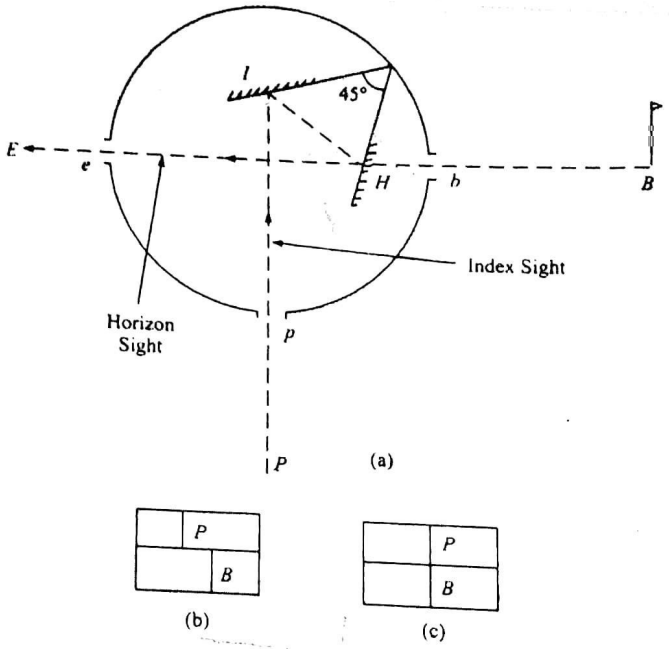

Fig. 2.23 Optical Square

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3.0 Angular Measurement and Compass Surveying

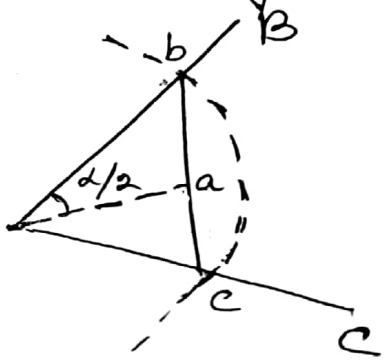
3.1 Measurement of angles with chain, tape & Compass :—

In chain traversing the entire work is done by a chain or tape and no angle measuring instrument is needed. The angles computed by the measurements is known as chain angles.

Derivation of formula for chain angles :—

First Method :—

Consider the angle BAC equal to α. With A as centre & radius equal to chain length i.e. 30m or 20m, draw an arc intersecting AB and AC at b & c respectively.



Bisect bc at a.

then, $\sin\frac{\alpha}{2} = \frac{ab}{Ab} = \frac{bc}{2Ab}$ $(\because ab = bc/2)$

$\Rightarrow \sin\left(\frac{\alpha}{2}\right) = \frac{bc}{40}$ (for a 20m chain).

$\because Ab$ = one chain length.

$$\left(\frac{\alpha}{2}\right) = \sin^{-1}\frac{bc}{40}.$$

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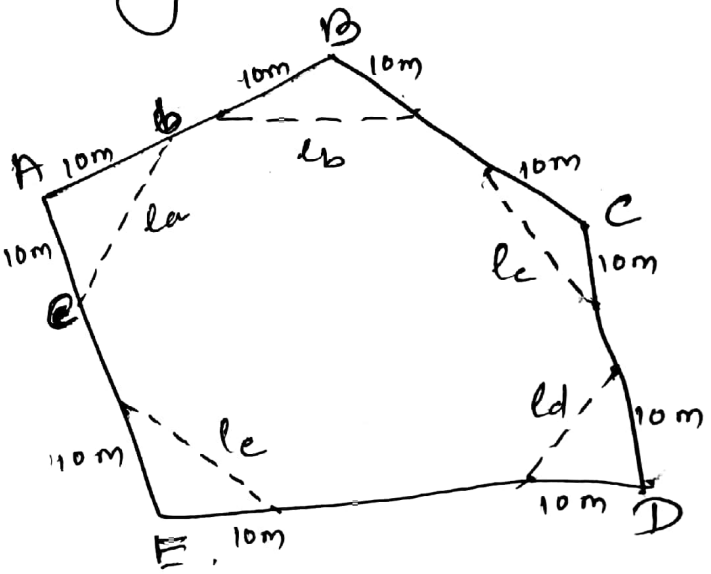
$\Rightarrow \alpha = 2\sin^{-1}\frac{bc}{40}$ for 20 m chain

$\alpha = 2\sin^{-1}\frac{bc}{60}$ for 30 m chain.

Second Method

Measurement of included angles of compass traverse :— When a compass is not available the included angles may be measured as below :—

Let ABCDE be a traverse whose stations A, B, C, D and E are marked on the ground after ascertaining the intervisibility of adjacent stations.



Let us measure angle BAE with tape only.

(i) Fix the position of traverse station A with a pencil mark on the top of a wooden peg driven into the ground.

(ii) Align the directing ranging rods held at B and E respectively.

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(iii) Measure the distance Ab on AB and Ae on AE equal to 10m & mark location of b and e with chain pins.

(iv) Measure the distance be with a tape correct to mm.

Let be = la.

By applying cosine formula,

$$\cos\alpha = \frac{(Ab)^2 + (Ac)^2 - (be)^2}{2 \cdot Ab \cdot Ac}, \text{ we get,}$$

$$\cos\alpha = \frac{10^2 + 10^2 - (la)^2}{2 \times 10 \times 10} = \frac{200 - la^2}{200}$$

$$\alpha = \cos^{-1}\frac{200 - la^2}{200} \text{ in degrees.}$$

(v) The included angles B, C, D & E may be measured in similare manner.

(vi) Apply geometrical check on the observed angle i.e.,

Sum of included angles = $(n-2) \times 180^\circ$.

n = no. of sides of the closed traverse

Q:- ABCD is a closed traverse where interior angles are ~~mea~~ to be measured. The details are as given below :—

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Station	Side	Distance (m)
A	la	11.90
B	lb	15.97
C	lc	11.76
D	ld	16.38.

Determine the value of each angle and provide a geometrical check.

$\angle A = \cos^{-1} \frac{200 - 11.9^2}{200} = 0.29195 = 73^\circ 1' 31''$

$\angle B = \cos^{-1} \frac{200 - 15.97^2}{200} = 0.27520 = 105^\circ 58' 26''$

$\angle C = \cos^{-1} \frac{200 - 11.76^2}{200} = 0.30851 = 72^\circ 01' 50''$

$\angle D = \cos^{-1} \frac{200 - 16.38^2}{200} = -0.34152 = 109^\circ 58' 10''$.

Sum $= 360^\circ 59' 57''$.

Interior angles $= (2n-4) \times 180$

$= 360^\circ$.

error $= 360^\circ - 360^\circ = 59' 57''$.

Correction to each angle $= \frac{59' 57''}{4} = 14' 59''$

say $15'$.

Setting out a 60° angle with chain & tape only

Let AX be the given line.

(i) Measured AB = 30m along AX and fix a ranging rod at B.

(ii) Fix mid point C at 15 m either from A or B.

(iii) Fix two points a and b equidistant from C.

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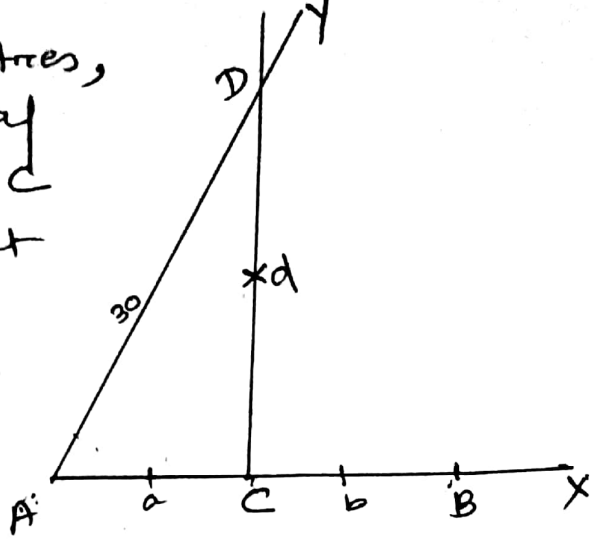
with a and b as centres, draw two arcs of equal radii (greater than ac or bc) to intersect at d.

(v) Join Cd and prolong it.

(vi) With A as centre, draw an arc of 30m radius to cut Cd at D.

(vii) Prolong AD to Y.

(viii) YAX is the required angle of 60°.



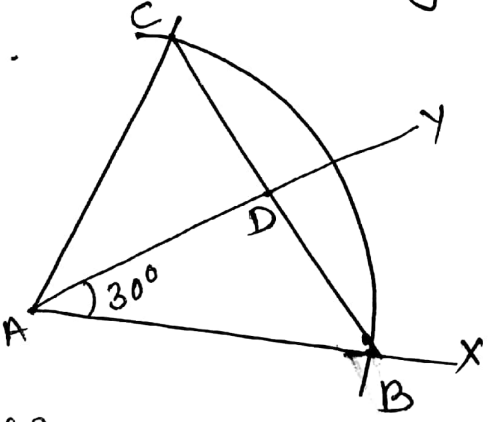
To check accuracy, proceed as under:—

(i) Measure CD which should be $= \sqrt{30^2 - 15^2} = 25.98$m

(ii) Measure BD which should be 30m.

Setting out 30° angle with chain & tape only

Let AX be the given line.



(i) Measure AB = 30m along AX & fix chain pin at B.

(ii) With A as centre & radius equal to AB, draw an arc.

(iii) With B as centre & radius equal to AB, draw another arc to cut at C.

(iv) Join BC & bisect it at D.

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(v) Join AD & prolong it to Y.

(vi) YAX' is required angle of 30°.

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3.2 Compass :—

1) Types of compasses :— there are 2 types of compasses :—

a) The prismatic compass
b) The surveyor's compass.

(a) Prismatic compass :— In this compass, the readings are taken with the help of a prism. Following are the essential parts of this compass:

(a) Compass Box :— The compass box is a circular metallic box (metal should be non-magnetic) of 8 to 10 cm diameter. A pivot with a sharp point is provided at the centre of the box.

(b) Magnetic needle and graduated ring :— The magnetic needle is made of a ~~broad~~ magnetised iron bar. The bar is pointed at both ends. The magnetic needle is attached to a graduated aluminium ring.

The ring is graduated from 0° to 360° clockwise, and graduations begin from the south end of the needle. Thus, 0° is marked at the south, 90° at the west, 180° at the north and 270° at the east.

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The degrees are again subdivided into half degrees. The figures are written upside down. The arrangement of the needle & ring contains an agate cap pivoted on the central pivot point. A rider of brass or silver coil is provided with the needle to counter balance its dip.

[dip of magnetic needle :— If a needle is perfectly balanced, it does not remain in balanced position after it is magnetised. This is due to the magnetic influence of earth. The needle is found to be inclined towards the pole. This inclination of needle with horizontal is known as dip.

It is found that north end of needle is deflected downwards & south end is deflected upward in northern hemisphere & vise versa].

(c) Sight vane and prism :— The sight vane and the reflecting prism are fixed diametrically opposite to the box. The sight vane is hinged with the metal box and consists of a horsehair at the centre. The prism consists of a sighting slit at the top & two smaller circular holes, one at the bottom of the prism & the other at the side of observer's eye.

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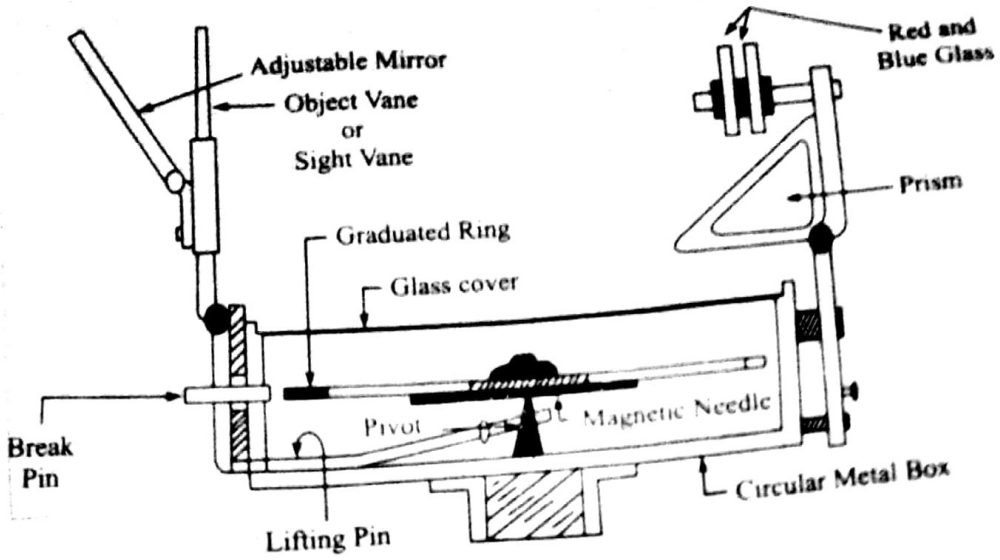

Fig. 3.12 Prismatic Compass

(d) Dark Glasses :— Two dark glasses are provided with the prism. The red glass is meant for sighting luminous objects at night and blue glass for reducing the strain on observer's eye in bright daylight.

(e) Adjustable Mirrors :— The mirrors is provided with the sight vane. The mirrors can be lowered or raised and can also be inclined. If any object is too low or too high w.r.t. the line of sight, the mirrors can be adjusted to observe it through reflection.

(f) Break Pin :- A break pin is provided just at the base of sight vane. If pressed gently, it stops oscillation of ring.

(g) Lifting pin :— It is provided just below the sight vane. When the sight vane is folded, it presses the lifting pin. Then

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the lifting pin lifts magnetic needle out of pivot point to prevent damage to the pivot head.

(h) Glass cover :— A glass cover is provided on the top of the box to protect the aluminium ring from dust.

(b) Surveyor's Compass :—

Surveyor's compass is similare to the prismatic compass except for the following points :—

(a) There is no prism on it. Readings are taken with naked eye.

(b) It consists of an eye-vane (in place of prism) with a fine slight slit.

(c) The graduated aluminium ring is attached to the circulare box. It is not fixed to the magnetic needle.

(d) The magnetic needle moves freely over the pivot. The needle shows the readings on the graduated ring.

(e) The ring is graduated from 0° to 90° in four or quadrants. 0° is marked at the north & south & 90° at the east and west. The letters E & W are interchanged from their true positions.

(f) No mirror is attached to the object vane

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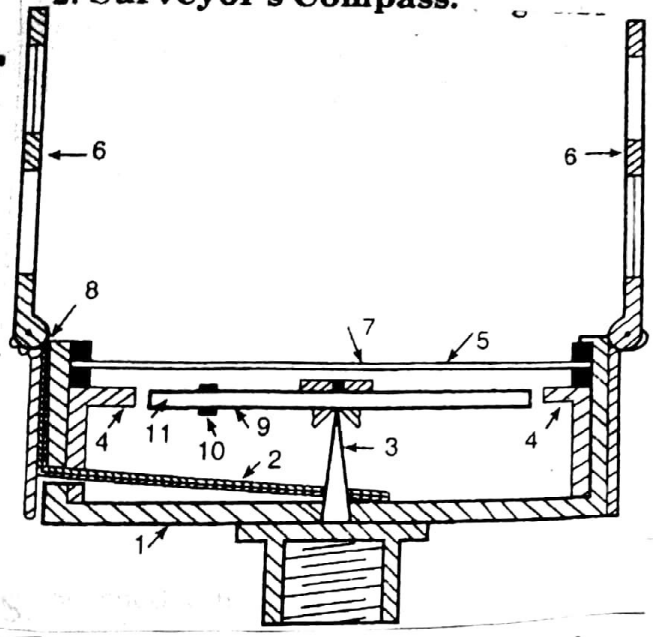
2. Surveyor's Compass.



1. Box
2. Lifting lever
3. Pivot
4. Circular graduated arc
5. Glass top
6. Sight vanes
7. Jewelbearing
8. Lifting pin
9. Magnetic needle
10. Rider
11. Metal pin

Temporary adjustment of Compass :-

1) **Fixing compass with tripod stand** :- The tripod stand is placed at required station with its legs well apart. The prismatic compass is held by the left hand & placed over the threaded top of the stand. After this, the compass box is turned clockwise by right hand. Thus the threaded base of the compass box is fixed with threaded top of stand.

2) **Centring** :- Normally the compass is centred by dropping a piece of stone from the bottom of compass box. Centring may also be done with a plumb bob held centrally below the compass box

3) **Levelling** :- It is done with the help of ball & socket arrangement provided on the top of tripod stand. This arrangement

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is loosened & the box is placed in such a way that the graduated ring rotates freely without touching either the bottom of the box or the glass cover on top.

4) Adjustment of Prism (for only prismatic compass) :— The prism is moved up and down till the figures on the graduated ring are seen sharp & clear.

5) Observation of Bearing :— After centring & levelling the compass box over the station, the ranging rod at the required station is bisected perfectly by sighting through the slit of prism & horsehair at the sight vane.

At this time, the graduated ring may rotate rapidly. The break pin is pressed very gently to stop the rotation. When, the ring comes to rest, the box is ~~stuck~~ struck very lightly to verify the horizontality of the ring. Then, the reading is taken from the graduated ring through the hole in the prism. The reading will be the magnetic bearing of the line.

Permanent Adjustment :- The adjustments which are made only if the fundamental relations between various parts of a compass are disturbed are called permanent adjustments.

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1) Adjustment of plate bubble :—

1) Set up the compass with a tripod on a firm ground.
2) Bring one of the levels parallel to a pair of levelling screws. Bring the bubble central by turning both the screws simultaneously either inward or outward.
3) Rotate the instrument through 90° till the level tube lies parallel to the line joining the third screw & mid point of first two screws. Centre the bubble with the third screw.
4) Rotate the instrument through 180° & if the bubble remains central, the instrument is in adjustment.

2) Adjustment of sight vane :—

1) Level the compass carefully.
2) Suspend a plumb bob roughly 25m away.
3) Move eye from to to bottom of the eye vane slit. Note whether the hair of the object vane remains in coincidence with the vertical plumb bob!

3) Adjustment of magnetic needle :—

1) Level the compass and read both the ends of the magnetic needle in any position.
2) Rotate the compass until the graduation originally opposite the north end of needle faces the south end.

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3) If the readings at the north end is now the same as that was originally at the south end, the needle is straight.

4) Adjustment of the pivot :-

1) If the difference of readings is variable, ascertain the position at which the discrepancy is maximum.

2) Remove the needle & bend the pivot at right angle to this position of the needle towards greater segment of the graduated ring formed by the needle.

Comparision

Sl. No.	Item.	Surveyor's Compass.	Prismatic Compass.
1)	Magnetic needle.	The needle is of edge bar type & also acts as an index.	The needle is broad type & doesnot act as an index.
2)	Graduated ring.	It is attached to the box & not to the needle. It rotates along with line of sight.	The graduated ring is attached with the needle. It doesnot rotate along with line of sight.
3)	Sighting vane.	The object vane consists of a metal vane with a vertical hair.	The object vane consists of a metal vane with a vertical hair.
4)	Reading system.	Readings are taken by directly looking through top of the glass.	Readings are taken with the help of prism.

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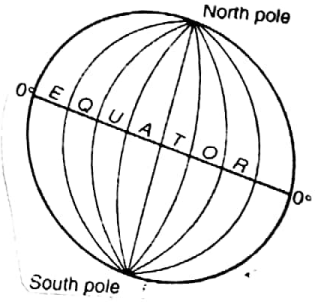
3.3 Designation of angles

1) **Concept of meridians** :- The fixed direction on the surface of earth, with reference to which, bearings of survey lines are expressed, is called meridian.

The meridians of reference of directions employed in surveying may be of following:-

(a) **True meridian** :- The line or plane passing through the geographical north pole, geographical south pole and any point on the surface of the earth, is known as the true meridian or geographical meridian.

The true meridian at a station is constant. The true meridians passing through different points on the earth's surface are not parallel, but converge towards the pole. But for survey in small areas, the true meridians passing through different points are assumed as parallel.



Angle between the true meridian & a line is known as true bearing of the line. It is also known as azimuth.

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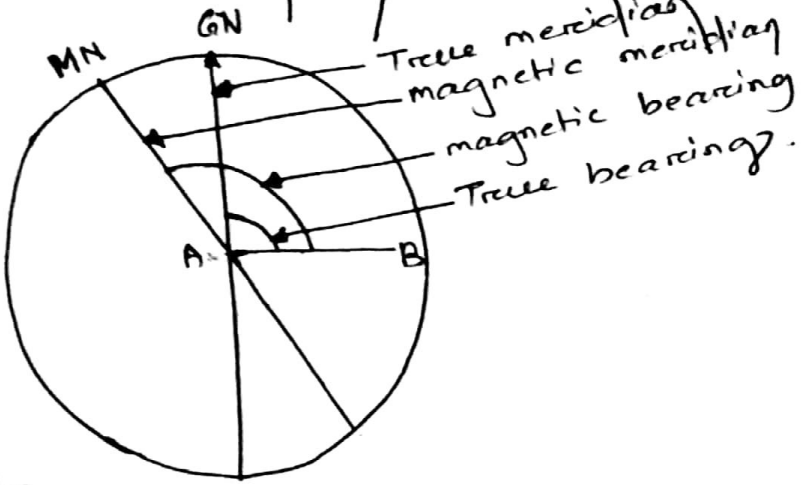
(b) Magnetic meridian :- When a magnetic needle is suspended freely & balanced properly, unaffected by magnetic substances, it indicates a direction. This direction is known as magnetic meridian.



The angle between magnetic meridian & a line is known as ~~mag~~ magnetic bearing or simply bearing of the line.

(c) Grid - meridian :- Sometimes, for preparing a map, some state agencies assume several lines parallel to the true meridian for a particular zone. These lines are termed as grid lines & central line is termed as grid meridian.

(d) Arbitary meridian :- Sometimes, for the survey of a small area, a convenient direction is assumed as meridian, known as arbitary meridian. An arbitary meridian has the merit of being invariable & its direction can be easily recovered at a future date, if required.

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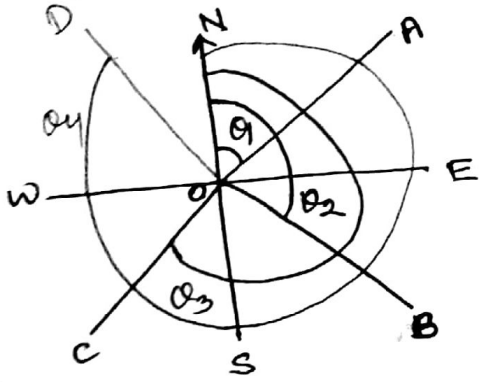
2) Concept of bearings :—

Magnetic bearings are designated by two systems :—

(a) Whole circle bearing (WCB)

(b) Quadrantal bearing (QB)

(a) **WCB** :— The magnetic bearing of a line measured clockwise from the north pole towards the line, is known as the whole circle bearing of that line. Such a bearing may have any value between 0° & 360°. The whole circle bearing of a line is obtained by prismatic compass.



WCB of OA = θ_1
WCB of OB = θ_2
WCB of OC = θ_3
WCB of OD = θ_4

(b) **QB** :— The magnetic bearing of a line measured clockwise or anticlockwise from the north pole or south pole (whichever is nearer the line) towards east or west is known as quadrantal bearing of the line. This system consists of 4 quadrants - NE, SE, SW & NW. The value of a quadrantal bearing lies between 0° and 90°; but the quadrants should always be mentioned. Quadrantal bearing

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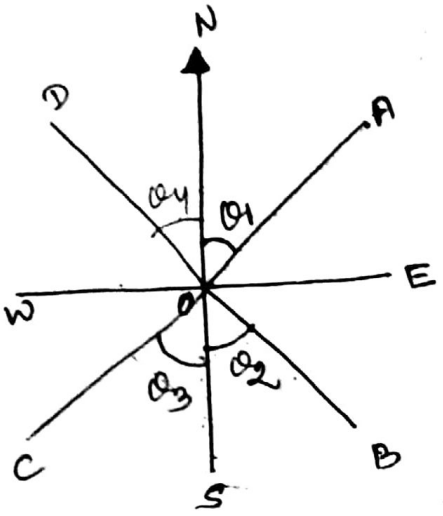
is obtained by surveyor's compass.



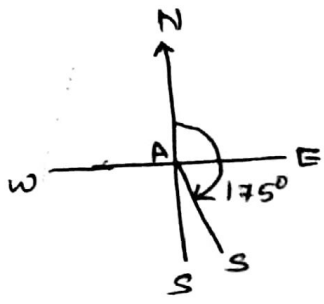
QB of OA = N θ_1 E
QB of OB = S θ_2 E
QB of OC = S θ_3 W
QB of OD = N θ_4 W.

Reduced bearing:- When the whole circle bearing of a line is converted to quadrantal bearing, it is termed as reduced bearing. The reduced bearing is similare to the quadrantal bearing. Its value lies between 0° & 90°, but the quadrants should be mentioned for proper designation.

WCB between	Corresponding RB	Quadrant
0° and 90°	RB = WCB	NE
90° and 180°	RB = 180° - WCB	SE
180° and 270°	RB = WCB - 180°	SW
270° and 360°	RB = 360° - WCB	NW.

eg:- Suppose WCB of a line = 175°

QB = 180° - 75° = 5°.



Hence QB or RB of the line is

S5°E.
south east.

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Q:- Convert the following WCBs to QBs.

(a) WCB of AB = 45° 30' - N 45° 30' E

(b) " " BC = 125° 45' - S 54° 15' E

(c) " " CD = 222° 15' - S 42° 15' W

(d) " " DE = 320° 30' - N 39° 30' W.

Q:- Convert following QBs to WCB.

(a) QB of AB = S 36° 30' W - 216° 30'

(b) " " BC = S 43° 30' E - 136° 30'

(c) " " CD = N 26° 45' E - 26° 45'

(d) " " DE = N 40° 15' W - 319° 45'

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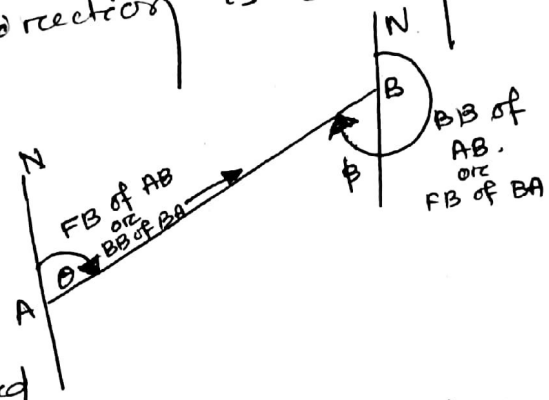
3.4 Use of compasses

1) Setting in field - Already discussed

2) Concept of forebearing & back bearing :-

Every line has two bearings : one is observed along the progress of survey or forward direction & is called fore bearing. The second is observed in the reverse or opposite direction is called back bearing.



Consider the line AB. Here, we consider the direction of meridian as upward and the bearing is measured clockwise from the meridian.

The bearing as measured at A along the progress of survey A to B is θ. So, the angle or bearing θ is fore bearing of line $\overline{AB}$.

Similarly the bearing as measured at B in the opposite direction of the progress of survey A to B, along clockwise direction is β. The bearing β is called the back bearing of line $\overline{AB}$.

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From fig, it is clear that fore bearing & back bearing of a line differ exactly by 180°.

i.e. **Back bearing = Fore bearing ± 180°**

(i) Use +ve sign when fore bearing is less than 180° & use negative sign when it is more than 180°.

(ii) In case of quadrantal bearing system, the numerical value of fore bearing and back bearing is equal but the quadrants are just opposite.

eg:- if FB = N 30° E, its BB = S30° W.

Q:- FBs of following lines are given. Find their BBs.

(a) FB of AB = 310° 30' → 130° 30'
BC = 145° 15' → 325° 15'
CD = 210° 30' → 30° 30'
DE = 60° 45' → 240° 45'

[FB ~ BB = 180° difference].

Problems On Included Angles

Q:- The bearings of lines OA, OB, OC, OD are 30° 30', 140° 15', 220° 45' & 310° 80', respectively. Find angles ∠AOB, ∠BOC and ∠COD.

Q:- FBs of lines AB, BC, CD and DE are 45° 30', 120° 15', 200° 30' & 280° 45' respectively. Find angles ∠B, ∠C and ∠D.

rest problems ----

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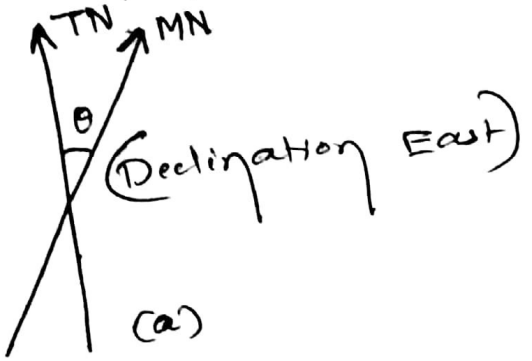
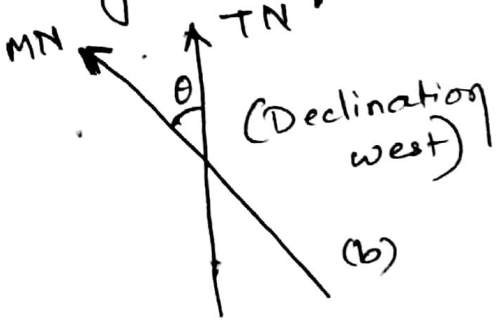
3.5 Effects of earth's magnetism

1) Dip of needle :- discussed earlier.

2) Magnetic declination :- the horizontal angle between magnetic meridian and true meridian is known as magnetic declination.





when the north end of magnetic needle is pointed towards west side of true meridian, the position is termed as Declination west (θ W).

when the north end of magnetic needle is pointed towards east side of true meridian, the position is termed as Declination east (θ E).

Variation of magnetic declination

Magnetic declination at a place is not constant. It varies due to following reasons :-

(a) Secular Variation :- the magnetic meridian behaves like a pendulum w.r.t. the true meridian. After every 100 years or so it swings from one direction to the opposite one & hence declination varies.

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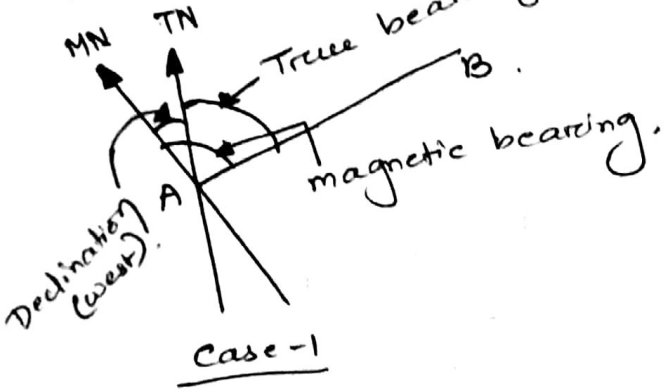
(b) Annual Variation :- Magnetic declination varies due to rotation of earth, with its axis inclined, in an elliptical path around the sun during a year. It is known as annual variation & amount is of 1 to 2 mins.

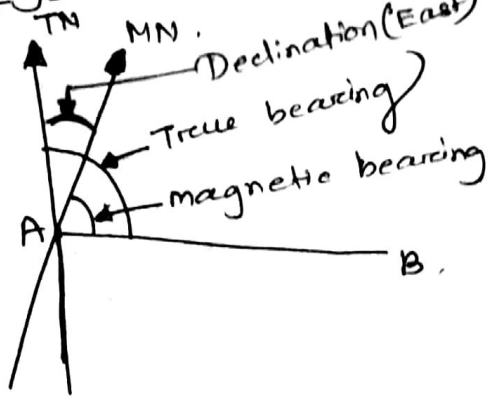
(c) Diurnal Variation :- Magnetic declination varies due to rotation of earth on its ocon axis is 24 hrs. It is known as diurnal variation & amount is 3 to 12 mins.

(d) Irregular variation :- Magnetic declination is found to vary suddenly due to some natural causes such as earthquake, volcanic eruptions & so on. It is known as irregular variation.

Application of correction to magnetic declination :-



Case-I



case-II

(a) True bearing = Magnetic bearing ± Declination

[Use +ve sign when declination is east
Use -ve " " " " west.]

(b) Magnetic bearing = True bearing ± Declination

[Use +ve sign when declination is west
" -ve " " " " east].

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Q:- The magnetic bearing of a line AB is 135° 30'. What will be the true bearing if declination is 5° 15' W.

Q:- True bearing of a line CD is 210° 45'. What will be its magnetic bearing, if declination is 8° 15' W.

3.6 Errores in angles measurement with compass :-

Sources :-

Following are the kinds of errors which may occure while taking readings with a compass.

1) Instrumental errors :-

(a) The needle may not be perfectly straight and might not be balanced properly.

(b) The pivot point may be eccentric.

(c) The graduations of the ring may not be uniform.

(d) The ring may not rotate freely on account of the pivot point being blunt. This may occure due to the head of pivot being broken because of careless handling.

(e) The sight vane may not be vertical

(f) Horse hair may not be straight & vertical

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2) Personal Errors :—

(a) The centring may not be done perfectly over the station.
(b) The graduated ring may not be levelled.
(c) The object may not be bisected properly.
(d) Readings may not be taken or entered correctly due to carelessness.
(e) The observer may be carrying magnetic substances.

3) Other sources of errors : —

(a) There may be local attraction due to presence of magnetic substances near the station.
(b) The magnetic field could vary on account of some natural cause.
(c) Magnetic declination may vary.
(d) Incorrect bisection of ranging rods.
(e) Incorrect reading of compass.

Precautions :—

The errors in compass surveying can be minimized by taking following precautions :—

1) Set up and level the compass carefully.
2) Stop the vibrations of the needle by gently pressing the break-pin, so that it may come to rest soon.
3) Always look along the needle & not across it,

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4) When the instrument is not in use, its magnetic needle should be kept off the pivot. If it is not done, the pivot is subjected to unnecessary wear which may cause sluggishness of the magnetic needle.

5) Readings should be taken along line of sight and not from any side.

6) When compass has to be shifted from one station to another, the sight vane should be folded over glass cover.

7) Compass box should be tapped gently before taking reading to find out whether the needle rotates freely.

8) The glass covers should not be dusted with a handkerchief, because the glass may be charged with electricity & needle may be deflected from its true direction. The glass covers should be dusted with moist finger.

3.7. Principles of traversing

As we already know, surveying involves a series of connected lines, known as traversing. Sides of traverse are known as traverse legs.

In traversing, the lengths of lines are measured by chain & directions are fixed by compass or theodolite or by forming glass with chain & tape.

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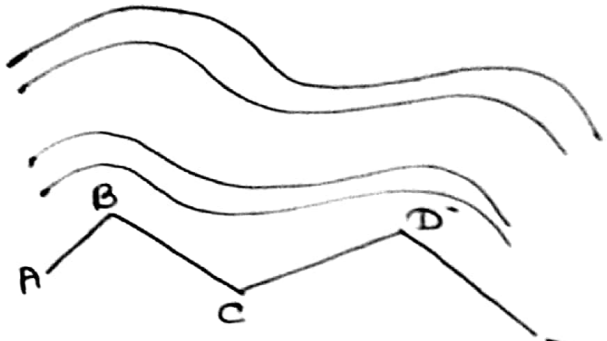
A traverse may be of two types :—
(a) Open traverse, (b) Closed traverse.

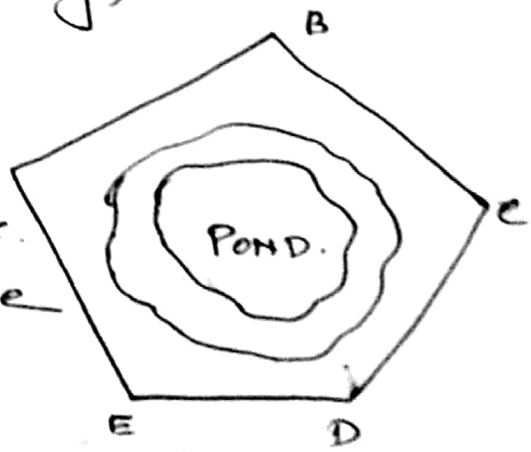
(a) Open traverse :- When a sequence of connected lines extends along a general direction & does not return to the starting point, is known as open traverse or unclosed traverse

Open traverse is suitable for the survey of roads, rivers, coast lines etc.



(b) Closed traverse :- When a series of a connected lines forms a closed circuit i.e. when the finishing point coincides with the starting point of a survey, it is called a 'close traverse'.

Here ABCDEA represents a closed traverse. Closed traverse is suitable for the survey of boundaries of ponds, forests, estates etc.



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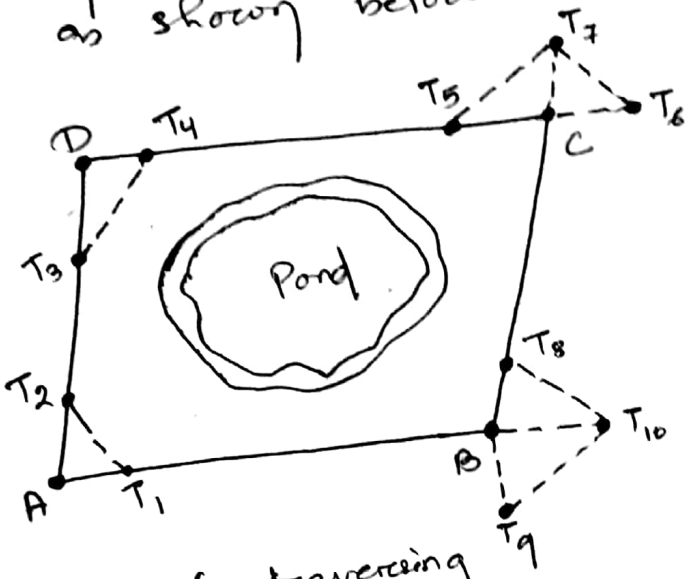
Methods of traversing :-

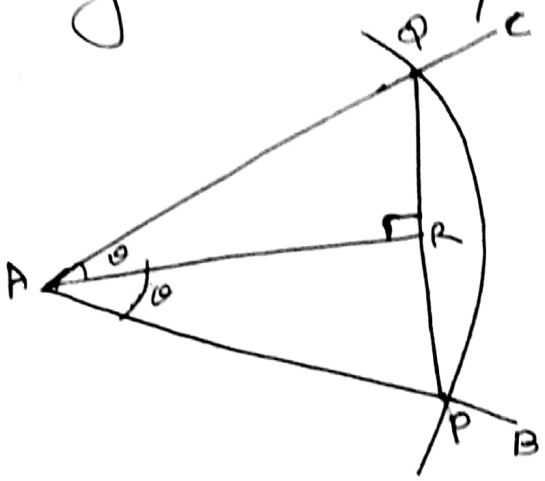
Traversing may be conducted by following methods:-

1) Chain traversing :-

Chain traversing is mainly conducted when it is not possible to adopt triangulation. In this method, the angles between adjacent sides are fixed by chain angles. The entire survey is conducted by chain & tape only and no angular measurements are taken. When it is not possible to form triangles as, for examples in a pond, chain traversing is conducted as shown below :-



(a) chain traversing



(a) First Method :- Suppose a chain angle is to be formed to fix the directions of sides AB & AD. Tie stations T_1 and T_2 are fixed on lines AB and AD. The distances AT_1, AT_2 & T_1T_2 are measured. Then the angle $\angle T_1AT_2$ is said to be chain angle. So chain angle is fixed by the line T_1T_2.

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(b) Second method:- Sometimes chain angles are fixed by a chord. Suppose the angle between the lines AB and AC is to be fixed. Taking A as centre and a radius equal one tape length (15m), an arc intersecting the lines AB and AC at points P and Q respectively is drawn. The chord PQ is measured and bisected at R.

Let $\angle PAR = \theta$.

$\angle BAC = 2\theta$.

$AP = AQ = 15m$.

In triangle PAR, $\sin\theta = \frac{PR}{AP} = \frac{2PR}{2AP}$

$= \frac{PQ}{30}$.

$\theta = \sin^{-1}\frac{PQ}{30}$.

The angle θ can be calculated from the above equation, and the chain angle $\angle BAC$ can be determined accordingly.

2) Compass traversing:-

In this method, the fore and back bearings of the traverse legs are measured by a prismatic compass and the sides of the traverse by chain and tape. Then the observed bearings are verified and necessary corrections ~~are~~ for local attraction are applied.

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3) Theodolite traversing :—

In such traversing, the horizontal angle between the traverse legs are measured by a theodolite. The lengths of the legs are measured by chain or by employing the stadia method. The magnetic bearing of the starting leg is measured by a theodolite. Then the magnetic bearings of other sides are calculated. This method is very accurate.

4) Plane Table traversing :—

In this method, a plane table is set at every traverse station in the clockwise or anticlockwise direction, and the circuit is finally closed. During traversing, the sides of traverse are plotted according to any suitable scale.

3.8 Local attraction

Causes :— A magnetic needle indicates the north direction when freely suspended or pivoted. But if the needle comes near some magnetic substances, such as iron ore, steel structures, electric cables conveying current etc., it is found to be deflected from its true direction. & doesnot show actual north. This disturbing influence of magnetic substance is known as local attraction.

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Detection :- To detect the presence of local attraction the fore and back bearing of a line should be taken. If the difference of the fore and back bearings of the line is exactly 180° then there is no local attraction.

If the FB and BB of a line donot differ by 180°, then needle is said to be affected by local attraction, provided there is no instrumental errors.

Errors :- To compensate for the effect of local attraction, the amount of errors is found out and is equally distributed between fore and back bearings of the line.

For eg :-

Observed FB of AB = 60° 30'

BB of AB = 240° 0'

Calculated BB of AB = 60° 30' + 180° 0'

= 240° 30'.

Errors = 240° 30' - 240° 0' = 30'.

Corrected BB of AB = 1/2 (240° 0' + 240° 30')

= 240° 15'.

Corrected FB of AB = 240° 15' - 180° 0'

= 60° 15'.

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Correction :-

(a) First Method :- The interior angles of a traverse are calculated from the observed bearings. Then an angular check is applied. The sum of the interior angles should be equal to $(2n-4) \times 90^\circ$ (n being the number of sides of the traverse). If it is not so, the total error is equally distributed among all the angles of the traverse.

Then, starting from the unaffected line, the bearings of all the lines may be corrected by using the corrected interior angles. This method is very laborious and is not generally employed.

(b) Second Method :- In this method, the interior angles are not calculated. From the given table, the unaffected line is first detected. Then, commencing from the unaffected line, the bearings of the other affected lines are corrected by finding the amount of correction at each station. This is an easy method, and one which is generally employed.

Q:- The following are the observed bearings of the lines of a traverse ABCDEA with a compass in a place where local attraction was suspected.

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Line	FB	BB.
AB	191° 45′	13° 0′
BC	39° 30′	222° 30′
CD	22° 15′	200° 30′
DE	242° 45′	62° 45′
EA	330° 15′	147° 45′

Find correcet bearings of the lines.

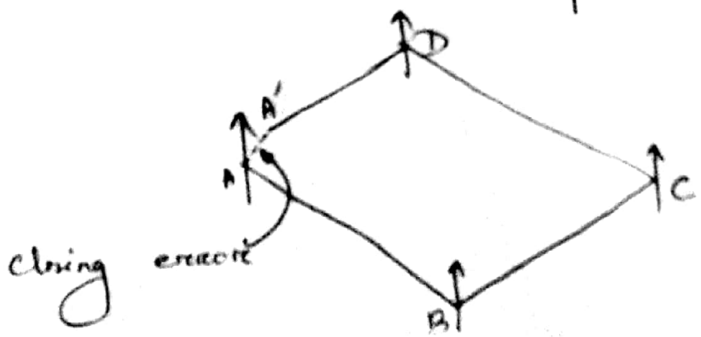
3.9 Plotting of traverse :-

1) By parallel meridian through each station :-

The starting point A is suitably selected on the paper & a line representating the north line. The bearing of line AB is plotted by protactor & it's length is plotted to any suitable scale.

At station B, north line is drawn parallel to to north line drawn at A. Then bearing of BC is plotted & it's length is marked according to previous scale.

Similarly all traverse legs are plotted. In case of closed traverse, there may be a closing error which should be adjusted graphically.



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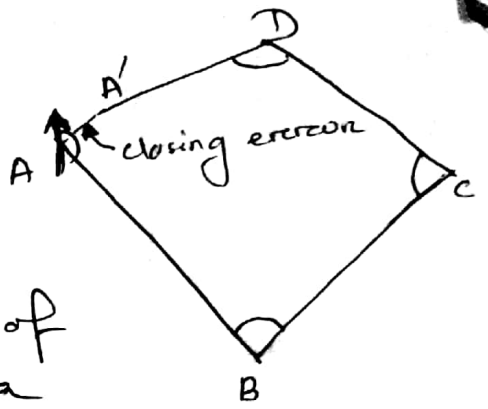
2) By considering included angles :-

The starting station A is suitably selected on the sheet. A line representating the north line is drawn through station A. The bearing of one AB is plotted by a protactor & distance AB is marked to a suitable scale.



At station B, angle B is plotted & distance BC is marked according to previous scale.

This process is continued untill all the lines have been plotted. On this case also, there may be a closing error which have to be adjusted graphically.

3) By considering central meridian

A suitable point O is selected at the centre of drawing sheet. A line representing the magnetic meridian is drawn through the point. Then a protactor is placed at O & all the lines namely ab, bc, cd & da are drawn according to their bearings.

Then a starting point A is suitably selected on the sheet. A line AB is drawn parallel to ab & length AB is plotted to

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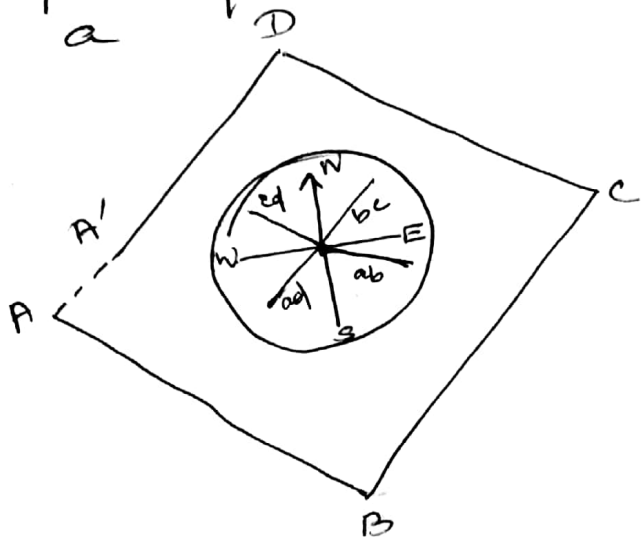
any suitable scale. Again from B, line BC is drawn parallel to line bc and length BC is plotted to previous scale.

This process is continued until all the lines have been drawn. In this case also, there may be a closing error which is adjusted graphically.



Adjustment of closing error

When a closed traverse is plotted, the finishing and starting points may not coincide. The distance by which the traverse fails to close is said to be the closing error.

Such an error may occur due to mistakes made in the measurement of lengths and bearings of the lines, or because of an error in plotting.

If the closing exceeds a certain permissible limit, the field work should

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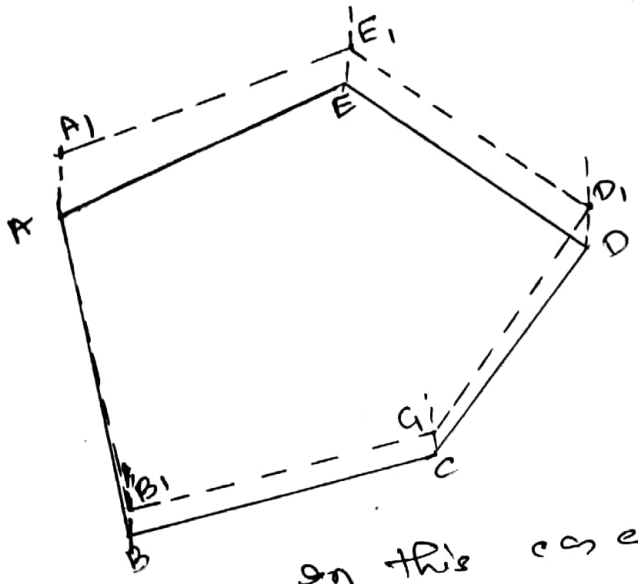
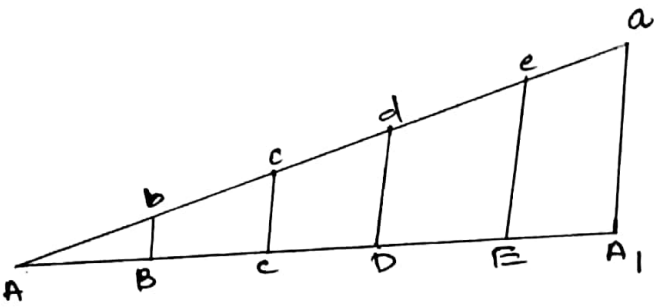
be repeated. But when the error is within the permissible limit, it is adjusted graphically by Bowditch's rule as explained below.

Suppose a traverse $AB_1C_1D_1E_1A_1$ is plotted to any suitable scale (RF = 1/400 (say)).





In this case, the traverse fails to close by a distance AA_1, which is the closing error.

To adjust this error, a horizontal line AA_1 is drawn to represent the perimeter of the traverse to another scale (RF = 1/2000). On this line distances AB_1, B_1C_1, C_1D_1, D_1E_1 & E_1A_1 are set off according to the corresponding measured lengths of the traverse legs.

A perpendicular A_1a is drawn equal to the amount of closing error, after

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which the line Aa is drawn. From the points B_1, C_1, D_1 and E_1 the lines B_1B, C_1C, D_1D and E_1E are drawn parallel to A_1a. These intercepts represents the amount by which the respective stations are to be shifted.

Lines are drawn parallel to the closing errore through stations B_1, C_1, D_1 and E_1. Then the intercepts B_1b, C_1c, D_1d and E_1e are set off along the parallel lines drawn through the respective stations. In this manner, the adjusted traverese ABCDEA is obtained.

Limits of closing errore

The angular errore of closeure should not exceed $15'\sqrt{N}$ minutes, where N is the number of sides of the traverese.

$$\text{Relative closing errore} = \frac{\text{amount of closing error}}{\text{perimeter of traverese}}$$

The value should not exceed 1/600.

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Map Reading, Cadastral Maps & Nomenclature

Cadastre:- A parcel based and up-to-date land information system containing a record of land that describes the geometry of parcels and linked to records such as nature of interests, ownership & value of the parcel & its improvements

Cadastral surveys document the boundaries of land ownership by the production of documents, diagrams, sketches, plans, charts and maps. They were originally used to ensure reliable facts for land valuation and taxation.

Mapping requirements:-

- Basic scale shall be in the range of 1:1000 to 1:5000.
- Follow cassini maps projection.
- Units shall be square meter.
- Map contents need to be standardized including symbols

Cadastral map:- A map showing the boundaries of subdivisions of land, usually with the bearing & lengths thereof and the areas of the individual tracts for the purpose of describing and recording ownership. A cadastral maps also shows culture, drainage and other features relating to the value and use of the land.

map:- A representation of all or a portion of the earth or other celestial body, showing relative size & position of features to some given scale or projection. A map may emphasize, generalize or omit the representation of certain features to satisfy specific requirement.

map scale:- A fractional scale, is the ratio that any small distance on the map bears to corresponding distance on the earth. It may be written in the form of a fraction 1/10,000 or as a proportion 1:10,000. It is representative in linear unit.

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Study of direction:- Bearing or direction of a line is measured from north or south to east or west, not exceeding 90°. Eg:- N 30° W or S 87° E.

Study of signs and symbols:- Standard symbols should be employed on the maps. Eg. of standard mapping symbols & cartographic specifications are shown in fig.

4.2 Cadastral map Preparation Methodology

~~Preparation is essential before undertaking a~~ map

Grid reference:-

It is defined locations in maps or indicates a location on the map in terms of a series of vertical & horizontal grid lines, identified by numbers or letters.

Grid square:-

The square of a map denoted by a grid formed by a series of numbered horizontal & vertical lines.

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FIGURE 2. Examples of Standard Mapping Symbols (*continued*)

12.7 CONTINUED 1"= 400' MAPS

SUBJECT	LEROY/EQUIVALENT PEN WEIGHT/ TEMPLATE	EXAMPLE
24. Road and Street Names	1 Pen/ 120 L Template	COUNTY ROAD
25. Alleys	0 Pen/ 80 L Template	ALLEY
26. Parcel Number	1 Pen/ 140 L Template	2
27. Original Lot Number	0 Pen/ 120 L Template Slant	20 | 21 | 22
28. Creeks, Streams Names	0 Pen/ 120 L Template Slant	DRAGOON CREEK
29. Rivers, Lakes Names	1 Pen/ 175 L Template Slant	CLINTON LAKE
30. Water Acreage	0 Pen/ 80 L Template Slant	35 AC. (c)
31. Deed Dimensions	0 Pen/ 80 L Template	175'
32. Scaled Dimensions	0 Pen/ 80 L Template	180' (s)
33. Deed Acreage	0 Pen/ 120 L Template	120 Ac. (d)
34. Calculated Acreage	0 Pen/ 120 L Template	127 Ac. (c)
35. Church, Cemetery, School, Etc.	0 Pen/ 80 L Template	SHILOH CEMETERY
36. Transmission Lines	0 Pen/ 80 L Template	K.P.&L. 100' R/W EASEMENT
37. Adjacent Map Reference	0 Pen/ 120 L Template	012
38. Easement Line	0 Pen	
39. Map Number	2 Pen/ 200 L Template	012-04-10
40. Conflict	0 Pen/ 120 L Template	CONFLICT
41. Road Dimensions	0 Pen/ 80 L Template	60' R/W
42. Vacated Street	0 Pen	
43. Leasehold Imp. Boundary Lines	0 Pen	13.01 L. I.
44. Leasehold Improvement	1 Pen/ 140 L Template	
45. Mineral Rights	1 Pen/ 140 L Template	12 320 Ac. M.R.

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FIGURE 2. Examples of Standard Mapping Symbols

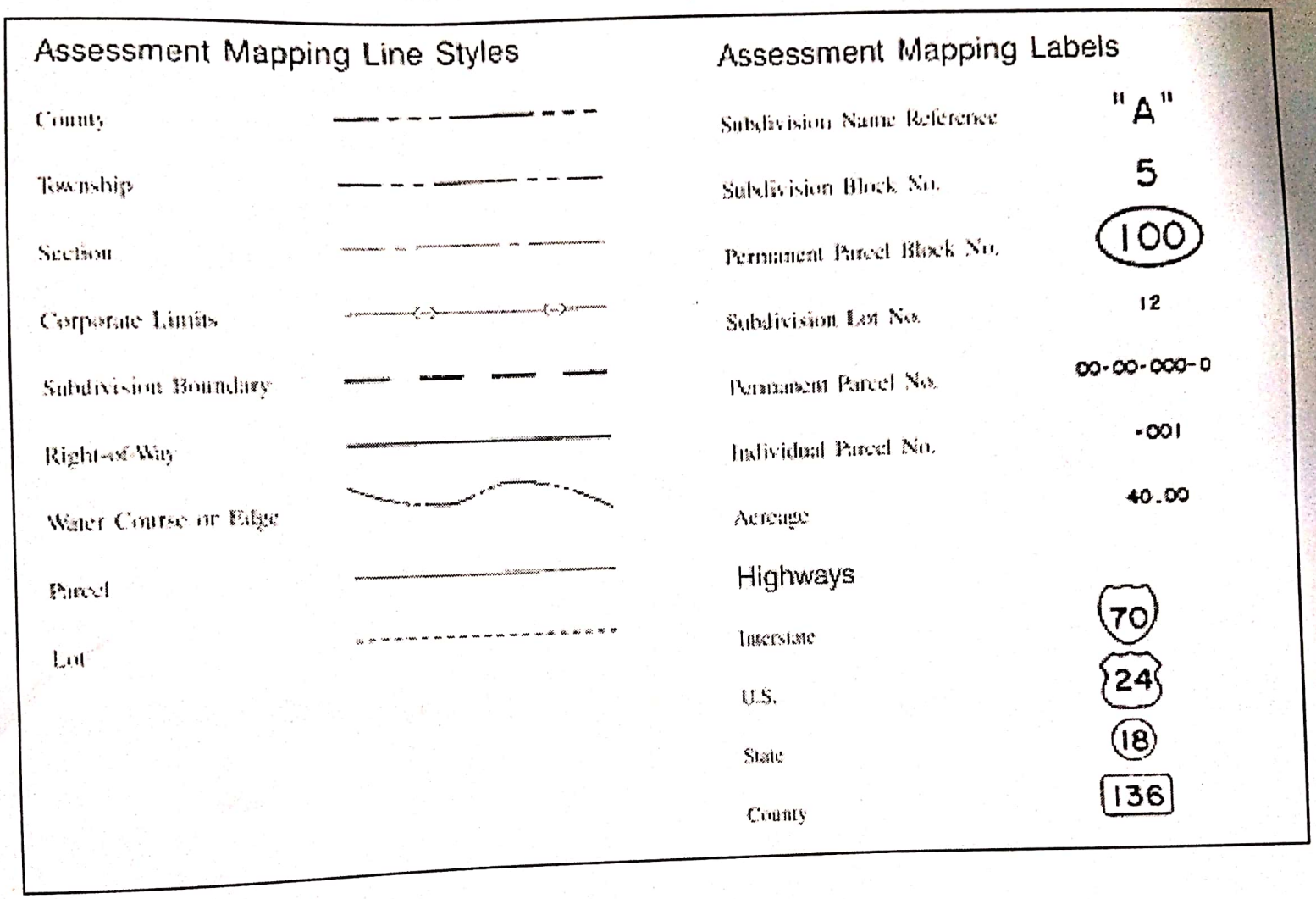

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FIGURE 2. Examples of Standard Mapping Symbols (*continued*)

12.6 PEN WEIGHT AND TEMPLATE GAUGE FOR 1" = 200', 1" = 100' MAPS

SUBJECT	LEROY/EQUIVALENT PEN WEIGHT/TEMPLATE	EXAMPLE
1. Road and Street Names	1 Pen / 120 L Template	TOPEKA AVE
2. Alleys	0 Pen / 80 L Template	ALLEY
3. Parcel Number	1 Pen / 140 L Template	4
4. Original Lot Number	0 Pen / 120 L Template Slant	22
5. Creeks, Streams, Etc.	0 Pen / 120 L Template Slant	RYE CREEK
6. Rivers, Lakes, Etc.	1 Pen / 175 L Template Slant	KANSAS RIVER LAKE PERRY
7. Deed Dimensions	0 Pen / 100 L Template	100
8. Scaled Dimensions	0 Pen / 100 L Template	105 (s)
9. Road Dimensions	0 Pen / 80 L Template	60' R/W
10. Deed Acreage	0 Pen / 100 L Template	40 Ac. (d)
11. Calculated Acreage	0 Pen / 100 L Template	44 Ac. (c)
12. Church, Cemetery, School Names Etc.	0 Pen / 80 L Template	SHAWNEE COUNTY COURTHOUSE
13. Ownership Block Number	2 Pen / 240 L Template	"04"
14. Original Block Number	3 Pen / 200 L Template	4
15. Transmission Lines	0 Pen / 80 L Template	K.P.&L. 100' R/W EASEMENT
16. See Note	0 Pen / 120 L Template	SEE 1" = 100' MAP 011-11-40
17. Easement Line	0 Pen	
18. Corner Dimension	0 Pen / 80 L Template	
19. Adjoining Map Number	0 Pen / 120 L Template	011-12-10
20. Conflict	0 Pen / 120 L Template	CONFLICT
21. Map Numbers	2 Pen / 200 L Template	011-12-14
22. State Line	4 Pen	NEBRASKA / KANSAS
23. County Line	4 Pen	SHAWNEE / OSAGE

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FIGURE 2. Examples of Standard Mapping Symbols (*continued*)

IP 6 CONTINUED 1"=200', 1"=100' MAPS

SUBJECT	LEROY/EQUIVALENT PEN WEIGHT/TEMPLATE	EXAMPLE
24. Township and Range Lines	4 Pen	
25. Section Lines	3 Pen	
26. Quarter Section Lines	1 Pen	
27. Corporate Limit Lines		TOPEKA CITY LIMITS
28. Railroad R/W	1 Pen	A.T.&S.F. R.R. 100' R/W
29. Highway R/W	1 Pen	50' R/W
30. Property Boundary Lines	1 Pen	P/L P/L
31. Original Lot Lines	0 Pen	
32. Water	0 Pen	
33. Land Hooks	0 Pen	
34. Transmission Lines	0 Pen	
35. State Name	2 Pen/ 200 L. Template	KANSAS
36. County Name	2 Pen/ 200 L. Template	SHAWNEE COUNTY
37. Township and Range Number	1 Pen/ 140 L. Template	T-22 S R-2W / T-23 S R-3W
38. Section Number	1 Pen/ 140 L. Template	2 1 / 11 12
39. Corporation Name	1 Pen/ 140 L. Template	TOPEKA CITY LIMITS
40. Railroad Name	0 Pen/ 120 L. Template Slant	BURLINGTON N. R.R.
41. Interstate Highway	0 Pen/ 140 L. Template	70
42. U.S. Highway	0 Pen/ 140 L. Template	24
43. State Highway	0 Pen/ 140 L. Template	18
44. County Highway	0 Pen/ 140 L. Template	136
45. S/D Limits	1 Pen	2
45-A S/D Limit Number	0 Pen/ 80 L. Template	
46. Vacated Street	0 Pen	
47. Leasehold Imp. Boundary Lines	0 Pen	13.01 L. I.
48. Leasehold Improvement	1 Pen/ 140 L. Template	
49. Mineral Rights	1 Pen/ 140 L. Template	12 320 Ac. M.R.

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IGURE 2. Examples of Standard Mapping Symbols (continued)

12.7 PEN WEIGHT AND TEMPLATE GAUGE FOR 1" = 400' MAPS

SUBJECT	LEROY/EQUIVALENT PEN WEIGHT/TEMPLATE	EXAMPLE
1. State Line	4 Pen	
2. County Line	4 Pen	
3. Township and Range Lines	4 Pen	
4. Section Lines	3 Pen	
5. Corporation Lines	3 Pen	
6. Railroad R/W	0 Pen	
7. Highway R/W	1 Pen	
8. Property Boundary Lines	1 Pen	
9. Original Lot Lines	0 Pen	
10. Water Line	0 Pen	
11. Land Hooks	0 Pen	
12. S/D Limits	1 Pen	
12-A S/D Limit Number	0 Pen/ 80L Template	
13. Transmission Lines	0 Pen	
14. State Name	2 Pen/ 200L Template	KANSAS
15. County Name	2 Pen/ 200L Template	SHAWNEE COUNTY
16. Township and Range Number	1 Pen/ 140L Template	T-14S R-2E / T-15S R-2E
17. Section Number	1 Pen/ 140L Template	6 5 / 7 8
18. Corporation Name	1 Pen/ 140L Template	TOPEKA CITY LIMITS
19. Railroad Name	0 Pen/ 80L Template Slant	BURLINGTON NORTHERN R.R.
20. Interstate Highway	0 Pen/ 140L Template	70
21. U.S. Highway	0 Pen/ 140L Template	40
22. State Highway	0 Pen/ 140L Template	6
23. County Highway	0 Pen/ 140L Template	127

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Cadastral map preparation methodology

Preparation is essential before undertaking a mapping program. The accessor should evaluate existing & needed personnel, facilities, data processing and technical administrative ~~procedure~~ support, investigate & determine funding available & establish preliminary schedules & completion dates.

Program Management

Mapping may be carried out in a variety of ways, depending on the scope and magnitude of the mapping project. Regardless of how mapping function is performed, responsibility for its management should be specifically assigned and a project plan developed as to serve as the organisation's blueprint for both the process of mapping & for the content & format of the final product.

Managers can expect to deal with contracts for photography, equipment & production, daily maps product, personnel training, budgeting, quality control & external standards.

Contracting for mapping services

When a mapping program or a major updating & remapping program is undertaken, consideration should be given to whether the project will be conducted in house or contracted to an appropriate vendor.

Many accessors office do not have experienced personnel necessary to produce & maintain cadastral maps & few have specialized equipment & expertise needed to produce base maps.

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Technical Specification

The jurisdiction must prepare a set of technical specifications that clearly defines the aerial photograph or mapping services to be performed.

The technical specifications should include :—

1) geographical areas to be mapped.
2) preliminary activities to be performed.
3) types of maps to be produced.
4) data to be displayed on the map.
5) Sources of data
6) Scale, size and layout of the map.
7) labeling
8) all other technical & accuracy requirements.

Evaluating mapping firms

A high level of technical expertise is required to evaluate mapping professionals & contract proposals. If the expertise is not readily available, consultants should be retained to assist in the effort.

Ownership & Map production & maintainance

Cadastral maps & ownership database should be maintained & published on a timely manner & on a continual basis as part of assessment roll production cycle & should be fully integrated into sales review.

Maps maintainance involves recording description, changes, making map corrections & notifying map users of the changes on a regular basis.

Maintainance also includes the constant correction & improvement of the maps from new &

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& more accurate survey data, documenting all records used in preparing the maps & in describing the locations of property corners.

Base Map Preparation

Cadastral maps should be prepared using base maps that have been produced photogrammetrically from aerial photography.

Aerial photographs and base maps should be prepared in accordance with established industry & governmental standards.

4.3 Parcel: — A land parcel or cadastral parcel is defined as a continuous area, or more appropriately volume i.e. identified by a unique set of homogeneous property rights.

Unique Identification number of Parcel:- A unique identification number should be assigned to each parcel or code that links the parcel with files containing data such as ownership, value, use and zoning.

Parcel identifiers provide a common index for all property records. They provide a means of referring legal descriptions in a uniform and more managable way.

Parcel identifiers make possible an efficient property record system for office and field use.

4.4 Control Points: —

It means determining the position of the control point w.r.t. latitude & longitude or well defined coordinates. There are 3 methods of positioning the control points :—

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(i) Point positioning :- Position of a stationary or moving point is determined w.r.t. well defined coordinates

(ii) Relative positioning :- The coordinates of an unknown point is determined w.r.t. known point.

(iii) Differentially positioning :- It is a system by which differences between observed & computed coordinates or ranges known as differential correction at a particular known point called reference station is made

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5.0 Plane Table Surveying

5.1 Objective :- Plane tabling is a graphical method of surveying where the objective is to do the field work & plotting simultaneously. Such survey doesnot involve field book.

Principle :- The principle of plane tabling is parallelism ; meaning that the rays drawn from stations to objects on the paper are parallel to the lines from the station to objects on the ground. The relative positions of the objects on the ground are represented by their plotted positions on the paper & lie on the respective rays.

Use of plane table surveying :- It is mainly suitable for filling interiore details when traversing is done by theodolite. Sometimes traversing by plane table may also be done. But this survey is recommended for the work where great accuracy is not required. As the fitting and fixing the arrangement of this instrument is not perfect.

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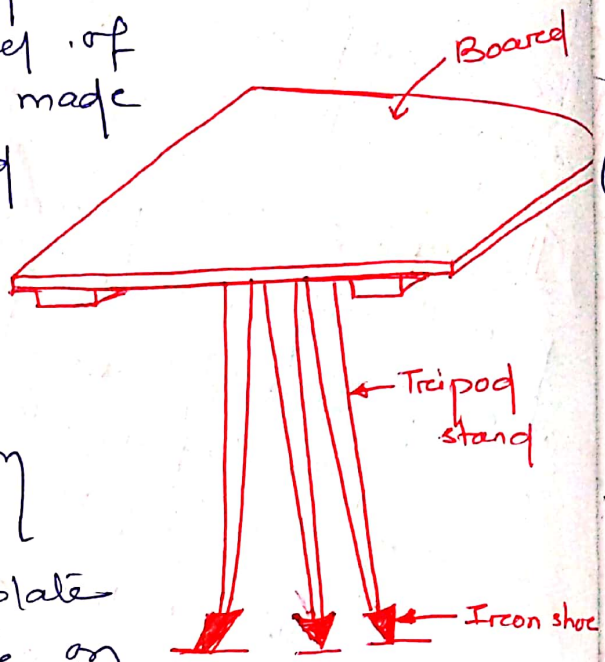
5.2 Instruments & accessories used in plane tabling surveying :—

1) **Plane table :—** The plane table is a drawing boarcel of 750 mm x 600 mm size, made of well seasoned wood like teak, pine etc. The top surface of the table is well levelled. The bottom surface consists of a threaded circular plate for fixing the table on the tripod stand by a wing nut.

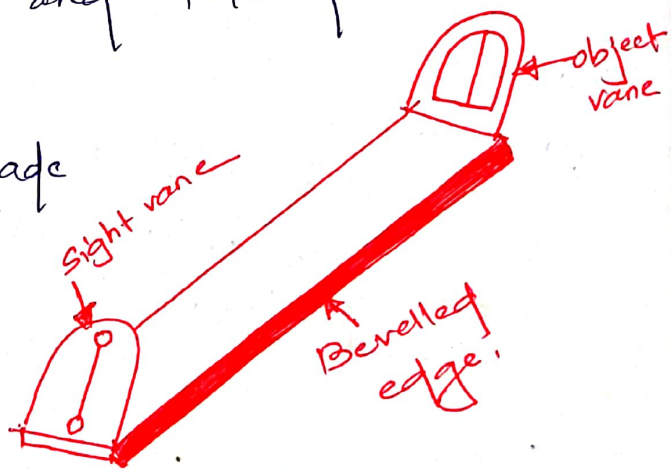


The plane table is meant for fixing a drawing sheet over it.

2) **The alidade :-** There are two types of alidades — plain and telescopic.

(a) **Plain alidade :-**

The plain alidade consists of a metal or wooden ruler of length about 50 cm. One of its edges is bevelled & is known as fiducial edge. It consists of 2 vanes at both ends of



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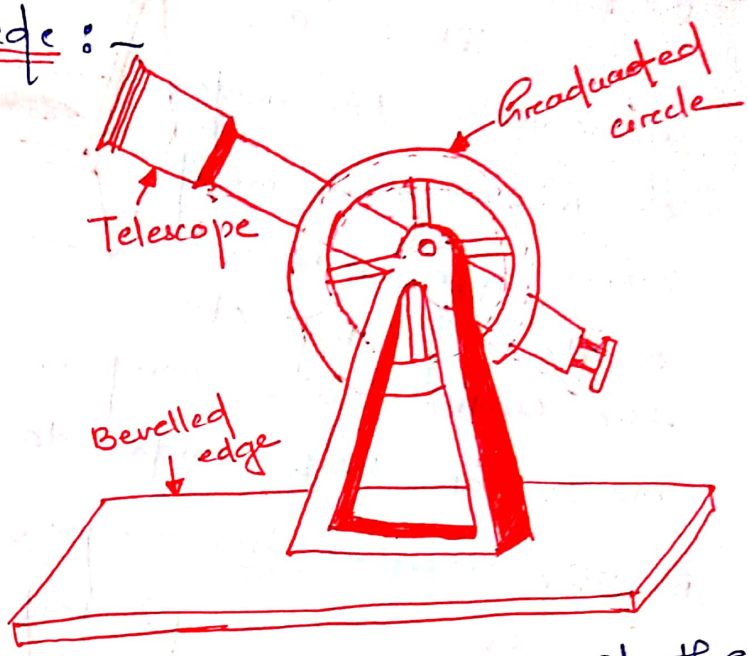
which are hinged with the rulers, One is known as object vane & carries a horse hair & the other is called sight vane it's provided with a narrow slit.

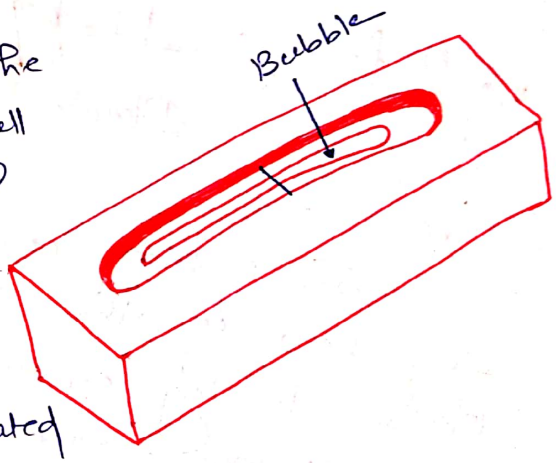
(b) Telescopic alidade :-

The telescopic alidade consists of a telescope meant for inclined sight or sighting distant objects clearly.



This alidade ~~consists~~ of a ~~telescope~~ has no vanes at the ends, but it's provided with fiducial edge. The function of the alidade is to sight objects. The rays should be drawn along the fiducial edge.

3(c) The spirit level :- The spirit level is a small metal tube containing a small bubble of spirit. The bubble is visible on the top along a graduated glass tube.



The spirit level is meant for levelling the plane table.

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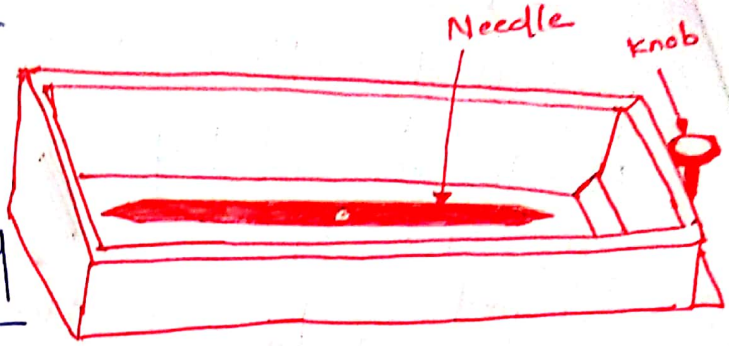
4) **The Compass:-** There are 2 kinds of compass.
(a) trough compass (b) circular box compass.

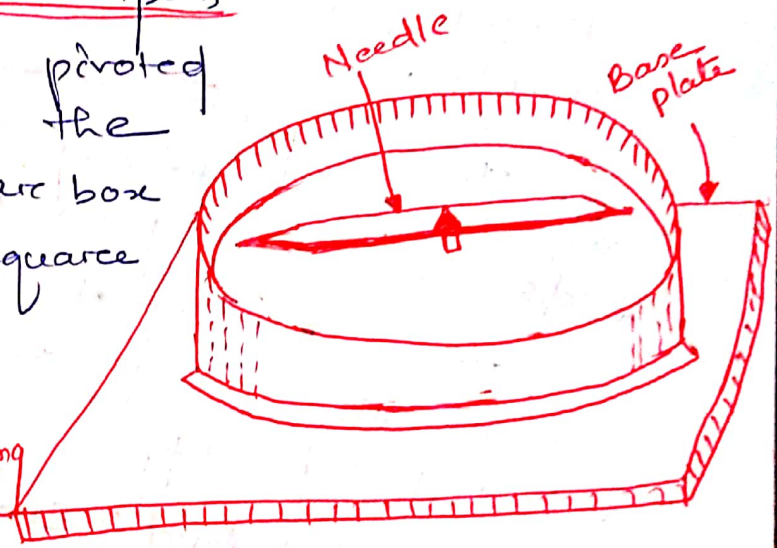
(a) **Trough compass:-**



It is a rectangular box made of non-magnetic metal consisting a magnetic needle pivoted at the centre. This compass consists of a 'o' mark at both ends to locate N-S direct.

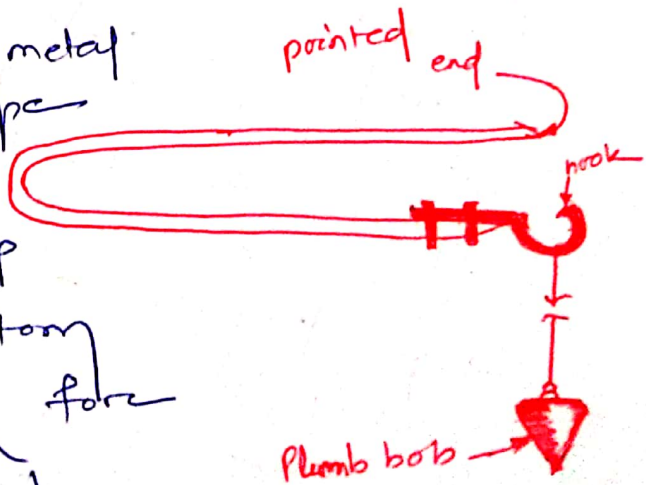
(b) **The circular box compass**

It carries a pivoted magnetic needle at the centre. The circular box is fitted on a square base plate.



5) **U-fork or Plumbing fork with Plumb bob:-**

The U-fork is a metal strip bent in the shape of 'U' having equal arm lengths. The top arm is pointed & bottom arm carries a hook for suspending a plumb bob.



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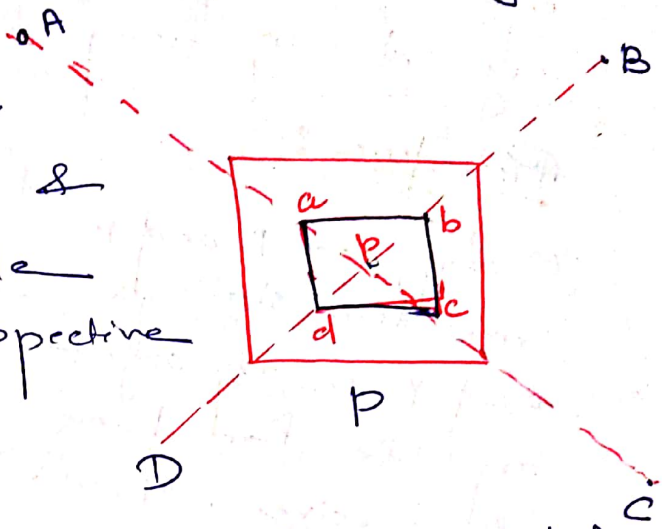
5.3 Methods of plane table surveying

1) **Radiation:-** This method is suitable for locating objects from a single station to the objects & the distances from the stations to the objects are measured & plotted to any suitable scale along the respective rays.



Procedure:-

(a) Suppose P is a station on the ground from where the objects A, B, C and D are visible.

(b) The plane table is set over the station P. A drawing sheet is fixed on the table which is then levelled & centred. A point p is selected on the sheet to represent the station P.

(c) The north line is marked on the right hand top corner of the sheet with trough compass or circular box compass.

(d) With the alidade touching p, the ranging rods at A, B, C and D are bisected and the rays are drawn.

(e) The distances PA, PB, PC and PD are measured & plotted to any suitable scale to obtain the points a, b, c and d representing the objects A, B, C and D on paper.

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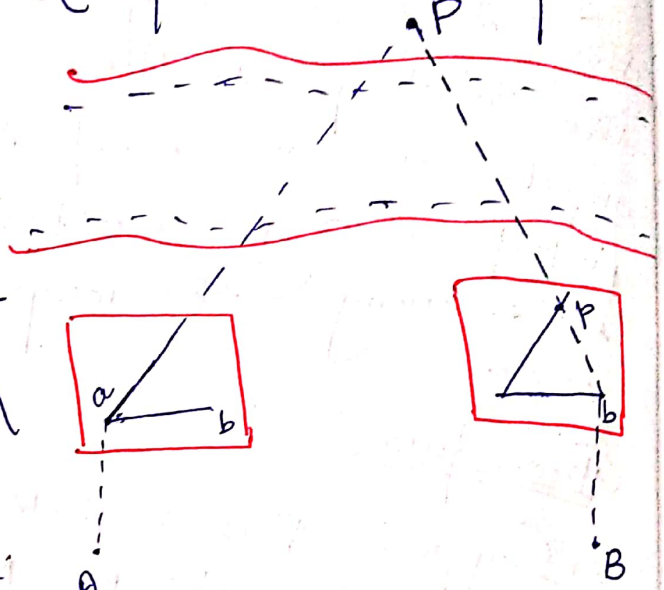
2) The intersection method

This method is suitable for locating inaccessible points by the intersection of rays drawn from 2 instrument stations.

Procedure:-



(a) Suppose A and B are two stations and P is an object on the far bank of a river. Now it is required to fix the position of P on the sheet by the intersection of rays drawn from A and B.

(b) The table is set up at A. It is levelled and centred so that a point a on the sheet is just over the station A. The north line is marked on the right-hand top corner. The table is then clamped.

(c) With the alidade touching a, the object P and ranging rod at B are bisected & rays are drawn through the fiducial edge of the alidade.

(d) The distance AB is measured & plotted to any suitable scale to obtain the point b.

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(c) The table is shifted & centred over B & levelled properly. Now the alidade is placed along the line ba and orientation is done by backsighting. At this time, it should be remembered that centering, levelling & orientation must be perfect simultaneously.

(d) With the alidade is touching b, the object P is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point p. This point p is the required plotted position.

The traversing method:—

This method is suitable for connecting the traverse stations. This is similar to compass traversing or theodolite traversing. But here, fielding and plotting are done simultaneously with the help of radiation & intersection methods.

Procedure:—

(a) Suppose A, B, C and D are the traverse stations.

(b) The table is set up at the station A. A suitable point a is selected on the sheet in such a way that the whole area may be plotted on the sheet. The table is centred, levelled and clamped. The north line is marked on the right-hand top corner of the sheet.

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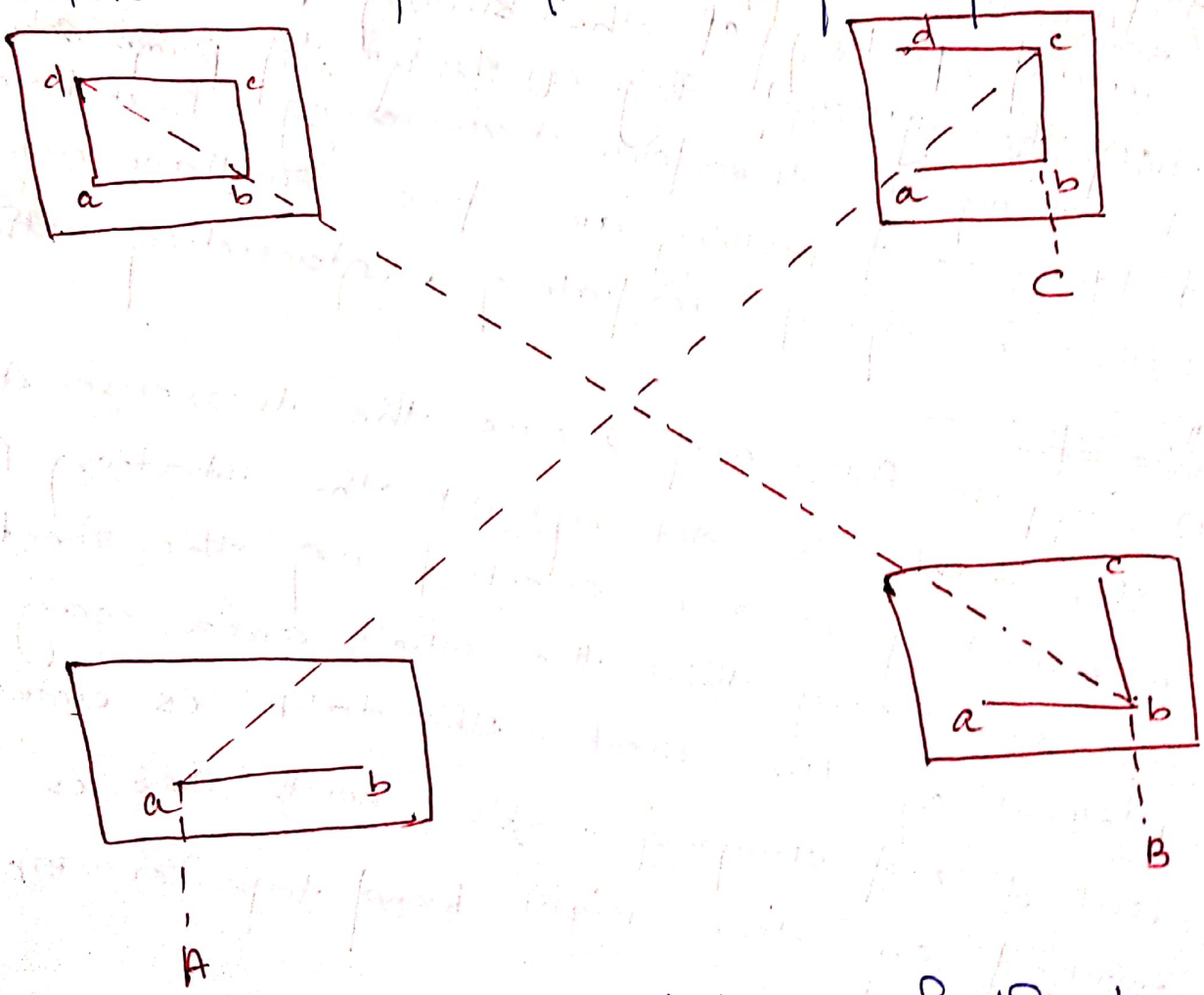
(c) With the alidade touching the point a, the ranging rod at B is bisected and a ray is drawn. The distance AB is measured & plotted to any suitable scale.

(d) The table is shifted and centred over B. It is then levelled, oriented by back sighting and clamped.

(e) With the alidade touching the point b, the ranging rod at C is bisected and a ray is drawn. The distance BC is measured and plotted to the same scale.

(f) The table is shifted and set up at C & the same procedure is repeated.



(g) In this manner, all stations of the traverse are connected.

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At the end, the finishing point may not coincide with the starting point and there may be some closing error. This error is adjusted graphically by Bowditch's rule.

(i) After making the corrections for closing error, the table is again set up at A. After centring, levelling and orientation, the surrounding details are located by radiation.

(ii) The table is then shifted and set up at all the stations of the traverse and after proper adjustments the details are located by the radiation and intersection methods.

4) The Resection Method.

This method is suitable for establishing new stations at a place in order to locate missing details.

Procedure :-

(a) Suppose it is required to establish a station at the position P. Let us select two points A and B on the ground. The distance AB is measured & plotted to any suitable scale. This line AB is known as the base line.

(b) The table is set up at A. It is levelled, centred & oriented by bisecting the ranging rod at B. The table is then clamped.

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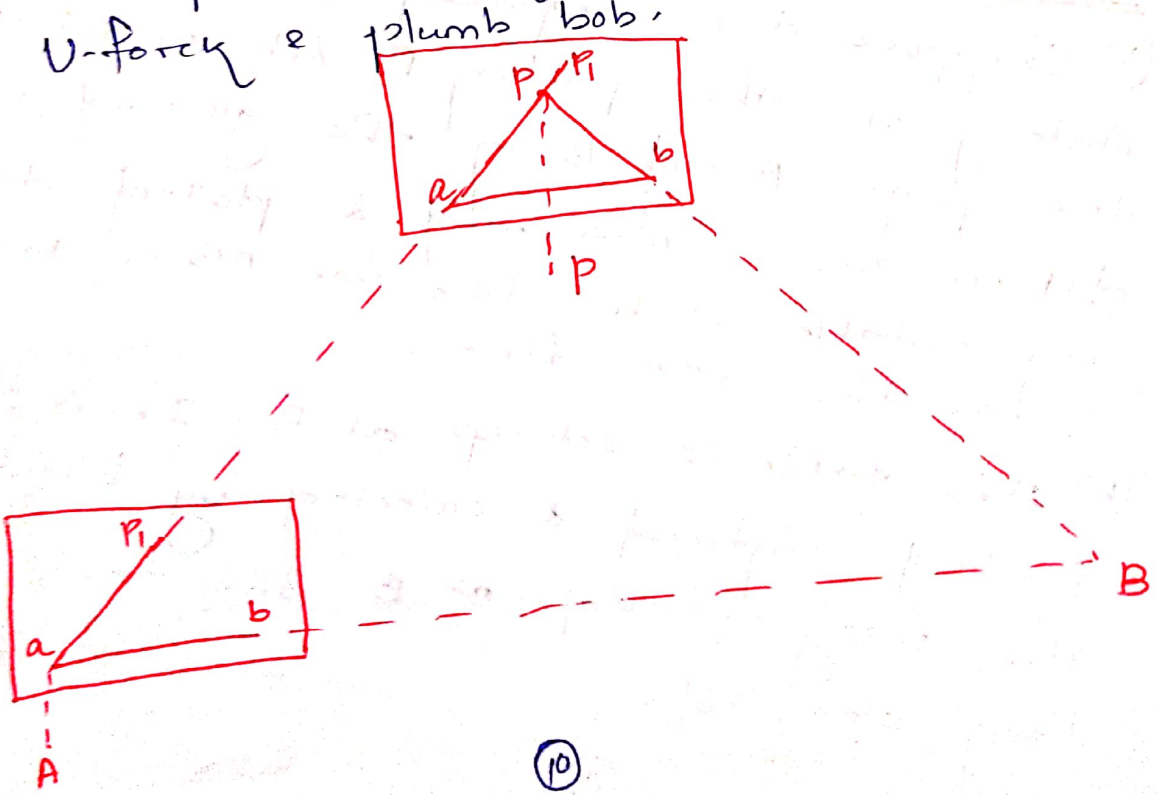
(c) With the alidade touching point a, the ranging rod at P is bisected & a ray is drawn. Then a point P_1 is marked on this ray by estimating with eye.

(d) The table is then shifted & centred in such a way that P_1 is just over P. It is then oriented by backsighting the ranging rod at A.

(e) With the alidade touching the point b, the ranging rod at B is bisected and a ray is drawn. Suppose this ray intersects the previous ray at a point p. This point represents the position of the station P on the sheet. Then the actual position of the station P is marked on the ground by U-fork & plumb bob.



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4 Two-Point Problem

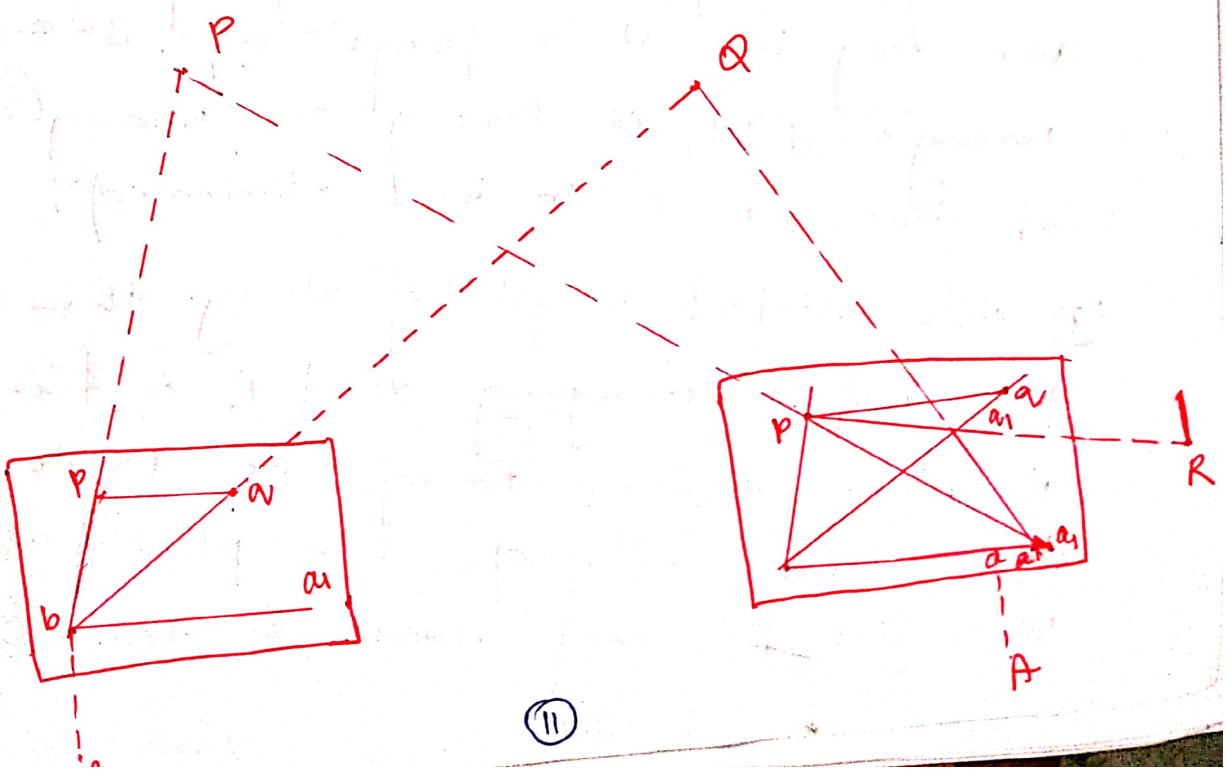
It is a special method of resection used to locate objects after plane table traversing is completed.

In two-point problem, two well-defined points whose positions have already been plotted on the plan are selected. Then, by perfectly bisecting these points, a new station is established at the required position.

Procedure :-

(a) Suppose P and Q are 2 well defined points whose positions are plotted on map as p and q. It is required to locate a new station at A by perfectly bisecting P & Q.

(b) An auxilliary station B is selected at a suitable position. The table is set up at B & levelled & oriented by eye-estimation. It is then clamped.



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(c) With the alidade touching p and q, the points P and Q are bisected & rays are drawn. Suppose these rays intersect at b.

(d) With the alidade centred at b, the ranging rod at A is bisected & a ray is drawn. Then, by eye estimation, a point a, is marked on this ray.

(e) The table is shifted and centred on A, with a, just over A. It is levelled & oriented by backsighting with the alidade touching p, the point P is bisected & a ray is drawn. Suppose this ray intersects the line ba, at the point a, as was assumed previously.

(f) With the alidade centred on a, the point Q is bisected and a ray is drawn. Suppose this ray intersects the ray ba at a point q_1. The triangle pqq_1 is known as triangle of error, & is to be eliminated.

(g) The alidade is placed along the line pq, and a ranging rod R is fixed at some distance from the table. Then, the alidade is placed along the line pq and table is turned

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to bisect R. At this position, the table is said to be perfectly oriented.

(k) Finally with the alidade centred on P and Q, the points P and Q are bisected & rays are drawn. Suppose these rays intersect at a point a. This would represent the exact position of the required station A. Then the station A is marked on the ground.

Three-point Problem

In this problem, 3 well-defined points are selected whose positions have already been plotted on the map. They, by perfectly bisecting these 3 well defined points, a new station is established at the required position.

No auxilliary station is required in order to solve this problem. The table is directly placed at the required position. The ~~may~~ problem may be solved by 3 methods:

(a) the graphical or Bessel's method
(b) the mechanical method
(c) the trial-and-error method.

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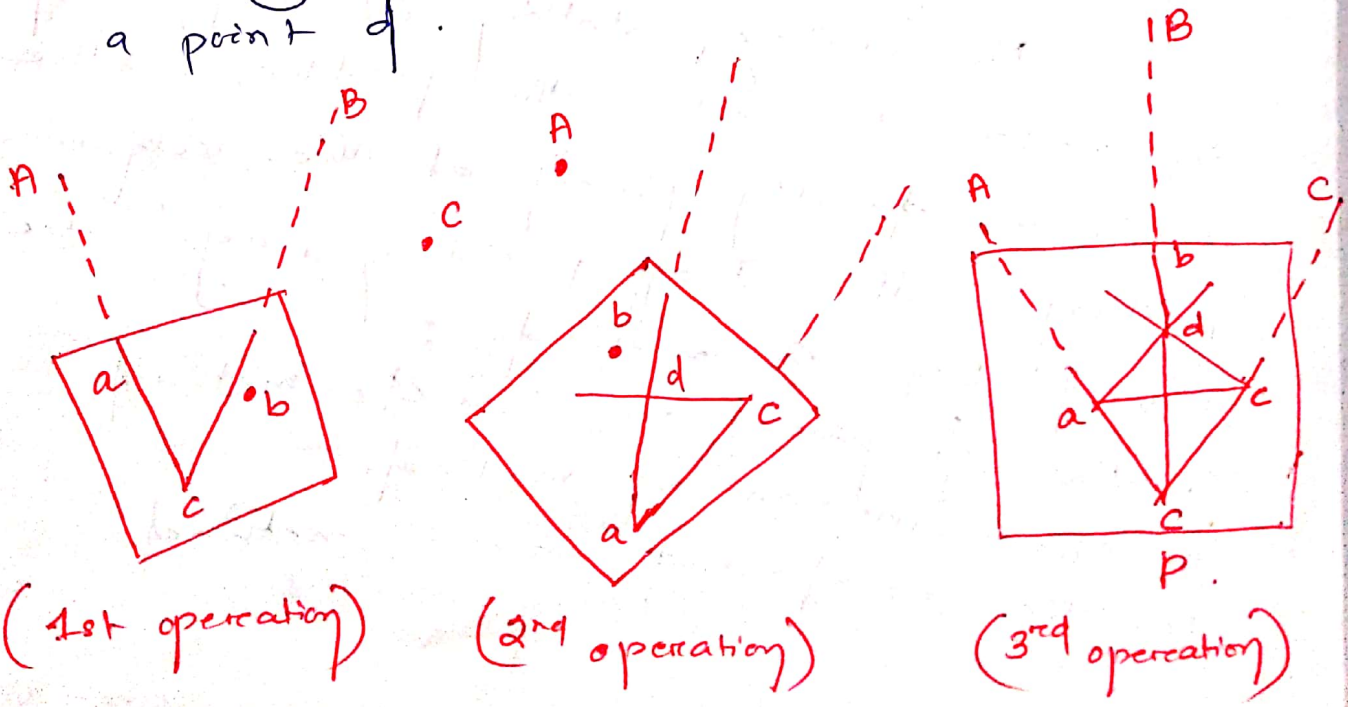
(a) The Graphical Method :-

(i) Suppose A, B and C are 3 well-defined points which have been plotted as a, b and c. Now it is required to locate a station P.

(ii) The table is placed at the required station P and levelled. The alidade is placed along the line ca and the point A is bisected. The table is clamped. With the alidade centred on C, the point B is bisected and ray is drawn.

(iii) Again the alidade is placed along the line ac and the point C is bisected and the table is clamped. With the alidade touching a, the point B is bisected & a ray is drawn. Suppose this ray intersects the previous ray at a point d.



(1st operation) (2nd operation) (3rd operation)

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(iv) The alidade is placed along db and the point B is bisected. At this position, the table is said to be perfectly oriented. Now, the rays Aa, Bb and Cc are drawn. These 3 rays must meet at a point p which is the required point on the map. This point is transferred to the ground by a U-fork & plumb bob.

(b) The Mechanical method

(i) Suppose A, B and C are 3 well-defined points which have been plotted on the map as a, b and c. It is required to locate a station at P.

(ii) The table is placed at P and levelled. A tracing paper is fixed on the map & a point 'p' is marked on it.

(iii) With the alidade centred on P, the points A, B and C are bisected and rays are drawn. These rays may not pass through the points a, b and c as the ~~oriented~~ orientation is done approximately.

(iv) Now the tracing paper is unfastened & moved over the map in such a way that the three rays simultaneously pass through the plotted positions a, b & c.

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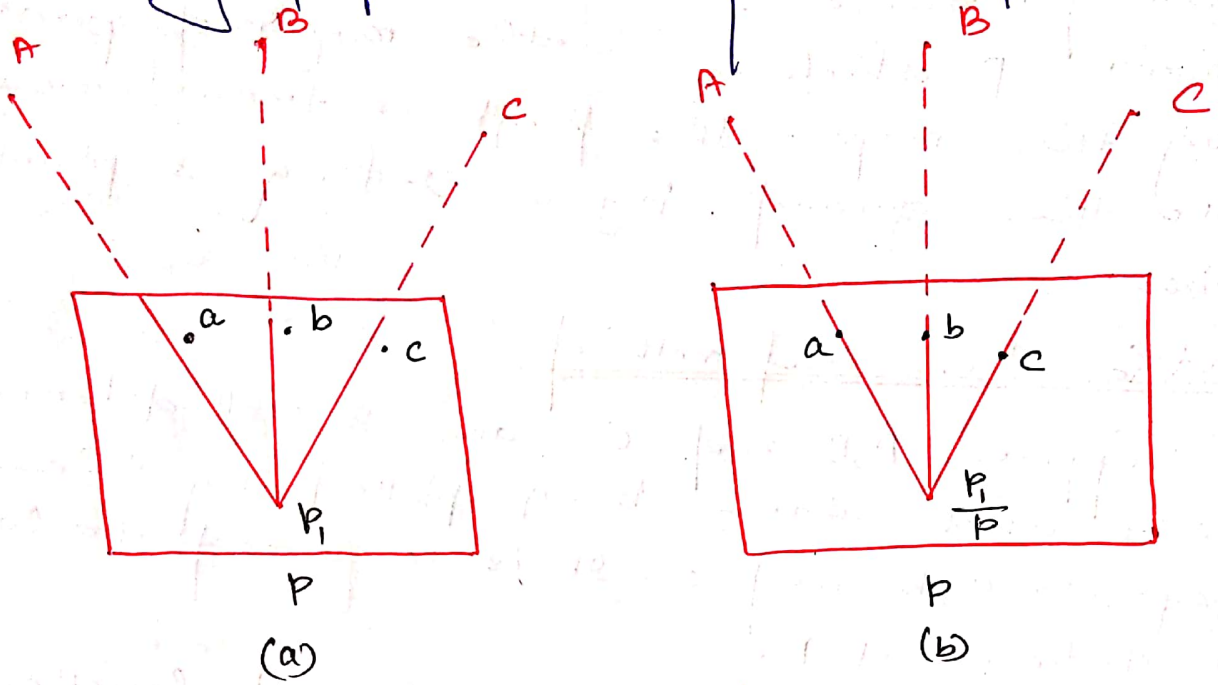
Then the point p is pricked with a pin to give an impression p on the map. p is the required point on the map. The tracing papere is then removed.



(v) Then the alidade is centred on p ← the rays are drawn towards A, B and C. These rays must pass through the points a, b and c.

(c) The method of trial and error

(a) Suppose A, B and C are three well-defined points which have been plotted as a, b and c on the map. Now it is required to establish a station at p.

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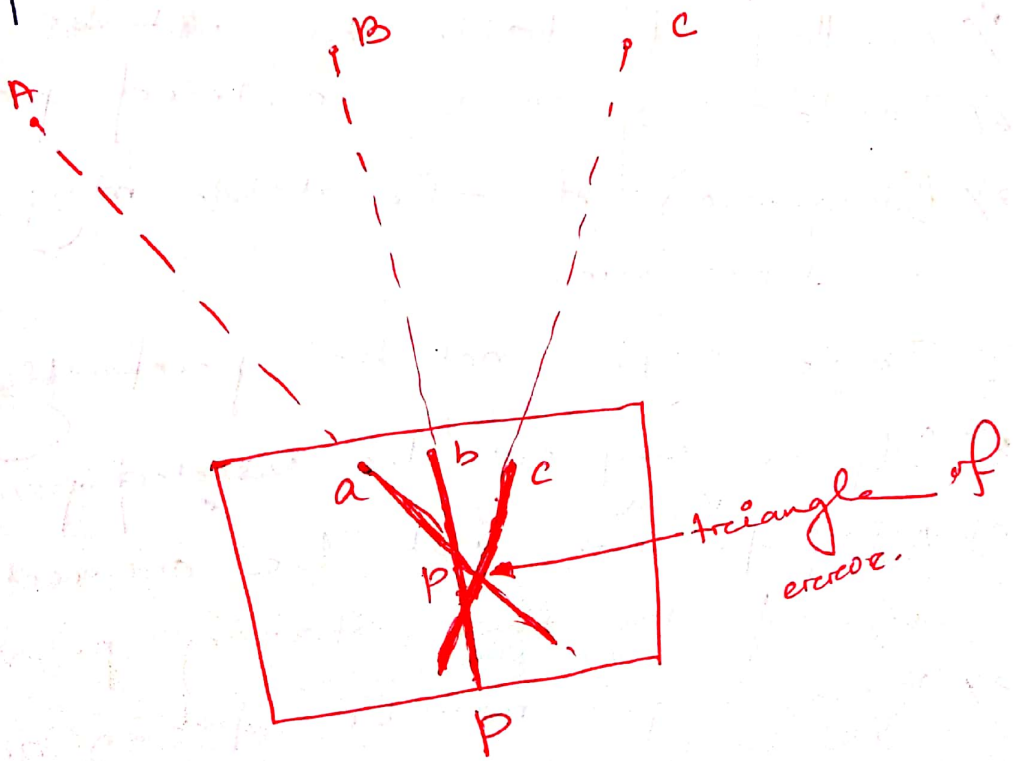
(b) The table is set up at P and levelled. Orientation is done by eye estimation.

(c) With the alidade, rays Aa, Bb and Cc are drawn. As the orientation is approximate, the rays may not intersect at a point, but may form a small triangle - the triangle of error.

(d) To get the actual point, this triangle of error is to be eliminated. By repeatedly turning the table clockwise or anticlockwise the triangle is eliminated in such a way that the rays Aa, Bb, Cc finally meet at point p. This is the required point on the map. This point is transferred to the ground by U-fork and plumb bob.



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Errors in plane table Surveying

A) Instrumental Error

1) The surface of table may not be perfectly level.
2) fiducial edge of alidade might not be straight.
3) Vanes may not be vertical.
4) Horsehair may be loose & inclined
5) Table may be loosely joined with the tripod stand.
6) Needle of trough compass may not be perfectly balanced.

B) Personal Error :-

1) Levelling of table may not be perfect
2) Table may not be centred properly
3) Orientation of the table may not be proper.
4) Table might not be perfectly clamped.
5) Objects may not be bisected perfectly.
6) Alidade may not be correctly centred on the station point.
7) Rays might not be drawn accurately

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3) Alidade may not be centred on the same side of the station throughout the work.

c) Plotting errors :-

1) A good quality pencil with very fine pointed end may not have used.

2) Incorrect scale may be used.

3) Errors may result from failure to observe correct measurements from the scale.

4) Unnecessary hurry at the time of plotting may lead to errors.

Precautions

1) Before starting the work, the equipments for survey work should be verified.

2) Centering should be perfect.

3) Levelling should be proper.

4) Orientation should be accurate.

5) Alidade should be centred on the same side of station pt'g until the work is completed.

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6) While shifting plane table from one station to another, tripod stand should be kept vertical to avoid damage.

7) Several accessories have to be carried, so care should be taken to ensure nothing is missing.

8) Pencil should have a sharp point.

9) Distances of objects are lines should be written temporarily along respective rays until plotting is completed.

10) Only selected scale should be on the table.

11) Measurements should be taken carefully from the scale while plotting.

12) Stations on the ground are marked A, B, C, D etc. while the station points on map are marked a, b, c, d etc.

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6.0 Theodolite Surveying & Traversing

6.1 Purpose & Definition of theodolite surveying :-

The instrument used for measuring horizontal & vertical angles accurately is known as theodolite.

Purpose :-

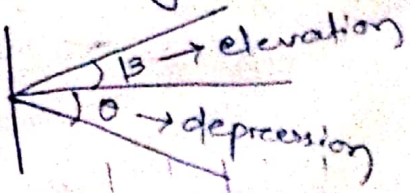
→ To measure horizontal angles.
→ " " vertical angles
→ " " deflection "
→ " " magnetic bearings.
→ " " horizontal distance between two points.
→ To find vertical height of an object.
→ Ranging a line.

Theodolites are of two types :—

1) Transit theodolite.
2) Non-transit theodolite.

1) Transit theodolite :— when the telescope can be revolved through a complete revolution about it's horizontal axis in vertical plane.

2) Non-transit theodolite :— when the telescope cannot be revolved through a complete revolution in vertical plane, but it can be revolved through certain extent in vertical plane, in order to measure angle of elevation or depression.



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→ Again theodolite may also be classified as
i) Vernier theodolite :- when fitted with vernier scale.
ii) Micrometere theodolite :- when fitted with micrometere (digital).

Definitions of terms :-

1) Centering :- The setting of theodolite exactly over the station with the help of plumb bob is known as centering.
2) Transiting :- The procen of turning the telescope in vertical plane through 180° about its horizontal axis is known as transiting.
3) Face left :- when vertical circle is on the left of the telescope at the time of observation, it is known as face left position. & observations taken are known as face left observations.
4) Face right :- when vertical circle is on the right of the telescope at the time of observation it is known as face right position & the observations taken are known as face right observations.
5) Telescope Normal :- The face left position is called telescope normal or telescope direct or bubble up position.
6) Telescope inverted :- the face right position is known as telescope inverted or bubble down position.

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7) Changing face :- The process of changing face of telescope from right to left & vice versa.

8) Swinging :- The turning of telescope about the vertical axis in horizontal plane is known as swinging.

It is right swing when telescope is turned clockwise & left swing when telescope is turned anti-clockwise.

9) Line of collimation :- It is an imaginary line passing through the intersection of the cross-hairs at the diaphragm & optical centre of object glass & its continuation.

10) Axis of telescope :- It is an imaginary line passing through the optical centre of object glass & optical centre of eye-piece.

11) Axis of bubble tube :- It is an imaginary line tangential to the longitudinal curve of the bubble tube at the middle point.

12) Vertical axis :- It is the axis about which the theodolite, may be rotated in a horizontal plane.

13) Horizontal axis :- It is the axis about which the theodolite, may be rotated in a vertical plane.

14) Temporary adjustment :- The adjustments which are required to be made at every

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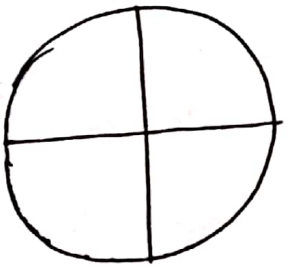
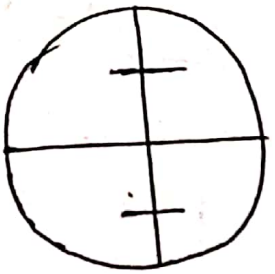
instrument stations before making observations are known as temporary adjustments.

15) **Permanent Adjustments** :— When desired relationship between the fundamental lines of theodolite is disturbed, then some procedures are adopted to establish this relationship. These adjustments are known as permanent adjustments.

16) **Diaphragm** :— It is a brass ring with cross-hairs in it. It is placed in position by turning the screws & can be moved upward or downward when required. It is placed in front of eye-piece. The cross-hairs may be made of spider web or fine platinum wire or fine scratch mark.





The vertical hair is meant to check the verticality of staff. The middle horizontal hair corresponds the collimation. The upper & lower short horizontal lines are called upper & lower stadia.

17) **Sensitiveness of bubble tube** :— The ability of bubble tube to show small deviations of bubble from horizontal position is called sensitiveness of bubble tube.

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5.2 TRANSIT THEODOLITE

Fundamental axes of theodolite

1) Line of collimation, Axis of the telescope, Axis of the bubble tube, Vertical axis & horizontal axis are called fundamental axes of theodolite.

Component Parts

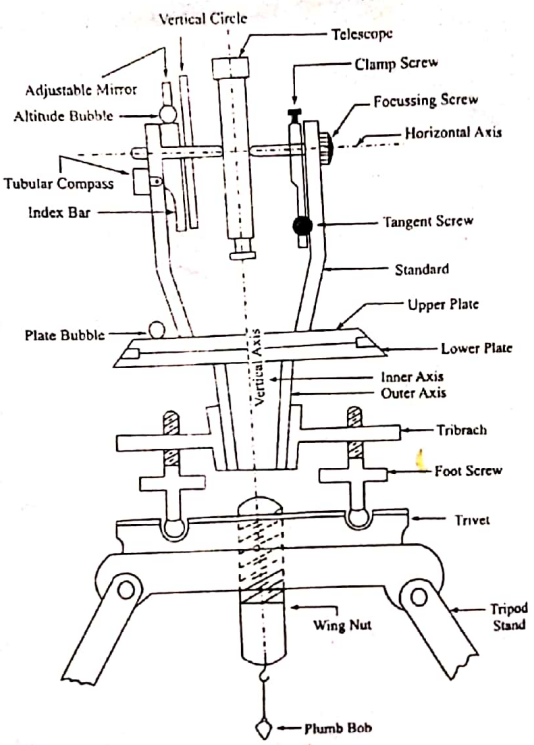

Fig. 9.2 Transit Theodolite

1) Trivet:- It is a circular plate having a central, threaded hole for fixing the theodolite on the tripod stand by a wing nut. It is also called the base plate. Three foot screws are secured to this plate by means of a ball-and-socket arrangement.

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2) Tribrach :- It is a triangular plate & carries three foot screws at its ends.

3) Foot screws :- These are meant for levelling the instrument. The lower part of the foot screws are secured in the trivet by means of a ball and socket arrangement, and the upper threaded part passes through the threaded hole in the tribrach plate.

4) Levelling Head :- The trivet, foot screws and tribrach constitute a body which is known as the levelling head.

5) Spindle :- Theodolite consists of 2 spindles or axes - one inner and the other outer. Inner axis is solid & conical & outer is hollow. The two spindles are co-axial.

6) Lower plate :- Lower plate is attached to the outer axis & is also known as the scale plate. It is bevelled & the scale is graduated from 0 to 360° in a clockwise direction. Each degree is again subdivided into 2, 3 or 4 divisions.

The lower plate is provided with a clamp screw & a tangent screw which controls its movements. When clamp screw is tightened, this plate is fixed with the outer axis. For fine adjustment

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the lower plate, the tangent screw is rotated to the extent required. The size of theodolite is designated according to the diameter of the lower plate.

7) Upper plate :— Upper plate contains the vernier A and B. It is attached to the inner axis, its motion is controlled by upper clamp screw & upper tangent screw. When clamp screw is tightened, the vernier scales are fixed with inner axis, & for fine adjustment of scales, tangent screw is rotated.

8) Plate Bubble :— Two plate bubbles are mounted at right angles to each other on the upper surface of the vernier plate. One bubble is kept parallel to the horizontal axis of the theodolite. Sometimes one plate bubble is provided on the vernier plate. The bubbles are meant for levelling the instrument.

9) Standard or A-frame :— Two frames are provided (shaped like letter - A) are provided on the upper plate to support the telescope, the vertical circle & the vernier scales. These frames are known as standards or A-frames.

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10) Telescope :- The telescope is pivoted between the standards at right angles to the horizontal axis. It can be rotated about its horizontal axis in a vertical plane. The telescope is provided with a focussing screw, clamping screw and tangent screw.

11) Vertical circle :- It is rigidly fixed with the telescope & moves with it. It is divided into four quadrants, Each quadrant is graduated from 0 to 90° in opposite directions, with the zero mark at the ends of the horizontal diameter of the vertical circle.

The line joining the zero marks, corresponds to the line of collimation. The subdivisions of the vertical circle are similar to those of horizontal circle. The vertical circle can be clamped or finely adjusted with the help of clamping screw provided along with the telescope.

12) Index bar or T-frame :- It is provided on the standard in front of the vertical circle. It carries two verniers (C & D) at two ends of horizontal arm. The vertical leg of index bar is provided with a clip screw at lower end by means of which altitude bubble can be brought to the centre.

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Altitude Bubble :- A long sensitive bubble is provided on the top of index bar. The bubble it contains is known as altitude bubble. This bubble is brought to the centre by clip screw at the time of measuring the vertical angle. A mirror is provided on the top of the bubble to help observe it when the instrument is set up above normal height.

14) **Compass** :- Sometimes a circular box (compass) is mounted on the vernier scale between the standards. In modern theodolites, an adjustable trough compass or tubular compass can be fitted with a screw to the standard. The compass is provided for taking magnetic bearing of the line.

Concept of vernier / Reading a Vernier :-

The least count of the vernier is to be determined first. The main divisions of the main scale is of one degree. Suppose it is divided into 3 parts. Then each part accounts for 20' (i.e. $d = 20'$).

The vernier scale has 20 big & 60 small divisions.

So least count $= \frac{d}{n} = \frac{20}{60} \times 60 = 20''$.

Least count for one small division $= 20''$

Least count for one big division $= (20'' \times 3) = 60'' = 1'$

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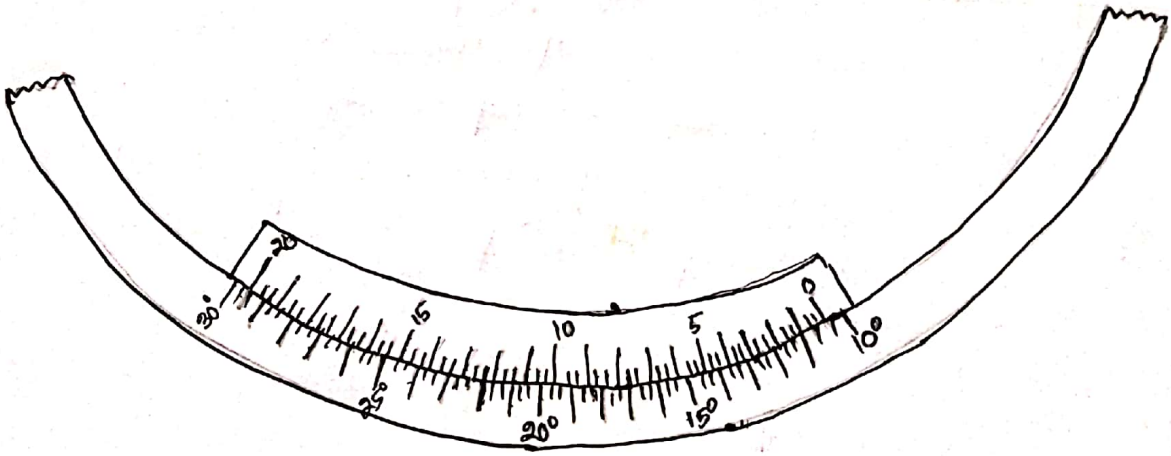
After making the final adjustment for measuring the angle, the position of the arrow of vernier scale is noted. Suppose the arrow crosses 10° 20', which is the direct reading obtained from ~~scale~~ main scale.

Suppose, again that the first small division after 12 big divisions exactly coincides with any of the main scale divisions. Then the vernier reading is 12' 20".

Final angle = 10° 20' + 12' 20"
= 10° 32' 20".



[Vernier theodolite]

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Temporary adjustment of Theodolite :-

1) **Setting theodolite over station** :- The tripod stand is fixed over any suitable station & theodolite is fixed on top of the stand by using nut arrangement or according to fixing arrangement provided in theodolite.

2) **Approximate levelling by tripod stand** :- The tripod legs are placed well apart & firmly fixed on ground. Approximate levelling is done by moving the 3rd leg of tripod stand inward or outward, clockwise or anticlockwise until the bubble comes approx. to centre of its ring.

3) **Centering** :- It is the process of setting the instrument exactly over the station. It should be ensured that the plumb bob suspended from hook under vertical axis lies approximately over station peg.

With the help of shifting head or movable capstan nut, centering is done accurately.

4) **Levelling** :- All the foot screws should be brought to the centre of their rang.
→ The plate bubble is kept parallel to any pair of foot screws; then by

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turning both these screws equally inward or outward, the bubble is brought to centre.

→ The plate bubble is turned through 90° to the previous position. Then turning the 3rd foot screw bubble is brought to centre.

→ The process is repeated several times so that the bubble remains in central position of plate bubble, both direction perpendicular to each other.

→ The instrument is rotated through 360° about vertical axis. If the bubble still remains in central position, the adjustment is perfect.

5) Focusing the eye piece :- For focusing the eye-piece, a piece of white paper is held in front of object glass & the eye piece is turned clockwise or anticlockwise until the cross-hairs look sharp & clear.

6) Focusing the object glass :- It is done to bring clear & sharp image of object in plane of cross-hair. For this, the telescope is turned clockwise or anti-clockwise until the image appears clear & sharp.

7) Setting the vernier :- The vernier A is set to zero & vernier B is set to 180°. To get this

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the lower clamp is fixed & upper clamp is loosened. the upper plate is rotated till the vernier A coincides with the 0 & vernier B coincides with 180. then upper clamp is fixed, by turning upper tangent screw fine adjustment can be made.

6.3 Concept of transiting

1) Measurement of horizontal angle:-

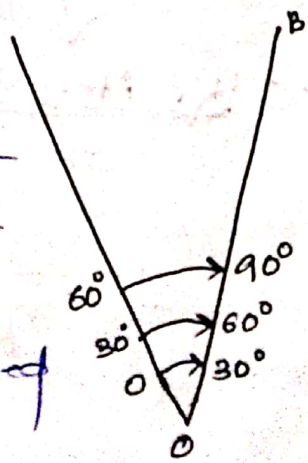
1) Repetition Method:-

In this method the same angle is added no. of times. the total angle is divided by no. of readings to get the angle. the ang. of face left & face right observation is taken with 3 repetition of each face.

→ Let's assume that we need to measure ∠AOB by this method. the instrument is set up at O & centred & levelled properly. Vernier A is set to 0 & B to 180°.

→ the upper clamp is fixed & lower one loosened. By turning the telescope, the ranging rod at A is perfectly bisected with the help of lower clamp screw & lower tangent screw. Here initial reading of vernier A is 0°.

→ the upper clamp is loosened & telescope is turned



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clockwise to perfectly bisect ranging rod at B. The upper clamp is fixed. Assume the reading on vernier A is 30°.

→ The lower clamp is loosened & telescope is turned anticlockwise to exactly bisect the ranging rod at A. The initial reading is 30° for 2nd observation.

→ The lower clamp is fixed. The upper one is loosened & telescope is turned clockwise to exactly bisect the ranging rod at B. Let the reading on vernier A be 60°.

→ The initial reading for 3rd observation is 60°. ∠AOB is again measured. The final reading on vernier A be 90°.

$$\angle AOB = \frac{\text{Accumulated angle}}{\text{No. of readings}} = \frac{90^\circ}{3} = 30^\circ$$

→ The face of instrument is changed & previous procedure is followed.

→ The mean of 2 observations give actual angle ∠AOB.

2) **Re iteration Method** :-

In this method, from a single station several angles are measured & horizon is closed. So the final reading on vernier should be

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same as initial reading. If discrepancy is small, error is equally distributed among all observed angles. If it is large, the reading shall be cancelled.

→ The theodolite is centred over O & levelled perfectly. The observation is taken in face left position & telescope is turned clockwise.

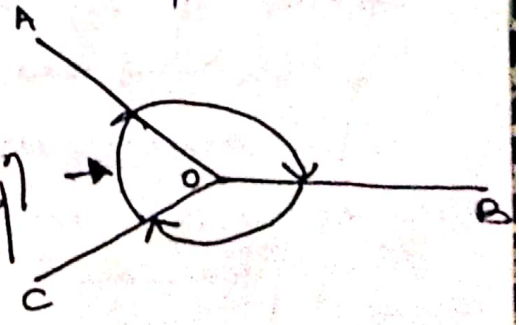
→ Vernier A is set to 0° & B to 180°.

→ The upper clamp is fixed & lower one is loosened. The ranging rod at A is bisected perfectly. The lower clamp is tightened.

→ The upper clamp is loosened & ranging rod at B is bisected. The reading on both verniers are taken. ∠AOB.

→ The object C is bisected & readings on verniers are noted. ∠BOC.

→ The last angle ∠COA is measured. The readings on leading vernier should show the initial reading. If not, the amount of discrepancy is noted. If discrepancy is small, the error is distributed among the angles & if the error is large, the observation should be taken again.



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→ The whole process is repeated by changing the face.

→ The mean of angles of two sets give the actual value of angles.

2) Measurement of vertical angle

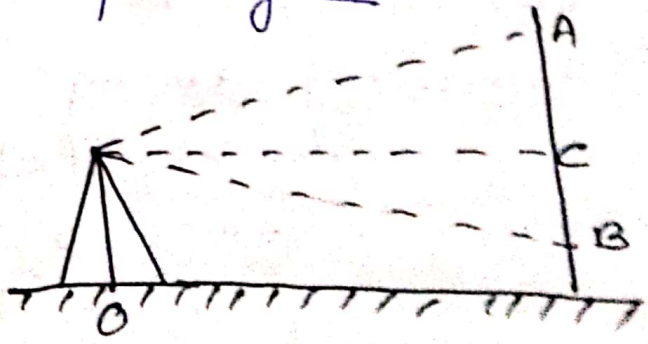
→ The theodolite is set up at O°. It is centred & levelled properly with vernier C & D set at 0° mark of vertical circle. The telescope is clamped.

→ The plate bubble is brought to centre with the help of foot screw as well as altitude bubble is brought to centre.

→ To measure angle of elevation, telescope is raised slowly to bisect point A & reading on both verniers are noted.

→ The face of instrument is changed & point A is again bisected. The reading on both verniers are noted. The mean of angle observed is correct angle of elevation.

→ To measure angle of depression, telescope is lowered slowly & point B is bisected. Reading on both verniers are noted for both faces. The mean angle of observation is correct angle of depression.



Vertical angle = angle of elevation + angle of depression.

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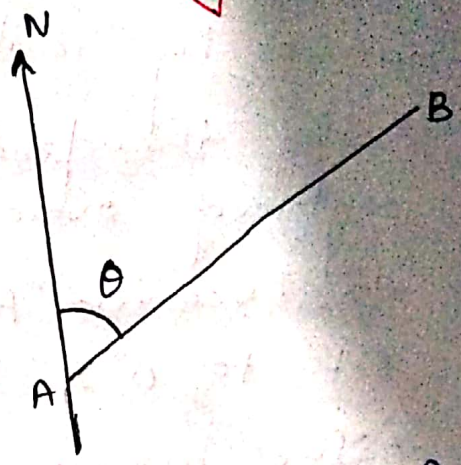
6.4.1 Measurement of magnetic bearings



→ The theodolite is placed at a suitable station A, centered & levelled properly. The vernier A & B are set to 0° & 180°.

→ In theodolites where compass is not already present a trough or tubular compass is fitted to theodolite.

→ The vernier A & B is set to 0° & 180°.

→ The lower clamp is loosened & telescope is rotated until the needle points to the north. At that position the telescope is perfectly oriented along magnetic meridian.

→ The lower clamp is fixed & upper clamp is loosened. By turning the telescope clockwise the ranging rod at B is perfectly bisected by upper tangent screw.

→ Reading on both verniers are recorded & mean is taken.

→ The face of instrument is changed & magnetic bearing of AB is taken as the avg. of both face readings.

6.4.2 Measurement of Deflection Angle

→ Deflection angle is the angle by which the survey line is deflected from original direction.

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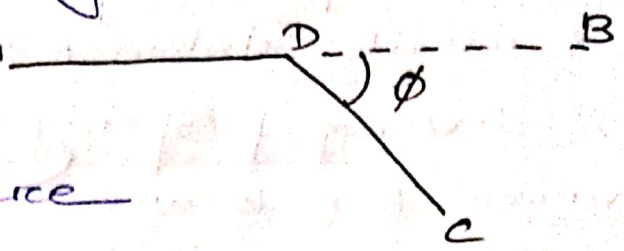
→ It is measured for designing horizontal curves in railways, highways etc.

→ The theodolite is centered over D & the verniers A & B are set to 0° & 180°.



→ The upper clamp is tightened & lower clamp is loosened & ranging rod at A is perfectly bisected.

→ The lower clamp is fixed at this position

→ The telescope is transited & ranging rod at B is visible through the telescope in opposite direction which is prolongation of AD.

→ The upper clamp is loosened & by turning the telescope in clockwise direct the ranging rod at C is perfectly bisected & the upper clamp is fixed & for precise bisection upper tangent screw is used.

→ The deflection angle Ø is measured from the vernier.

→ The lower clamp screw is loosened & by turning the telescope clockwise, the ranging rod at A is again bisected & lower clamp is fixed.

→ The telescope is transited. The upper clamp is loosened and the telescope

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is rotated in clockwise direction & ranging rod at C is perfectly bisected & upper clamp is fixed.

→ The deflection angle is now doubled & the reading is taken from the vernier.

→ Half of the value will given the correct value of deflection angle.

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3) Measurement of direct angle :-

→ The angle measured clockwise from the preceding line to the following line is called direct angle. These angles are also some times known as azimuths from the back line, or angles to the right and may varey from 0° to 360°.



Procedure :-

→ To measure a direct angle say BCD, proceed as under :-

(i) Set up the theodolite over station C, centre and level it carefully.

(ii) Keeping the face of instrument left, set the vernier to read zero degree by turning the upper plate.

(iii) Unclamp the lower clamp & bisect the station B accurately using the lower tangent screw. Clamp the lower plate.

(iv) Unclamp the upper plate, swing the telescope clockwise and bisect the forward station D. Read both the verniers.

(v) Plunge the telescope, unclamp the lower clamp & bisect the preceding station B,

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without disturbing the vernicre readings.

(vi) Unclamp the uppere plate, roving the telescope clockwise and bisect the station D accurately. Read both vernieres again.

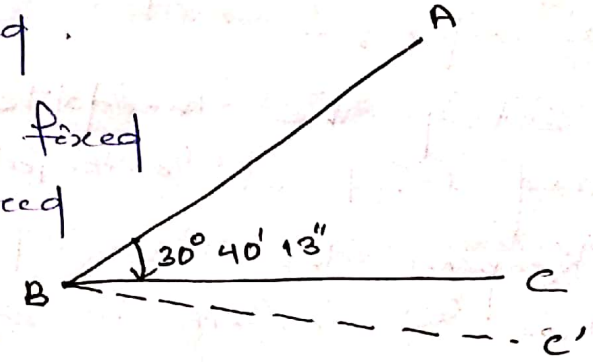
(vii) Take the mean of the vernier readings. Note, angle is doubled & hence average value gives value of direct angle BCD.

Direct angles are generally used in theodolite traverse.

6.4.4 Setting out angles

The method of repetition is generally used to lay off angles whenever greatere precesion is reqeired.

Let BA be a fixed line and it is reqeired to lay off BC making an angle ABC equal to 30° 40' 13" with a theodolite having a 20" least cocent.



Procedure:-

1) Centre the instrument over B and level it carefully.

2) Sight A accurately and make the vernicre to read zero.

3) Unclamp the uppere plate and roving the telescope till reading is approximately,

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equal to the required angle. Use the upper tangent screw to set the angle exactly equal to 30° 40' 20".

4) Fix c' in the line of sight at the required distance away from B.

5) Measure the angle ABC' by the method of repetition. Let the number of repetition be 5 and the accumulated angle AB'c' be 153° 22' 20". The average value of angle ABC = $\frac{1}{5}$ (153° 22' 20")
= 30° 40' 28".

6) Calculate the required correction to be applied to the angle ABC i.e. 30° 40' 28" — 30° 40' 13" = 15".

7) Since correction is very small, it may be applied linearly by making offset c'c = BC' tan ∠CBC".

8) Measure the length of BC'. Let it be 250m.

9) Calculate the distance c'c = 250 tan 15"
= 0.018m assuming tan 1' = 0.0003 nearly.

10) Establish C by setting a perpendicular c'c to BC'. Measure c'c equal to 0.018m.

11) Check the accuracy of the angle ABC by measuring its value by the method of repetition. Apply further correction, if found necessary.

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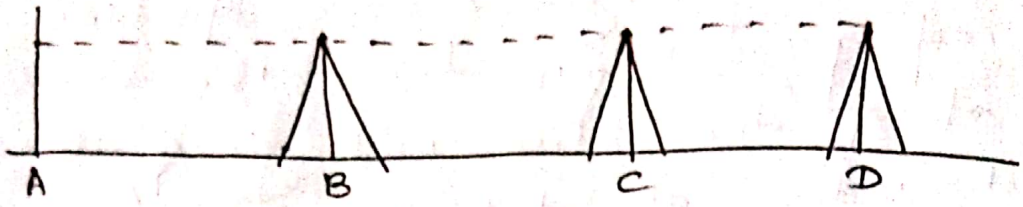
6.4.5 Predonging a straight line



→ The theodolite is placed at B, centered & levelled properly.

→ The upper & lower clamp screws are tightened & the ranging rod at A is bisected perfectly with tangent screw.

→ The telescope is transited & ranging rod at C is perfectly bisected & marked.

→ The theodolite is shifted & centered over C. It is centered & levelled. The ranging rod at B is bisected & upper & lower tangent screws are fixed.

→ The telescope is transited & the ranging rod is placed in the perfect line B, C & D.

→ Similarly other points are marked in the line.

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→ It is measured

6.4.6 Errors in theodolite observations :-

1) Instrumental error :-

(a) Non-adjustment of plate bubble :- The axis of plate bubble may not be perpendicular to vertical axis. The error may be eliminated by levelling the instrument with reference to altitude bubble also.

(b) Line of collimation not being perpendicular to horizontal axis :- A cone is formed when the telescope is rotated in vertical plane. This error is eliminated by taking reading from both faces & taking average of them.

(c) Horizontal axis not being perpendicular to vertical axis :- In this case there will be angular error. It is eliminated by reading the angle from both faces.

(d) Line of collimation not being parallel to axis of telescope :- It causes error in vertical angle measurement. It is eliminated by taking reading from both faces.

(e) Eccentricity of inner & outer axes :- It affects in vernier readings. It is eliminated by taking readings from both verniers & taking avg. of them.

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(8) Graduations not being uniform :— It is eliminated by taking the same angle several times but on different parts of the horizontal circle..

(9) Verniers being eccentric :— In vernier A, it is set to 0° & vernier B to 180°, but in some case it doesn't occur, in that case, the error is eliminated by reading both verniers & taking average.

2) Personal Errors :—

→ The centering may not be done perfectly due to carelessness.
→ The levelling may not be done carefully
→ If clamp screws are not properly fixed, the instrument may slip.
→ The tangent screws may not be operated properly.
→ Focussing may not be done perfectly to avoid parallax.
→ Object or ranging rod may not be bisected perfectly.
→ Verniers may not be set in proper place
→ Errors would also result, if verniers are not read because of oversight.

3) Natural Errors :—

↳ High temperature causes irregular refraction.
↳ High wind causes vibration in the instrument.

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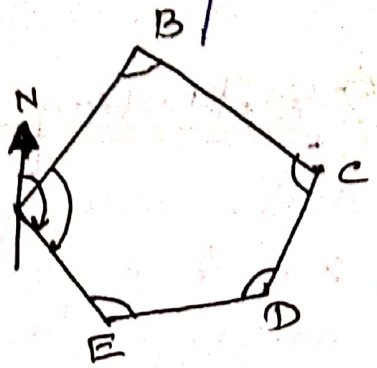
6.5 Theodolite Traversing

Methods :-

1) Inclined Angle Method :-

→ This method is suitable for closed traverse. The traversing can be done either in clockwise or anticlockwise direction.

→ The theodolite is taken & placed over station A. At this point theodolite is centered & levelled perfectly. Vernier A is set to 0° & B is set to 180°. Upper clamp is fixed.



→ Lower clamp is loosened & telescope is oriented along north line with the help of tubular compass fitted to telescope. Magnetic bearing of AB is measured.

→ Vernier A is again set to 0° & B to 180° & upper clamp is fixed.

→ Lower clamp is loosened & ranging rod at E is bisected. The lower clamp is fixed & upper clamp is opened & telescope is rotated to bisect the ranging rod at B. Reading on verniers are noted which gives ∠A.

→ The face of instrument is changed & again the same angle is measured. The mean of both readings give ∠A.

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→ Other angles are measured by centering theodolite over B, C, D & E.

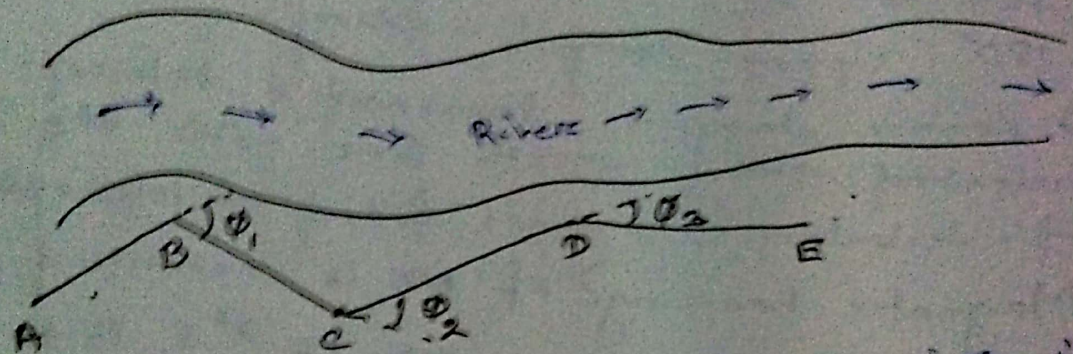
→ Arithmetical check is applied :-

$(2n-4) \times 90^\circ$ = Sum of interior angles.

If discrepancy is small, the error is distributed among the angles.

2) **Deflection angle method :-**

→ This method is suitable for open traverse & it is employed in survey of rivers, roads, railways etc.



→ In case of river, the traversing is carried out by following procedure :-

(i) Theodolite is set up at A & centered & levelled. Bearing of AB is measured.

(ii) Theodolite is then shifted to B & centered. Plate bubble is levelled & Vernier A & B are set to 0° & 180°.

(iii) Telescope is directed towards A & transited. The theodolite is then turned clockwise to bisect the ranging rod at C. The vernier readings are taken.

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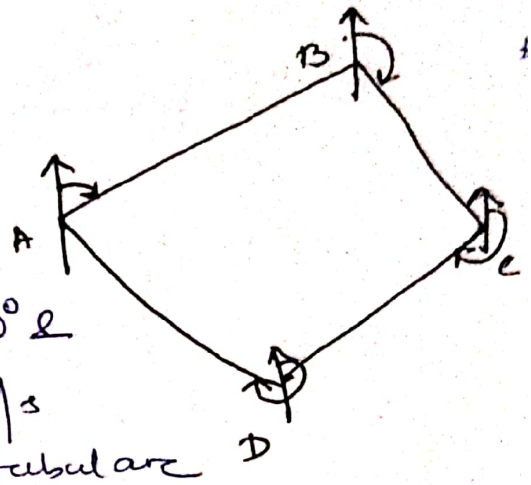
The deflection angle ϕ is obtained from avg. value of angles from vernier's A & B. Other deflection angles ϕ_2 & ϕ_3 are obtained similarly.

3) **Bearing Method / Fast Needle Method:-**

→ This method is used to measure magnetic bearing & length of traverse legs. The angle between the lines are not measured measured.



Let us assume ABCDA is a closed traverse.

→ The theodolite is set up at A. Vernier A is set to 0° & telescope is directed towards north line with help of tubular compass. The lower clamp is fixed.

→ Upper clamp is loosened & ranging rod at B is bisected. Reading on vernier A gives bearing of AB. Let it be 30°. The back bearing of line DA is measured from A upper clamp is fixed.

→ The instrument is shifted & set up at E with vernier A fixed at reading 30°. The lower clamp is loosened & ranging rod at A is bisected. The telescope is now transited. The upper clamp is released & ranging rod at C is bisected Vernier A reading is 100°.

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→ The instrument is shifted, centered & level at C with vernier A reading as 100°.
→ The bearing of CD is calculated with the same procedure.
→ The fore bearing of remaining sides are measured.
→ Finally the FB & BB of DA should differ by 180°.

Plotting the traverse by co-ordinate Method

In this method, survey stations are plotted by calculating their coordinates. This method is most practical & accurate one for plotting traverse or any other extensive system of horizontal control. The biggest advantage in this method is that, the closing error can be eliminated by balancing prior to plotting.

Checks for open & closed traverse :—

1) Check in open traverse :-

When a series of lines connected but extends along general direction & doesn't return to starting point is known as open traverse or unclosed traverse. It cannot be checked in field & following procedure is adopted :-

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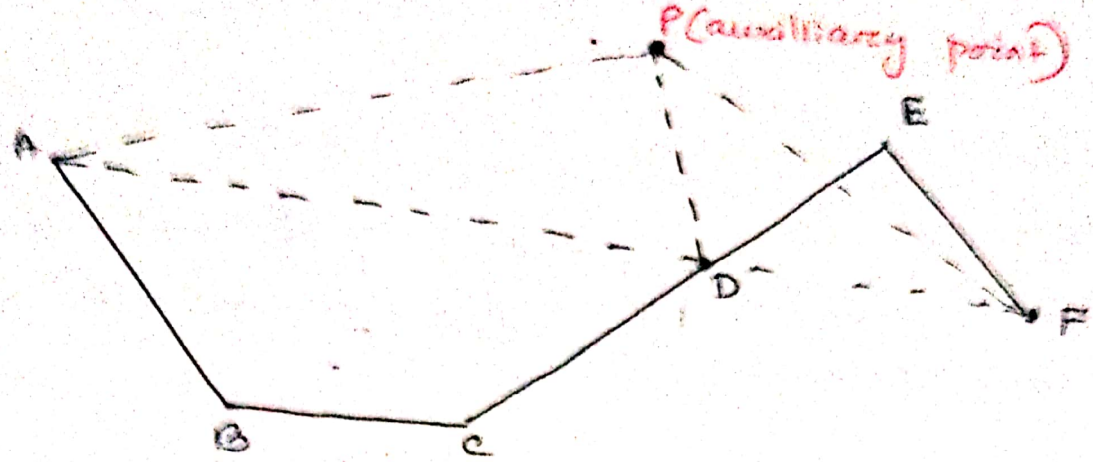
(i) Tie line or cut-off line :-



In case of open traverse ABCDEF, cut-off lines AD & DF are taken. PB & BB of lines AD and DF are measured alongwith distance of AD & DF. If after plotting the traverse, the distances, FB & BB of cut-off lines tally with field measurement, the traverse is said to be correct.

(ii) Auxilliary Point :-

An auxilliary point P is selected on one side of traverse. Then magnetic bearing of points A, D & F is taken. If traverse & plotting have been done accurately, the bearings must meet at P when plotted from the stations.

2) Check in closed traverse :-

→ The sum of measured interior angles should be equal to $(2n-4)\,90°$.

→ Sum of measured exterior angle should be equal to $(2n+4) \times 90°$, where n is no. of sides.

→ Algebraic sum of deflection angle should be equal to 360°, considering right hand deflec

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to be positive & left hand deflection negative.

→ The FB & BB of finishing line should differ by 180°.

→ The sum of northing should be equal to that of southings & sum of easting should be equal to that of westings.

6.6 Traverse Computations

1) Consecutive Coordinates (Latitude & Departure)

→ The latitude & departure of a point calculated with reference to the preceding point is called consecutive coordinate.

→ The latitude of a survey line may be defined as its co-ordinate measured parallel to the assumed meridian. Sometimes latitudes are also called meridians.

→ The departure of a survey line is defined as its coordinate measured perpendicular to the meridian. Sometimes departures are also called perpendiculars.

Latitude - Northing or Southing
Departure - Easting or westing.

→ The latitude (L) of the line is positive when measured northward or upward & termed as northing.

→ The latitude (L) of the line is negative when measured southward or downward & termed as southing.

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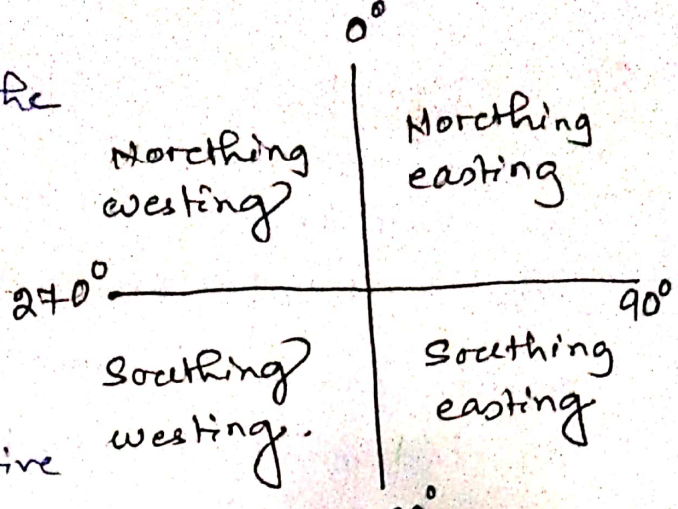
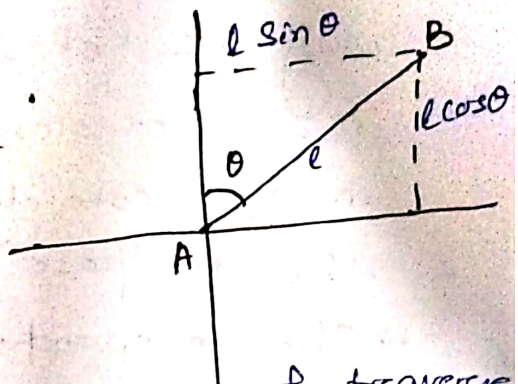
The departure (D) of the line is positive when measured eastward & termed as easting.

The departure (D) of the line is negative when measured westward & termed as westing.



→ The latitude & departure of the line AB of length l & reduced bearing θ is,



$L = +l\cos\theta$

$D = +l\sin\theta$.

→ To calculate latitude & departure of traverse lines, the bearing is first reduced to quadrantal bearing system.

→ Sign of latitude & departure will depend upon the reduced bearing of a line.

W.C.B.	R.B.	Sign of Latitude	Sign of Departure
0° to 90°	NOE : I	+	+
90° to 180°	SOE : II	−	+
180° to 270°	SOW : III	−	−
270° to 360°	NOW : IV	+	−

→ The latitude & departure of any station w.r.t. preceeding one is known as consecutive or dependent coordinates.

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2) Independent Coordinates :-

→ The coordinates of traverse stations can be calculated w.r.t. common origin.

→ The total latitude & departure of any point w.r.t. a common origin are known as independent coordinates or total coordinates of the point.

→ The two reference axes in this case may be chosen to pass through any of the traverse station but generally a most westerly station is choosen for this purpose.

→ The independent coordinates of any point may be obtained by adding algebraically the latitude & departure of lines between that point & origin.

→ In Gales table, the independent coordinates of all points are made positive, by suitably selecting the coordinates of starting station & all points are brought to first quadrant.

For example, the coordinate of starting station A are assumed to be (+150, +50).

Point	Line	Consecutive coordinates			Independent coordinate			Remark
		Northing	Southing	Easting	Westing	Northing	Easting	
A	—	—	—	—	—	150.00	50.00	A is starting point of traverse
B	AB	—	115.00	—	40.00	35.00	10.00	
C	BC	05.00	—	50.00	—	40.00	60.00	
D	CD	80.00	—	25.00	—	120.00	85.00	
A	DA	30.00	—	—	35.00	150.00	50.00	

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→ Traverse computation are done in tabular form. Gales traverse table is widely used because of its simplicity.

Following steps are involved in theodolite traversing

→ The included angles are adjusted to satisfy the geometrical condition i.e. the sum of included angles should be $(2n \pm 4)90°$ where n is no. of sides.

→ Starting with the observed bearings of one line, calculate the bearings of all other lines. Reduce all bearing to quadrantal bearing system,

→ Calculate the consecutive co-ordinates i.e. latitude & departures.

→ Calculate ΣL & ΣD.

→ Apply necessary corrections to the latitude & departures of the lines so that $\Sigma L = 0$ & $\Sigma D = 0$. The corrections may be applied to the methods.

→ Using the corrected consecutive co-ordinates, calculate the independent co-ordinate to the points so that they are all positive, the whole of the traverse thus lying in north east quadrant

(1)	(2)	(3)	(4)	(5)	(6)	(7)	Consecutive coordinates				Correction				Corrected consecutive coordinates				Independent coordinates	
							N	S	E	W	N	S	E	W	N	S	E	W	N	E
Line & length	Point	Angle	Correction	Corrected angle	W.C.B.	R.B.														

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Q.1 The following records are obtained in a traverse survey, where the length & bearing of the last line were not recorded :-

Line	length (m)	Bearing
AB	75.5	30°24′
BC	180.5	110°36′
CD	60.25	210°30′
DA	?	?

Compute the length & bearing of DA.

Ans :-

Let L = length of DA , θ = Bearing of DA.

line	length in m (L)	WCB	RB	Latitude (L cos θ)	Departure (L sin θ)
AB	75.5	30°24′	N 30°24′ E	75.5 × cos 30°24′ = +65.12	75.5 × sin 30°24′ = 38.21
BC	180.5	110°36′	S 69°24′ E	180.5 × cos 69°24′ = −63.51	180.5 × sin 69°24′ = 168.95
CD	60.25	210°30′	S 30°30′ W	60.25 × cos 30°30′ = −51.91	60.25 × sin 30°30′ = −30.6
DA	L	–	θ	L cos θ	L sin θ

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$\Sigma L = 0$

$\Rightarrow 65.12 - 63.51 - 51.91 + L\cos\theta = 0$

$\Rightarrow L\cos\theta = 50.3$

$\Sigma D = 0$

$38.21 + 168.95 - 30.6 + L\sin\theta = 0$

$\Rightarrow L\sin\theta = -176.56$

As latitude is positive & departure is negative, the line DA is in NW quadrant.

$$\frac{L\sin\theta}{L\cos\theta} = \frac{176.56}{50.3}$$

$\Rightarrow \tan\theta = 3.501$

$\Rightarrow \theta = 74°5'$

Bearing of DA = N74°5' W

Distance DA $= \sqrt{50.3^2 + 176.56^2}$

$= 183.58$ m.

Q.2

Line	length	Bearing
AB →	100 →	?
BC →	80.5 →	140°30'
CD →	60 →	220°30'
DA →	? →	310°15'

Calculate length of DA & Bearing of AB.

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Let length of DA = L & Bearing of AB = θ.

Line	length	WCB	RB	Latitude	Departure
AB	100	-	θ	$100\cos\theta$	$100\sin\theta$
BC	80.5	140°30′	S 39°30′ E	$80.5\times\cos 39°30'$ $= -62.12$	$80.5\times\sin 39°30'$ $= 51.2$
CD	60.0	220°30′	S 40°30′ W	$60\times\cos 40°30'$ $= -45.62$	$60\times\sin 40°30'$ $= -38.97$
DA	L	310°15′	N 49°45′ W	$L\cos 49°45'$ $= 0.646L$	$L\sin 49°45'$ $= -0.763L$

$\Sigma L = 0$

$\Rightarrow 100\cos\theta - 62.12 - 45.62 + 0.646L = 0$

$\Rightarrow 100\cos\theta = 107.74 - 0.646L$ ——— (1)

$\Sigma D = 0$

$\Rightarrow 100\sin\theta + 51.2 - 38.97 - 0.763L = 0$

$\Rightarrow 100\sin\theta = 0.763L - 12.23$ ——— (2)

Squaring & adding eqn. (1) & (2),

$100^2 = (107.74 - 0.646L)^2 + (0.763L - 12.23)^2$

$= 11607.9 - 139.2L + 0.42L^2 + 0.582L^2 - 18.66L + 149.6$

$= 11757.5 - 157.86L + L^2$

$\Rightarrow L^2 - 157.86L + 1757.5 = 0$

$$L = \frac{157.86 \pm \sqrt{(157.86)^2 - 4\times1\times1757.5}}{2}$$

$$= \frac{157.86 \pm 139.75}{2} = 145.8\text{ m or } 12.04\text{ m}$$

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When, length of DA, L = 145.8 m.

Putting in eqn. (1),

$100 \cos\theta = 107.74 - 0.646 \times 145.8 = 13.55$

$\cos\theta = 0.1355$ (+ve)

Putting in eqn. (2),

$100 \sin\theta = 0.763 \times 145.8 - 12.23 = 99.01$

$\sin\theta = 0.9901$

As cosθ & sinθ are positive, line AB is in NE quadrant

$$\frac{\sin\theta}{\cos\theta} = \frac{0.9901}{0.1355} = 7.307$$

$\theta = 82°12'$

Bearing of AB = N 82°12' E

When, length of DA, L = 12.04 m.

Putting in eqn. (1),

$100 \cos\theta = 107.74 - 0.646 \times 12.04 = 99.96$

$\cos\theta = 0.9996$

$100 \sin\theta = 0.763 \times 12.04 - 12.23 = -3.04$

$\sin\theta = -0.0304$

As cosθ is positive & sinθ is negative, AB is in NW quadrant

$$\tan\theta = \frac{0.0304}{0.9996} = 0.0304$$

$\theta = 1°44'$

Bearing of AB = N 1°44' W.

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Q.3

Line	length (m)	Bearing
AB	100.5	N 30°30' E
BC	?	S 45° E
CD	75.0	S 40°30' W
DE	50.5	S 60°0' W
EA	?	N 40°15' W

Calculate length of BC & EA.

Ans: Let Length of BC = l_1 & length of EA = l_2

Line	length	RB	Latitude	Departure
AB	100.5	N 30°30' E	$100.5 \times \cos 30°30'$ $= 86.59$	$100.5 \times \sin 30°30'$ $= 51$
BC	l_1	S 45° E	$l_1 \cos 45°$ $= -0.707\, l_1$	$l_1 \times \sin 45°$ $= 0.707\, l_1$
CD	75.0	S 40°30' W	$75 \times \cos 40°30'$ $= -57.03$	$75 \times \sin 40°30'$ $= -48.71$
DE	50.5	S 60° W	$50.5 \times \cos 60°$ $= -25.25$	$50.5 \times \sin 60°$ $= -43.73$
EA	l_2	N 40°15' W	$l_2 \cos 45°15'$ $= 0.763\, l_2$	$l_2 \sin 40°15'$ $= -0.646\, l_2$

$\Sigma L = 0$

$\Rightarrow 86.59 - 0.707\, l_1 - 57.03 - 25.25 + 0.763\, l_2 = 0$

$\Rightarrow 0.763\, l_2 - 0.707\, l_1 = -4.31$ ——— (1)

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$\Sigma D = 0$

$\Rightarrow 51 + 0.707 l_1 - 48.71 - 43.73 - 0.646 l_2 = 0$

$\Rightarrow 0.707 l_1 - 0.646 l_2 = 41.44$ ——————— (2)

Adding (1) & (2),

$0.117 l_2 = 37.13$

$\Rightarrow l_2 = 317.35 \text{ m}$

Putting in eqn. (1), we get

$0.707 l_1 = 0.763 \times 317.35 + 4.31$

$l_1 = 348.58 \text{ m}$.

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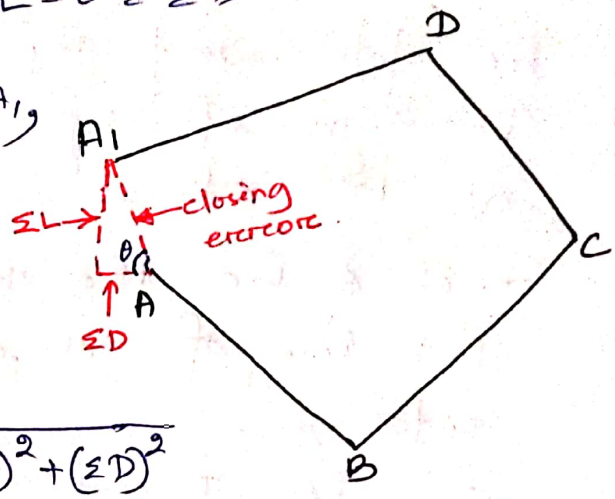
6.7 Closing Error :-

If a closed traverse is plotted as per field measurements, and the end point of the traverse does not coincide with the starting point, such error is known as closing error.

In closed traverse $\Sigma L = 0$ & $\Sigma D = 0$.

The traverse ABCDA, fails to close by a distance AA_1, which is the closing error of the traverse.



Closing error, $AA_1 = \sqrt{(\Sigma L)^2 + (\Sigma D)^2}$

Direction of closing error; $\tan\theta = \frac{\Sigma D}{\Sigma L}$.

Relative closing error = $\frac{\text{closing error}}{\text{Perimeter of traverse}}$.

The sign of ΣD & ΣL will decide the quadrant in which closing error lies.

Adjustment of Angular Error

- Before calculating latitudes & departures, the traverse angles should be adjusted to satisfy geometric conditions.
- → In closed traverse, the sum of interior angles should be equal to $(2n-4) \times 90°$.
- → The angular error of closure is distributed equally among all the angles if measured with equal precision.

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→ The angular error of closure should not exceed the least count multiplied by $\sqrt{n}$.

Adjustment of Bearing

In closed traverse the bearings are observed, the closing error in bearing may be determined by comparing the 2 bearings of the last line as observed at the first & last station of traverse.

→ Let e be the closing error in bearing of last line of closed traverse having N sides.

Correction for first line = $\frac{e}{N}$

" " 2nd " = $\frac{2e}{N}$

" " 3rd " = $\frac{3e}{N}$

" " last line = $\frac{Ne}{N} = e$

6.8 Balancing a traverse

In closed traverse, sum of latitude must be equal to 0 & that of departure must be equal to 0.

In actual practice, some closing error is always found to exist while computing latitude & departure of traverse stations.

The total error in latitude & departure are determined. These errors are distributed among the traverse stations.

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The following are common methods of adjusting a traverse:-

(a) Bowditch's method (c) Graphical method
(b) Transit Method (d) Axis method

(a) Bowditch's Method:-

This method is based on the assumption that the errors in linear measurements are proportional to $\sqrt{l}$ & the errors in angular measurements are inversely proportional to $\sqrt{l}$ where l is length of line.

This method is used where linear & angular measurements are of equal precision. The total error in latitude & in departure is proportion to length of side.

Correction to latitude or departure of any side = Total error in latitude or departure × (length of that side / Perimeter of traverse)

$$C_L = \Sigma L \cdot \frac{l}{\Sigma l} \quad \& \quad C_D = \Sigma D \frac{l}{\Sigma l}$$

C_L = Correction to latitude of any side
C_D = " " departure " " "
ΣL = total error in latitude
ΣD = " " " departure
Σl = length of perimeter
l = length of any side.

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(b) Transit Method :-

It is employed where angular measurements are more precise than linear measurements.

→ The total error in latitude & departure is distributed in proportion to the latitudes & departures of the sides.

Correction to latitude or departure of any side = total error in latitude or departure × (lat. or departure of that line / Arithmatic sum of latitude or departure.)

$$C_L = \Sigma L \cdot \frac{L}{L_T} \quad \& \quad C_D = \Sigma D \cdot \frac{D}{D_T}.$$

L = Latitude of any line
D = departure " " "
L_T = arithmatic sum of latitudes
D_T = " " " departures.

(c) Graphical Method :-

→ Bowditch's rule may be applied graphically without doing theoretical calculation.
→ In this method calculation of latitude & departure is not required.
→ Let ABCDEA be a closed traverse but plotted as $AB_1C_1D_1E_1A_1$ with A as starting point & A_1 as closing point.
→ Draw a straight line AA_1 equal to the perimeter of the plotted traverse. Mark point B_1, C_1, D_1, E_1 on the line equal to the length of traverse legs i.e. AB_1, B_1C_1, C_1D_1, D_1E_1 & E_1A_1.

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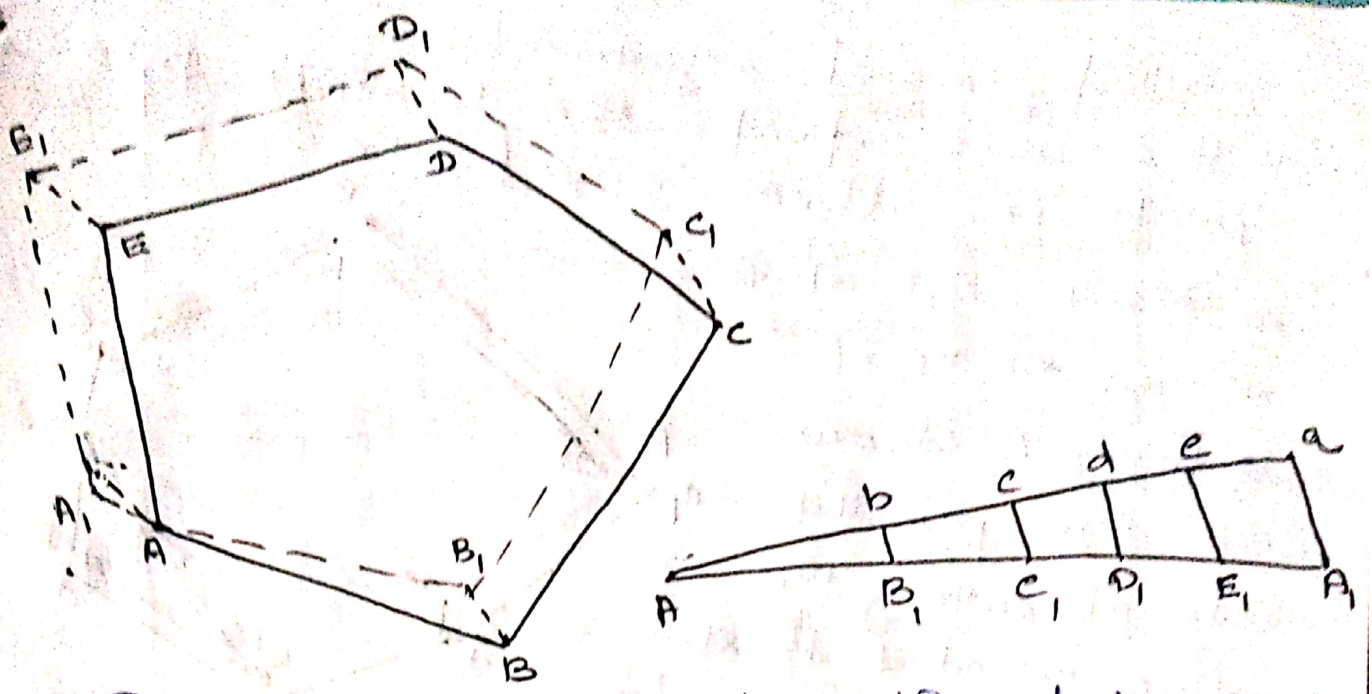

→ Draw a line A_1a equal to the closing error & parallel to the traverse. Connect Aa & from B_1, C_1, D_1 & E_1 draw B_1b, C_1c, D_1d & E_1e parallel to A_1a, intersecting the lines Aa at b, c, d & e.

→ The intercepts B_1b, C_1c, etc represent the corrections both in magnitude & direction at B, C etc

→ These corrections give the direction & distance through which the stations B, C etc. are to be shifted.

→ Join points A, B, C, D & E to get the traverse.

(d) **Axis Method** :-

This method is adopted when angles measured are correct & accurate but the lengths need correction.

→ Suppose abcdefa is a traverse with aa_1 as closing error.

→ aa_1 is connected & the line is extended which touches cd at x. The line a_1x is

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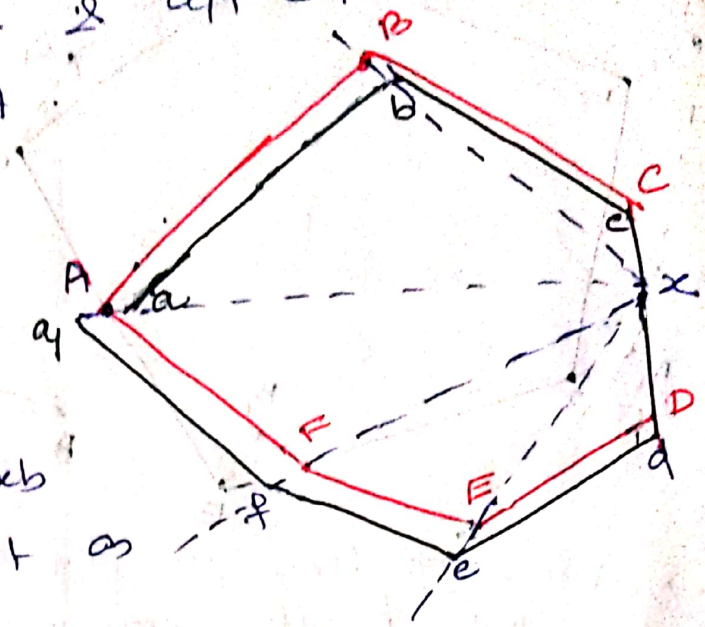
is called axis of adjustment.

→ This axis divides the traverse into 2 parts i.e. abcax & afedx.

→ Bisect aa at A.

→ Join xb, xe, xf

→ From A, draw a line parallel to ab cutting the line extended of xb at B and name it as AB.



→ From B, draw a line BC parallel to be cutting the line xe when produced in C.

→ Similarly, from A, AF is drawn parallel to AF to cut xf at F. From F, draw FE parallel to fe to cut xe in E. Likewise, from E, draw ED parallel to ed to cut xd in D.

ABCDEF is the adjusted traverse.

Correction to any length =

$$= \text{that length} \times \frac{\frac{1}{2} \times \text{closing}}{\text{length of axis}}$$

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Calculation of area of closed traverse :-

i) The coordinates
ii) Latitude & Double meridian distance.
iii) Departure & total latitude.

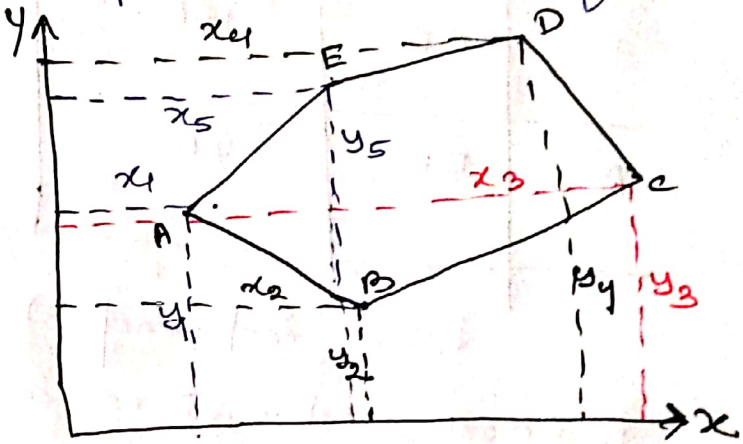
i) Calculation of area from Co-ordinates :-

The consecutive co-ordinates are converted into independent coordinates. Thus the whole traverse is transferred to the first quadrant

$$\Sigma P = (y_1x_2 + y_2x_3 + y_3x_4 + y_4x_5 + y_5x_1).$$

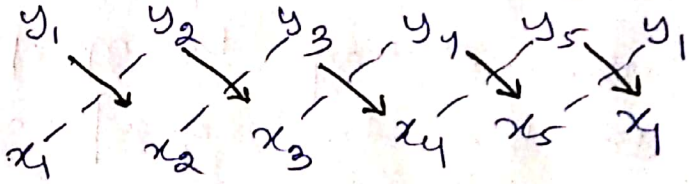
$$\Sigma Q = (x_1y_2 + x_2y_3 + x_3y_4 + x_4y_5 + x_5y_1).$$

Double area = $\Sigma P - \Sigma Q$



Required area

$$= \frac{1}{2} \times (\Sigma P - \Sigma Q)$$



Q :- Find the area of closed traverse by coordinate method.

Side	Latitude	Departure
AB	225.5	120.5
BC	-245	210
CD	-150.5	-110.5
DA	170	-220.

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Station	Side	Consecutive Coordinates		Independent Coordinates	
		Latitude (y)	Departure (x)	Latitude	Departure
A				200	100
B →	AB	+ 225.5	120.5	425.5	220.5
C	BC	− 245	210	180.5	430.5
D	CD	−150.5	−110.5	30	320
A	DA	170	− 220	200	100.

The independent coordinates of station A (most westerly) is assumed to be (+200, +100) Thus, the independent coordinates of all the stations become positive (i.e. they came to the first quadrant).

The coordinates are now arranged in determinant form as follows:—

200 → 425.5 → 180.5 → 30 → 200

100 → 220.5 → 430.5 → 320 → 100.

$\Sigma P = (200 \times 220.5) + (425.5 \times 430.5) + (180.5 \times 320) + (30 \times 100) = 288,037.75$.

$\Sigma Q = (100 \times 425.5) + (220.5 \times 180.5) + (430.5 \times 30) + (320 \times 200) = 159,265.5$

Area $= \frac{1}{2}(\Sigma P - \Sigma Q) = 64,386.25\ m^2$.

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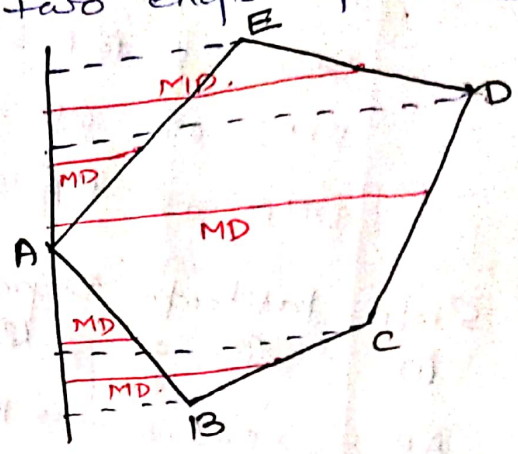
Calculation of area from latitude & Double meridian distance (DMD):—

Meridian distance is the perpendicular distance between the midpoint of any line & reference meridian.

DMD is the distance equal to the sum of meridian distance of two ends of the line.



i) DMD of first line
= departure of first line

ii) DMD of second line
= DMD of first line
+ departure of first line
+ departure of 2nd line.

iii) DMD of any succeeding line =
= DMD of preceding line
+ departure of preceding line
+ departure of line itself

iv) DMD of last line = departure of last line with opposite sign.

Procedure of Calculating area:—

→ Each DMD is multiplied by the latitude of that line.
→ The algebraic sum of these products is worked out.
→ The sum is equal to twice the area.
→ Half of this sum gives the required area of the traverse.

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Q). Side latitude Departure.

AB → 225.5 → 120.5
BC → -245.0 → 210.0
CD → -150.5 → -110.5
DA → 170.0 → -220.0.

Ans:-

DMD of AB = 120.5
DMD of BC = 120.5 + 120.5 + 210 = 451
DMD of CD = 451 + 210 - 110.5 = 550.5
DMD of DA = 550.5 - 110.5 - 220 = 220

Side (1)	latitude (2)	Departure (3)	DMD (4)	Double area = (2)×(4) (5)(+) 6(-)
AB	225.5	120.5	120.5	27172.75
BC	-245	210	451	— 110,495.0
CD	-150.5	-110.5	550.5	— 82850.25
DA	170.0	-220	220	37400
			Total =	+64572.75 (-) 193,345.

Algebraic sum = -128,772.5

Twice area = algebraic sum

Required area = $\frac{1}{2} \times (128772.5)$

= 64,386.25 m^2

(Negative sign is neglected).

(iii) Calculation of area from departure & total latitude:-

Consider A is the most westerly station & the reference meridian is assumed to pass through it. The procedure is as follows:-

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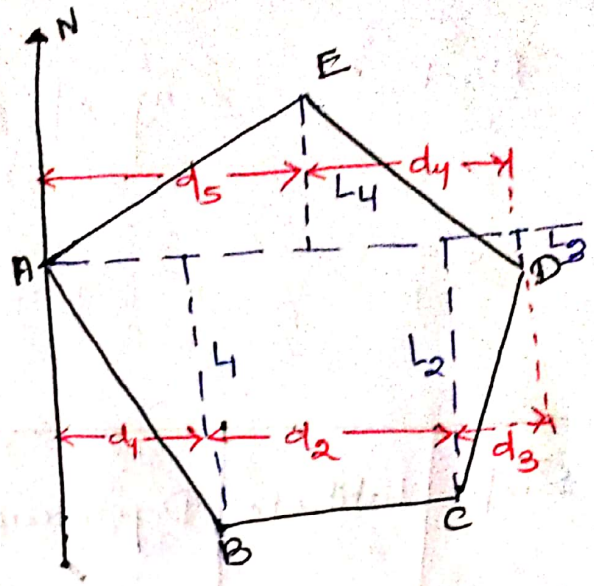

→ The total latitude of each station of the traverse is found out.

→ The algebraic sum of departures of two lines meeting at a station is determined.

→ The total latitude is multiplied by algebraic sum of departures, for each individual point.

→ The algebraic sum of this product gives twice the area.

→ Half of this sum gives the required area.

Q:-

Side	latitude	Departure
AB	225.5	120.5
BC	-245	210
CD	-150	-110.5
DA	170.0	-220.0

total latitude of B = 225.5

" " " C = 225.5 - 245 = -19.5

" " " D = 225.5 - 245 - 150 = -170.0

" " " A = 225.5 - 245 - 150.5 + 170

= 0.

Algebraic sum of departure at B = AB + BC

= 120.5 + 210.0

= 330.5

" " " C = BC + CD

= 210.0 - 110.5

= 99.5

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Algebraic sum of departure at D = CD + DA
= -110.5 - 220.0
= -330.5

" " " " " A = DA + AB
= -220 + 120.5
= -99.5.

Side (1)	latitude (2)	Departure (3)	Station (4)	Total Latitude (5)	Algebraic sum of adjoining departure (6)	Double area (5)×(6)
AB	225.5	120.5	B	225.5	330.5	(+) 74631.75
BC	-245	210	C	-19.5	99.5	(-) 1940.25
CD	-150.5	-110.5	D	-170.0	-330.5	(+) 56185
DA	170	-220	A	0.0	-99.5	0

Algebraic sum = 128772.5

So, Required area = $\frac{128772.5}{2}$ = 64386.25 m^2 (Ans).

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7.0 Levelling & Contouring

7.1.1 LEVELLING

It is the art of determining the relative heights of different points on or below the surface of earth.

7.1.2 Purpose of Levelling :-

→ To prepare contour map for fixing sites for reservoirs, dams, barrages, etc to fix the alignment of roads, railways, irrigation canals & so on.

→ To determine the altitudes of different important points on a hill or to know the reduced levels of different points on or below the surface of earth.

→ To prepare a longitudinal section & cross section of a project in order to determine the volume of earthwork.

→ To prepare a layout map for water supply, sanitary or drainage schemes.

7.1.3 Types of level

1) Dumpy level :- The telescope of the dumpy level is rigidly fixed to it's support. It cannot be removed from its support nor can it be rotated about its longitudinal axis. The instrument is stable & retains its permanent adjustment for a long time.

2) The WYE level (Y-Level) :- The telescope is held in two Y-supports. It can be removed from the supports & reversed from one end of the telescope to the other end. The Y support

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consists of two curved clips which may be raised. Thus, the telescope can be rotated about its longitudinal axis.

3) **Cooke's Reversible level** :- It is a combination of the dumpy level & Y-level. It is supported by 2 rigid sockets. The telescope can be rotated about its longitudinal axis.

4) **Cushing level** :- The telescope cannot be removed from the sockets & rotated about its longitudinal axis. The eye-piece & object glass are removable & can be interchanged from one end to other end of telescope.

5) **Modern tilting level** :- The telescope can be tilted slightly about its horizontal axis with the help of a tilting screw. In this instrument, the line of collimation is made horizontal of each observation by means of tilting screw.

6) **Automatic level** :- It is also known as self-aligning level. This instrument is levelled automatically within a certain tilt range by compensating device.

7.1.4 Definition of terms Used in Levelling :-

(1) **Level Surface** :- Any surface parallel to the mean spheroidal surface of earth is known as level surface.

(2) **Level line** :- Any line lying on a level surface is known as level line. This line is normal to the plumb line.

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(3) **Horizontal plane** :- Any plane tangential to the level surface is known as horizontal plane.

It is perpendicular to plumb line which indicates direction of gravity.

(4) **Horizontal line** :- Any line lying on the horizontal plane is known as horizontal line.

It is a straight line tangential to level line.

(5) **Vertical line** :- The direction indicated by plumb line is known as vertical line.

(6) **Vertical plane** :- Any plane paning through vertical line

(7) **Datum Surface or Datum line** :- It is an imaginared level surface or level line from which the ~~surface~~ distances of different points are vertical measured.

(8) **Reduced level (R.L.)** :- The vertical distance of a point above or below the datum line is known as R.L. It can be +ve or -ve accordingly if the point is above or below the datum line.

(9) **Bench Mark (B.M.)** :- These are fixed points or marks of known R.L. determined with reference to the datum line.

It serve as reference point for finding the RL of new points or for conducting levelling operations on projects involving roads, railways etc.

Bench marks are of four types :-

(i) **GTS Bench mark** :-

- It is great trigonometrical survey
- These BM are established by Survey of India Dept.

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→ The values of reduced level, their position & the no. of bench marks are given in a catalogue published by this dept.

(i) Permanent Bench marks :-

↳ These are established by diff. Govt. dept. like PWD, railways, irrigation etc.

↳ RL of these points are determined with reference to the GTS bench mark & are kept on permanent point like plinth of building, parapet of a bridge or culvert.

(iii) Temporary Bench marks :-

↳ when the bench marks are established temporarily at the end of a day's work, it is temporary B.M.
eg:- root of a tree, parapet of nearby culvert.

(iv) Arbitary Bench-marks :-

↳ when RL of some fixed points are assumed, they are termed as arbitary BM. It is adopted in small survey operations when only the undulation of ground surface is required to be determined.

7.2.1 Instruments used for Levelling :- Discussed in 7.1.3.

7.2.2 Line of collimation :- It is an imaginary line passing through the intersection of cross hairs at the diaphragm & optical of the object glass & its continuation.

7.2.3 Axis of telescope :- It is an imaginary line passing through the optical centre of the object glass & optical centre of eye piece.

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7.2.4 Axis of Bubble tube :— It is an imaginary line tangential to the longitudinal curve of the bubble tube at its middle point.

7.3 Levelling Staff

It is a graduated wooden rod used for measuring the vertical distances between the points on the ground & line of collimation. It is of two groups:—

i) Target Staff.
ii) Self Reading staff.

(i) Target Staff :— This staff consists of movable target. The target is provided with a vernier which is adjusted by the staffman, according to the direction from the levelman, so that the target coincides with the collimation line. After this reading is taken by either staffman or levelman. It is used for long sighting.

(ii) Self-Reading Staff :— There are different types of self-reading staff :—

a) Sop-with telescopic staff :- This staff is arranged in 3 lengths placed one into the other. It can be extended to its full length by pulling. The top portion is solid & of length 1.25m, central portion is hollow & of length 1.25m & bottom portion is hollow & 1.5m long. The total length is 4m & held in vertical position by brass spring catcher.

Staff is graduated with smallest division of 5mm. The values are marked in red & black

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b) Folding Metric staff :- It is made up of well seasoned wood of width 75mm, thickness 18mm & length 4m. It is divided into 2 parts of length 2m having a locking arrangement. It can be folded or detached when required.

c) One-length staff :- It is solid & made of seasoned timber. It is 3m long & graduated in same way as telescopic staff.

d) Invar Staff :- It is also 3m long. An invar band is fitted to wooden staff. The band is graduated by mm. It is used for precise levelling work.

DIAPHRAGM :-

It is a brass-ring fitted inside the telescope, just in front of the eye piece. It can be adjusted by four screws. The ring carries the cross-hair, which gets magnified when viewed through eye-piece.

The cross-hairs may be marked in the following ways :-

→ With spider webs stretched across the ring.
→ By very fine scratch marks made in glass fitted with the ring.
→ The means of platinum wires or silk threads stretched across the ring.

The cross hair consists of following lines :-

→ Two vertical hairs meant for maintaining the verticality of the staff.

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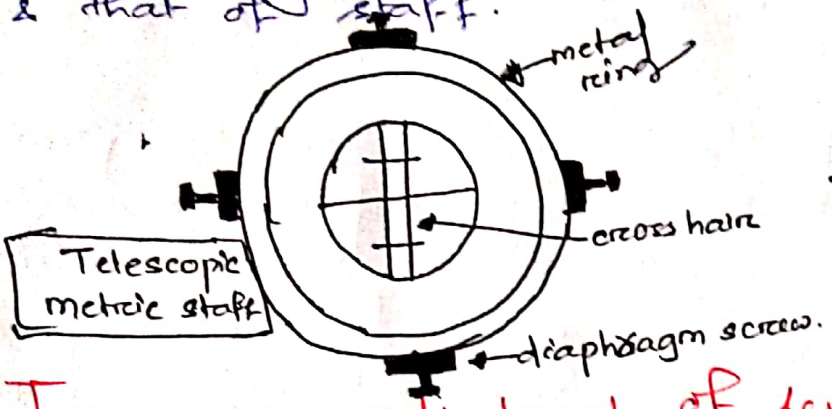
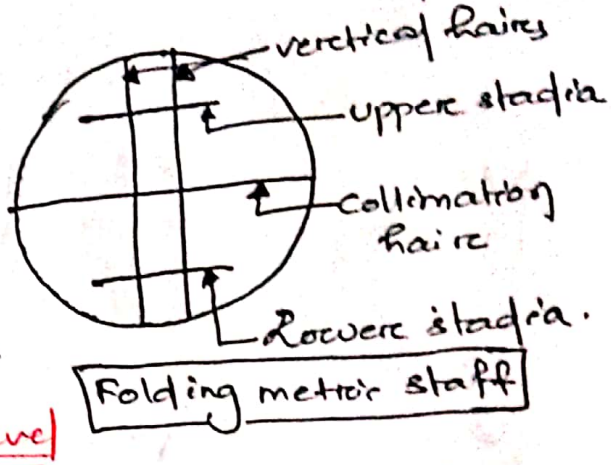
→ Middle horizontal hair representing the line of collimation
→ Upper stadia hair & lower stadia hair, both horizontal & short in length. The stadia hairs are meant for determining horizontal distance between position of level & that of staff.





Temporary adjustment of level

The adjustment made at every set up of the level before taking staff readings are known as temporary adjustment.

(1) **Selection of Suitable Station** :- A suitable position is selected for setting the level. From this position, greatest no. of observations should be possible to take without any difficulty. The ground should be level & firm.

(2) **Fixing level with tripod Stand** :- The tripod stand is placed at required position with legs well apart & firmly pressed into the ground.
The level is fixed on top of tripod stand as per fixing arrangement of the level

(3) **Approximate levelling by legs of tripod stand** :-
→ The foot screws of the level are brought to the centre of their run.
→ Two legs of the tripod stand are firmly fixed into the ground & 3rd leg is moved to the left or right, in or out until bubble is at it's centre

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(4) Perfect levelling by foot screws :-

→ The longitudinal bubble is on top of the telescope so the telescope is placed parallel to any pair of foot screws and bubble is brought to the centre by turning both foot screws either inward or outward.

→ Next the telescope is turned 90° to the previous position & 3rd foot screw is rotated either clockwise or anticlockwise to bring the bubble to the centre.

→ The telescope is brought to its first position & the same process is repeated to bring the bubble to the centre.

→ The process is repeated until bubble remains at its centre in all position.

→ The telescope is rotated through 180° to check whether the bubble remains at centre or not.

→ If the bubble remains at its centre, temporary & permanent adjustments are perfect.

→ If bubble remains deflected from centre, the permanent adjustment is not perfect & needs to be modified.

(5) Focussing of eye-piece :- A white paper is held in front of object glass & eye piece is rotated clockwise or anticlockwise until cross-hairs are seen clearly.

(6) Focussing the object glass :- The telescope is directed towards levelling staff while looking through the eye-piece, the focusing screw is turned clockwise or anticlockwise until the graduation of staff is clearly visible.

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Taking Staff Readings

Levelling is verified by turning the telescope in all directions when bubble remains in the centered position of the bubble tube of the telescope, the staff readings are taken.

BS (Backsight reading) :-

It is the first staff reading in any set up of the instrument after the levelling has been done. It is always taken on a point of known RL i.e. on the bench-mark or change point.

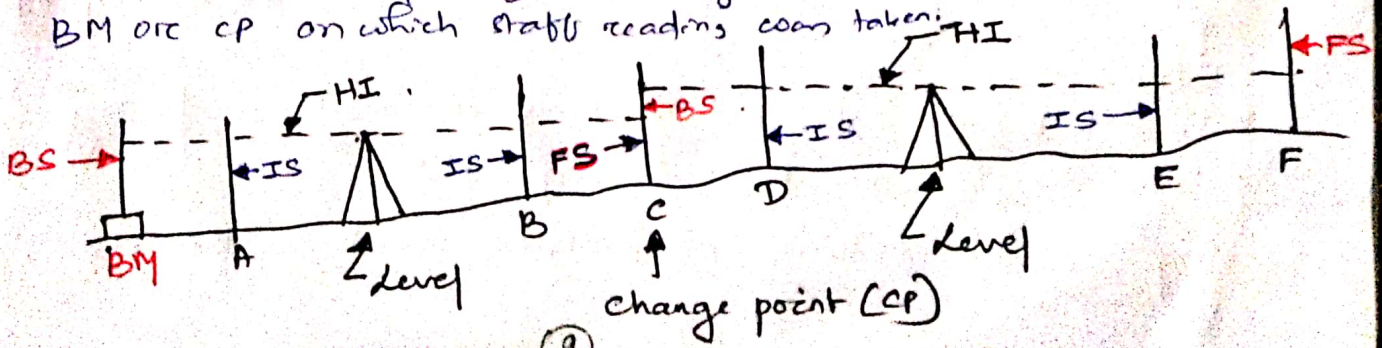
FS (Foresight Reading) :-

It is the last staff reading in any set up of the instrument & indicates shifting of instrument.

IS (Intermediate sight reading) :- It is any other staff reading between the back sight & FS in the same set up of the instrument.

CP (Change Point) :- This point represents the shifting of instrument. Here, an FS is taken from one setting & BS from the next setting.

HI (Height of Instrument) :- After perfect levelling of instrument, the RL of the line of collimation is known as height of instrument (HI). It is obtained by adding BS reading to RL of BM or CP on which staff readings were taken.



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7.4 Field data entry in Level Book :-

Field book or level book calculations are carried out on the readings noted during a levelling operation & then they are entered.

Following points has to be remembered while entering :-

(1) The first reading of any set up is entered in the BS column, the last reading in the FS column & other readings in IS column.

(2) A page always starts with a BS reading & finishes with an FS reading.

(3) If a page finishes with an IS reading, the reading is entered in the IS & FS columns of that page & brought forward to the next page by entering it in the BS and IS columns.

(4) The FS and BS of any change point are entered in the same horizontal line.

(5) The RL of the line of collimation is entered in the same horizontal line in which the corresponding BS was entered.

(6) Important note, bench-marks and change points should be clearly described in the remarks column.

(1) Height of Collimation Method :-

→ The H.I. is calculated for each setting of the instrument by adding the BS to the elevation of BM.

→ The R.L. of first station is obtained by subtracting its FS from HI.

→ In second setting of instrument, the H.I is

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obtained by adding the back sight taken on first station to its reduced level.

→ R.L. of last point is obtained by subtracting the fore-sight of the last point from HI at the last setting.

→ In case of any intermediate sight observed at an intermediate station, its reduced level is obtained by subtracting its fore-sight from the ht. of instrument fore the setting.

Arithmetic check :- $\Sigma B.S. - \Sigma F.S. = \text{Last R.L} - \text{first R.L.}$

(2) Rise & Fall Method :-

→ In this method, the difference of level between 2 consecutive points for each setting of the instrument is obtained by comparing their staff reading.

→ The difference of these staff readings indicate a rise if back sight reading is more than the fore sight & a fall if it is less than fore sight.

→ If RL of Back staff (reading) point is known, then R.L. of following points may be obtained by adding its rise or subtracting its fall from the R.L of preceeding point.

Arithmetic Checks :-

$\Sigma B.S - \Sigma F.S. = \Sigma \text{Rise} - \Sigma \text{fall} = \text{Last R.L.} - \text{First R.L.}$

COMPARISON

Height of collimation Method	Rise & Fall method
1) It is more rapid & saves time & labour.	1) It is tabore as the staff reading of each station is compared to get a rise or fall.

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Height of collimation Method	Rise & Fall Method
2) It is adopted for reduction of level for construction work such as longitudinal & cross-section levelling operation.	2) It is adopted for determining the difference of level between 2 points where precession is required.
3) No check on reduction of R.L. of intermediate stations.	3) Complete check on the reduction of R.L. of intermediate station.
4) Only 2 arithmatic checks i.e. the difference bet^n the sum of BS & FS must be equal to difference in R.L. of last & first station.	4) 3 arithmetic checks i.e. diff. bet^n sum of BS & sum of FS must be equal to diff. bet^n sum of rises & sum of falls as well as it must be equal to diff of RL of last station & first station.
5) Errors in any in intermediate sights are not detected.	5) Errors in intermediate sight are noticed as these are used for finding out rises & falls.

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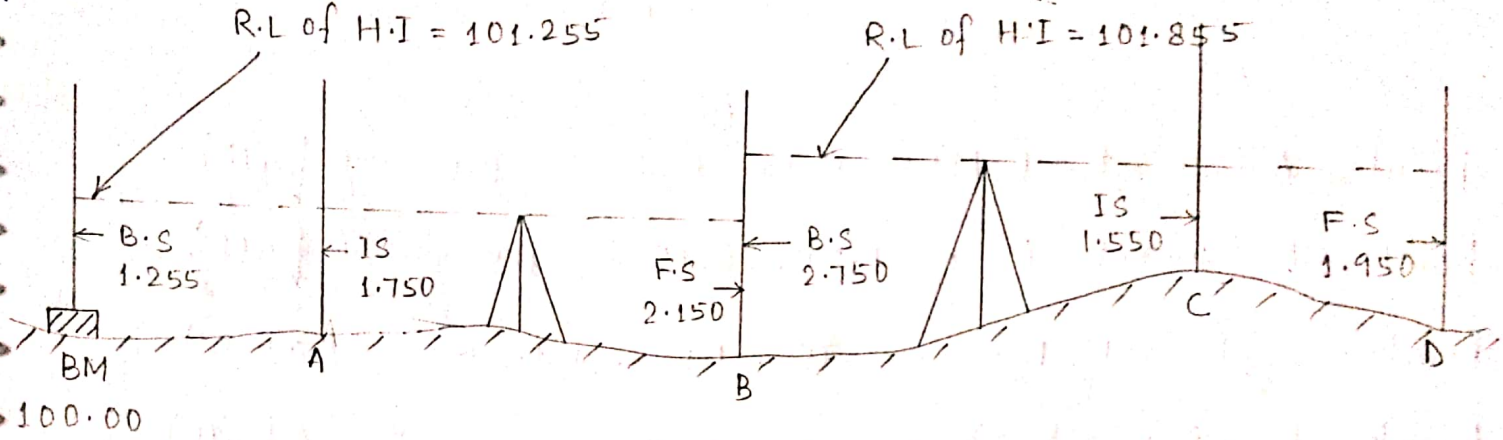
NUMERICAL PROBLEMS :-

Ex : 1



R.L of H.I in 1st setting = 100 + 1.255 = 101.255

R.L of A = 101.255 - 1.750 = 99.505

R.L of B = 101.255 - 2.150 = 99.105

R.L of H.I in 2nd setting = 99.105 + 2.75 = 101.855

R.L of C = 101.855 - 1.550 = 100.305

R.L of D = 101.855 - 1.950 = 99.905

Arithmetic Check : Σ B.S - Σ F.S = last R.L - first R.L

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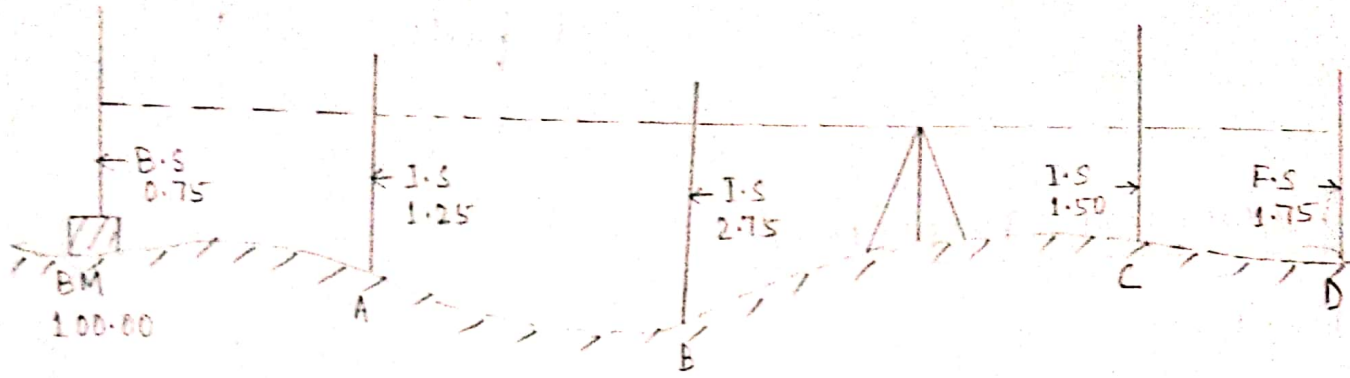
Ex : 2



Point A (w.r.t B.M) = 0.75 - 1.25 = - 0.5 (fall)

Point B (w.r.t A) = 1.25 - 2.75 = - 1.5 (fall)

Point C (w.r.t B) = 2.75 - 1.5 = + 1.25 (rise)

Point D (w.r.t C) = 1.5 - 1.75 = - 0.25 (fall)

R.L of B.M = 100.00

R.L of A = 100 - 0.5 = 99.50

R.L of B = 99.5 - 1.5 = 98.00

R.L of C = 98.00 + 1.25 = 99.25

RL of D = 99.25 - 0.25 = 99.00

Arithmetic Check :-

Σ B.S - Σ F.S = Σ Rise - Σ fall = last R.L - first R.L

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Q.1 The following consecutive readings were taken with a dumpy level along a chain line at a common interval of 15m. The first reading was at a chainage of 165m where the RL is 98.085. The instrument was shifted after fourth & ninth readings.

3.150, 2.245, 1.125, 0.860, 3.125, 2.76, 1.835, 1.47, 1.965, 1.255, 2.39 and 3.035 m.

Find the R.L of all points by :-

→ Collimation System

→ Rise & fall system

Collimation System :-

Station Point	Chainage	B.S	I.S	F.S	R.L of collimation line (H.I)	R.L	Remark
1.	165	3.15			101.235	98.085	
2.	180		2.245			98.99	
3.	195		1.125			100.11	
4.	210	3.125		0.86	103.5	100.375	Change Point
5.	225		2.76			100.74	
6.	240		1.835			101.665	
7.	255		1.47			102.03	
8.	270	1.225		1.965	102.76	101.535	Change point
9.	285		2.39			100.37	
10.	300			3.035		99.725	
Total		7.5		5.86			

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Arithmetic Check :-

$\Sigma B.S - \Sigma F.S = 7.5 - 5.86 = 1.64$.

Last R.L - First R.L = 99.725 − 99.085 = 1.64.

Rise & Fall System :-

Station Point	Chainage	B.S	I.S	F.S	Rise (+)	Fall (−)	R.L	Remark
1.	165	3.15					98.085	
2.	180		2.245		0.905		98.99	
3.	195		1.125		1.12		100.11	
4.	210	3.125		0.86	0.265		100.375	Change Point
5.	225		2.76		0.365		100.74	
6.	240		1.835		0.925		101.665	
7.	255		1.47		0.365		102.03	
8.	270	1.225		1.965		0.495	101.535	Change Point
9.	285		2.39			1.165	100.37	
10.	300			3.035		0.645	99.725	
Total		7.5		5.86	3.945	2.305		

Arithmetic Check :-

$\Sigma B.S - \Sigma F.S = 7.5 - 5.86 = 1.64$

$\Sigma Rise - \Sigma fall = 3.945 - 2.305 = 1.64$

Last RL - first RL = 99.725 − 98.085 = 1.64

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Q.2 The following consecutive readings were taken with a levelling instrument at interval of 20m.

2.375, 1.73, 0.615, 3.45, 2.835, 2.070, 1.835, 0.985, 0.435, 1.63, 2.255 and 3.63 m.

The instrument was shifted after the fourth & eighth readings. The last reading was taken on a BM of RL 110.200 m. Find RL of all points.

Station Point	Chainage	B.S	I.S	F.S	Rise (+)	Fall (-)	RL	Remark
1.	0	2.375					112.62	
2.	20		1.73		0.645		113.265	
3.	40		0.615		1.115		114.38	
4.	60	2.835		3.45		2.835	111.545	Change Point
5.	80		2.070		0.765		112.31	
6.	100		1.835		0.235		112.545	
7.	120	0.435		0.985	0.85		113.395	Change Point
8.	140		1.63			1.195	112.20	
9.	160		2.255			0.625	111.575	
10.	180			3.63		1.375	110.2	on BM
Total =		5.645		8.065	3.61	6.03		

$\Sigma B.S - \Sigma F.S = 5.645 - 8.065 = -2.42$

$\Sigma Rise - \Sigma Fall = 3.61 - 6.03 = -2.42$

Last R.L - first R.L = -2.42

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Q.3 The following successive readings were taken with a dumpy level along a chain line at common intervals of 20m. The first reading was taken on a chainage 140m. The R.L of second change point was 107.215m. The instrument was shifted after the third & seventh readings. Calculate the R.L of all the points.

3.15, 2.245, 1.125, 3.86, 2.125, 0.76, 2.235, 0.47, 1.935, 3.225 & 3.89m.

Station Point	Chainage	B.S	I.S	F.S	Rise (+)	Fall (−)	R.L	Remark
1	140	3.15					103.565	
2	160		2.245		0.905		104.47	
3	180	3.86		1.125	1.120		105.59	1st CP
4	200		2.125		1.735		107.325	
5	220		0.76		1.365		108.69	
6	240	0.47		2.235		1.475	107.215	2nd CP
7	260		1.935			1.465	105.75	
8	280		3.225			1.29	104.46	
9	300			3.89		0.665	103.795	
Total		7.48		7.25	5.125	4.895		

$\Sigma B.S - \Sigma F.S = 7.48 - 7.25 = +0.23$

$\Sigma Rise - \Sigma Fall = 5.125 - 4.895 = 0.23$.

Last R.L - first R.L = 0.23

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Q.4 The following consecutive readings were taken with a level & a 4m levelling staff on a continuously sloping ground at common interval of 30m.

0.855 (on A), 1.545 m, 2.335, 3.115, 3.825, 0.455, 1.38, 2.055, 2.855, 3.455, 0.585, 1.015, 1.85, 2.755, 3.845 (on B).

The R.L of A was 380.5. Make entries in the level book & apply the usual checks. Determine the gradient of AB.

Station Point	Chainage	B.S	I.S	F.S	Rise (+)	Fall (-)	R.L	Remark
A	0	0.855					380.5	
	30		1.545			0.69	379.81	
	60		2.335			0.79	379.02	
	90		3.115			0.78	378.24	
	120	0.455		3.825		0.71	377.53	Change Point
	150		1.38			0.925	376.605	
	180		2.055			0.675	375.93	
	210		2.855			0.80	375.13	
	240	0.585		3.455		0.60	374.53	Change Point
	270		1.015			0.43	374.1	
	300		1.85			0.835	373.265	
	330		2.755			0.905	372.36	
B	360			3.845		1.09	371.27	
Total=		1.895		11.125	0	9.230		

$\Sigma BS - \Sigma FS = 1.895 - 11.125 = -9.23.$

$\Sigma Rise - \Sigma fall = 0 - 9.23 = -9.23.$

Last R.L - first R.L = 371.27 - 380.5 = -9.23.

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Falling gradient of AB = $\frac{\text{difference of level}}{\text{Horizontal distance}}$

$= \frac{9.23}{360} = \frac{1}{39}$.

5. In fly levelling from the BM of R.L 140.605, the following readings were observed.

Backsight :- 1.545, 2.695, 1.415, 2.925

Foresight :- 0.575, 1.235, 0.595

From the last position of instrument, six pegs at 20m interval are to be set out on a uniformly rising gradient of 1 in 50, the first peg is to have an R.L of 144.00. Find the staff readings & RL's of the pegs.

Station	Chainage	B.S	I.S	F.S	Rise (+)	Fall (−)	R.L	Remark
BM		1.545					140.605	on BM
		2.695		0.575	0.970		141.575	
		1.415		1.235	1.46		143.035	
		2.925		0.595	0.82		143.855	
peg 1	0		2.780		0.145		144.00	
peg 2	20		2.380		0.40		144.40	
peg 3	40		1.980		0.40		144.80	
peg 4	60		1.580		0.40		145.20	
peg 5	80		1.180		0.40		145.6	
peg 6	100			0.780	0.40		146.0	
Total		8.580		3.185	5.395	0		

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Rise per 20m = $\frac{20}{50}$ = 0.40:

6. During a fly levelling operation, the following observations were made :-

BS - 0.65, 2.155, 1.405, 2.655, 2.435 m.

FS - 2.455, 1.305, 0.555, 2.405 m.

The first BS was taken on a BM of RL 90.5m. From the last BS, it is required to set four pegs each at a distance of 30m on a falling gradient of 1 in 100. Calculate the RL of these four pege.

Station	Chainage	B.S	I.S	F.S	Rise (+)	Fall (-)	R.L	Remark
BM		0.65					90.5	on B.M
		2.155		2.455		1.805	88.695	
		1.405		1.305	0.85		89.545	
		2.655		0.555	0.85		90.395	
	0	2.435		2.405	0.25		90.645	
Peg 1	30		2.735			0.30	90.345	
Peg 2	60		3.035			0.30	90.045	
Peg 3	90		3.335			0.30	89.745	
Peg 4	120			3.635		0.30	89.445	
		9.30		10.355	1.95	3.005		

Fall per 30m = $\frac{30}{100}$ = 0.3

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7. The following is the page of a level book, where some readings were missing. Fill in the missing readings & calculate the reduced levels of all the points.

Station	B.S	I.S	F.S	Rise (+)	Fall (-)	R.L	Remark
1	3.25					2,49.26	BM
2.	1.755		? 4.0		0.75	248.51	CP
3.		1.95			0.195	248.315	
4.	? 3.84		1.92	0.30		248.345	CP
5.		2.34		1.5		249.845	
6.		? 1.34		1.0		250.845	
7.	1.85		2.185		0.845	250.0	CP
8.		1.575		0.275		250.275	
9.		? 3.545			1.97	248.305	
10.	? 2.1		1.895	1.65		149.955	CP
11.			1.350	0.75		250.705	

8. A page of a level book with some missing readings is reproduced below. Fill in the missing details along with the necessary arithmetical checks.

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Station	B.S	I.S.	F.S	Rise (+)	Fall (-)	R.L	Remark
1	3.65					108.26	
		X 0.9		2.75		111.01	
		2.83			1.93	109.08	
		3.64			0.81	108.27	
	X 1.09		2.42	1.22		109.49	CP
		2.41			1.32	108.17	
		2.32		0.09		108.26	
		3.0			0.68	107.58	
		-2.17		5.17		112.75	Negative sign indicates staff held against ceiling.
	X 1.75		X 0.58		2.75	110.0	CP
			X 3.07		1.32	108.68	

Check:-

$\Sigma BS - \Sigma FS = 6.49 - 6.07 = 0.42$

$\Sigma Rise - \Sigma fall = 9.23 - 8.81 = 0.42$

last RL - first RL $= 108.68 - 108.26 = 0.42$

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DESCRIPTION OF DUMPY LEVEL :-

1. TRIPOD STAND :-

It consist of three legs which may be solid or framed. The legs are made up of light & hard wood. The lower ends of the legs are fitted with steel shoe.

2. LEVELLING HEAD :-

It consist of two parallel triangular plates having three grooves to support the foot screws.

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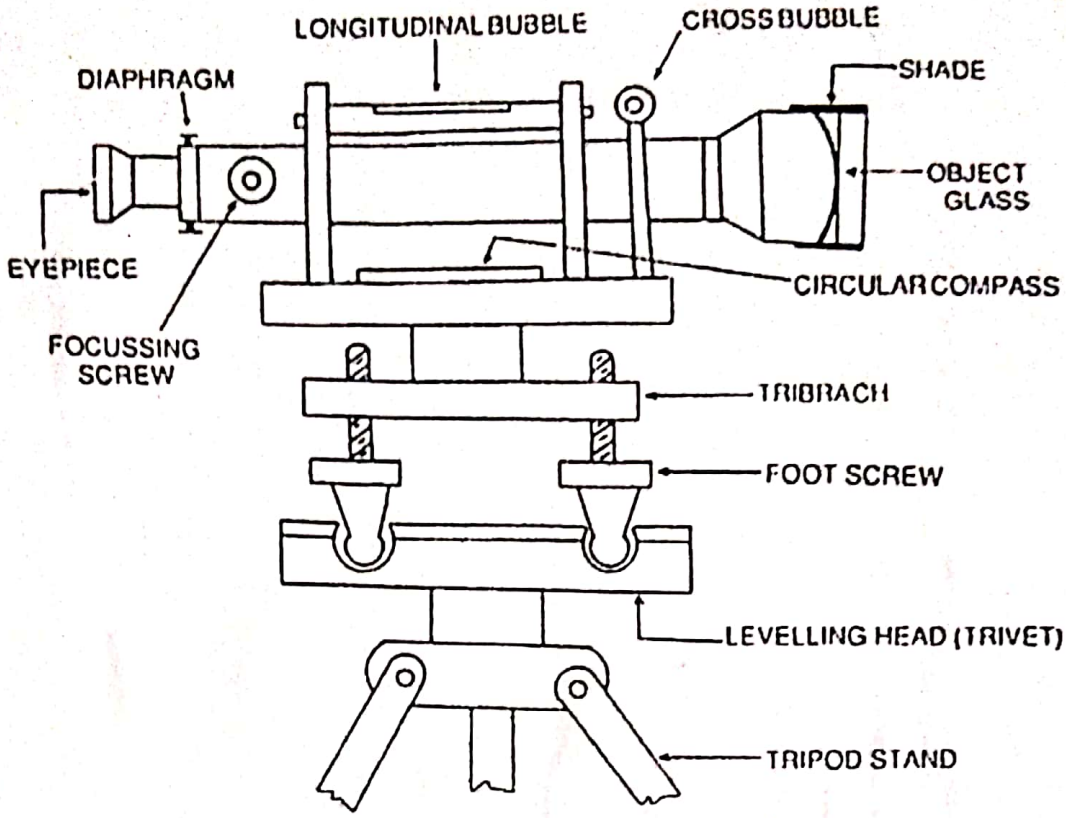

3. FOOT SCREW :-

Three foot screws are provided between the trivet and tribrach. By turning the foot screw the tribrach can be raised or lowered to bring the bubble to the center of its run.

4. TELESCOPE :-

It consist of two metal tubes, one moving within the other. It also contain an object glass & an eye piece on opposite ends. A diaphragm is fixed with the telescope just in front of the eye-piece. The diaphragm carriers cross-hairs. Focussing of telescope is done by means of focussing screw.

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5. BUBBLE TUBES :-

Two bubble tubes, one called the longitudinal bubble tube and other the cross-bubble tube, are placed at right angle to each other. These tubes contain spirit bubble. Bubble is brought to the center with the help of foot screws. The bubble tubes are fixed on the top of telescope.

6. COMPASS :-

It is provided just below the telescope for taking readings or magnetic bearings of the line when required.

The compass is graduated in such a way that a pointer, which is fixed to the body of compass, indicates 0° when the telescope is directed along north line.

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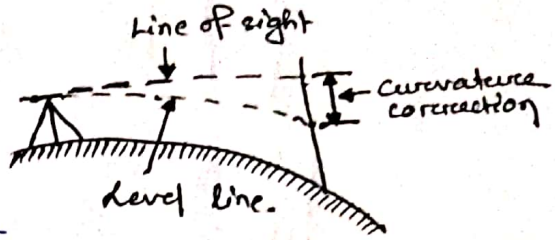
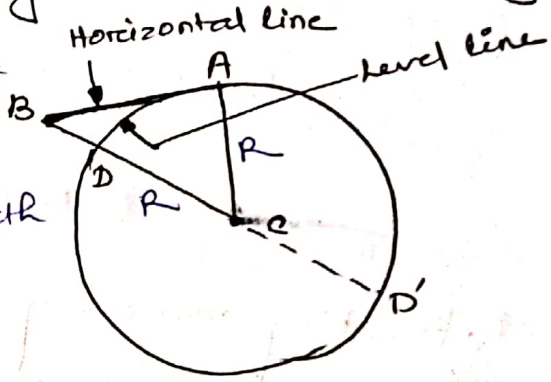
2.5. Effect of Curvature :-

For long sights, the curvature of the earth surface affects staff readings. The line of sight is horizontal, but the level line is curved and parallel to the mean spheroidal surface of earth.



The vertical distance between the line of sight & the level line at a particular place is called the curvature correction. Due to curvature, objects appear lower than they really are.

Curvature correction is always subtractive (i.e. negative)



Let AB = D = horizontal distance (in km).

BD = C_c = curvature correct°

DC = AC = R = radius of earth

DD' = diameter = 12,742 km.

From right angle triangle, ABC in fig.,

$BC^2 = AC^2 + AB^2$

$\Rightarrow (R + C_c)^2 = R^2 + D^2$

$\Rightarrow R^2 + 2RC_c + C_c^2 = R^2 + D^2$

C_c^2 is neglected as it is very very small in comparision to diameter of earth (DD' = 2R)

$2RC_c = \quad = C_c = \frac{D^2}{2R} = \frac{D^2 \times 1000}{12,742}$

$\Rightarrow$

$\Rightarrow C_c = 0.0785 D^2$ m. (negative)

So, True staff reading = Observed staff reading − Curvature correction (C_c)

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7.5.2 Effect of Refraction

Rays of light are refracted when they pass through layers of air of varying density. So, when long sights are taken, the line of sight is refracted towards the surface of earth in a curved path. The radius of this curve is 7 times that of earth under normal atmospheric conditions. Due to this effect of refraction, objects appear higher than they really are.

However on an average, the refraction correction is taken as $1/7^{th}$ of curvature correction.

$$C_r = \frac{1}{7} \times C_c = \frac{1}{7} \times \frac{D^2}{2R}$$

$$\therefore C_r = \frac{1}{7} \times 0.0785 D^2 = 0.0112 D^2 \, m \, (+ve)$$

Refraction correction is always positive.

So, True staff reading = Observed staff reading + Refraction correction.

7.5.3 Combined correction :-

The combined effect of curvature and refraction is as follows :—

Combined correction = Curvature correct^n + Refraction correct^n

$$= -0.0785 D^2 + 0.0112 D^2$$

$$= -0.0673 D^2 \, m.$$

So, combined correction is always subtractive.

True staff reading = Observed staff reading − Combined correct^n.

It is also expressed as

$$\frac{D^2}{2R} - \frac{1}{7}\frac{D^2}{2R} = \frac{6}{7}\frac{D^2}{2R}$$

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7.6 Reciprocal Levelling

Principle :-

The principle is based on the fact that when the instrument is kept equidistant from the back & forward staff stations, the difference in elevation of two stations is equal to the difference of staff reading.

By setting the level mid way, the errors due to curvature & refraction, & also collimation error if any, are eliminated.

In case of river or valley, it is not possible to set up the level mid-way between two points on opposite banks. In such case, method of reciprocal levelling is adopted.

In reciprocal levelling, the level is set up on both banks of the river or valley & 2 sets of staff readings are taken by holding the staff on both banks.

The errors are completely eliminated & true diff. of level is equal to the mean of the two differences of level.

Method :-

1) Suppose A & B are 2 points on opposite bank of a river. The level is set up near A & after proper temporary adjustment, staff readings are taken at A & B. Let the readings are a_1 & b_1.

2) The level is shifted & set up near B &

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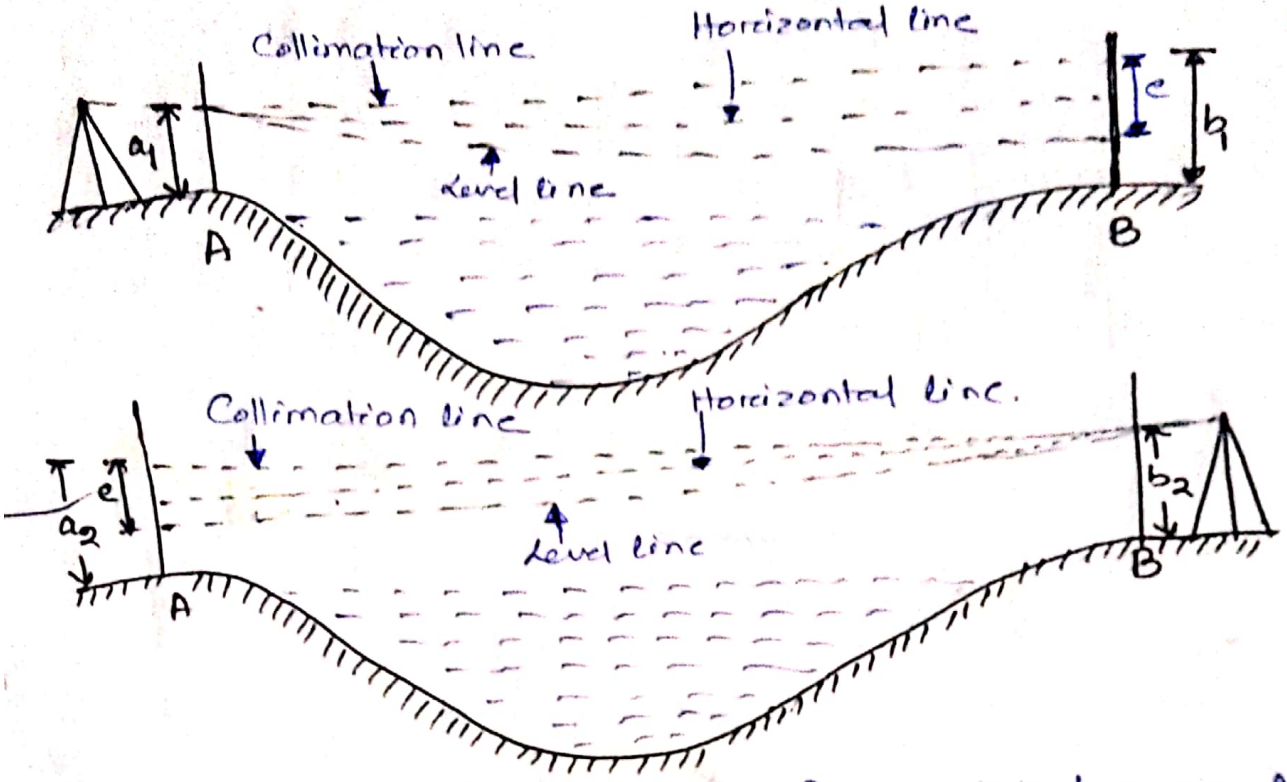
after proper adjustment, staff readings are taken at A & B. Suppose the readings are a_2 & b_2.



Let, h = true difference of level between A & B
e = combined error due to curvature, refract^ & collimation.

In 1st case,

Correct staff reading at A = a_1

Correct staff reading at B = $b_1 - e$

True difference of level between A & B,

$h = a_1 - (b_1 - e)$ (fall from B to A) ——— ①

In 2nd case,

Correct staff reading at B = b_2

Correct staff reading at A = $a_2 - e$.

True diff. of level,

$h = (a_2 - e) - b_2$ ——— ②.

From eq^ ① & ②

$2h = a_1 - (b_1 - e) + (a_2 - e) - b_2$

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$\Rightarrow 2h = a_1 - b + e + a_2 - e - b_2 = (a_1 - b_1) + (a_2 - b_2)$.

$\Rightarrow \boxed{h = \frac{(a_1 - b_1) + (a_2 - b_2)}{2}}$.

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Q.1 In reciprocal levelling, two points A & B are taken on opp. banks of a river. When the level was set up near A, the staff readings on A & B were 2.245 & 3.375 respectively. When the level was set up near B, the respective staff readings were 1.955 & 3.055. Find the true diff. of level between A & B. What is RL of B, if that of A is 125.55?

Ans :- In 1st setting,

Apparent diff. of level bet. A & B

$= 3.375 - 2.245 = 1.13$ m

(fall from A to B).

In 2nd setting,

Apparent diff. of level between A & B

$= 3.055 - 1.955 = 1.1$ m

True diff. of level $= \frac{1.13 + 1.1}{2} = 1.115$ m.

R.L of B $= 125.55 - 1.115 = 124.435$ m.

Q.2 In levelling across river, two pegs A & B were fixed on opp. banks. The following readings were taken.

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Position of level	Staff reading at	
	A	B
level at A	1.871	1.469
level at B	1.664	0.706

If R.L of A is 50.865, find R.L of point B.

Ans :- 1st setting,

Apparent diff. in elevation = 1.871 - 1.469

= 0.402 (fall from B to A)

2nd setting,

Apparent diff. in elevation = 1.664 - 0.706

= 0.958

True difference in elevation,

$$= \frac{0.402 + 0.958}{2} = 0.68 \text{ m.}$$

The R.L of B = R.L of A + 0.68

= 50.865 + 0.68 = 51.545 m.

Q.3 Reciprocal levels were taken with a dumpy level & following observations were recorded :-

Instrument near station	Staff readings at station	
	A	B
A	1.225	1.375
B	0.85	0.5

R.L of station A is 626.155. Calculate the R.L of station B. Also calculate the error in line of collimation & state clearly whether it is inclined

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Upward or downwards.

Sol:- In 1st setting,

Apparent diff. of level = 1.375 − 1.225

= 0.15 m (Fall from A to B).

In 2nd setting,

Apparent diff. of level = 0.5 − 0.85

= − 0.35 m.

True diff. of level between A & B,

$$\frac{0.15 - 0.35}{2} = -0.1 \text{ m}$$

Negative sign shows A is lower than B.

R.L of B = 626.155 + 0.1 = 626.255 m.

At 2nd setting,

Correct reading on A = Reading on station B + true diff. in level

= 0.5 + 0.1 = 0.6 m

Observed reading on station A = 0.85 m.

Error of collimation = 0.85 − 0.6

= 0.25 m

As observed reading on station A is greater than its true reading, the line of collimation is inclined upwards.

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Q.4. The following records refer to an operation involving reciprocal levelling.

Instrument At	Staff reading on		Remarks
	A	B	
A	1.155	2.595	Distance AB = 500m,
B	0.985	2.415	RL of A = 525.50

Find:-

a) True R.L of B.

b) The combined correction for curvature & refraction.

c) Collimation error.

d) Whether the line of collimation is inclined upwards or downward.

Ans:- a) True difference of level between A & B

$$= \frac{(2.595 - 1.155) + (2.415 - 0.985)}{2}$$

$$= 1.435 \text{ m (fall from A to B)}. \quad \text{——— (1)}$$

R.L of B = 525.5 − 1.435 = 524.065m.

b) Combined correction for 500m = $0.0673 \times 0.5^2 = 0.0168$m (−)

c) Let us assume line of collimation is inclined upward.

Collimation error in 500m = e . (+, as it is inclined upward)

When error is positive, correction is negative.

When instrument at A,

Correct staff reading at A = 1.155m

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Correct staff reading at B = 2.595 − 0.0168 − e

True diff. of level between A & B

= (2.595 − 0.0168 − e) − 1.155

= 1.4232 − e ——— (2)

Equating (1) & (2), we get

1.4232 − e = 1.4350

⇒ e = −0.0118

Collimation error per 100m = $-\frac{0.0118 \times 100}{500}$

= −0.0023m.

d) The collimation error was assumed positive, but the result is negative. So the assumption is wrong. The line of collimation is actually inclined downwards.

Q.5 In testing a dumpy level, the following records were noted while undertaking reciprocal levelling :-

Instrument at	Reading at	
	A	B
A	1.725	1.370
B	1.56	1.235

Is the line of collimation in adjustment? What should be the correct staff readings at A, during second set-up to make the line of collimation truly horizontal? Find amount of collimation error also.

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Ans:- True diff. of level between A & B $= \frac{(1.725 - 1.37) + (1.56 - 1.235)}{2}$

$= 0.34\,m$ (fall from B to A).

In 2nd setting,

Correct reading at B = 1.235m

Correct staff reading at A = 1.235 + 0.34 = 1.575 m.

The observed staff reading (1.56) at A is less than correct staff reading (1.575m).

So line of collimation is inclined downwards.

Amount of collimation error = 1.56 - 1.575

$= -0.015\,m$.

Q.6 The following observations were made during the testing of a dumpy level.

Instrument At	Staff Reading at		Remark
	A	B	
A	1.725	2.245	R.L of A = 450.000 m
B	2.145	3.045	

Distance between A & B = 200m.

Is the instrument in adjustment? To what reading should the line of collimation be adjusted when the instrument is at B? Find the R.L of B.

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Ans :-

True diff. of level $= \frac{(2.245 - 1.725) + (3.045 - 2.145)}{2}$

$= 0.71\,m$

(fall from A to B).

When instrument at B,

Correct staff reading at B = 3.045 m

Correct staff reading at A = 3.045 − 0.710

= 2.335 m.

Observed staff reading at A (2.145 m) is less than the true staff reading (2.335 m), so the line of collimation is inclined downwards.

Collimation error = 2.145 − 2.335 = − 0.19 m.

So, R.L of B = 450 − 0.710 = 449.29 m

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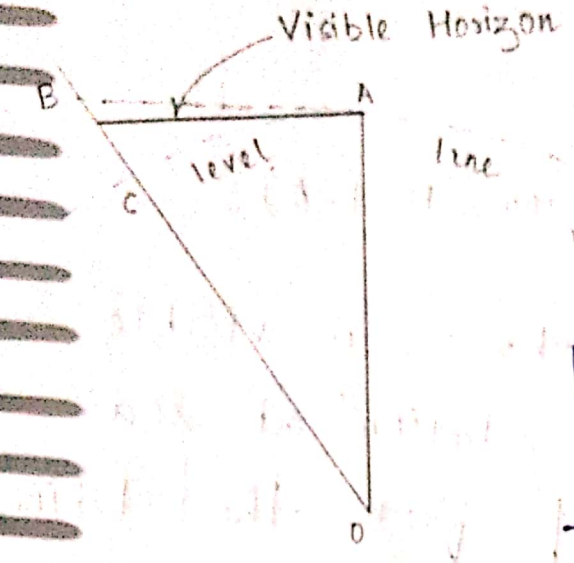
DISTANCE TO THE VISIBLE HORIZON :-



let,

C = staff station.

BC = h, staff reading.

d = distance of visible horizon.

let, the horizon meets the earth surface at A, the point where the level line CA meets the horizon of point B.

$h = 0.0673\, d^2$ m, where d is in km.

$$d = \sqrt{\frac{h}{0.0673}} \text{ km}$$

Questions :-

1. A level is set up at a point 150 m from A & 100 m from B, the observed staff readings at A & B are 2.525 & 1.755 respectively. Find the true difference of level between A & B.

Ans :- Combined Correction to staff reading at A $= 0.0673 \times D^2$

$$= 0.0673 \times \left(\frac{150}{1000}\right)^2$$

$$= 0.0015 \text{ m}$$

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Correct Reading on A = 2.525 − 0.0015

= 2.5235 m.

Combined Correction to staff reading at B

$$= 0.0673 \times \left(\frac{100}{1000}\right)^2$$

= 0.000673 m = 0.0007 m (−ve)

Correct Reading at B = 1.755 − 0.0007 = 1.7543 m

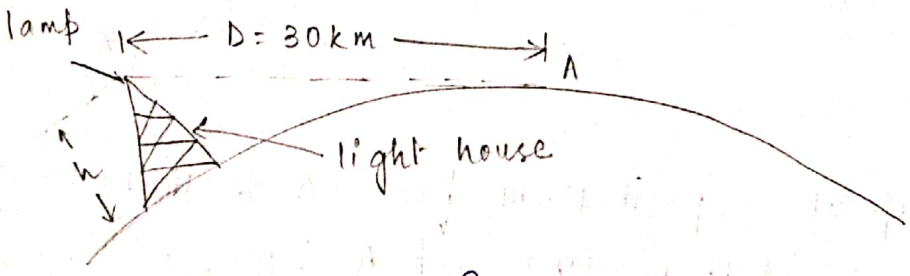
True difference of level between

A & B = 2.5235 − 1.7543

= 0.7692 m (fall from B to A)

2. A lamp at the top of a lighthouse is visible just above the horizon from a station at sea level. The distance of the lamp from the station is 30 km. Find the ht. of lighthouse

Ans :-



$$h = 0.0673 d^2$$

$$= 0.0673 \times 30^2 = 60.57 \text{ m.}$$

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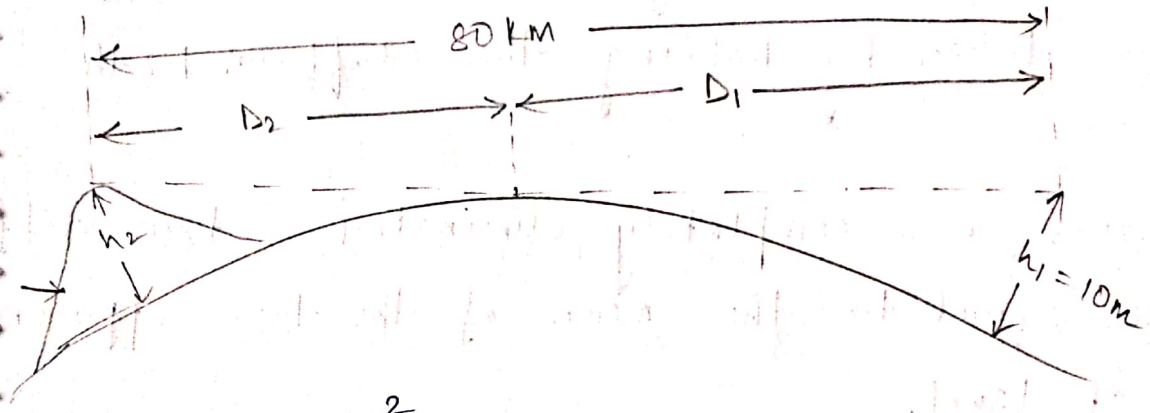
3. A man at a position 10m above sea level observes the peak of a hill. The distance between the man & the hill is 80 km. Find ht. of hill.



$h_1 = 0.0673\, d_1^2$

$\Rightarrow d_1 = 12.19$ km

$d_1 + d_2 = 80$

$\Rightarrow d_2 = 67.81$ km

$h_2 = 0.0673 d_2^2$

$\Rightarrow h_2 = 0.0673 \times (67.81)^2$

$= 309.46$ m.

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= 449.29 m

1.14 DIFFICULTIES IN LEVELING

The following are some of common difficulties faced in levelling :-

1. Levelling in Undulating terrain :-

At the time of levelling in undulating terrain, the level should never be set up on the top of summit or bottoms so shallows. If level is set up at summit large no. of stations will be required & progress of work will be slow

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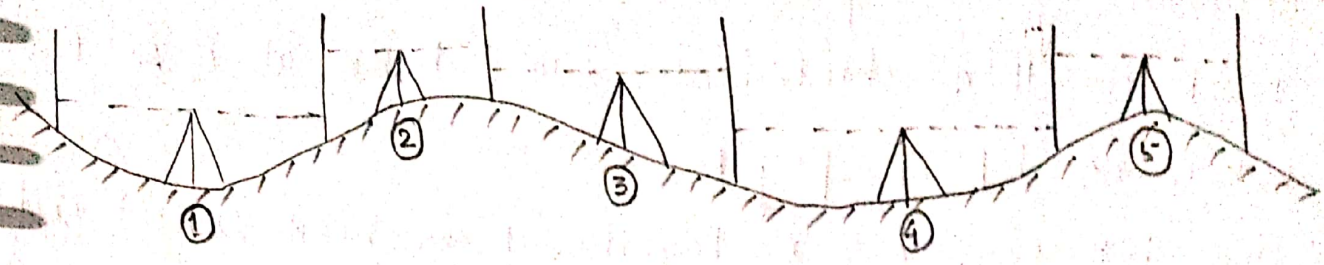
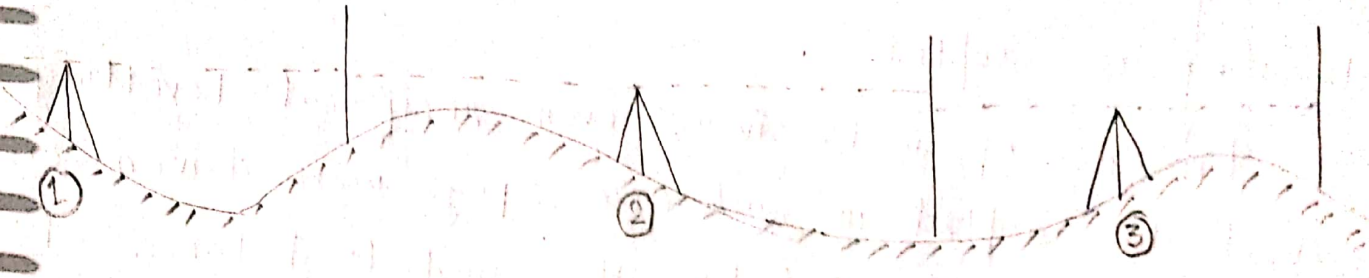


In order to avoid short sights & to equalise fore & back sight- readings, the instrument may first be set up on the slope. In first case the no. of stations are 5 where as in second case the no. of station is reduced to 3, for same length.



2. Levelling Across Lake :

When staff cannot be read due to great distance, the difficulty may be overcome as follows :-



Drive two pegs A & B flushed with the water level at surface on the opposite bank of lake.

As the water surface is level that means have same R.L on all points of water surface. The level of top of pegs A & B is same. R.L of top of peg A

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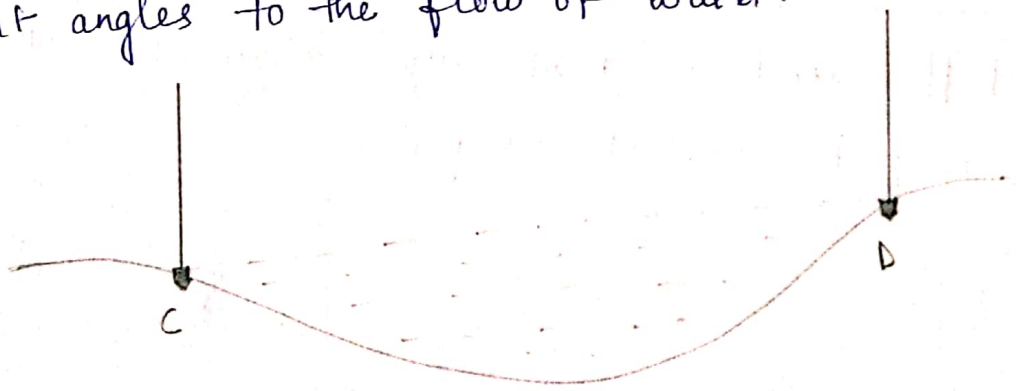
is obtained by taking F.S & instrument station O_1. The level is then shifted to other bank at O_2 & B.S is taken on top of peg B.

The two pegs A & B together have been considered as a single change point.

3. Levelling Across River :-

In case of rivers where water flows in rivers with appreciable velocity, the water level on opp. banks or edges will be different. In such case reciprocal levelling is adopted.

In case of wide rivers even reciprocal levelling cannot be adopted in such case, 2 pegs were driven on opposite banks separated by the shortest distance at right angles to the flow of water.



The RL of C is obtained in usual way. Then the true difference of level between C & D is obtained by reciprocal levelling. Thus the RL of B is calculated & levelling is continued.

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4. Levelling Across Solid Wall:-

While levelling across a solid wall, mark two pegs A & B and insert them on either side of wall, just touching it. The level is set up at O_1 & staff reading is taken on A. Let this reading be AC. The height of wall is measured by staff. Let the ht. be AE. The HI is found by taking BS on any BM or CP.

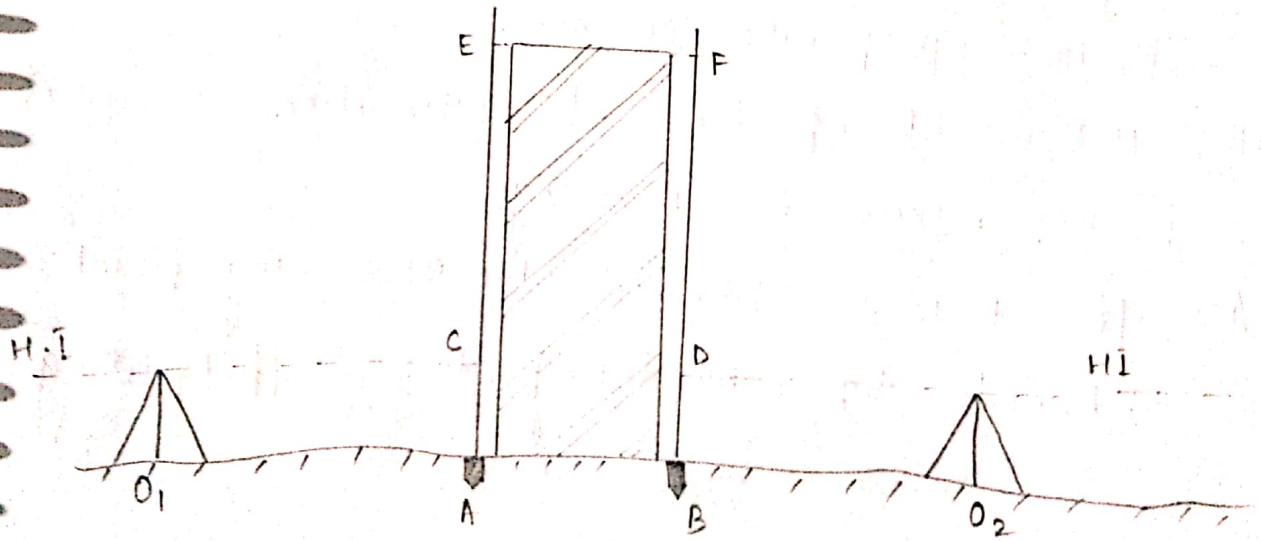
RL of A = HI - AC

RL of E = RL of A + AE = RL of F (same level)

The level is shifted & set up at O_2. The staff readings BD is noted & ht. BF is measured.

RL of B = RL of F - BF

HI at O_2 = RL of B + BD.



5. When a BM is above the line of Collimation:-

This case occur when B.M is on underside of a culvert. It generally remains much above the line of sight of a level set up on the floor.

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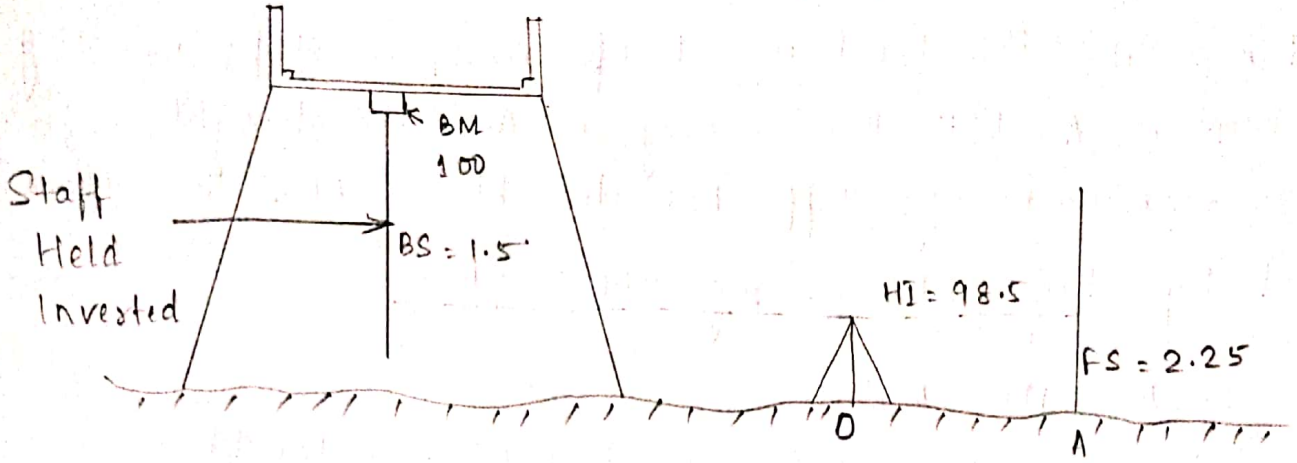
Following steps are followed to overcome such difficulty



Suppose BM exists on the bottom surface of culvert, & it is required to find out RL of A. The level is set up at O & staff is held inverted on BM. The staff reading is taken & noted with a negative sign. The remark "staff held inverted" should be entered in appropriate column.

The H.I or ht. of line of collimation is equal to RL of BM minus B.S reading.

As the H.I is obtained, the RL of any point can be obtained by subtracting the staff reading from H.I.

For example, R.L of BM = 100m

B.S of inverted staff = +1.5m. (-ve in nature)

H.I = R.L of BM − 1.5

= 100 − 1.5 = 98.5 m.

R.L of A = HI − FS = 98.5 − 2.25 = 95.25 m.

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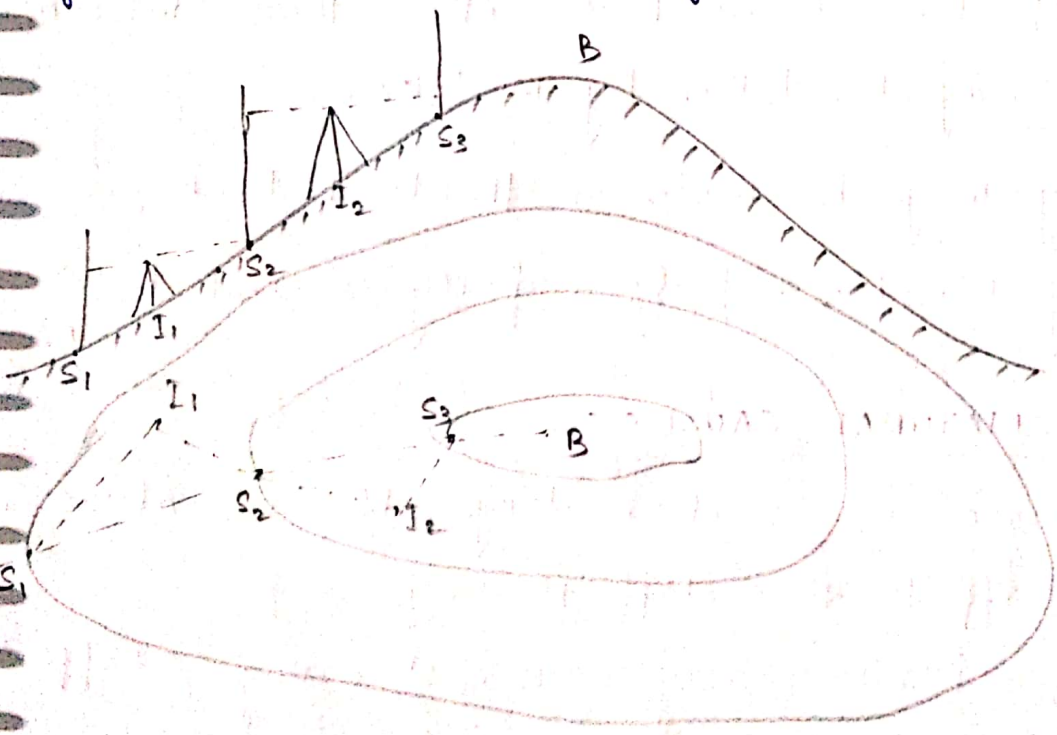
6. Levelling On steep Slope :-

When levelling is done on steep slopes, B.S & F.S may not be equal unless short lengths are adopted. So, it is more convenient to set up the instrument away from the line joining the staff positions so that



BS & FS distances are kept equal.

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7.7 Errors in Levelling

The errors in levelling are of following types:-

1) Personal error
2) Error due to natural causes
3) Instrumental error

1) Personal error:-

- The instrument may not be perfectly levelled.
- The focussing of eye-piece & objective glass may not be perfect & parallax may not be eliminated entirely.
- The position of staff may be displaced at the change point at the time of taking FS & BS.
- The staff may appear inverted when viewed through the telescope. By mistake, the staff readings may be taken upwards instead of downwards.
- The readings of the stadia hair rather than the central collimation hair may be taken by mistake.
- A wrong entry may be made in level book.
- The staff may not be properly & fully extended.

2) Error due to natural causes:-

- When the distance of sight is long, the curvature of earth may affect staff reading.
- The effect of refraction may cause a wrong staff reading.
- The effect of high wind & a shimmering sun may result in a wrong staff reading.

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3) Instrumental errors :-

- The permanent adjustment of the instrument may not be perfect. The line of collimation may not be parallel to axis of bubble tube.
- The internal arrangement of focussing tube is not perfect.
- Graduation of levelling staff may not be perfect.

Precise Levelling

→ It is used for establishing a network of bench marks throughout the country by state agency as Survey of India.

→ Instruments of highest precision are used & all precautions taken to minimize & to eliminate all the sources of errors.

→ The standards for levelling of high precision were fixed by international Geodetic Association.

The specifications of precise levelling are :-

* Probable accidental error $= \pm 1\sqrt{K}$ mm
* Probable systematic error $= \pm 0.2\sqrt{K}$ m.

where K is in Kilometer.

Instruments :-

Levels manufactured by Carl Zeiss are Wild Heerbrug are mostly used for precise levelling. Invar precision levelling staves are used in place of ordinary levelling staves. With the parallel plate micrometers, reading upto $\frac{1}{1000}$ of a metre can be taken accurately.

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Precautions

a) Adjustment of the instrument should be carefully tested & perfectly done.

b) A constant watch is made to ensure that the bubble remains central while observations are made.

c) The level is protected from the sun, wind and rain by umbrella.

d) Levelling staff is provided with a level to check its verticality.

e) The level is set up on hard & firm ground exactly mid way between the forward staff & backward staff.

f) Lengths of sights are kept within 50m.

g) Change points are taken on a steel plate firmly fixed.

h) The fore sight & back sights are taken in quick succession to eliminate the chances of any settlement of instrument or staff.

i) The levelling is carried out either in the morning or late evening, to avoid errors due to refraction.

j) Check levelling is done in opposite directions by 2 surveyors on different days & with diff. change points.

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Permanent adjustment of different types of level

All instruments are properly adjusted by manufacturers. Their fundamental lines get disturbed due to mishandling of instrument in field.

The main objective of permanent adjustment is to fix the relationship between fundamental lines of level.

Various fundamental lines of level :-

- Axis of bubble tube
- Vertical axis
- Axis of telescope
- line of collimation

Relationship of fundamental lines :-

→ The line of collimation should be parallel to the axis of bubble tube. It is applicable to every type of levelling instruments.

→ The axis of telescope & line of collimation should coincide.

→ The axis of bubble tube should be perpendicular to the vertical axis of level.

Adjustment of a Dumpy Level is of following steps-

i) To make the axis of bubble tube Lee to vertical axis of level.

ii) To make line of collimation parallel to the axis of bubble tube.

Adjustment (i) (to make axis of bubble tube Lee to vertical axis)

(a) Set up the level on firm ground & level it. The bubble need remains central only in a position

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3rd foot screws.

b) Turn the telescope through 180° in azimuth so that the ends of the telescope are reversed.

c) If the bubble still remains central, the instrument is in perfect adjustment.

Adjustment

a) If the bubble does not remain central, note down the deviation of centre of bubble

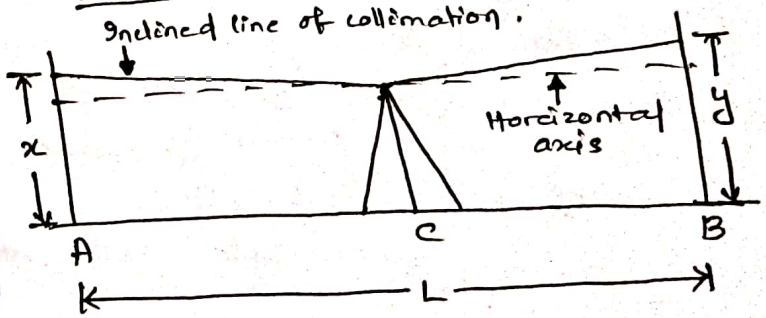
b) Bring the bubble half way back by means of the Capstan nut & the other half with the foot screw beneath the telescope.

c) Turn the telescope through 90° so that it becomes parallel to the pair of the foot screws. Bring the bubble central by means of this pair of foot screws.

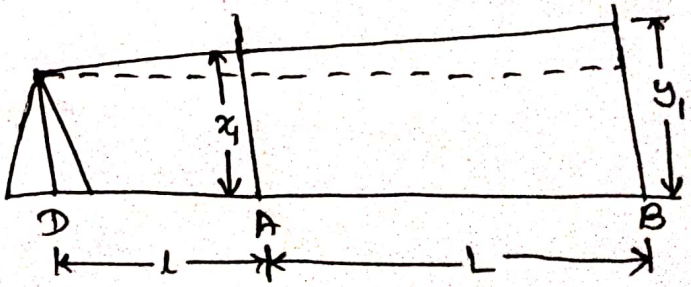
d) Rotate the telescope & observe if bubble remains central. If not, repeat the whole process until the adjustment is correct.

Adjustment (ii) (to make line of collimation ||el to axis of bubble tube.

a) Fix two pegs A & B at a distance L on a fairly level ground.



b) Set up the level exactly at the mid point C of AB. Take the staff readings on A & B, keeping the bubble central of its run. Let staff readings be x & y respectively.



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c) Shift the level to a point D at a distance l from A on the line BA produced,

d) Read the staff readings on A & B. Let the staff readings be x_1 & y_1 respectively.

e) Calculate true difference of level by subtracting the reading x from reading y when the level was at C.

f) Calculate the apparent difference of level by substracting the reading x_1 from y_1.

If two diff. are equal, instrument is in adjustment.

g) If not, find whether there is a rise or fall from peg A to B. If x is greater than y, B is highere than A & vice versa.

h) Calculate the reading y_2 on peg B at the same level as of x_1 i.e.

Reading on peg B = Reading on A ± true diff. (+ for fall & − for rise)

$y_2 = x_1 \pm$ true diff.

i) If y_1 is greater than y_2, line of collimation is inclined upwards. If y_1 is ~~no~~ smaller than y_2, line of collimation is inclined downwards.

j) Calculate; net collimation error on distance L

i.e., Collimation error on distance L

$= y_1 - y_2$

∴ Collimation error per unit distance

$= \frac{y_1 - y_2}{L}$.

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k) Calculate the required correction for the readings on pegs A & B i.e.

Collimation correction to reading on peg A

$$= \frac{y_1 - y_2}{L}.$$

Collimation correction to reading on peg B

$$= \frac{l+L}{L}(y_1 - y_2).$$

2) Add the collimation correction to the staff readings if the line of sight is inclined downwards & substract it, in case it is inclined upwards.

m) Loosen the capstan headed screw & shift the Horizontal read the correct reading of B. Check the reading of A whether it agrees with calculated reading. Repeat the adjustment, if found necessary.

Adjustment of Y-level

As, the telescope of Wye-level can be lifted from the collars during adjustment, one more geometrical condition is to be fulfilled in addition to those required for a dumpy level.

The fundamental relationship between diff axes to be established are:—

i) The line of collimation should coincide with the axis of collars. It is the manufacturer's responsibility to ensure that the axis of the collars also coincides with optical axis.

ii) Line of collimation of telescope should be parallel to the axis of bubble tube.

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iii) Axis of bubble tube & vertical axis of level should be mutually perpendicular.

Adjustment 1 (Adjustment of line of collimation)

Aim: To make the line of collimation coincide with the axis of collares.

Necessity: This adjustment is very important. The telescope should be clipped in one position. If not, the rotation of line of collimation about its axis would generate a cone. The fundamental requirement of parallelism between the axis of bubble tube & the line of collimation is obtained only for a particular position of telescope by collares.

Test & Adjustment :-

i) Set up the level firmly on the ground.

ii) Sight a definite small mark on a wall at a distance approximately 50-100 m.

iii) Fix both the clamps.

iv) Loosen the clips & rotate the telescope through 180° about its longitudinal axis.

v) Sight the point again, if the line of sight still strikes the same point, the instrument is in perfect adjustment. If not, it requires adjustment.

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vi) By means of the four capstan screws, make the intersection of hairs half-way towards the point.

vii) Repeat the test & adjustment until correct.

Principle of the test & Adjustment :-

Let, the line of collimation be inclined by e'' upward. The mark sighted will be at P_1. After giving the 180° rotation, the line of collimation inclined upwards will now be inclined downwards by the same amount & the point sighted by the line sight of will be at P_2.

The apparent error is therefore twice the actual. Hence, the necessary correction to be applied is only half the apparent error.

Adjustment - 2 (Adjustment of level tube)

Aim :- To make the axis of level tube parallel to the line of collimation.

The main function of a level is to furnish a horizontal line of collimation. It is obtained only if the axis of the bubble is parallel to the line of collimation, when the bubble of the level tube is at the centre of its run.

Test & Adjustment :-

Step-1 : To bring the axis of bubble tube in the plane of collimation.

i) Level up the instrument carefully.

ii) Rotate the telescope in the collares through a small angle. If the bubble remains central, the adjustment is correct.

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iii) If not, bring the bubble central by adjusting the screw & controlling the level tube carefully.
iv) Repeat the test & adjustment till the required condition is obtained.

Step-2: To make the axis of the bubble parallel to the line of collimation.
i) Set the telescope parallel to any pair of levelling screws.
ii) Open the collars & level the instrument carefully.
iii) Lift the telescope from the collars & replace it end to end.
iv) If the level remains central, the adjustment is correct.
v) If not, move the bubble half-way towards the centre by screw provided at one end of the level tube.
vi) Repeat the test till the required condition is achieved.

Adjustment-3 (Adjustment of perpendicularity of a vertical axis & axis of the level tube)

Aim: To ensure that the instrument once levelled, the bubble remains central for all positions of the telescope.

Necessity: The adjustment is not very important. It is carried out only for the surveyor's convinience in using the level as under:—

i) Set up the instrument on firm ground.
ii) Bring the level tube parallel to a pair of levelling screws.

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iii) Turning both the screws at same time, bring the bubble central.
iv) Rotate the telescope through 90°. With the help of the 3rd levelling screw, bring the bubble central.
v) Rotate the telescope through 180°. If the bubble remains central, the adjustment is correct.
vi) If not, bring the bubble half-way back by the capstan screw.
vii) Level up & repeat the test & adjustment till the required condition is obtained.

Adjustment of a Tilting Level

In tilting levels only one adjustment is of prime importance i.e. the line of collimation is horizontal when the bubble is central at its run.

Test & Adjustment:—

i) Calculate the correct staff reading.
ii) Raise or lower the line of sight with the help of tilting screw to read the calculated reading.
iii) Now, the bubble ~~cept~~ becomes out of center.
iv) Bring the bubble central with the help of the capstan screw provided at one end of the level tube.
v) Repeat the test & adjustment till the instrument is perfectly adjusted.

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CONTOURING

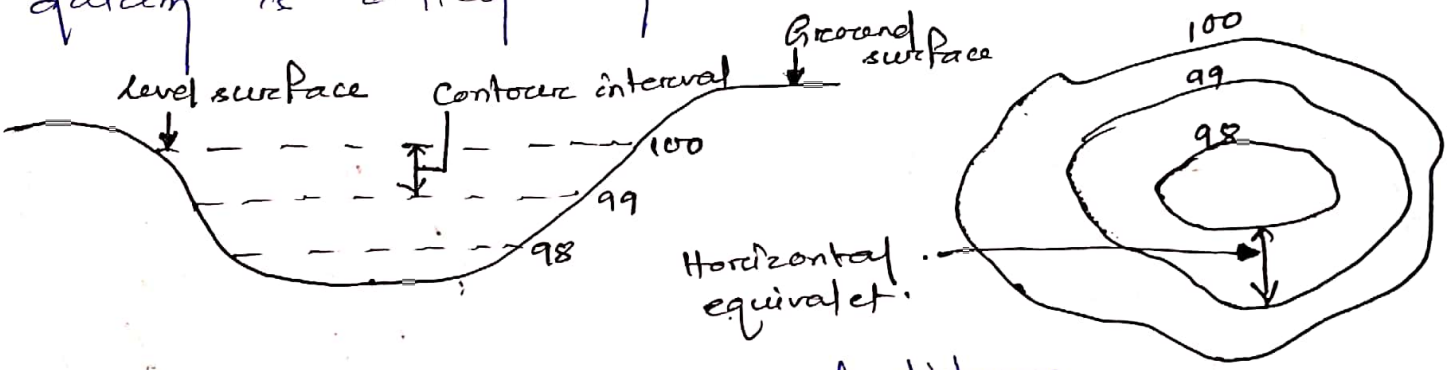
7.8 Definition, Concept & Characteristics

→ The line of intersection of a level surface with the ground surface is known as contour line or contour.

→ The line passing through points of equal reduced level.

→ A map showing only contour lines of an area is called a contour map.

→ An imaginary line, on the ground, joining the points of equal elevation above the assumed datum is called contour.



Contour Interval:- The vertical distance between any two consecutive contours, is called contour interval.

Horizontal Equivalent:- The least horizontal distance between 2 consecutive contours is called horizontal equivalent.

It is diff. for diff. contours & is dependent on the slope of ground surface. It is comparatively less in hilly areas than in plains.

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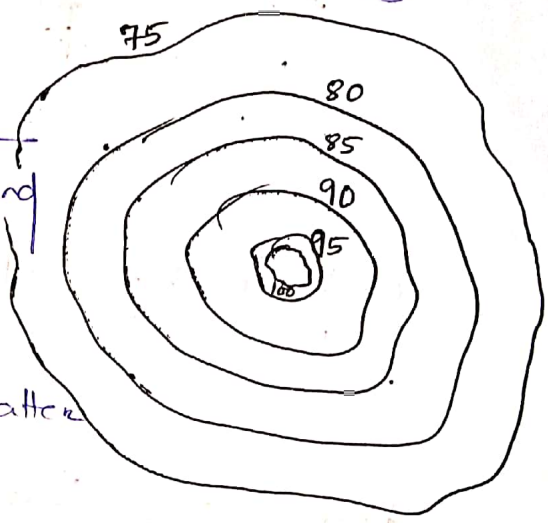
Objective of preparing contour map :- For all engg. projects involving roads, railways etc. a knowledge of nature of ground surface is required for locating alignments & estimating vol^{m} of earth work.

Uses of Contour map :-

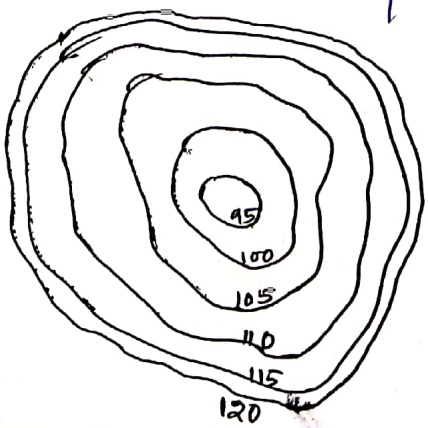
1) The nature of ground surface of a country can be understand by studying a contour maps.
2) A suitable site can be selected for any engg. project.
3) The capacity of a reservoir or area of catchment can be approximately computed.
4) A suitable route for a given gradient can be marked on map.
5) Quantities of earthwork can be approximately computed.

Characteristics of Contours

1) The contour lines are closure near top of a hill or high ground & wide apart near the foot. This indicates a very steep slope towards the peak & flatter slope towards the foot.



2) The contour lines are closer near the bank of a pond or depression & wide apart towards the centre. This indicates a steep slope near the bank & a flatter slope at the centre.



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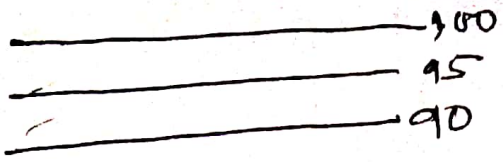
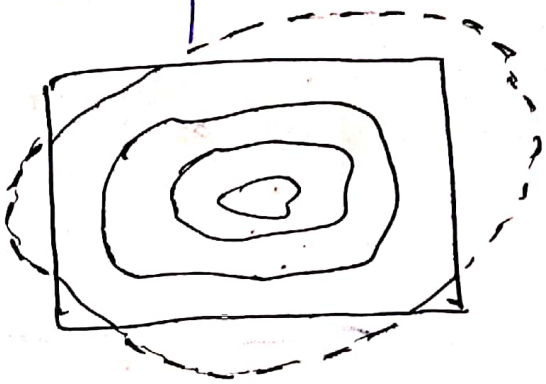
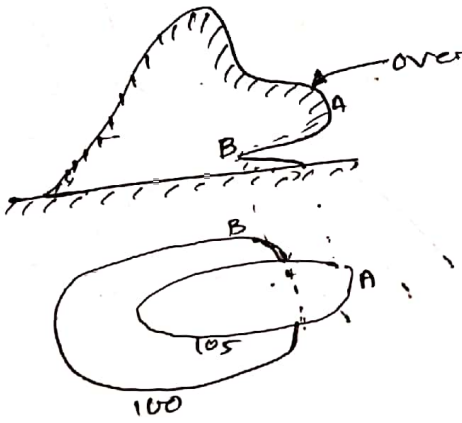
3) Uniformly spaced contour lines indicate a uniform slope.



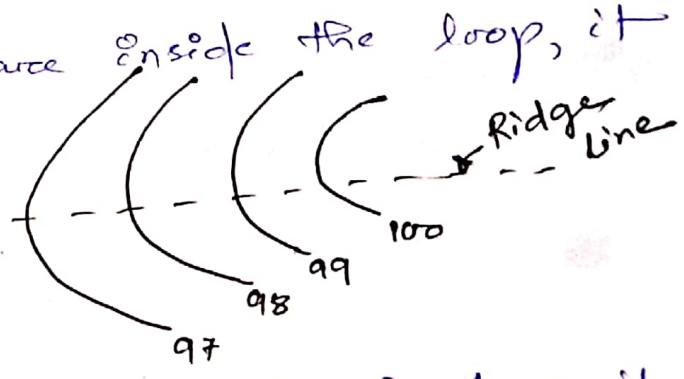
4) Contour lines always form a closed circuit. But these lines may be within or outside the limits of the map.



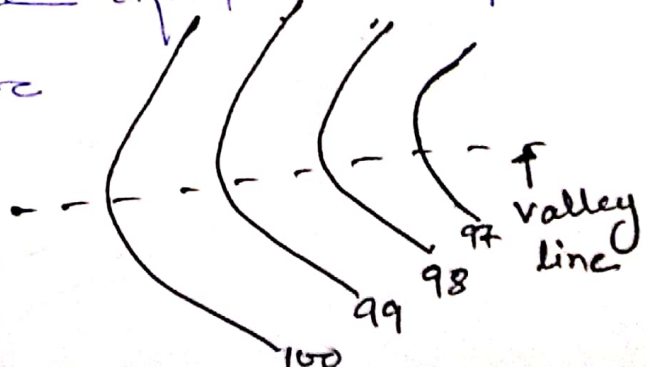
5) Contour lines cannot cross one another, except in the case of an overhanging cliff. But the over-lapping portion must be shown within or outside the limits of the map.



6) When the higher values are inside the loop, it indicates a ridge line. Contour lines cross ridge lines at right angle.



7) When the lower values are inside the loop, it indicates a valley line. Contour lines cross valley line at right angles.



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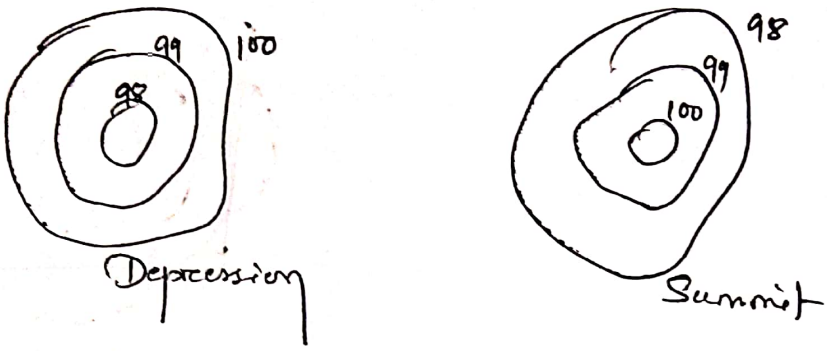
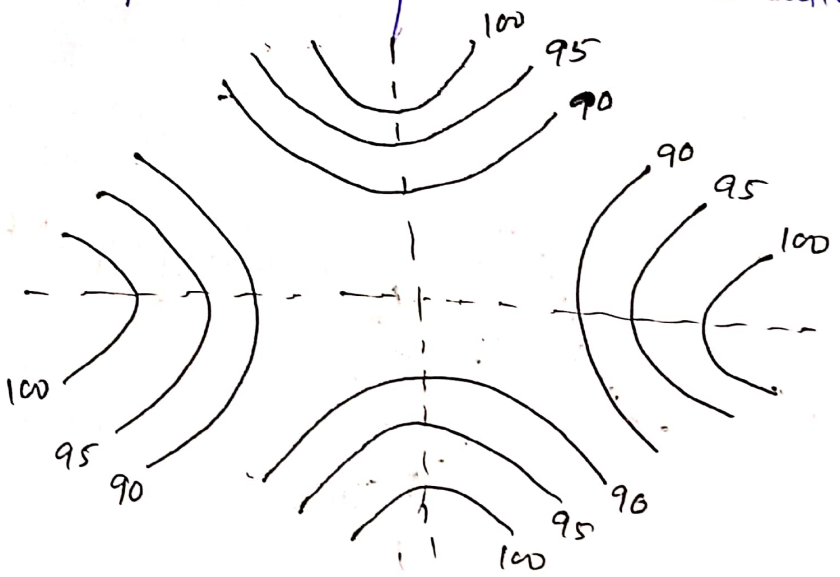
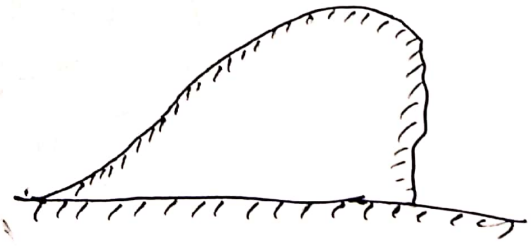
8) A series of closed contours always indicate a depression or summit. The lower values being inside the loop indicates a depression & higher values inside the loop indicates a summit.

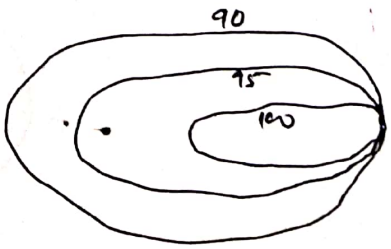


9) Depressions between summits are called saddles.



10) Contour lines meeting at a point indicates a vertical cliff





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9.1 Methods of contouring

There are two methods of contouring :-

(1) Direct Method (2) Indirect Method

(1) Direct Method

There are 2 cases in direct method.

Case-I When the area is oblong & cannot be controlled from a single station :-

The various points on any contour are located on the ground by taking levels. These points are marked by pegs. The points are plotted on the map, to any suitable scale, by plane table. This method is very slow & tedious. It gives accurate contour lines.

Procedure :-

1) A temporary bench mark is set up near the site by taking fly level readings from a permanent bench mark.

2) The level is then set up at a suitable position L from where maximum area can be covered.

3) The plane table is set up at a suitable station P from where the above area can be plotted.

4) A backsight reading is taken on the TBM. Suppose the RL of the TBM is 249.50 m & the BS reading is 2.25 m. Then the RL of HI is 251.75 m. If a contour of 250.00 m is required, the staff reading should be 1.75 m. If a contour of 249.00 m is required, the staff reading should be 2.75 m & so on.

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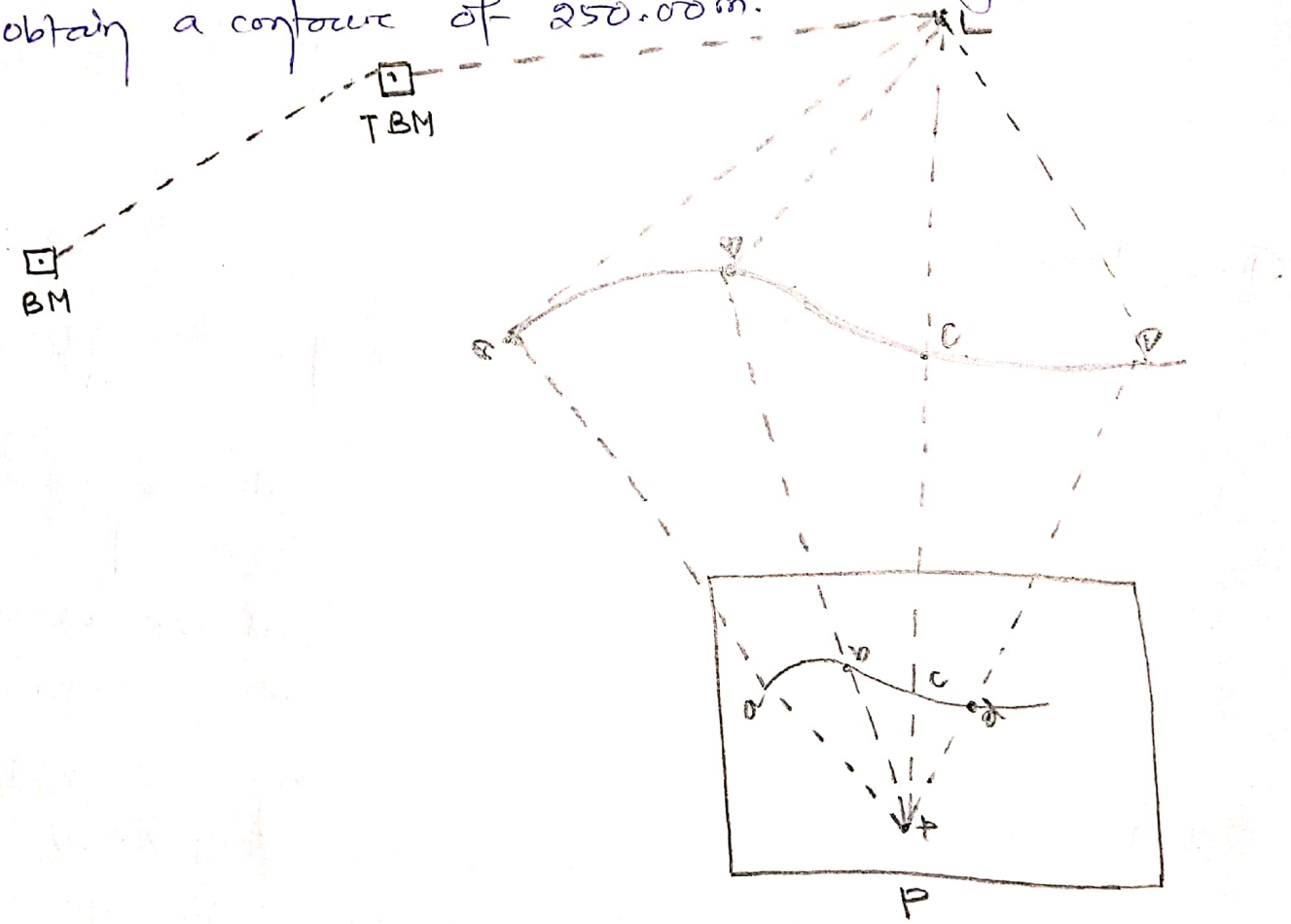
5) The staffman holds the staff at different points of the area by moving up & down, on left & right, until the staff reading is exactly 1.75. Then the points are marked by pegs. Suppose these points are A, B, C, D - - - - .

6) A suitable point P is selected on the sheet to represent the station P. Then, with the alidade touching P, rays are drawn to A, B, C, & D. The distances PA, PB, PC & PD are measured & plotted to a suitable scale. In this manner the points a, b, c & d of the contour line of RL 250.00 m are obtained. These points are joined by line to obtain a contour of 250.00 m.



7) Similarly, points of other contours are located.

8) When required, the levelling instrument & plane table are shifted & set up in a new position in order to continue the operation along the oblong area.

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Part II: when the area is small & can be controlled from single station.

In this case, the method of radial lines is adopted to obtain contours map. It is very slow & tedious, but gives the actual contour lines.

Procedure:

1) The plane table is set up at a suitable station P from where the whole area can be commanded.
2) A point P is suitably selected on the sheet to represent the station P. Radial lines are drawn in different directions.
3) A temporary bench-mark is established near the site. The level is set-up at a suitable position L & BS reading is taken on the TBM. Let the HI in this setting be 153.25 m. So, to find the contour of RL 152 m a staff reading of 1.25m is required at a particular point, so that the RL of contour of that point comes to 152.00 m,

$$RL = HI - \text{staff reading}$$
$$= 153.25 - 1.25 = 152.00 \text{ m}.$$

4) The staff man holds the staff along the rays drawn from the plane table station in such a way that the staff reading on that point is exactly 1.25. In this manner, points A, B, C, D & E are located on the ground, where the staff readings are exactly 1.25.
5) The distances PA, PB, PC, PD & PE are measured & plotted to any suitable scale. Thus the points a, b, c, d & e are obtained which are joined in order to obtain a contour of 152.00

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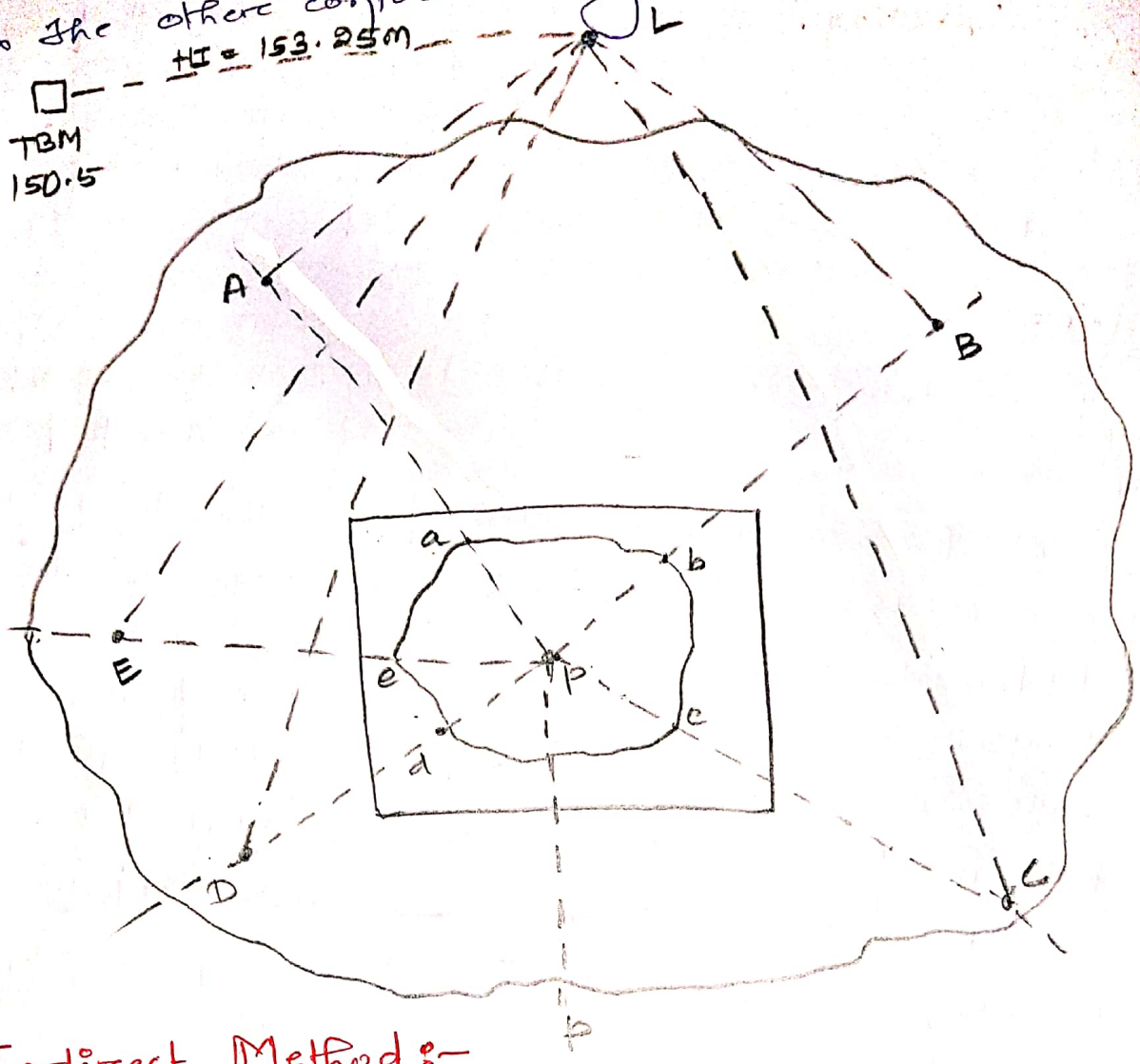
8) The other contours may be located in similar fashion



2) Indirect Method :-

The RL of diff. points are taken at regular intervals along a series of lines set up on the ground. The position of these points are plotted on a sheet to any suitable scale. The levels are noted at respective points.

The points of contour lines are found out by inter-polation, after which they are joined to get the required contour lines.

It is quick but the method gives only the approx. position of contour lines.

This method can be done in 2 ways :—

i) Cross-sections

ii) Squares.

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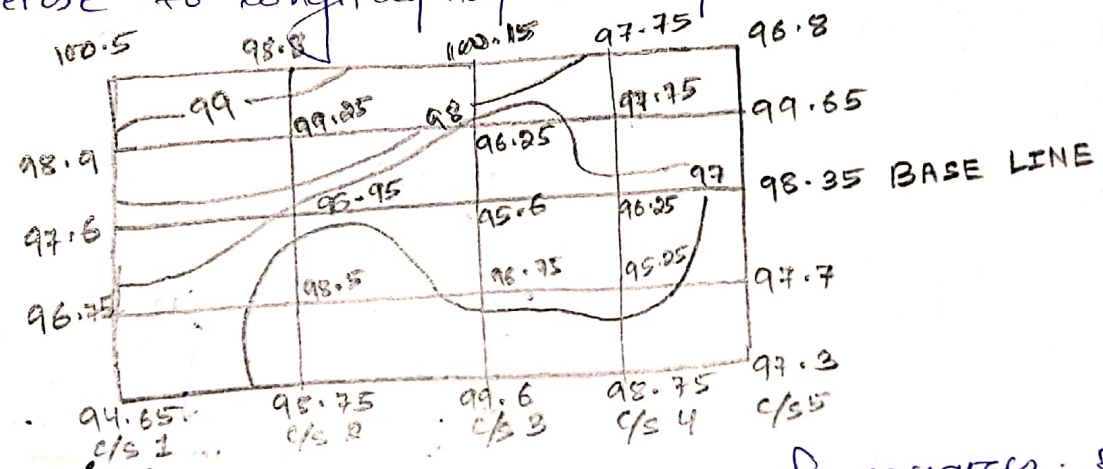
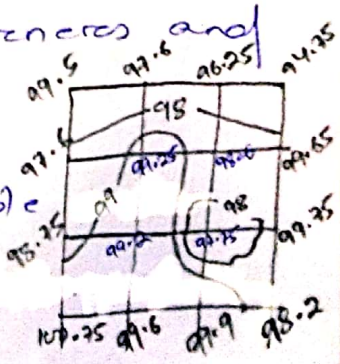
Cross-Sections

→ In this method a base line, centre line or profile line is considered. Cross-sections are taken perpendicular to this line at regular interval.
→ Points are marked along the cross-sections at a regular intervals.
→ A temporary BM is set-up near site.
→ Staff readings are taken along base lines & c/s.
→ R.L. of each point is calculated.
→ The required contour line is drawn by interpolation.
→ It is useful for route survey, where c/s are taken transverse to longitudinal sections.



ii) Squares:

→ The whole area is divided into no. of squares. The size of square depends on nature & extend of the ground.
→ The corners are numbered serially.
→ A temporary BM is set up near site & level is set up at a suitable position.
→ Staff readings are taken on the corners and RL of these points are computed.
→ The square is plotted to a suitable scale
→ The contour lines are plotted by interpolation



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7.9.2 Plotting of Contours maps

The process of locating the contours proportional between the plotted points is termed as interpolation.

It is done in following 2 methods :—

a) Arithemetical Calculation.

b) Graphical method.

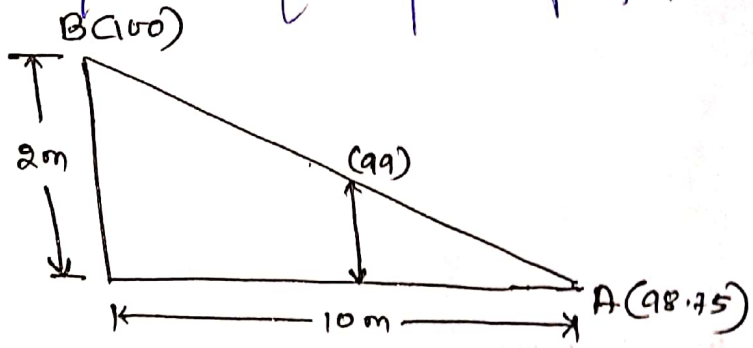
a) Arithemetical Calculation :—

Suppose A & B are two points having RL 98.75m & 100.75m respectively. Horizontal distance between A & B is 10m.

Vertical diff. between A & B = 100.75 − 98.75 = 2m.

If a contour of 99.00m is required to plot.

Difference of level between A & 99m contour is = 99 − 98.75 = 0.25m



Horizontal distance of 99m contour from A = $\frac{10}{2} \times 0.25 = 1.25$m.

b) Graphical Method :-

→ In a paper, a line AB is drawn & divided into equal parts. AB is bisected at C.

→ A point O is selected on the perpendicular.

→ The radial lines are drawn from O to 4 divisions on AB.

→ These parallel lines are drawn parallel to AB namely 1-1, 2-2 & 3-3. ....

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boundary lines & every fifth one is marked thick.

Contour Gradient :-

An imaginary one on the surface of earth having constant slope or inclination with the horizontal is known as contour gradient.

The inclination of contour gradient is either as rising gradient are falling gradient.

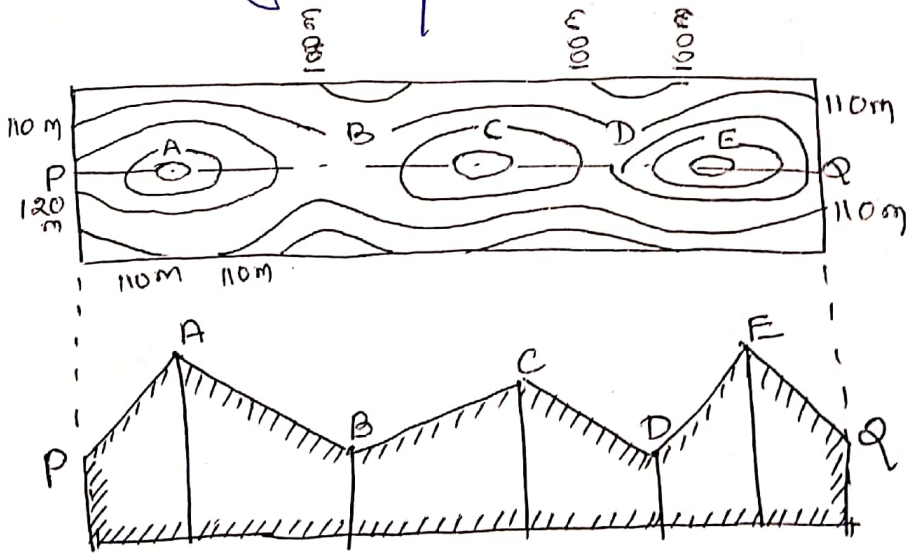
7.9.3 Toposheet :-

It is a map which provides, information of all main topographical features with symbols & with various scales of maps according to users requirement.

7.10 Use of Contour maps

1) Drawing cross-sections from contour maps

The sections of a contour map help to study the general shape of ground. Such sections are useful for calculating the earth work for road, railways or irrigation structures.



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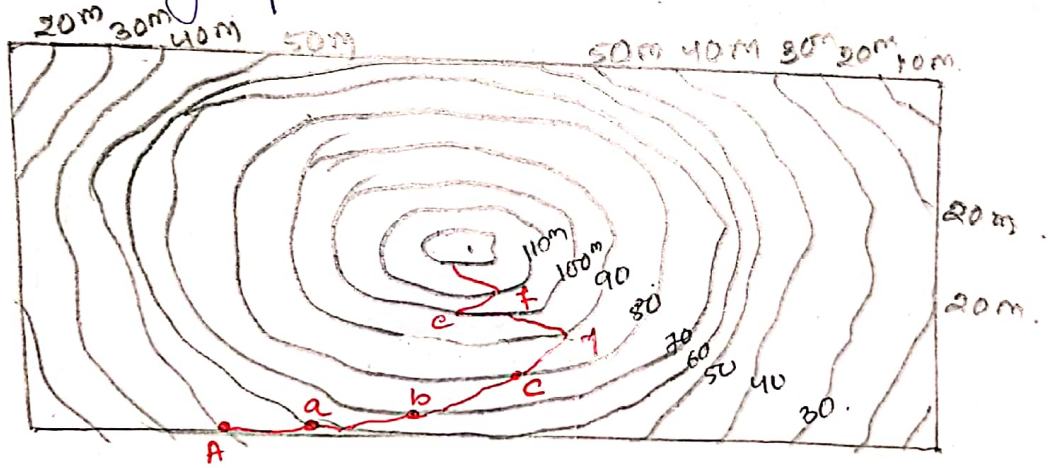
From the given contoured map, a cross section is to be drawn along the line PQ.

→ The points of intersection of contoured lines & line PQ are represented in x-axis & corresponding height are plotted along y-axis.

→ The top of these perpendiculars which represent the contoured height are joined by straight lines, to give an idea about ground surface.

2) Locating proposal routes of roads/railways

On a contoured map, the alignment of a proposed road or railway may be decided & located by drawing a contoured gradient.



Let, the contour interval of supplied contour map, be 10m & upward gradient 1 in 30 is required to be drawn from A to B.

Ground horizontal equivalent

= contoured interval × gradient = 10 × 30
= 300m.

With A as centre & a radius representing 300m on the scale of map, draw an arc to cut the 60m contour at a. Similarly with a as centre

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with the same radius, draw another arc to cut 90m contour at b. By a similar manner points c and d are obtained. When an arc with d, as centre e is drawn, it cuts 100m contour at ee'.

Point e' though in the continuance of the route, is rejected because d & e' when joined cuts the 100m contour again & thus deep cutting is involved.

At e, the arc of the same radius if drawn cuts 110m contour nowhere. To overcome this difficulty, the 105m contour is interpolated & half the radius is used to obtain the point f. Location of a no. of points are made till the last point is reached.

During the process of drawing arcs, it may be noted that it cuts next. Where contours at 2 points. Only the point suitable for the proposed alignment, considering general layout & geographical features, is accepted & other points are ignored.

During construction, the route of the road is made to follow the contour gradient line so obtained as closely as possible.

3) Selection of a canal alignment.

Suppose a reservoir is to be constructed at R across a river. An irrigating canal RL is required to be constructed to irrigate the areas of 2 villages Rampur & Shyampur located on opposite slope of the ridge.

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→ We know that waters in canals flow under gravity. From the contours map it may be noted that the elevation of the reservoir R is 547m approx.

→ The canal alignment should therefore be kept below this level.

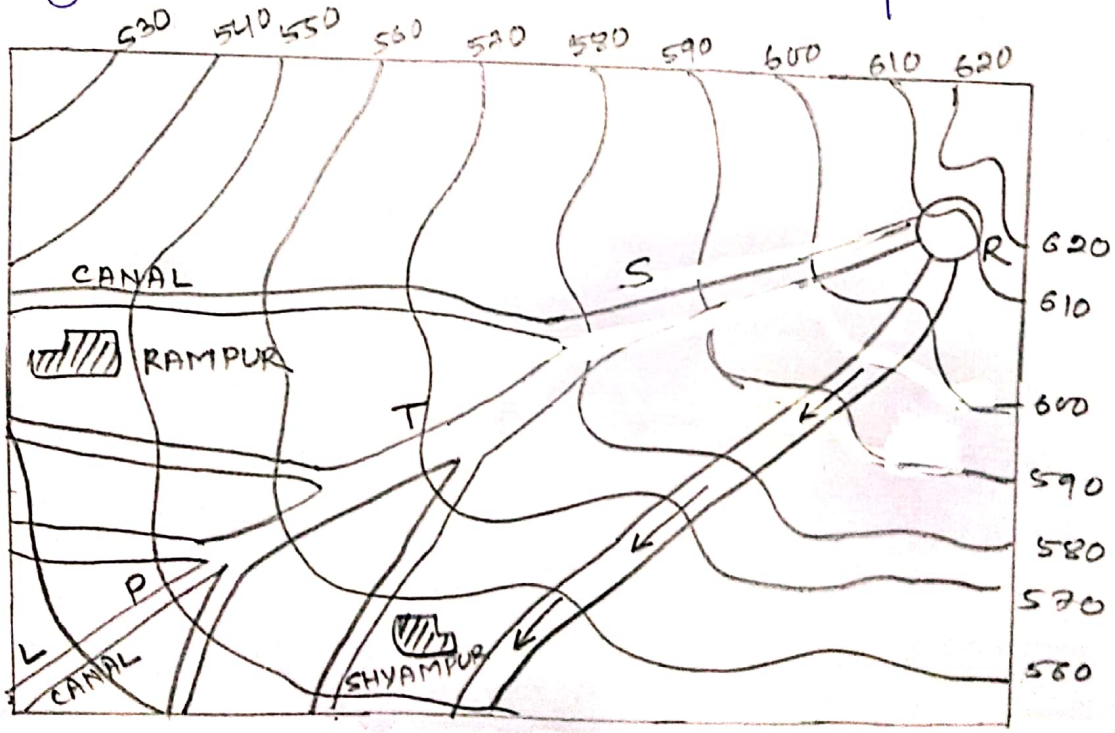
→ The watershed or ridge line is first marked by a dotted line. From R, the canal is taken along RS where S is a point on watershed & its height is approx. 542m.

→ Points may be chosen by considering other factors to avoid construction of aqueducts over intercepting drainage enroute.

→ From S onboard canal may be aligned along the watershed line S-T-P-L ensuring proper gradient.

→ Branch canals may then be constructed on either slope so that areas of both villages may be irrigated.

It may be noted that in absence of contour map, layout of canal cannot be decided economically.



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Volume of earthwork

With the help of contour map, earthwork calculation, capacity of reservoir can be found by computing volume.

In case of reservoir, max. water level of dam & contour interval are obtained. The capacity of reservoir may be obtained by using:—

1) Trapezoidal formula
2) Prismoidal formula.

Trapezoidal formula :-

$$\text{Volume } (V) = h\left[\frac{A_1 + A_n}{2} + A_2 + A_3 + \dots + A_{n-1}\right]$$

Prismoidal formula :-

$$V = \frac{h}{3}\left[A_1 + A_2 + 4(A_2 + A_4 + \dots) + 2(A_3 + A_5 + \dots)\right]$$

where,

A_1, A_2, A_3 = areas enclosed between successive contours

h = vertical contour interval.

(Problems ---).

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8.0 Computation Of Area

8.1 Determination of areas

Area is defined as the area of tract of land as projected upon a horizontal plane & not the actual area of the surface of the land.

Units of area: m^2, hectares (1 hectare = 10,000 m^2), Sq. ft, Acres (1 acre = 43.560 sq ft).

Depending upon the topography of the terrain & required accuracy, the area of land portion, may be determined by following methods:

(i) From the field notes

(ii) From plotted plan or maps.

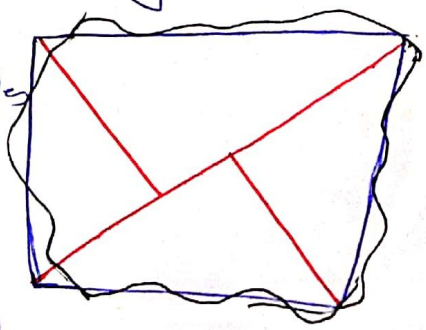
1) Computation of area from Plans or map

The area may be calculated in two ways:—

Case-1 - Considering the entire area:-

The entire area is divided into regions of a convenient shape and calculated as follows:—

(a) **By dividing areas into triangles:—** The triangles are so drawn as to equalise the irregular boundary line.



Then the bases and altitudes of the triangles are determined according to the scale to which the plan was drawn. After this, the areas of these triangles

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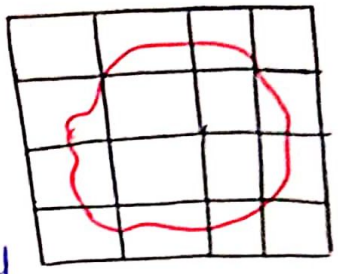
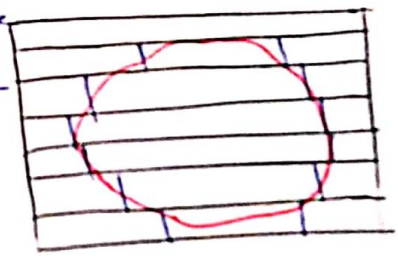
are calculated (area = ½ × base × altitude). The areas are then added to obtain the total area.

(b) By dividing the area into squares :- In this method, squares of equal size are ruled out on a piece of tracing paper. Each square represents a unit area, which could be $1 cm^2$ or $1 m^2$. The tracing paper is placed over the plan and number of full squares are counted. The total area is then calculated by multiplying the number of squares by the unit area



(c) By drawing parallel lines & Converting them to Rectangles :- In this method, a series of equidistant parallel lines are drawn on a tracing paper. The constant distance represents a metre or centimeter. The tracing paper is placed over the plan in such a way that the area is enclosed between the two parallel lines at the top and bottom. Thus the area is divided into a no. of strips. The curved ends of the strips are replaced by perpendicular lines (by give & take principle) & a no. of rectangles are formed. The sum of the lengths of the rectangles is then calculated.



Then, required area = Σ Length of rectangles × constant distance.

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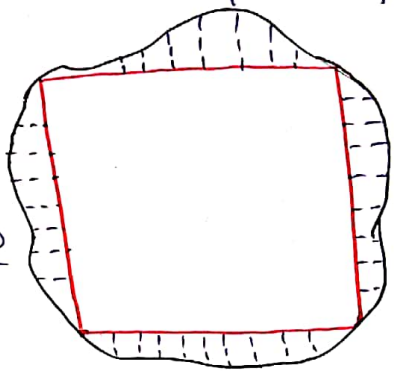
Case-II

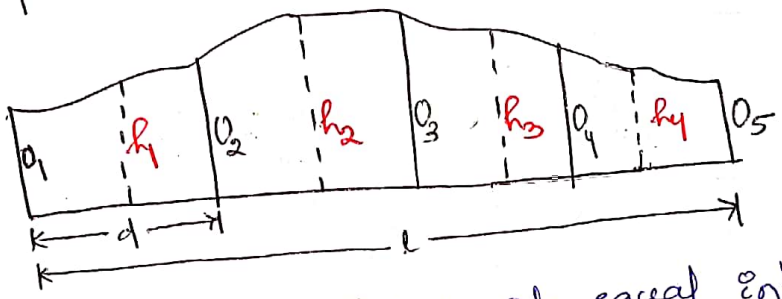
On this method, a large square or rectangle is formed within the area of the plan. Then ordinates are drawn at regular intervals from the side of the square to the curved boundary. The middle area is calculated in the usual way. The boundary area is calculated according to one of the following rules:—



1) The mid-ordinate rule
2) The average-ordinate rule
3) The trapezoidal rule
4) Simpson's rule.

8.2 Calculation of area by using ordinate rule, trapezoidal rule, Simpson's rule.

1) Mid Ordinate rule



$O_1, O_2, O_3, \ldots O_n$ = ordinates at equal intervals.

l = length of base line

d = common distance between ordinates

$h_1, h_2, h_3, \ldots h_n$ = mid-ordinates.

Area of plot $= h_1 \times d + h_2 \times d + \ldots + h_n \times d$

$= d(h_1 + h_2 + h_3 + \ldots + h_n)$

Area = Common distance × Sum of mid-ordinates

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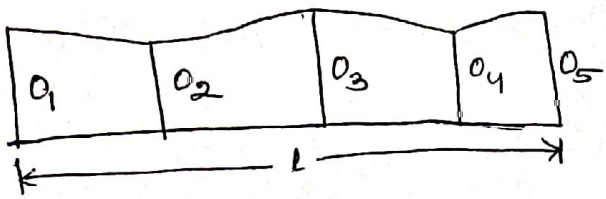
2) **Average Ordinate rule :—**

Let, $O_1, O_2, O_3 \cdots O_n$ = ordinates or offsets at regular intervals

l = length of base line

n = number of divisions.

$n+1$ = number of ordinates.

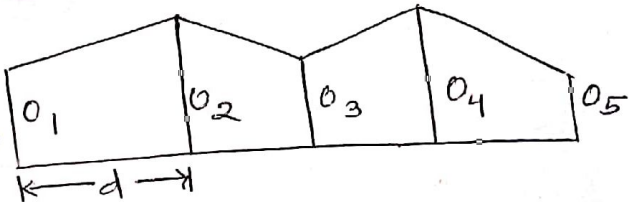


$$\text{Area} = \frac{O_1 + O_2 + \cdots + O_{n+1}}{n+1} \times l.$$

$$= \frac{\text{Sum of ordinates}}{\text{no. of ordinates}} \times \text{length of base line.}$$

3) **Trapezoidal rule :—**

While applying the trapezoidal rule, boundaries between the ends of ordinates are assumed to be straight.



Thus the areas enclosed between the base line & irregular boundary line are considered as trapezoids.

$O_1, O_2, O_3 \cdots O_n$ = ordinates at equal intervals

d = common distance

$$\text{First area} = \frac{O_1 + O_2}{2} \times d$$

$$2^{nd} \text{ area} = \frac{O_2 + O_3}{2} \times d$$

$$\cdots\cdots\cdots$$

$$\text{Last area} = \frac{O_{n-1} + O_n}{2} \times d$$

$$\text{Total area} = \frac{d}{2}(O_1 + 2O_2 + \cdots + 2O_{n-1} + O_n)$$

$$= \frac{\text{common distance}}{2}[\text{1st ordinate} + \text{last ordinate} + 2(\text{sum of other ordinates})]$$

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Thus the trapezoidal rule may be stated as follows :

To the sum of the first & the last ordinate, twice the sum of intermediate ordinates is added. This total sum is multiplied by the common distance. Half of this product is the required area.

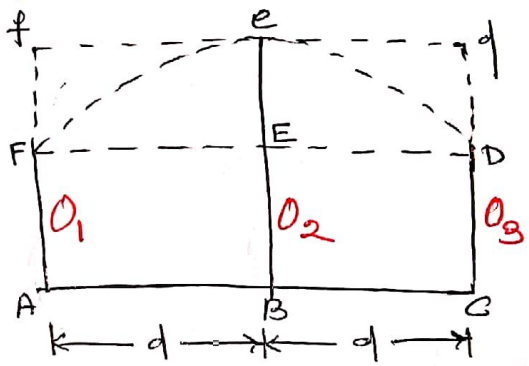
4) **Simpson's Rule :—**

In this rule, the boundaries between the ends of ordinates are assumed to form an arc of a parabola. Hence, Simpson's rule is sometimes called the parabolic rule.

Let,

O_1, O_2, O_3 = Three consecutive ordinates

d = common distance between ordinates.



Area AFeDC = area of trapezium AFDC + area of segment FeDEF.

Hence, Area of trapezium = $\frac{O_1+O_3}{2} \times 2d$

Area of segment = $\frac{2}{3}$ × area of parallelogram FfdD

$= \frac{2}{3} \times Ee \times 2d$

$= \frac{2}{3} \times \left\{ O_2 - \frac{O_1+O_3}{2} \right\} \times 2d$

So, the area between first 2 divisions,

$$\Delta_1 = \frac{O_1+O_3}{2} \times 2d + \frac{2}{3}\left\{ O_2 - \frac{O_1+O_3}{2} \right\} \times 2d$$

$$\Rightarrow \Delta_1 = \frac{d}{3}(O_1 + 4O_2 + O_3)$$

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Similarly, the area between next 2 divisions,

$\Delta_2 = \frac{d}{3}(O_3 + 4O_4 + O_5)$ & so on,

Total area $= \frac{d}{3}(O_1 + 4O_2 + 2O_3 + 4O_4 + \cdots + O_n)$

$= \frac{d}{3}\{O_1 + O_n + 4(O_2 + O_4 + \cdots) + 2(O_3 + O_5 + \cdots)\}$.

$\Rightarrow$ Total area $= \frac{\text{common distance}}{3}\{$1st ordinate + last ordinate + 4 (sum of even ordinates) + 2 (sum of remaining odd ordinates)$\}$

Thus, the rule may be stated as follows :—

To the sum of first & last ordinates, 4 times sum of even ordinates & twice the sum of remaining odd ordinates are added. This total sum is multiplied by the common distance One third of this product is the required area.

Trapezoidal rule

1) Boundary between ordinates is considered straight
2) It can be applied for any no. of ordinates.
3) It gives approximate result

Simpson's rule

1) Boundary between ordinates is considered parabola's arc.
2) To apply this rule, the no. of ordinates must be odd.
3) It gives more accurate result

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Computation of Volume

The volume of earthwork between cross-sections may be calculated by one of following methods:-

i) Prismoidal formula

ii) Trapezoidal formula / End area formula.

Prismoidal Formula:-

A prismoid is a solid bounded by planes of which two called end faces are parallel. The end faces may be both polygon not necessarily similar or with same no. of sides. The other faces called longitudinal face.

"The volume of a prismoid is equal to sum of areas of end parallel faces plus four times the area of central section, & multiplied by 1/6th of the perpendicular distance between end sections.

$$V = \frac{D}{3}\left[A_1 + A_n + 2\Sigma Odds + 4\Sigma Evens\right]$$

D = distance between sections

Prismoidal formula is also known as Simpson's rule.

Trapezoidal Formula:-

$$V = L \times \frac{A_1 + A_2}{2}$$

where, V = volume of prismoid

A_1 = area of one end section

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A_2 = area of other end section

L = distance between section

For series of cross sections,

$$V = L\left(\frac{A_1 + A_n}{2} + A_2 + A_3 + \ldots + A_{n-1}\right)$$

Prismoidal Correction :-

The volume obtained by trapezoidal method is not as accurate as prismoidal formula. The accuracy of result obtained by trapezoidal formula can be increased by applying a correction called prismoidal correction. As volume of any prismoid calculated by trapezoidal formula is somewhat larger than that obtained by prismoidal formula, prismoidal correction is always negative.

Prismoidal Correction = Volume by trapezoidal formula − Volume by prismoidal formula

$$P.C = \frac{L}{2}(A_1 + A_2) - \frac{L}{6}(A_1 + 4A_m + A_2)$$

$$= \frac{L}{3}(A_1 + A_2 - 2A_m)$$

Formula for obtaining prismoidal correction for different section

a) Level section : $P.C = \frac{bs}{6}(h_1 - h_2)^2$

b) Two level section : $P.C = \frac{L}{6s}(d - d')(d_1 - d_1')$

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c) Three level section : $P.C = \frac{L}{12}(D-D')(h_1-h_2)$

d) Side hill two level section :-

→ Center line being in excavation

$$P.C = \frac{L}{12s}(w-w')(d-d')$$

→ Center line being in embankment-

$$P.C = \frac{L}{12s}(w_1-w_1')(d-d')$$

Curvature Correction :-

The prismoidal & trapezoidal formula were derived on the assumption that the end sections are in parallel planes. When center line of cutting or embankment is curved in plan, it is common practice to calculate the volume as if the end sections were in parallel planes, & then apply the corrections for curvature.

i) level section : No correction is necessary since the area is symmetrical about center line.

ii) Two level section & three level section :

$$C_c = \frac{Ql}{6R}(w_1^2 - w_2^2)\left(h + \frac{b}{2n}\right)$$

where, R = radius of curve.

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iii) For two level section:-

The curvature correction to area,

$$= \frac{Ae}{A} \text{ per unit length}$$

e = eccentricity = horizontal distance from the center line to centroid of area.

$$= \frac{w_1 w_2 (w_1 + w_2)}{3An}$$

The correction is positive if centroid & center of curvature are to opp. side of center line while it is negative if the centroid & center of curvature are to the same side of center line.

iv) for side hill two level section:-

Correction to area = $\frac{Ae}{R}$ per unit length

$e = \frac{1}{3}\left(w_1 + \frac{b}{2} - nh\right)$ for the larger area

$e = \frac{1}{3}\left(w_2 + \frac{b}{2} + nh\right)$ for smaller area

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HIGHWAY ENGINEERING

1-0 Introduction :

The branch of Transportation Engineering which deals with the design, construction and maintenance of different types of road is called Road Engineering or Highway Engineering.

1-1 SCOPE OF HIGHWAY ENGG.

Apart from the design, construction and maintenance of different types of roads, Highway Engg. also includes the study of following topics

- → Development, planning, alignment
- → Highway geometric design & location
- → traffic operation and its control
- → Materials, pavement design, Economic consideration, finance &
- → Administration etc.
- → special aspects like problems of hill road construction, road-side development, landscaping and legal issues are also covered under highway engineering.

Technical Terms

Road :- A public thoroughfare (ସାଧାରଣ ଲୋକଙ୍କ ଚଲାଚଲ ପଥ) over which vehicles, cyclists, pedestrians, etc may lawfully move from one place to another is called a road or highway.
Usually the term highway is used for an important road of National or State importance in a country.

Traffic :- The vehicles, cycles, carts, pedestrians, etc travelling together on a road constitute the traffic.

Side walk :- The portion of roadway of an urban road reserved only for pedestrians is called foot path, side walk or foot way.
The minimum width of side walk should be 1.5m

Cycle track :- The portion of roadway of an urban road reserved only for pedal bicycles is called cycle track.
The min^{m} width of cycle track should be 2m.

Motor way :- The portion of roadway of an urban road reserved for use only by high speed and power driven vehicles is called motor way, express way or super highway.
The land width for an Express way is recommended as 50-60
The function of Express way is to cater for movement of heavy volumes of motor traffic at high speeds.

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Importance of Highway Transportation

Considering the utility of roads anywhere in the different parts of a country, thy can be rightly compared to arteries in a human body.

Just as arteries (ଧମନୀ ସମ୍ବନ୍ଧୀୟ) maintain man's health by providing circulation of blood; similarly roads promote (ପ୍ରୋତ୍ସାହିତ କରିବା) nation's wealth by keeping its people and goods moving.

Thus, we see that progress and well-being (ମଙ୍ଗଳ, ସୁସ୍ଥାନ ସ୍ଥା) of a nation depends much on roads. In fact, roads are the life lines of nations economy.

The importance or necessity of highway transportation can be easily judged from the following purposes or advantages of roads:-

1. They facilitate conveyance of people, goods, raw-materials, manufactured articles, etc. speedily and easily in the different parts of a country.
2. They act as the only source of communication in regions of high altitude i.e in mountainous regions.
3. They help in growth of trade and other economic activities in and outside the villages and towns by establishing contact between town & villages.
4. They help in providing efficient distribution of agricultural products and natural resources all over the country.
5. They help in price stabilization of commodities due to mobility of products all over the country.
6. They help in cultural & social advancement of people and making the villagers active and alert members of the community.
7. They help in promoting the cultural & social ties among people living in different part of a country and thus strengthen the national unity.
8. They help in providing improved medical facilities quickly to human beings, especially to those who live in rural areas.
9. They provide more employment opportunities.

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10. They enhance land value and thus bring better revenue
11. They serve as feeders for Airways, waterways & Railways
12. They help in reducing distress (కష్టం, దుఃఖం) among the people, caused due to famine, by supplying them food & clothing quickly.
13. The help in maintaining better law & order in a country.
14. They play a very important role in the defence of a country during war days.

Lastly, it can be said 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 its road system.

History of Highway Development in India

Indian civilization, being one of the oldest in the world (4000 to 3000 B.C), saw the development and growth of roads alongwith its own development. Thus, while tracing out the history of development of roads in India, one is to study it alongwith the development in political, economic and cultural life of this country.

Pre-historic Period

The history of roads is as old as the history of man on earth. The pre-historic man traced out a narrow way for going out for hunting the food. The narrow way was named as footpath or pathway. This pathway is considered as the first roadmark laid on the surface of earth. The utility & necessity of the pathway gradually developed with the introduction of wheeled carts. The pathway was widened into a roadway which was the beginning of road as a means of communication & transport.

Roads under early Indian Rulers

Ancient history of India reveals that long long ago, Indians were knowing the science of road construction. The excavations at Mohenjodaro and Harappa (Pakistan) have established that even 3500 years B.C, there was a well designed network of roads, and streets were paved at that time.

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(a) Aryan Period

During the Aryan period, there are references in Rig Veda (Part 1, para 5) about 'Mahapathas' as a means of communication. About 600 year B.C, a pucca road (6.1m to 7.3m wide) was built in Rajgir (ancient Rajagriha) of Patna District by king Bimbisara. This road was made of stones and is still in existence.

(b) Mauryan period

During this period, roads were developed on technical bases. Specifications were laid down for width of roads, road surfaces, types of roads for different purposes, etc. * camber was given to the surface of roads and the convexity of road surface was compared to the back of a tortoise. Artha Shasthra, the well known treatise on administration, gives a good deal of information regarding roads along with specifications adopted during Mauryan period. The book of Artha shasthra was written in about 300 years B.C by **Kautilya**, the first Prime Minister of Emperor Chandragupta Maurya.

Chandragupta Maurya (322-298 B.C) took keen interest in the maintenance & development of roads. He had a separate department of communications to look after the public roads. He got constructed the Grand Trunk road connecting North-West Frontier with capital palliputra, the modern patna. He has also fixed some sign post in the form of pillars and mile stones along the road side at regular intervals.

Emperor Ashoka took special interest in the improvement of roads & provided facilities to the travellers. Such facilities were in the form of plantation of trees, digging wells and constructing rest houses at about 4.8 to 6.4 km distance along the roads. The famous chinese traveller **Fahien** had spoken very highly of the roads of that time in the record of his travel.

Roads during the Mughal period

The roads were very greatly improved in India during the Mughal period. **Chahar Gulshan**, which was written in eighteenth century, gives an information regarding 24 important roads which formed the network of roads in India during the Mughal period.

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The road system in those days was considered as one of the best road system in the world.

The road from Delhi to Daultabad was constructed by Mohamad Tughlag. Sher shah Suri got constructed the longest road i.e the road from Punjab to Bengal. The present Grand Trunk Road forms the greater part of the old shershahi road also called Badshahi Sarak. The road from Agra to Allahabad and that from Ujjain to Bijapur were also got constructed by Muslim Emperors. Many of the roads, constructed during Mughal period, exist even today.

Roads during the British rule

The economic and political shifts caused much damages in the maintenance of road transportation. Thus, with the fall of Mughal Empire, the condition of roads became deteriorated.

At the beginning of the British period, a number of old Mughal roads, connecting important military & business centres, were metalled and some new roads were constructed by Military boards during the time of **Lord William Bentick**. But the administration of roads under Military Boards was not a satisfactory arrangement. It was only during the administration of **Lord Dalhausie** that the central public works Department was established to look after the construction & maintenance of roads. Later, such departments were created in other provinces also. **Lord Mayo** and **Lord Rippon** contributed a lot in the development of roads because the affairs of construction & maintenance of roads came directly under the control of Local Bodies.

With the development of Railways in India, the road development received a serious set back. The work of road construction & maintenance was given a secondary importance and thus the roads gradually lost the interest of the Government. Major roads, except those of military importance, were altogether neglected & attention was mainly concentrated on the feeder roads to Railways. Thus, the outlook on road development was completely changed and they were considered to be only of local importance. According to Government of India Act of 1919, the affairs of all the roads, except those of military importance & certain other roads of national importance, were transferred from the

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central Government to the provincial Governments. The provincial Governments, in their turn, took over direct responsibility of construction and maintenance of roads of provincial importance & placed the greater part of road mileage in the charge of Local Bodies.

After World War-I, motor transport came to the ~~fore-f~~ fore-front which created revolution in India's transportation system. Under the continued effect of high speed motor transport, the existing roads soon got deteriorated. The Local Bodies, with their limited financial & meagre technical resources, could not deal with the situation properly & with the increased motor traffic, the condition of roads went from bad to worse. Then the central Government took the following steps towards the development of roads:-

(a) Appointment of Jayakar committee
(b) creation of central Road Fund
(c) Indian Roads Congress
(d) Nagpur plan

(a) Appointment of Jayakar Committee

In 1927, the central Government appointed the Jayakar committee under the chairmanship of Dr. M.R. Jayakar to report on the condition of existing roads and to suggest ways and means for their future development. In 1928, the Jayakar committee recommended that since the Provincial Governments and the Local Bodies were unable to look after all the roads and, therefore, the central Government should look after all the important roads of national importance.

(b) Creation of central Road Fund

On recommendation of the Jayakar committee, the central Road Fund was enforced on 1st March 1929. The petrol tax surcharge at the rate of 2 annas per gallon (2.64 paisa/litre) of the petrol consumed by motor traffic was imposed to build the road development Fund. Out of annual revenue, thus collected, 20 percent was to be retained by central Government for meeting expenses on the administration purpose, research

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and the development of roads under its charge. The balance 80 percent of the Central Road Fund was to be distributed among the provinces, according to their petrol consumption, for maintenance & construction of roads

(c) Indian Roads Congress

In December 1934, a semi-official technical body (semi-govt body) known as Indian Road Congress (I.R.C) was established by the central Govt. as per recommendation of the Jayakar committee. This body was formed of national importance for controlling standardisation, specification and recommendation regarding design & construction of roads & bridges. But the economic depression during that time delayed the road development programmes.

After World war-II, there was a revolution in respect of automobiles using the roads in our country. The road development at that time could not keep pace with the rapid increase in road vehicles & therefore, the existing road started deteriorating fast. This necessitated proper highway planning by the authorities.

(d) Nagpur plan

In 1943, a conference of the chief Engineers of central & state Governments was conveened by the central Govt at Nagpur. It is a landmark in the history of road development in India since it was the first attempt to prepare road development programme in a planned manner. That conference finalised a Twenty year Road Development plan (1943-1963), popularly known as the Nagpur plan. According to that plan, all roads were classified into four broad categories namely National Highways, State Highways, District Roads and Village Roads. It was also recommended that the central Goverement should assume complete financial liability for construction and maintenance of roads classified as National Highways and the construction of roads of national importance was made the responsibility of the central Government.

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Roads during the Post-independence period

After independence, the Govt. of India started taking much interest towards the development of roads in this country. The Nagpur plan targets were mostly achieved by 1960 through the first and second five year plans (1951-56 & 1956-1961)

The various steps taken by the Government of India towards the development of roads in the country after independence are

(a) Central Road Research Institute
(b) National Highway Act
(c) Road Development plan
(d) Highway Research Board

(a) Central Road Research Institute

In 1950, Central Road Research Institute (CRRI) was started at New Delhi. This Institute is considered as one of the National Laboratories of the Council of Scientific and Industrial Research in India. This institute is mainly engaged in applied research and offers technical advice to state Govt. on various problems concerning to roads.

(b) National Highway Act

In 1956, the National Highway Act was passed. According to this act, the responsibility of development and maintenance of National Highways was given provisionally to the central Govt.

(c) Road Development plan (1961-81)

In 1958, the next Twenty years Road Development plan (1961-81) was finalised at the meeting of Chief Engineers of the states. This is popularly known as the Chief Engineer's plan. In this plan, due consideration was given to the future developments in various fields of our country.

According to this Road Development plan, the total length was almost double to that of Nagpur plan target.

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(d) Highway Research Board

On October, 1973, a Highway Research Board has been set up by the Govt. of India on the recommendations of the I.R.C council. This board is a separate organisation for roads just as the Railway Board in our country.

The Highway Research Board has been entrusted with functions to identify the nature, extent and priority for road research & to advice ~~the~~ and recommend to the Govt. items of priority research. This Board is to spread the results of research at home and abroad and also to apply these results in the actual highway construction.

Let us hope that with increased allocation of funds, better organisational arrangements and through intensive future plannings, India will not make up the deficiency in roads but she will lead many other countries in this respect in near future.

Indian Roads Congress (I.R.C) (semi-Govt body)

The Indian Roads Congress (I.R.C) is a distinguished association of all the engineers who are in charge of roads from strategic importance to the rural ones. It constitutes the rank engineers from the central & state Govt, Military Engg services, Boarder Roads, Engg colleges and commercial fields. All these members of I.R.C have a common platform to pool in their ideas on road research, construction, maintenance and standardisation.

The I.R.C was established by the central Govt in 1934 as per recommendation of the Jayakar Committee. This technical body was having 73 members at that time, where as now there are ~~more than~~ about 13500 members of this association.

The affairs(ફાળવાર) of the Indian Roads congress are controlled by a Governing Body known as the council. It constitutes 55 members nominated by the Govt. of India, Central P.W.D, E.I.C's branch, C.R.R.I, the state Govts and also elected from the ordinary & the Associate Members.

The Executive committee of I.R.C constitutes the following officials:

(a) President;
(b) vice-President (4);
(c) Honorary Treasurer;
(d) Honorary Secretary

The Director-General (Road development) and Additional Secretary to the Govt. of India, Ministry of shipping and Transport (Road wing)

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is the permanent treasurer of the Executive committee of the I.R.C.

Functions of Indian Roads Congress

Before 1934, there were no generally accepted road standards or bridge specifications of any kind. The Indian Roads Congress was, therefore, formed to perform the following functions: -

(i) To provide forum for regular pooling up the information, knowledge and experience for all matters affecting the construction and maintenance of roads in India.

(ii) To recommend standard specifications regarding design and construction of road bridges in India

(iii) To provide platform for the expression of professional opinion on matters relating to road engg.

Indian Roads Congress holds its sessions at different places in the country, problems pertaining to various aspects of road & bridge construction are discussed and results are further disseminated (প্রচার করা) through paper readings. This programme of the association has helped in promoting research and publicising the economic techniques adaptable to road construction and development in India.

Thus, the activities of I.R.C have been ultimately linked with the Govts. economic development programme. In fact, this technical body is devoted to the cause of better roads in India.

Classification of Roads

Roads are classified into different categories as ...

1. According to location
2. According to importance
3. According to traffic
4. According to tonnage

1. According to location

According to location and financial responsibility, non-urban roads in India are classified into following 5 categories

(a) N.H (b) S.H (c) M.D.Rs (d) O.D.Rs (e) V.R

This classification of roads was done as per recommendations made in the Nagpur plan finalised by the I.R.C in 1943.

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This classification is, therefore popularly known as I·R·C classification of road

(a) National Highways

The main highways runing through the length and breadth of the country connecting major ports, foreign highways and capitals of states, etc are known as National Highways.

These highways constitute the main arteries (মুখ্য ধমনী) of transport in the country and are also of military importance. National Highways should have carriageway of atleast two lane width. They should have modern type of surfacing. The responsibility of construction and maintenance of these roads lies with the central Govt.

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(b) State Highways

The highways linking district headquaters and important cities within the state or connecting them with National Highways or with highways of the neighbouring states are known as State Highways (SHs)

The highways are also called Provincial Highways. These highways serve as arterial routes of traffic to and from District Roads within the state. State Highways should preferably be of two lane width. They should also have a modern type of surfacing. The responsibility of construction and maintenance of these roads lies with state Govts.
However, the central Govt. gives grant for the development of these roads.

(c) Major District Roads

The important roads within a district serving areas of production and markets and connecting these places with each other or with the main highways are known as Major District Roads (M.D.Rs)

M.D.Rs should have atleast metalled single lane carriageway. The responsibility of construction & maintenance of these roads lies with the District Authorities. However the state Govt gives grant for development of these roads.

(d) Other District Roads

The roads serving rural areas of production & providing them with outlet to market centres, Tehsil headquarters, block development headquarters, railway stations, etc are known as the O.D.Rs.

(e) Village Roads

The roads connecting villages or group of villages with each other or with the nearest road of higher category are known as V.Rs.

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The roads are very important from the rural area development point of view. They are generally unmetalled & should have single lane width of stabilized soil or gravel. The responsibility of construction and maintenance of these roads lies with the Local District Authorities.

② According to importance

According to importance of connecting holy places(ତୀର୍ଥ), stations of strategic importance, roads are classified into the following categories.

(a) class I Roads
(b) class II Roads
(c) class III Roads

③ According to traffic

According to intensity of traffic, roads are classified into following categories

(a) very heavy traffic roads : which carry above 600 vehicles per day.

(b) Heavy traffic roads : which carry 251 to 600 vehicles per day

(c) Medium traffic roads : which carry 70 to 250 vehicles per day

(d) Light traffic roads : which carry below 70 vehicles per day.

④ According to tonnage

According to total tonnage per day, roads are classified into the following categories -

(a) very heavy traffic roads : which carry over 1524 metric tonnes per day.

(b) Heavy traffic roads : which carry 1017 to 1524 metric tonnes per day.

(c) Medium traffic roads : which carry 508 to 1016 metric tonnes per day

(d) Light traffic roads : which carry below 508 metric tonnes per day

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ORGANISATION OF THE STATE HIGHWAY DEPT.

There are different departments organised by the Govt. for its proper & smooth working. One such department in a state is the (P.W.D) public works Department.

The B&R or R&B branch, R.W (Rural works) is the concerned organised body of the state Highway Department. This branches of PWD looks after construction & maintenance of roads and government buildings in a state.

The minister is the over-all incharge of his Department. He is assisted by Principal secretary (Administration), Deputy secretary & Under secretary (Administration), Joint secretary, Engineer-in-Chief, Special secreary, Chief Engineers, Deputy secretary (Technical) in each branch/[redacted] department of the state Govt. or in a state.

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Defⁿ :- The physical features of a road are known as road geometrics.

These physical features have direct connection with the highway users. These are provided according to their geometrical design in order to facilitate safe and economical operation of vehicles.

With the development of fast moving traffic, the speed, safety and comfort have become the main requirement of road users now-a-days. Every road users wants to reach his destination within the shortest possible to time & that too in a comfortable manner without meeting with an accident. In order to fulfil these requirements, physical features of a road such as pavement width, formation width, right of way, curves, etc. The success and failure of road construction depends much upon the design of the physical features of the road.

Road geometrics include the following elements of a highways -

1. cross-sectional elements such as right-of-way, road margins, formation width, carriageway width, shoulders, side slopes, kerbs formation level, camber, gradients etc.
2. Speed of road vehicles i.e design speed, average running speed, etc.
3. Sight distances such as stopping or non-passing sight distance, passing or overtaking sight distance, intermediate sight-distance and lateral sight distance.
4. Curves such as horizontal and vertical curves
5. Superelevation, ...etc

Road geometrics are greatly influenced by the topography of the area, traffic characteristics and requirements of road users.

Geometrical Design of Highways

The phase of highways design which deals with the visible elements of different highways is called geometrical design of highways.

A proper geometrical design of a highway is essential before its actual construction to provide speed, safety and comfort to the road users.

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Ideals to be kept in view while doing the geometrical design of a highway:-

Following ideals should be kept in view while doing the geometrical design of a highway:

(i) There should be proper record of survey work done in the field.

(ii) There should be sufficient cross-sectional dimensions such as carriageway width, formation width, right of way, etc.

(iii) The road alignment should be economical

(iv) There should be easy gradient

(v) There should be adequate camber on straight portion of the road.

(vi) There should be proper super-elevation on curved portion of the road.

(vii) There should be efficient drainage system

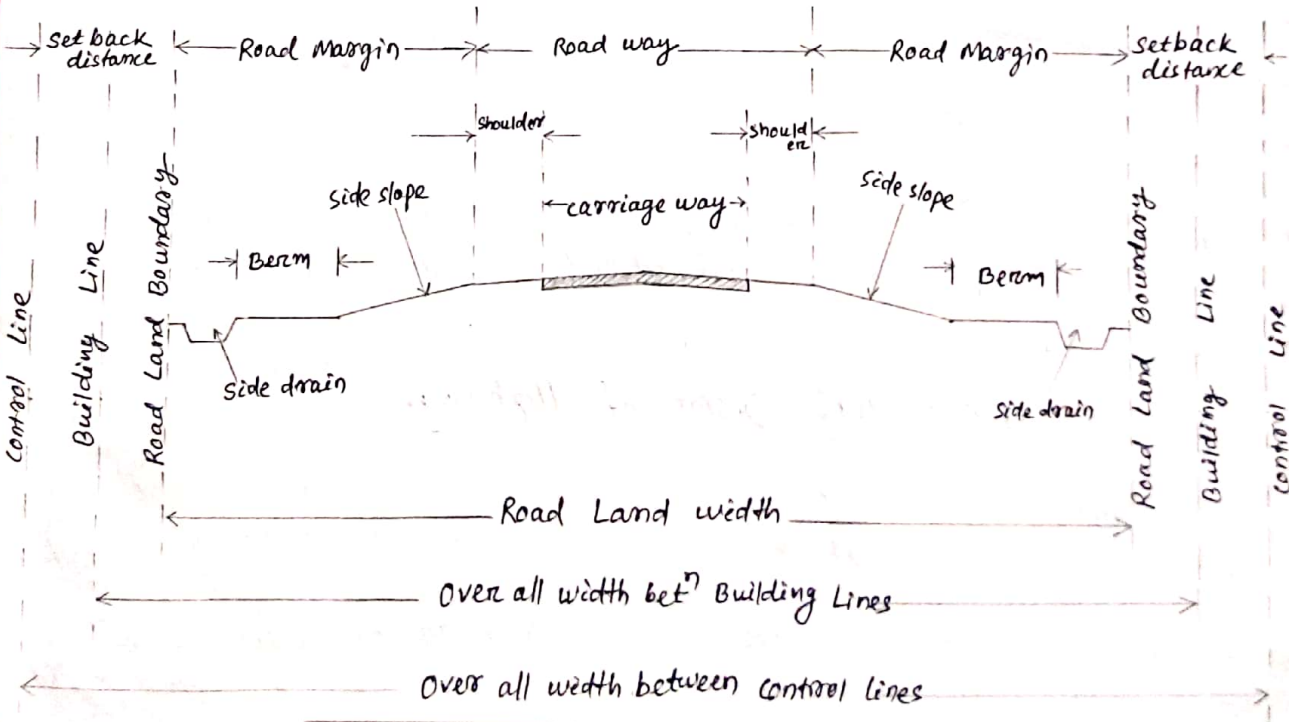
(viii) There should be provision of proper curves.

(ix) There should be provision of extra road width on curves.

(x) There should be provision of sufficient sight distances available on curves.

(xi) There should be provision of road signals, particularly at road junctions.

(xii) Estimated cost of the project should also be kept in view,



A ROAD IN EMBANKMENT

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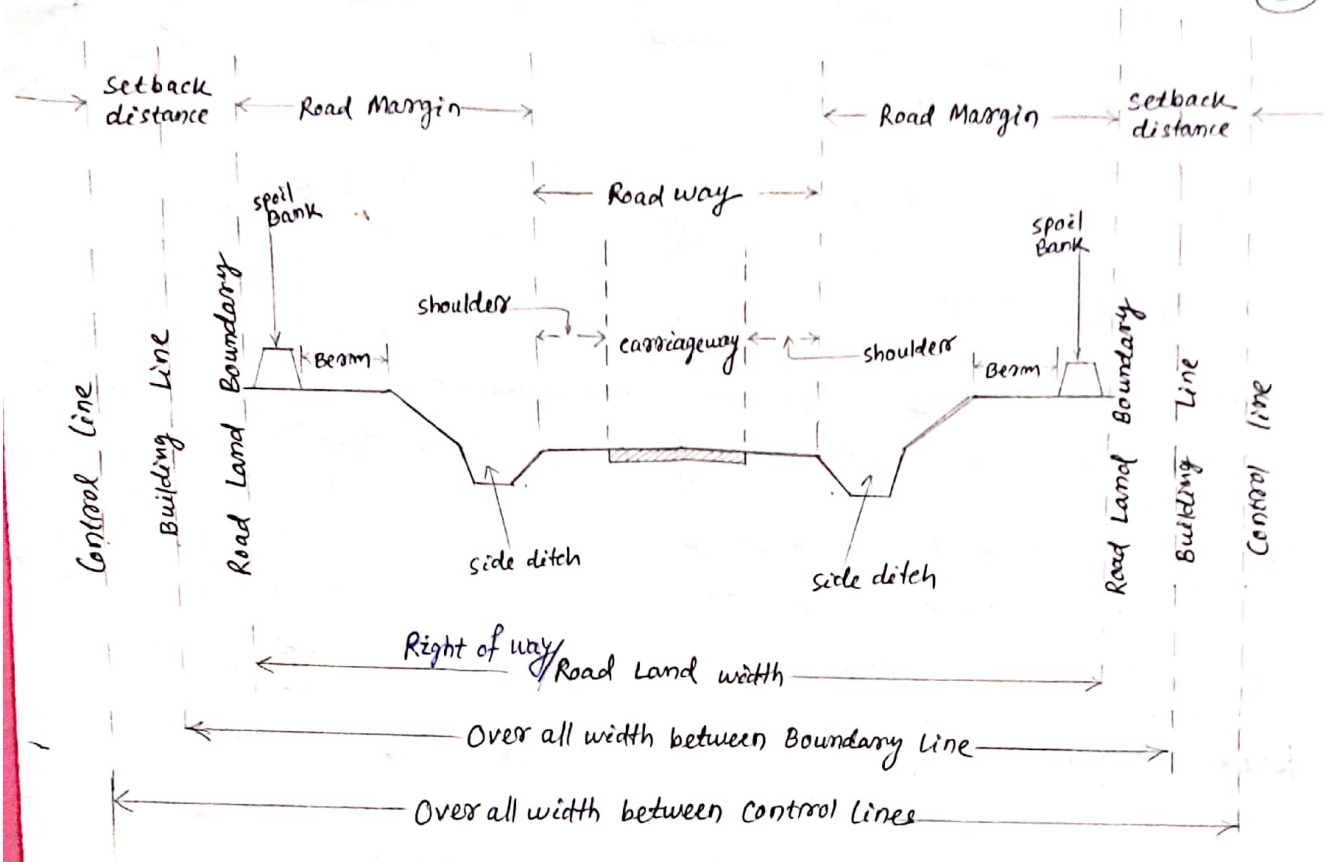

A ROAD IN CUTTING

Right of way :

The area of land required and reserved for construction and development of a road along its alignment is called right of way / permanent land /.

→ The width of right-of-way is called permanent land width or road land width.

→ The width of right of way of a road depends upon the importance of the road and its possible future development. It is determined by the width of formation, slope of embankment or cutting, depth of embankment or cutting, possible future widening of the road and the min^m sight distance required.

→ At the time of acquiring land for the right-of-way for a new road, the traffic requirements, topographical features, design needs, future developments and ultimate economy should be considered.

→ It is always better to acquire more width near cities, towns and industrial areas because the cost of the adjoining land always increases due to the opening of the new communications route and also ribbon development starts taking place making it difficult, later on, to acquire more land.

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Ribbon development : The constructional development, occuring on the side of a road in the form of shops, hotels or other buildings is known as ribbon development.

On curves, the building line on the inner side shall be at a greater distance from the centre line of road pavement in order to provide adequate sight distance.

Thus, right of way meets the present traffic requirements, design needs and also allows for future development of a road. If sufficient land width is acquired at the time of planning and constructing a new road, the same can be widened & other improvements can be made as & when desired in future.

Road Margin

The portion of land width on either side of the roadway of a road are known as road margins.

The various elements included in the road margins are parking lane, frontage road, driveway, cycle track, footpath, guard rail, slopes and side drains.

The road margins are secured and reserved to meet the present & future demands of development of the road.

Roadway width / Formation width

The top width of a highway embankment or bottom width of highway cutting excluding the side drains is called roadway width or formation width.

Roadway width comprises of the width of carriageway including traffic separator, if any, plus the shoulders on either side.

Roadway width is decided and constructed to meet the present traffic requirement, topographical features, design needs and ultimate economy of the road.

The roadway width for different categories of roads has finalised by I.R.C

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Carriageway / Pavement / Crust

The portion of roadway constructed for movement of vehicular traffic is called carriageway / pavement / crust.

The width of carriageway or pavement depends on the width of traffic lane and number of lanes required.

→ According to I.R.C specifications, the carriageway width for single lane traffic is 3.75m.

The number of traffic lanes required for a highway depends on the following factors :-

(i) Type of existing traffic and expected in next 10 years
(ii) Intensity of traffic expected in next 10 years
(iii) Over all maximum expected width of a vehicle
(iv) Minimum side clearance required for safe driving.

Thus, carriageway width is a very important factor in the safe design of a highway. The position selected by a driver while meeting an incoming vehicle or passing a slow moving vehicle safely depends mostly upon the carriageway width.

Shoulders

The portion of the roadway between the outer edges of the pavement and edges of the top surface of embankment or inner edges of the side drains in cutting are known as shoulders.

The width of shoulders varies from 0.5m to 4.0m according to the nature of the area and type of road. They form an essential part of the roadway. They may be consisting of asphaltic, water bound macadam or simply earthen surface according to the type of pavement. They are always given an outward slope to drain off the rain water quickly towards the sides.

The outward slopes to be given to the surface of shoulders as recommended by the I.R.C are as follows

(a) Paved surfaces :- 1 in 24 to 48
(b) Earth Surfaces :- 1 in 16 to 24

Objects of providing shoulders

(i) They act as components of the roadway
(ii) They provide lateral stability to the carriageway.

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(iii) They serve as parking places for vehicles in case of emergency.

(iv) They provide space for erecting road signals.

(v) They provides pace for animal drawn vehicles, cyclist & pedestraims to retreat when a fast moving vehicles tends to cross or over-take them.

(vi) They provide space for accidental repairs to vehicles when they become unservicable on the road.

Side slopes

The slopes given to the sides of earthwork of a road in embankment or in cutting for its stability are called side slopes.

Side slopes should be so designed as to keep the earthwork stable in embankment or in cutting. Flatter side slopes would make the earthwork uneconomical whereas steeper side slopes would result into erosion & slips.

The main factors affecting the design of slopes are :-

(i) The nature of soil constituting the road embankment or cutting
(ii) climatic conditions
(iii) The method of drainage provided
(iv) The method of protecting the side slopes from erosion & slips.

Earth embankments of usual height can stand safely with side slope of 1 in 1.5. In cutting, a side slope of 1:1 can serve the purpose. In hard rock stretches, side slopes varying from 1 in $\frac{1}{2}$ (batter 1 in 2) to 1 in $\frac{1}{4}$ (batter 1 in 4) can be provided safely.

(*) But modern tendency is to provide flatter slopes as they result in safe movements of vehicles and low maintenance cost.

I.R.C has recommended the following side slopes for road embankments or cuttings.

In Filling :-

Generally 2:1 slopes of embankments when,

(i) the height of the embankment is over 0.8 m

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(ii) the filling material / subsoil strata consists of heavy clay.

(iii) the height of the embankment is more than 3m & subject to weathering.

In cutting :-

(i) Ordinary soil = 1:1 to $\frac{1}{2}$:1

(ii) Disintegrated rock or conglomerate = $\frac{1}{2}$:1 to $\frac{1}{4}$:1

(iii) Soft rock and shale = $\frac{1}{4}$:1 to $\frac{1}{8}$:1

(iv) Medium rock = $\frac{1}{12}$:1 to $\frac{1}{16}$:1

(v) Hard rock = Nearly vertical

Berms :

The portions of land width left in between the toe of road embankment and the inner edges of borrow pits or the portions in between the top edge of road cutting and the nearest edge of spoil banks on either side are known as berms.

They prevent side slopes of the road embankment from damage by erosion or the cutting from damage by lateral thrust due to spoil bank.

Kerbs

The boundaries between the pavement and shoulders or footpaths are known as kerbs.

The kerbs may also be provided between the pavement & traffic separator or dividing ~~signal~~ island.

Kerbs are generally constructed of cut stone or cement concrete slabs which are usually chamfered or rounded off. The kerb & the road surface near the edge together form a side channel which carries away rain water that comes to it after falling on the surface of road.

The height of kerbs is about 10 to 20 cm above the surface of pavement edge where footpath is provided next to it.

On urban roads, the function of kerbs is to prevent the traffic from getting off carriageway, whereas in rural roads, submerged kerbs, consisting of bricks on edge, are constructed between the pavement & its shoulders to provide lateral stability to the road pavement.

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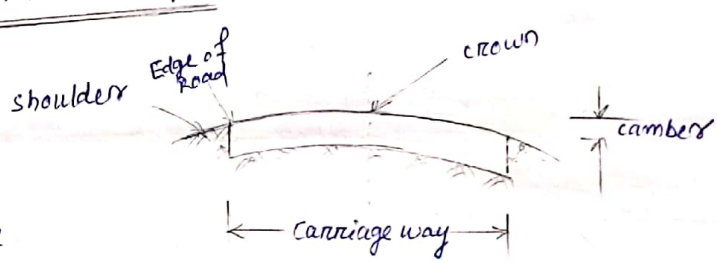
Formation level

The reduced level of the finished surface of earthwork for a road in embankment or in cutting is known as formation level.

Formation level of a highway should be decided such as to provide economical earthwork in the road project. When the road is to be constructed in embankment, the formation level should be kept above the highest flood level in the area. In case of road in cutting, the formation level should be sufficiently above the sub-soil water table.

CAMBER / CROSS FALL / transverse slope



The convexity provided to the surface of carriageway or the rise given to the centre of carriageway above its edges on straight portion of a road is called camber or cross fall.

→ camber is provided on the straight reaches of a road.
→ It is usually expressed as the **percentage of rise** given to the crown of the carriage way above its edges.
→ It is also expressed as slope of the line joining the crown with the edges of the carriageway.

Thus, a camber of 2.5 percent (1 in 40) means that for a 7m wide carriageway, its crown lies $\frac{2.5}{100} \times \frac{7}{2} \times 100 = 8.75\text{cm}$

The amount of road camber depends on the intensity of rainfall in the locality & the permeability of the road surfacing material. Thus, a hard or less permeable smooth surface will require less camber than a soft or more permeable rough surface.

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Objects of providing camber

(i) To drain off rain water from the surface of carriageway towards the sides of a road as quickly as possible

(ii) To regulate the vehicles to their proper lanes

(iii) To improve the architectural appearance of the roadway.

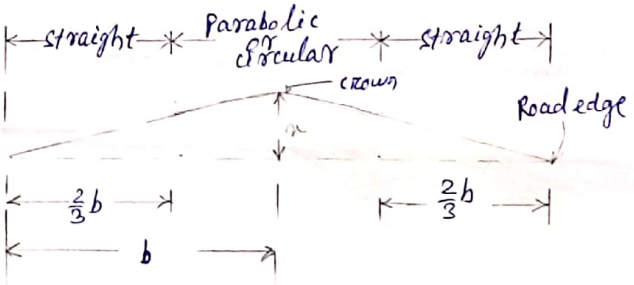
Types of Camber

Following types of camber are generally provided to the road surface.

(a) Composite camber,
(b) Sloped or straight camber
(c) Two straight line camber
(d) Barrel camber

(a) Composite camber

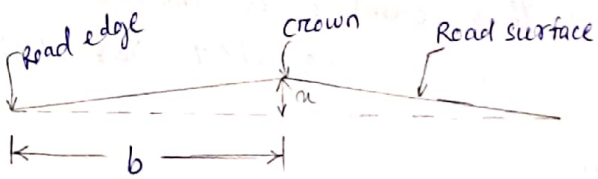
It consists of two straight slopes from the edges with a parabolic or circular crown at the centre of carriageway as shown....



→ This type of camber can be easily constructed & maintained.

(b) Sloped or straight camber

It consist of two straight slopes from the edges joining at the centre of carriageway.



→ This type of camber is very simple and can be easily constructed & maintained.

→ This type of camber is provided when very flat cross slope is required as in the case of cement concrete pavement.

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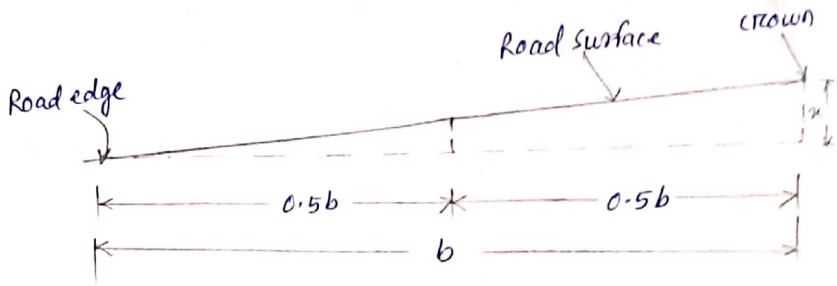
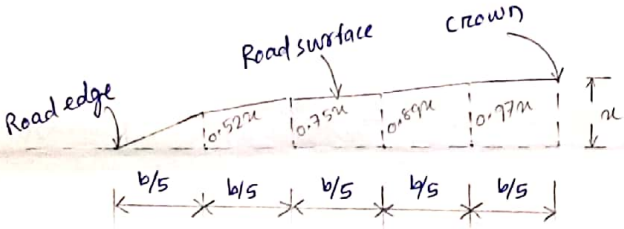
(c) Two straight line camber



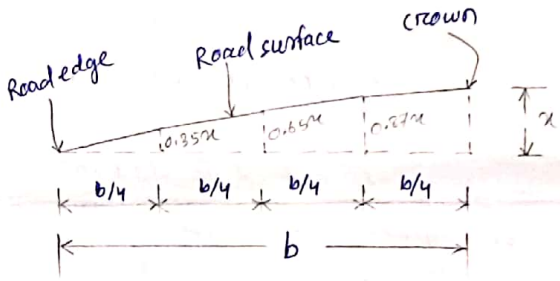
It consists of two straight lines steeper near the edges & flatter near the crown of carriageway.

* This type of camber is considered to be the best for Indian Roads because it provides more contact area of the tyres with the road surface than in other types of camber. Thus, it provides less damage to the road surface.

(d) Barrel camber



Parabolic Barrel Camber



Elliptical Barrel Camber

It consists of a continuous curve either elliptical or parabolic. It provides a flat road surface at the middle & steeper towards the edges. On account of steeper edges, this type of camber provides better drainage property.

→ This camber is, therefore, preferred by fast moving vehicles and is **suggested for Urban roads.**

* This type of camber is difficult to construct & maintain. The barrel camber has more steeper edges which are inconvenient to use. Moreover, the steep edges are eroded quickly and hence additional kerbs are to be provided.

Advantages of excessive camber

(i) Excessive camber provides quick drainage of rain water to the sides and thus saves the foundation course of the road structure from weakening by percolation of rain water to it through the

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road surface.

(ii) This prevents rain water to accumulate in local sinkages or depressions and forming water pool on the road surface, which are disagreeable to public as well as to the road structure.

Dis-advantages of excessive camber

(i) The road surface with excessive camber wears out excessively and quickly at the central portion because the drivers have a tendency to run along the centre of the road to avoid camber pull & thus results in more maintenance cost.

(ii) This results in more number of accidents because the vehicles, moving at high speed, have a tendency to slip towards the edges.

(iii) This causes problems of toppling (ଭୂମିଶାଇ କରିବା) over of heavily ladden bullock carts.

(iv) Excessive camber results into formation of cross ruts (ଚକ ଚକ୍ର) and the shoulders may be washed away due to rapid flow of water

Methods of providing camber

The required camber is provided to the subgrade during consolidation. For providing the desired amount and shape of camber, templates or camber boards are prepared conforming to the specified camber. The material to be used in the subgrade is spread & hand packed to the shape of the desired camber. The rolling is then started from the edges and shifted towards the centre. Overlapping equal to half the width of the roller should be given while shifting the roller towards the centre of the road after rolling each strip. Then checking of the top surface of the subgrade should be done by means of camber boards and deficiency, if any, is rectified by spreading or removing the material as required.

GRADIENT / GRADE / LONGITUDINAL SLOPE

The ground is never dead flat, and hence the road to be provided will also have rise and falls along the length of the road.

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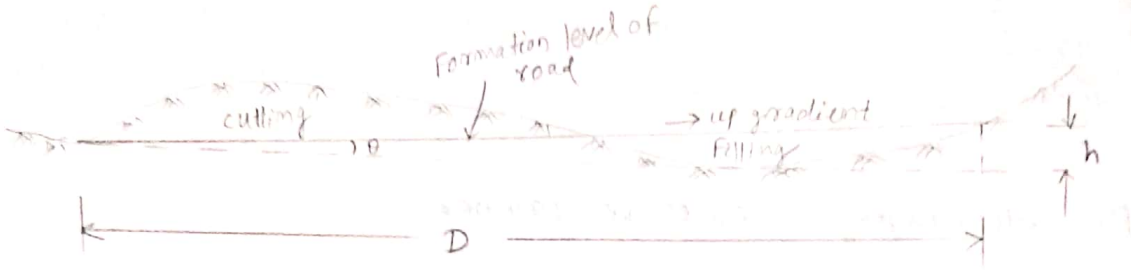
The rate of rise or fall provided to the formation of a road along its alignment is called grade or gradient or longitudinal slope.



It is expressed as the ratio of rise or fall to the horizontal distance or as percentage rise or fall.

* In India (ratio of fall or rise to the horizontal distance) being mostly adopted.

Thus, if a road ascends or descends one metre for every 50m the gradient is said to be 1 in 50 or 2%.

* The gradient may also be expressed in degrees of elevation or depression above or below a horizontal plane.

* A gradient of 1° represents a rise or fall of 1m in a horizontal distance of 57.3 m (as π rad = 180°, 1 rad = $\frac{180}{\pi}$ = 57.2957...)

Gradient to a road is necessary to drain off surface water easily through the side drains. Even in plains, flat gradients of 1 in 300 to 500 are provided for satisfactory drainage and to keep the roads in good conditions with less maintenance cost.

Object of providing gradient

1. To connect the terminal stations situated at different levels.
2. To provide effective drainage of rain water falling over the road surface, particularly when the pavement is provided with kerbs.
3. To make the earth work of the road project economical since a perfectly level road involves more cutting & filling.
4. To construct side drains economically with convenient depths below ground level.

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(5) To reduce the maintenance cost of road surface. It has been experienced that maintenance cost is reduced by 15% to 25% on a road with slight gradient than that with level surface.

Factors affecting gradient selection

Following are the various factors which govern the selection of gradient in the alignment of a road...

(i) Nature of ground
(ii) Nature of traffic
(iii) Drainage required
(iv) Type of road surface
(v) Total height to be covered
(vi) Road and railway intersections and bridge approaches
(vii) Safety required.
(viii) Type & importance of highway

Types of gradient

The following are the different types of road gradient

(a) Ruling gradient / permissible gradient
(b) Limiting gradient / Maximum gradient / momentum grade.
(c) Exceptional gradient / [redacted] /
(d) Average gradient
(e) Floating gradient
(f) Minimum gradient

There are no such universal classification and limits for the various types of gradients. The different categories of the gradients depends on the type of terrain of the country such as flat or mountainous & the type of conveyance such as heavy transportation vehicles, goods transporters, passenger buses, jeeps, animal drawn carts, pedestrians. with this reservation, gradient may be classified as .....

(a) Ruling gradient

This is such a gradient that all vehicles, whether drawn by power or by animals can traverse long length of the road without undue consumption of fuel or much fatigue.

* This gradient is generally adopted for designing vertical profile of a road
* This gradient should never be exceeded in any part of a road in the normal course.

→ The gradient usually adopted while making the alignment of a road is called ruling gradient.

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The choice of ruling gradient depends on the following factors:-

→ The nature of vehicles
→ The type of road surface
→ The local topographical conditions.

The ruling gradient is also defined as the permissible gradient on the alignment of the highway which the engineer must endeavour (प्रयत्न) to design and stick to.

(b) Limiting / Maximum / Momentum Gradient

This gradient is slightly steeper than the ruling gradient. It is provided when the natural ground profile is slightly steeper than the ruling gradient. It is necessary to adopt such a steeper gradient, to keep the earth work operation minimum. This gradient is generally adopted to connect plain & rolling terrain or to reach hilly areas.

(c) Exceptional Gradient

The gradient steeper than the limiting ~~which may be used in~~ short lengths of the road, only in extraordinary situations, is called exceptional gradient.

This type of gradient is adopted only in very difficult situations & for short lengths not exceeding 100m at a stretch. In mountainous & steep terrain, successive stretches of exceptional gradients must be separated by a minimum length of 100m having gentler gradient.

(d) Average Gradient

The total rise or fall between any two points along the alignment of a road divided by the horizontal distance between them is called average gradient.

(A) This gradient is determined to carry out the preliminary surveys i.e proper location, reconnaissance survey, etc

(e) Floating Gradient

The gradient on which a motor vehicle, moving with a constant speed, continues to descend with the same speed

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without any application of power or brakes is called floating gradient.

In case of floating gradient, the downward component of the weight of the moving vehicle remains equal to the frictional resistance of the road surface.

(7) Minimum Gradient

The minimum desirable slope essential for effective drainage of rain water from the road surface is called minimum gradient.

On unkerbed pavements in embankment, near-level grades are not objectionable when the pavement has sufficient camber to drain the rain water laterally. But, in cut sections or where the pavement is provided with kerbs, it becomes necessary that the road should have minimum gradient for efficient drainage. Desirable minimum gradient for this purpose is 0.5 percent if the side drains are lined and 1.0 percent if the drains are unlined.

Design Speed

The maximum safe speed of vehicles assumed for geometrical design of a highway is known as design speed.

This is the approximately uniform speed that can be maintained by majority of the drivers over the specified category of road. This speed is assumed for co-relation of geometrics (physical features) of a road that influence the operation of vehicles. Since the overall geometrical design of any road depends on design speed, it is essential that the assumed design speed should be in conformity with the high standards of mobility, safety and efficiency desired on different categories of road.

The design speed depends upon the following factors

(a) Type and condition of the road surface
(b) Structure of the road
(c) Nature, type and intensity of traffic
(d) Type of curve along the road.

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(e) Sight distance required

(f) Nature of the country

Design speed on different categories of Roads as per Recommendation of I.R.C in km/h

	Plain Area		Rolling Area		Hilly Area		Steep Area	
	Rulling	Minm	Rulling	Minm	Rulling	Minm	Rulling	Minm
N.H & S.H	100	80	80	65	50	40	40	30
M.D.R	80	65	65	60	40	30	30	20
O.D.R	65	50	50	40	30	25	25	20
V.R	50	40	40	35	25	20	25	20

The Economic Commission for Asia and Far East (E.C.A.F.E) has recommended the following design speed for the low cost roads.

(i) plain or rolling area : 48 km/hr

(ii) Hilly area : 32 km/hr

These design speeds have been recommended on the possibility of further development of the low cost roads and future expansion of traffic.

Average Running Speed

The speed maintained by vehicles over the particular section of a road is called average running speed.

- It is less than the design speed.

This is calculated by dividing the distance covered by a vehicle with the actual running time (excluding any stopping time).

It is varies from 70% to 90% of the design speed. If the design speed is 100 $\frac{km}{hr}$, then the average running speed is 75 km/h i.e 75 percent of the design speed.

$$\text{Average running speed} = \frac{\text{Distance covered by vehicle}}{\text{Actual running time.}}$$

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Approximate Relationship between Design Speed and Average Running speed of vehicles.

Design speed	Average Running Speed	
	Percent of design speed	Km/h
50	90	45
65	85	55
80	80	64
100	75	75
110	70	77

Sight Distance

The distance along the centre line of a road at which a driver has visibility of an object, stationary or moving, at a specified height above the carriageway is known as sight distance.

or

Sight distance is the length of road visible ahead to the driver at any instance.

or

The longest distance a driver can see in front of him, may be termed as sight distance.

or

Sight distance is the maximum distance visible to the driver along the centre line of a highway.

(A) When a fast moving vehicle negotiates a horizontal or vertical curve on approaches a road intersection, a certain distance of visibility through which the driver can see the opposite vehicle, a pedestrian or some fixed object on the road in the direction of travel is essential so that the driver may react and avoid any collision or accident. Therefore, provision of such a distance of visibility, known as the sight distance, becomes necessary to provide safe & efficient movement of vehicles on the road.

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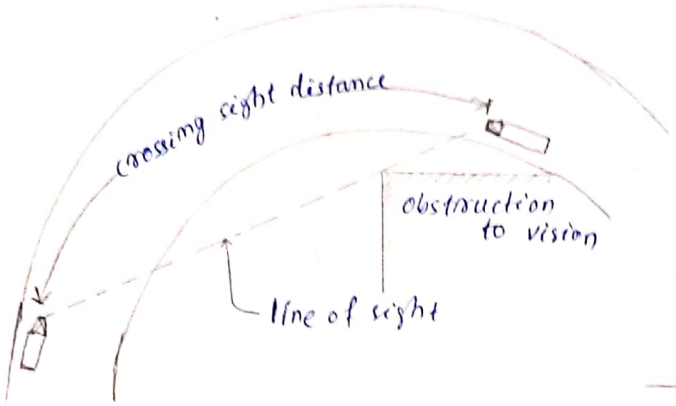
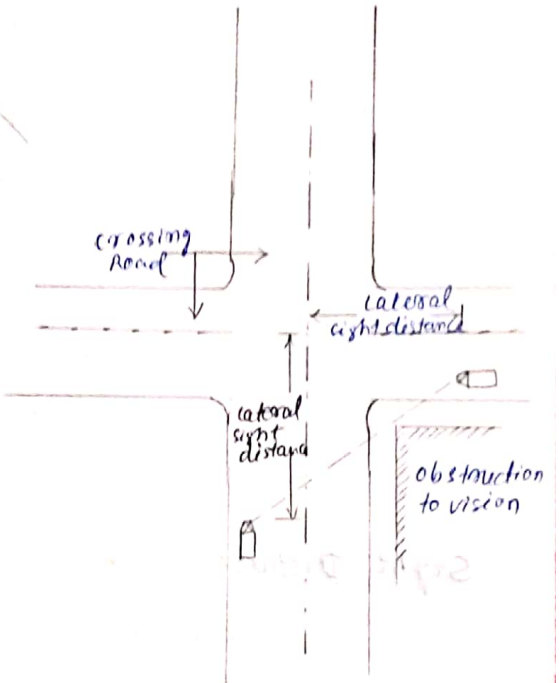
Moreover, the geometrical design of highways and traffic control both are affected much by the sight distance required by the drivers.

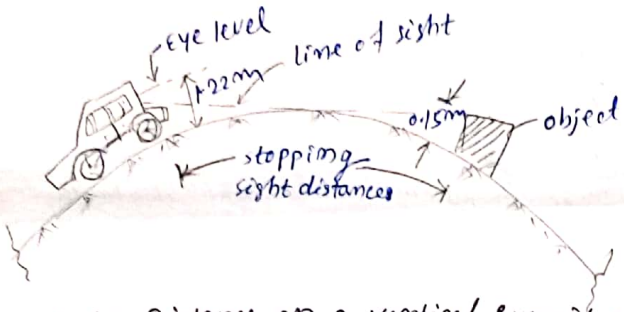
Restrictions to sight distance



Sight Distance at a Horizontal curve



Sight Distance at a Road Intersection (Lateral sight Distance)



Sight Distance on a vertical summit curve (stopping sight Distance)

Restriction to sight distance may be caused at horizontal curves, by objects obstructing vision at the inner side of the road, on the vertical summit curves or at road intersection.

But the sight distance at such locations should be such that the drivers & pedestrians get sufficient time to react to an emergency to avoid not only accident but also to extend road courtesy (ಇತ್ತ, ಇಲ್ಲಿಂದ, ಕಳುಹಿ) to each other.

The design standards for sight distance must satisfy the following 3 conditions

(i) The driver travelling at a design speed should have sufficient road length visible to enable him/her to stop his/her vehicle in case of obstruction on the road ahead, without causing collision

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or accident.

(ii) The driver travelling at a design speed should be able to over take the slow moving vehicles at reasonable intervals during his journey without endangering the traffic from opposite direction.

(iii) The driver entering an uncontrolled road intersection should have sufficient visibility to enable him/her to control his vehicle & avoid collision with other vehicle approaching the intersection.

Factors affecting the sight Distance

The safe sight distance depends upon the following factors :-

(a) speed of the vehicle
(b) perception time and brake reaction time.
(c) Efficiency of brakes of the vehicle
(d) The frictional resistance of the road surface
(e) Height of the driver's eye.
(f) Slope of the road surface.

In addition to these factors, the following factors also affect the sight distance of a road.

(i) Eye sight of the driver
(ii) Efficiency of the screen wipers during rain
(iii) Efficient lighting system during night hours.
(iv) weather conditions.

Types of Sight Distance

The various sight distances which are considered for geometrical design of a road can be splitted up into the following types

(a) safe stopping sight distance (SSD) or Absolute min^m sight distance or Non-passing sight distance

(b) Safe overtaking sight distance (OSD) or passing sight distance

(c) Intermediate Sight distance

(d) Lateral Sight distance.

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(a) Safe-stopping / Non-passing / Absolute Minm sight Distance

distance required for an emergency stop.

The clear distance ahead needed by a driver to bring his vehicle to a stop before meeting a stationary object on the road is called stopping or non-passing sight distance.

→ Minm stopping sight distance is the distance travelled by the vehicle during perception and brake reaction time plus the braking distance.

In case of a summit curve, minm stopping sight distance is the distance measured along the centre line of a road at which a driver whose eye sight is 1.22m above the road surface can see the top of an object 15cm high on the road as shown in fig:

Vehicle speeds as recommended by I.R.C are given in following table

Speed	Perception & Brake Reaction		Braking		Safe stopping sight Distance (m)	
V (km/h)	Time, t(sec)	Distance(m) $d_1 = 0.278\,Vt$	co-efficient of Longitudinal friction (f)	Distance (m) $d_2 = \frac{V^2}{254f}$	calculated values (d_1+d_2)	Design values (mts)
20	2.5	14 (13.9)	0.4	4.0 (3.937)	18 (17.837)	20
25	2.5	18 (17.375)	0.4	6.[illegible] (6.15)	24	25
30	2.5	21 (20.85)	0.4	9.0 (8.858)	30	30
40	2.5	28 (27.8)	0.38	17.0 (16.57)	45	45
50	2.5	35 (34.75)	0.37	27.0 (26.60)	62	60
60	2.5	42 (41.7)	0.36	39.0 (39.37)	81	80
65	2.5	45 (45.175)	0.36	46.0 (46.20)	91	90
80	2.5	56 (55.6)	0.35	72 (71.991)	128	130
100	2.5	70 (69.5)	0.35	112 (112.485)	182	180

All these value mentioned are based on perception and brake reaction time of 2.5 seconds & co-efficient of longitudinal friction varying from 0.40 at 20 $\frac{km}{h}$ to 0.35 at 100 $\frac{km}{h}$

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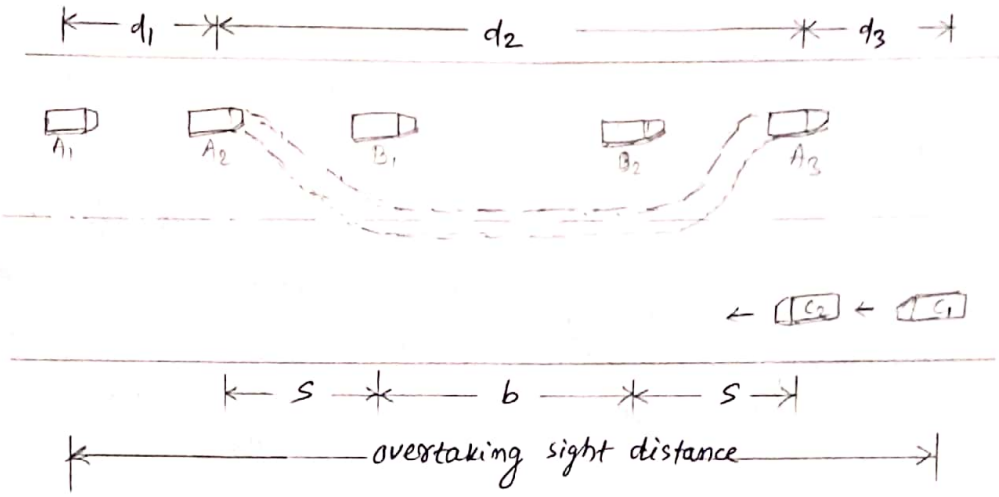
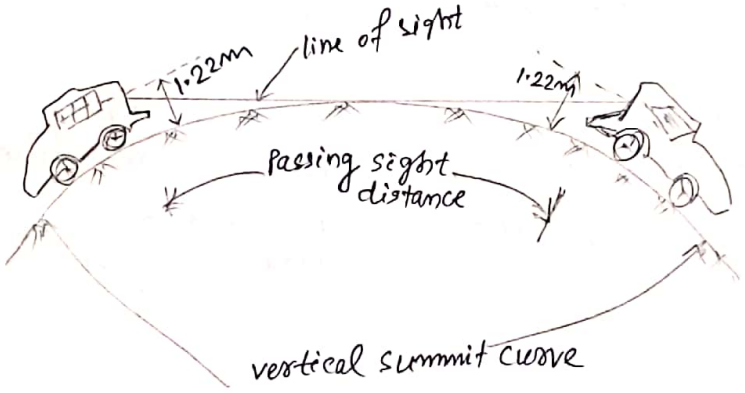
(b) Safe Overtaking / Passing sight Distance / Non-stopping sight Distan

distance required when vehicles can overtake & pass each other.





The min^m sight distance needed by a driver on a two way road to enable him to overtake another vehicle ahead with safety against the traffic from opposite direction is called overtaking or passing sight distance.

The min^m overtaking sight distance depends upon the following factors:

(i) Speed of overtaking, overtaken and that of the vehicle coming from the opposite direction.

(ii) Rate of acceleration of the overtaking vehicle

(iii) spacing between vehicles

(iv) skill and reaction time of the driver.

In case the driver of vehicle A running at design speed tries to overtake a slow moving vehicle B on a two lane road

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while the third vehicle 'c' comes from the opposite direction, its overtaking manoeuvre (S_1, S_2, S_3) ... is shown in fig ...

Then, the overtaking sight distance required for vehicle A

$= d_1 + d_2 + d_3$

where

d_1 = the distance travelled by the overtaking vehicle A during the reaction time A_1 to A_2

d_2 = the distance travelled by vehicle A from A_2 to A_3 during the actual overtaking operation.

d_3 = the distance travelled by vehicle C coming from the opposite direction i.e from C_1 to C_2 during the overtaking operation by vehicle A.

→ Design values for overtaking sight distance as per recommendations of I.R.C are given in table.

→ These values are based on time component of 9 to 14 seconds for the actual overtaking manoeuvre depending upon design speed, increased by about $\frac{2}{3}$rd to take into account the distance travelled by a vehicle from the opposite direction travelling at the same speed.

Safe overtaking Sight Distance for various Design speeds

Speed in km/h	Time component in seconds			Safe overtaking Sight Distance (mtrs)
	For overtaking manoeuvre	for opposing vehicle	Total	
40	9	6	15	165
50	10	7	17	235
60	10.8	7.2	18	300
65	11.5	7.5	19	340
80	12.5	8.5	21	470
100	14	9	23	640

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(c) Intermediate sight distance

The distance which affords reasonable opportunities to drivers to overtake the vehicle ahead with caution is known as intermediate sight distance.

At approaches of intersections between the main highway & intermediate or single or unimportant roads, intermediate sight distance is provided. Intermediate sight is also used when provision of OSD is not possible due to economic reasons or space limitation at site.

→ According to Indian practice, intermediate sight distance is taken as two times (twice) the value of safe stopping sight distance.

→ The place where the overtaking sight distance or as much length of the road as feasible is not available, intermediate sight distance should be adopted as the next best alternative.

→ Because of practical reasons, intermediate sight distances are not provided on summits & horizontal curves. If there is no space for SSD, restrictive pavement markings or 'no overtaking' sign boards should be installed at appropriate places along the road.

Design values of intermediate sight distance as recommended by I.R.C are given below

Intermediate Sight Distance for various Design Speeds.

Speed Km/h	Intermediate sight Distance (mtrs)	Speed Km/h	Intermediate sight Distance (mtrs)
20	40	50	120
25	50	60	160
30	60	65	180
35	80	80	240
40	90	100	360

(d) Lateral sight distance

The sight distance needed by the driver of a vehicle who sees another vehicle approaching the intersection, reacts and applies brakes to bring his vehicle to dead stop at the intersection.

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without any collision or accident is called safe sight distance for entering into an intersection or lateral sight distance.

The line of sight is usually obstructed by structures or other objects at the corners of road intersections, it is, therefore, important that on all approaches of road intersections, there should be a clear views across the corners from a sufficient distance so as to avoid collision of vehicles. This is more important in case of unchannelised road intersections. This sight distance should be equal to the stopping sight distance along each intersecting road.

The lateral sight distance should be sufficient to satisfy the following three(3) conditions: -

(i) To enable either one or both the approaching vehicles to change their speeds to avoid collision.

(ii) To bring either one or both the approaching vehicles to stop before reaching a point of collision.

(iii) To enable the stopped vehicles on minor road (controlled by a stop sign) to start, accelerate and cross the main road before the approaching vehicle, travelling at design speed on main road, reaches the intersection.

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REACTION TIME - PIEV THEORY

The time required by the driver, say, to apply the brake on seeing a vehicle or obstruction, is not an instantaneous action, but is a "time consuming" phenomena based on the psychological (ମନସ୍ତାତ୍ତ୍ୱିକ) processes involved. These processes may be divided as those due to Perception, intellection, emotion and volition.

According to PIEV theory, the driver perceives (ଉପଲବ୍ଧି କରିଥା) a situation on the road, analyses it using his intelligence (ଜ୍ଞାନ, ମେଧା, ବୁଦ୍ଧି ଶକ୍ତି), emotionally (ଆବେଗପୂର୍ଣ୍ଣ ଭାବରେ) feels it and reacts with volition (ଇଚ୍ଛା ଶକ୍ତି)

Perception time :- The time required to perceive an object or situation

or

The time taken by the driver of a vehicle in motion to realise a danger before trying to take any preventive measure to avoid accident is known as perception time.

or

It is the time required for a driver, while driving the vehicle, to observe, notice, assess and mentally analyse a given situation - the driver will then decide whether to react, apply the brake or steer away. In other words, during driving, it is the minimum time required for a driver to react instantly and excercise control.

Intellection time :- The time required for comparing the different thoughts, regrouping and registering new sensations.

Emotion time :- The time elapsed during emotional sensation and disturbances.

Volition time :- The time required for the final actions.

The PIEV time depends upon

(a) Physical fitness/characteristics of driver
(b) Psychological factors (intelligence, age....)
(c) Environmental condition
(d) Purpose of trip
(e) Speed of vehicle

[The total reaction time of an average driver may be as low as 0.5 second for Situations to as high as 3 to 4 seconds for complex problems.]

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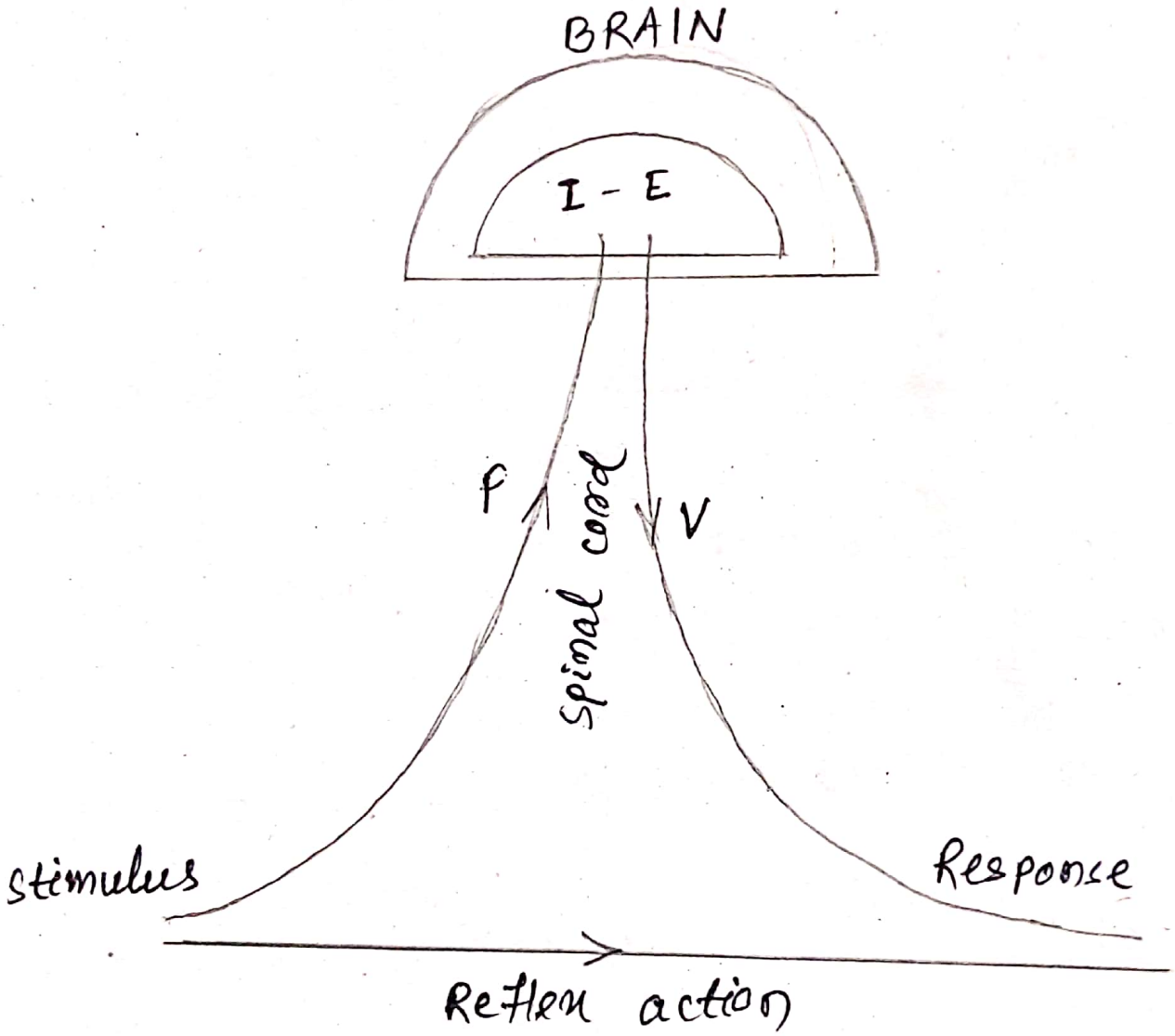
Diagramatic representation of PIEV Theory



P - perception
I - Intellection
E - Emotion
V - volition

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According to I.R.C recommendations, total reaction time is considered as 2.5 seconds.
or
However for design purposes, the total time required from perception to reaction (say application of breaks) is taken to be 2.5 seconds.

* Brake reaction time :- The brief interval between the perception of danger and the effective application of brakes is called brake reaction time.

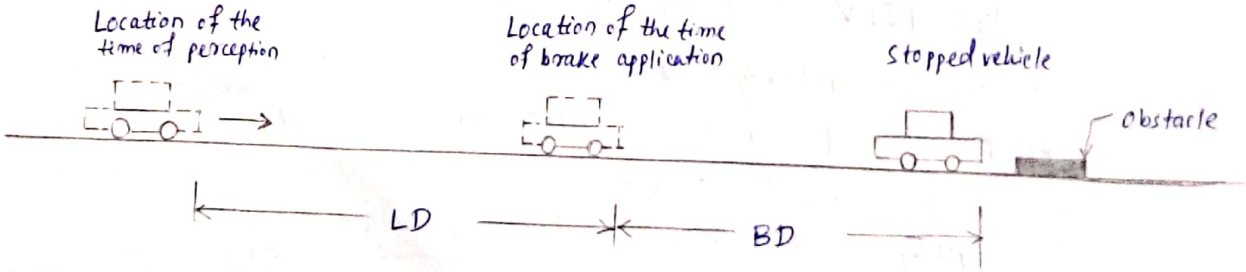
* Reaction time / Total reaction time :-
The sum of perception time and brake reaction time is called total reaction time or simply reaction time.

* Lag distance :- It is the distance travelled by a vehicle from perception of a situation to application of brakes by the driver. or The distance travelled by a vehicle from perception of a situation to application of brakes by the driver.

* Braking distance :-
It is the distance travelled by a vehicle just after application of brakes, either to bring the vehicle to a complete halt/stop or steer it away.

A vehicle moves further even after application of brakes since kinetic energy of the vehicle cannot be reduced to zero instantly.

Following are the factors affecting braking distance
- velocity of the travelling vehicle and its inertial force
- Brake efficiency of vehicle and type of braking system
- Surface friction or road surface characteristics
- Longitudinal slope or gradient of a road.



LD :→ Lag distance
BD :→ Braking distance

SSD = LD + B.D

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Lag distance / Reaction distance

During the total reaction time, 't' seconds the vehicle may be assumed to move forward with a uniform speed at which the vehicle has been moving and this speed may be taken as the design speed.

If 'v' is the design speed in m/sec and 't' is the total reaction time of the driver in seconds, then

Lag distance = vt metre

If the design speed is V $\frac{km}{h}$, then Lag distance = $Vt \times \frac{1000}{60 \times 60}$

$= 0.278 Vt$ metre

Braking distance

On level surface

Assuming a level road, the braking distance may be obtained by equating the work done by the frictional force (F) in stopping the vehicle & the kinetic energy of the vehicle moving at design speed.

The kinetic energy of the vehicle of mass 'm' in motion at a speed of 'v' m/s is $\frac{1}{2}mv^2$ or $\frac{Wv^2}{2g}$ [if weight of vehicle = W, $W = mg$]

$\frac{1}{2}mv^2 = f_s W l$

$= FXl$

$= Wfl$

where F = Frictional force developed

Force x distance = work done

$F = Wf$ where W = total weight in kg

f = co-efficient of friction or skid resistance (0.35 to 0.4)

l = braking distance in m

v = speed of vehicle $\frac{m}{sec}$

g = acceleration due to gravity $= 9.81 \frac{m}{sec^2}$

$\Rightarrow \frac{1}{2}\frac{W}{g}v^2 = Wfl$

$\Rightarrow \boxed{l = \frac{v^2}{2gf}}$

When v in $\frac{km}{hr}$ $= 0.278 V$ $\frac{m}{sec}$

$\therefore l = \frac{(0.287V)^2}{2 \times 9.81 f} = \frac{0.07728 v^2}{19.62 f}$ * Dividing 0.07728 in numerator & denominator

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$$\frac{0.077284}{0.077284} = \frac{V^2}{253.868\,f} \approx \frac{V^2}{254\,f}$$

$$\frac{19.62\,f}{0.077284}$$

$$\therefore l = \frac{V^2}{254f}$$

Stopping distance on level surface/road / Plane road

SSD or SD in m = Lag distance or Reaction distance + Braking distance

SD or SSD in m = L.D + B.D

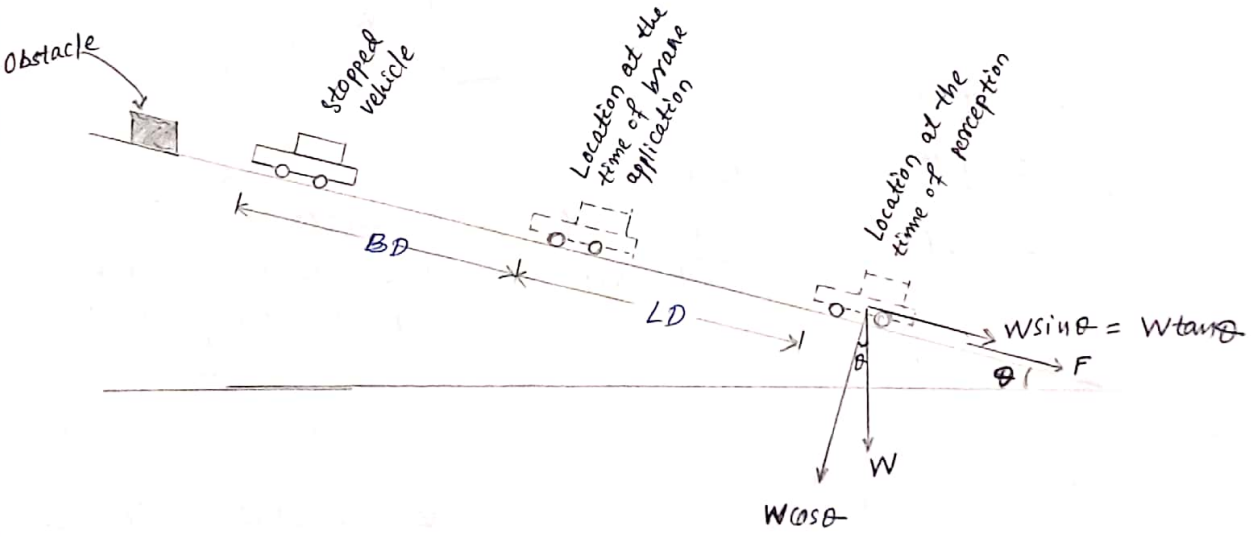
$$= 0.278\,Vt + \frac{V^2}{254f}$$

{ Value of 'f' ranging from 0.35 to 0.4

'V' in km/h

't' in second

SSD on a sloped road



When there is an ascending gradient of say, n% the component of gravity adds to the braking action and hence the braking distance is decreased.

The component of gravity acting parallel to the surface which adds to the braking force = $W\sin\theta = W\tan\theta = W\left(\frac{n}{100}\right)$

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Equating the kinetic energy and work done

$$\frac{1}{2}mv^2 = f_s WL + WL\tan\theta \qquad (f_s = f)$$

$$\frac{W}{g}\frac{v^2}{2} = f_s WL + WL\tan\theta$$

$$\frac{v^2}{2g} = f_s L + \tan\theta \quad \text{(cancelling 'W' on both sides)}$$

$$\frac{v^2}{2g} = L(f + \tan\theta)$$

$$L = \frac{v^2}{2g(f + \tan\theta)}$$

$\tan\theta = \frac{n}{100}$
$= +0.01n$ or $\frac{1}{n}$ in n

$$\therefore L = \frac{v^2}{2g(f + 0.01n)} \quad \text{— (A)}$$

Similarly, in descending gradient of $-n\%$, the braking distance increases, as the component of gravity now opposes the braking force.

$$\therefore L = \frac{v^2}{2g(f - 0.01n)} \quad \text{— (B)}$$

Hence the above eqns A & B, for SSD may now be written as

$$SSD = Vt + \frac{v^2}{2g(f \pm 0.01n)}$$

in 'm' if v in m/s

or

$$SSD = 0.278Vt + \frac{V^2}{254(f \pm 0.01n)}$$

if V is in $\frac{km}{hr}$

On any road stretch if the stopping sight distance for the design speed is not available, the speed should be restricted by installing suitable speed limit regulation sign alongwith warning signs.

However the option of speed restriction should be considered only as a temporary measure and wherever possible the stretch of the road

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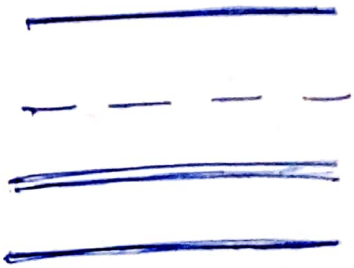
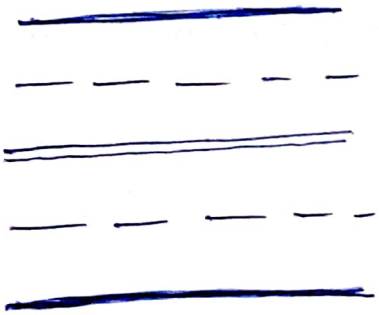
should be re-aligned or the obstruction to visibility removed so as to provide at least safe stopping sight distance for the design speed.



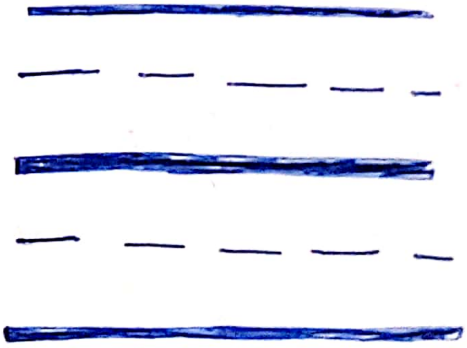
Single carriageway with 2 lane



Single carriageway with 3 lane



Single carriageway with 4 lanes



Duel carriageway with 2 lanes in each direction

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Pb Calculate the stopping sight distance for
(i) two lane highway having two way traffic.
(ii) single lane road having two way traffic.
Design speed = 40 km/hr, $f = 0.35$, $t = 3$ seconds

Soln:

Stopping distance $= Vt + \frac{V^2}{2gf}$ $\left\{ 40 \frac{km}{hr} = 11.11 \frac{m}{s} \right.$

$= (11.11 \times 3) + \frac{(11.11)^2}{2 \times 9.81 \times 0.35}$

$= 51.30$ m

(i) when there are two lane two way traffic = 51.30 m

(ii) when single lane two way traffic = $2 \times 51.30 = 102.6$ m

when V in km/hr

SD/SSD $= 0.278 Vt + \frac{V^2}{254f}$

$= (0.278 \times 40 \times 3) + \frac{40^2}{254 \times 0.35}$

$= 51.35$ m

(i) 51.35 m

(ii) $2 \times 51.35 = 102.7$ m

(i) stopping sight distance in one-way traffic lane and also in two-way traffic lane, should be equal to the stopping distance

(ii) On single lane road, when two-way movement of traffic is permitted then stopping sight distance should be equal to twice the stopping distance.

Pb calculate the stopping sight distance for
(a) two lane road having two-way traffic
(b) single lane road having two-way traffic.
The design speed of road is 50 $\frac{km}{hr}$. The coefficient of friction between the road surface and the tyres may be taken as 0.4 and the reaction time of the driver may be assumed as 3 seconds. The roadway is level.

Soln V in km/hr

SD = 66.30 m

(a) 66.30 m

(b) $2 \times 66.30 = 132.6$ m

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Pb// Calculate the stopping sight distance for a road for which the design speed is 40 km/hr. The brake efficiency is 40% and the reaction time of the driver may be assumed as 3 s.

Soln// Reaction time = 3 seconds

co-efficient of friction = ? brake efficiency = 40%.

~~[illegible]~~

$$SD = 0.278\,Vt + \frac{v^2}{254f}$$

$$= (0.278 \times 40 \times 3) + \frac{40^2}{254 \times 0.4} = 49.10\ m \simeq 50\ m$$

Pb// Calculate the min^m sight distance required to avoid a head on collision of two cars approaching from the opposite directions at 90 and 60 kmph. Assume a reaction time of 2.5 seconds, co-efficient of friction of 0.7 & a brake-efficiency of 50 percent for both the cars.

Soln// Reaction time = 2.5 seconds

Effective co-efficient of friction f, with 50% brake efficiency during stopping $= 0.5 \times 0.7 = 0.35$ ✓

1st car speed 90 $\frac{km}{hr}$ $V_1 = 90\ \frac{km}{hr}$

2nd " speed 60 $\frac{km}{hr}$ $V_2 = 60\ \frac{km}{hr}$

SSD for 1st car $= 0.278\,Vt + \frac{v^2}{254f} = 153.66\ m$

SSD for 2nd car $= 0.278\,Vt + \frac{v^2}{254f} = 82.194\ m$

) Total sight distance required to avoid head on collision of the two approaching cars $= SSD_1 + SSD_2$

$= 153.66 + 82.194$

$= 235.86 \simeq 236\ m$

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(3) Head-light sight distance :-

This is the distance visible to a driver during night driving under the illumination of the vehicle head light. The sight distance is critical at up-gradients and at the ascending stretch of the valley curves.

Pb later calculate the stopping sight distance on a highway at a descending gradient of 2% for a design speed of 80 kmph. Assume other data as per IRC recommendations.

Soln gradient is 2%, reaction time = 2.5 second, design coefficient of friction $f = 0.35$

$V = 80$ kmph, $n = -2\% = -0.02$

$$SSD = 0.278Vt + \frac{V^2}{254(f \pm 0.01n)}$$

$$= 0.278Vt + \frac{V^2}{254(f \pm 0.02f)}$$

$$= (0.278 \times 80 \times 2.5) + \frac{80^2}{254(0.02 \times 0.35)}$$

$$= (0.278 \times 80 \times 2.5) + \frac{80^2}{254(0.35 - 0.02)}$$

$$= 131.95\,m \simeq 132\,m$$

Pb The design speed of a road vehicle is 65 kmph, the friction coefficient is 0.36 and reaction time of driver is 2.5 sec. Calculate (i) Head light sight distance
(ii) Intermediate sight distance required for the road.

Soln $V = 65$ kmph, $f = 0.36$, $t = 2.5$ second

$$SSD = 0.278Vt + \frac{V^2}{254f}$$

$$= (0.278 \times 65 \times 2.5) + \frac{65^2}{(254 \times 0.36)}$$

$$= 91.38 \simeq 91.4\,m$$

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(i) SSD = 91.4m

(ii) 2SSD = 182.8 ≃ 183m

Pb Calculate the min^m non-passing sight distance on a highway at a descending gradient of 6%. Given the following data (i) Design speed = 80 kmph

(ii) Reaction time of driver = 2.5 s

(iii) coefficient of friction bet^n tyre & road surface = 0.4

Soln

$$0.278Vt + \frac{V^2}{254(f - 0.01n)}$$

$$= (0.278 \times 80 \times 2.5) + \frac{80^2}{254(0.4 - 0.06)}$$

$$= 129.7 \simeq 130\ m$$

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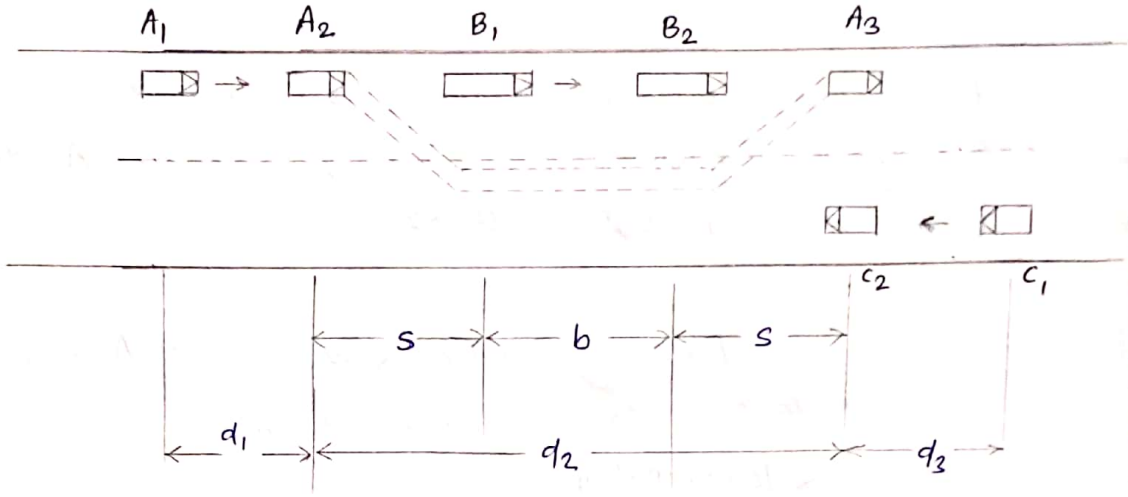
Over taking Sight Distance (OSD) / Passing Sight Distance (PSD) / Non-stopping Sight Distance



two-directional

Analysis of overtaking sight distance (OSD) on a two-way road

Simple overtaking process on a two-lane highway with two-way traffic movement /

vehicle 'A' travelling at the design speed $V \frac{m}{sec}$ or V kmph desires to overtake another slower vehicle B moving at a speed of $V_b \frac{m}{sec}$ or V_b kmph. The vehicle 'A' has to accelerate, shift to the adjacent right side lane, complete the overtaking manoeuvre and return to the left lane, before the on-coming vehicle 'C' approaches the overtaking stretch as shown....

The overtaking manoeuvre may be split up into three operations, thus dividing the overtaking sight distance, OSD into three(3) parts, d_1, d_2 and d_3

d_1 is the distance (m) travelled by the overtaking vehicle A during the reaction time 't' (sec) of the driver from position A_1 to A_2 before starting to overtake the slow vehicle B.

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d_2 is the distance (m) travelled by the vehicle A during the actual overtaking operation during T (secs) from position A_2 to A_3

d_3 is the distance (m) travelled by on-coming vehicle C during the actual overtaking operation of A during T (secs) from position C_1 to C_2

(2 lane undivided straight road)

Thus on a 2-lane road with 2-way traffic the overtaking sight distance, OSD = $d_1 + d_2 + d_3$ (m)

determination

Assumptions made in the analysis

Assumptions made to calculate the values of d_1, d_2, d_3 in mtr as given below,...

- the overtaking vehicle A is forced to reduce its speed from the design speed V (m/sec) to V_b (m/s) of the slow moving vehicle B and move behind it, allowing a space 's' (m), till there is an opportunity for safe overtaking operation.

or

d_1 = distance travelled by the vehicle 'A' (from position A_1 to A_2) during the time in which vehicle 'A' at speed V_b m/s decides whether or not, he should take over the slow moving vehicle B i.e perception time 't' seconds

$$d_1 = V_b t \text{ (m)}$$

The IRC suggests that this reaction time 't' of the driver may be taken as 2.0 seconds as an average value, as the aim of the driver is only to find an opportunity to overtake.

$$\therefore d_1 = 2V_b \text{ (m)}$$

- From position A_2, the vehicle A starts accelerating, shifts to the adjoining lane, overtakes the vehicle B, and shifts back to its original lane ahead of B in position A_3 during

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the overtaking time, T (sec)

The straight distance between position A_2 and A_3 is taken as d_2 (m), which is further splited into three parts,

$$d_2 = (s + b + s)\ m$$

→ The min^m distance between position A_2 and B_1 may be taken as the min^m spacing S (m) between the two vehicles while moving with the speed V_b (m/sec).

The min^m spacing between vehicles depends on their speed and is given by formula $\boxed{S = (0.7V_b + 6)\ m}$

→ The min^m distance between B_2 and A_3 may also be assumed equal to S(m).

If the overtaking time by vehicle A for the overtaking operation from position A_2 to A_3 is T (sec), the distance covered by the slow moving vehicle B travelling at a speed of V_b (m/s) = $b = V_b T$.

Thus the distance = $d_2 = (b + 2s)$

→ Now the time 'T' depends on speed of over taken vehicle B and the average acceleration 'a' (m/sec^2) for overtaking vehicle A

The overtaking time T (sec) may be calculated by equating the distance $d_2 = \left(V_b T + \frac{1}{2} a T^2\right)$ ....

using the general formula for the distance travelled by an uniformly accerating body with initial speed V_b $\frac{m}{sec}$ and 'a' is the average acceleration during overtaking in m/sec^2

$d_2 = b + 2s$

$= \left(V_b T + \quad 2s\right)$

or $b = V_b T$, $2s = \frac{aT^2}{2}$, $T = \sqrt{\frac{4s}{a}}$ where $s = (0.7V_b + 6)m$

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$$d_2 = (V_b T + 2s) \quad m$$

→ The distance travelled by vehicle 'C' moving at design speed V(m/s) during the overtaking operation of vehicle A i.e during time T(sec) is the distance d_2(m) between positions C_1 to C_2

Hence $d_3 = VT$ (m)

$= VT$ ✓

Thus $OSD/PSD = d_1 + d_2 + d_3$

$$= \boxed{(V_b t + V_b T + 2s + VT)\ m}$$

In kmph the above values may be written

$$\boxed{PSD/OSD = (0.28 V_b t + 0.28 V_b T + 2s + 0.28 VT)\ \frac{km}{hr}}$$

Here

V_b = initial speed of overtaking vehicle in $\frac{km}{hr}$

t = reaction time of driver = 2 sec

V = speed of overtaking vehicle or design speed in $\frac{km}{hr}$

$$T = \sqrt{\frac{4s}{a}}$$

$$= \sqrt{\frac{4(0.7V_b + 6)}{a}}$$

$$= \sqrt{\frac{4(0.7 \times 0.28 V_b + 6)}{a}}$$

$$= \sqrt{\frac{4(0.196 V_b + 6)}{a}}$$

$$= \sqrt{\frac{4(\simeq 0.2 V_b + 6)}{a}}$$

$$= \sqrt{\frac{4(0.2 V_b + 6)}{a}}$$

$$= \sqrt{\frac{4(0.2 V + 6)}{a}}$$

where V is the design speed in $\frac{m}{sec}$

$s = (0.7 V_b + 6)$ m

$= (0.7 \times 0.28 V_b + 6)$

$= (0.196 V_b + 6)$

$= (\simeq 0.2 V_b + 6)$

$= (0.2 V_b + 6)$ m

* V is the design speed in kmph.

* In case the speed of overtaken vehicle (V_b or V_b) is not given, the same may be assumed as 4.5 $\frac{m}{sec}$ or 16 $\frac{km}{hr}$, less than the design speed of the highway. Therefore $V_b = (V - 4.5)\frac{m}{sec}$ or $V_b = (V - 16)\frac{km}{hr}$

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$T = \sqrt{\frac{4S}{a}}$ sec if 'a' is in $\frac{km}{hr/sec}$ $\left(\frac{m}{sec^2}\right)$ ← initial

$= \sqrt{\frac{4 \times S}{0.278\ A/sec}}$ $= \frac{1000}{3600}$ /sec

$= \sqrt{\frac{14.388\ S}{A}} \simeq$

$= \sqrt{\frac{14.4\ S}{A}}$

where A is average acceleration during overtaking, kmph/sec

(*) when, the road is unidirectional & there is no interference of the opposite traffic, then d_3 becomes zero, & $P.S.D = d_1 + d_2$

Criteria for overtaking Sight Distance Requirement on Highways

It is desirable that adequate sight is available on most of the road stretches such that the vehicles travelling at the design speed can overtake slow vehicles at the earliest opportunity.

→ On road stretches with two way traffic movement, the min^m overtaking distance should be $(d_1 + d_2 + d_3)$ where overtaking is not prohibited.

→ on divided highway and on roads with one way traffic regulation, the overtaking distance need only be $(d_1 + d_2)$ as no vehicle is expected from the opposite direction.

→ On divided highways with four or more lanes, it is not essential to provide the usual OSD.

→ On horizontal curves the overtaking sight distance requirements cannot always be fulfilled especially on sharp curves, if the safe overtaking sight distance requirements are high. In such cases overtaking should be prohibited by regulatory signs.

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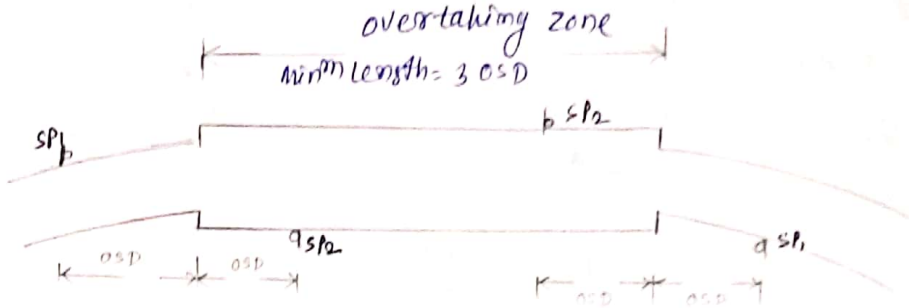
Criteria for overtaking zones



SP_1 → Sign post "overtaking zone ahead"

SP_2 → Sign post "End of overtaking zone"

$OSD = 3(d_1 + d_2)$ for one way traffic

$OSD = 3(d_1 + d_2 + d_3)$ for two way traffic

It is desirable to construct highways in such a way that the length of road visible ahead at every point is sufficient for safe overtaking. This is seldom practicable and there may be stretches where the safe overtaking distance cannot be provided.

→ In such zones where overtaking or passing is not safe Or is not possible, sign posts should be installed indicating "No passing" or "overtaking prohibited" before such restricted zones start.
However overtaking opportunity for vehicles moving at design speed should be given at as frequent intervals as possible. These zones which are meant for overtaking are called "overtaking zones"

→ Figures above shows an overtaking zone with specification for the positions of the sign posts. The width of carriageway and the length of overtaking zone should sufficient for safe overtaking. Sign posts should be installed at sufficient distance in advance to indicate the start of the overtaking zones. Similarly the end of the overtaking zones should also be indicated by appropriate sign posts installed ahead at similar distances specified above.

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The minimum length of overtaking zone should be 3 times the safe overtaking sight distance i.e OSD.

(A) Thus min^m length of overtaking zone should be $3(d_1+d_2+d_3)$ for two way roads and $3(d_1+d_2)$ for one way roads.

(A) It is desirable that the length of overtaking zones is kept 5 times the overtaking sight distance i.e 5(OSD).

Pb The speed of overtaking & overtaken vehicles are 70 and 40 kmph, respectively - on a two way traffic road. The average acceleration during overtaking may be assumed as $0.99 \frac{m}{s^2}$.

(a) calculate safe overtaking sight distance

(b) what is the min^m length of overtaking zone?

(c) Draw a neat sketch of the overtaking zone & position of the sign posts.

Soln Speed of overtaking vehicle, V = 70 kmph

$$v = \frac{70}{3.6} = 19.44 \text{ m/sec}$$

or

$$\frac{70000}{3600} = 19.44 \text{ m/sec}$$

Speed of overtaken vehicle V_b = 40 kmph, $V_b = \frac{40}{3.6} = 11.11 \frac{m}{sec}$

Average acceleration during overtaking $a = 0.99 \frac{m}{sec^2}$

(a) OSD for 2 way traffic

$$= d_1+d_2+d_3$$

$$= \left(V_b t + V_b T + 2s + vT\right) m$$

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$d_1 = V_b t = 11.11 \times 2 = 22.2\ m$

$d_2 = S = (0.7V_b + 6) = (0.7 \times 11.11 + 6)$

$= 13.77$

$$T = \sqrt{\frac{4S}{a}} = \sqrt{\frac{4 \times 13.77}{0.99}} = \sqrt{55.63636363636}$$

$= 7.46\ sec$

$d_2 = V_b T + 2S$

$= (11.11 \times 7.46) + 2(13.77)$

$= 82.89 + 27.54$

$= 110.43\ m$

$d_3 = VT = 19.44 \times 7.46$

$= 145.022\ m$

$OSD = d_1 + d_2 + d_3$

$= 22.2 + 110.43 + 145.022$

$= 277.652 \cong 278\ m$

(b) $Min^m = 3OSD$

$= 3 \times 278 = 834\ m$

Desirable $= 5OSD = 5 \times 278 = 1390\ m$

(c)

BEFORE GOING TO PROBLEM → should complete the Table work in Page 45, 46...

Pb. Calculate the safe overtaking sight distance for a design speed of 96 kmph. Assume all other data suitably.

Ans: V = speed of overtaking vehicle

∴ speed of overtaken vehicle = 96 − 16 = 80 kmph (assumed)

reaction time $t = 2$ sec

A = Average acceleration assumed = 2.56 kmph/sec

$= 0.72\ m/sec^2$

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96 km/hr = $\frac{96 \times 1000}{3600}$ = 26.67 m/sec

80 km/hr = $\frac{80 \times 1000}{3600}$ = 22.23 m/sec

$d_1 = V_b t = 22.23 \times 2 = 44.46$ m

$d_2 = b + 2s = V_b T + 43.122$ $\left(\because b = V_b T,\ 2s = 2(0.7V_b + 6)\right.$

$= \left(22.23 \times 10.95\right) + 43.122$ $= 2(0.7 \times 22.23 + 6)$

$= 286.54$ m $= 43.122$

$T = \sqrt{\frac{4s}{a}} = \sqrt{\frac{4 \times 21.561}{0.72 \text{ (from table)}}}$

$= 10.95$ sec

$d_3 = VT = 26.67 \times 10.95 = 292.03$ m

$\therefore OSD = d_1 + d_2 + d_3 = 44.46 + 286.54 + 292.03$

$= 623.03$ m

Maxm overtaking acceleration at different speeds

Speed		Maxm overtaking acceleration	
V, Kmph	V, m/sec	A, kmph/sec	a, m/sec^2
25	6.94	5.0	1.41
30	8.33	4.8	1.30
40	11.10	4.45	1.24
50	13.88	4.0	1.11
65	18.05	3.28	0.92
80	22.22	2.56	0.72
100	27.78	1.92	0.53

- About 9 to 14 seconds of time duration is taken for completion of an overtaking manoeuvre (ಓವರ್ ಟೇಕಿಂಗ್)
- The time taken for initial manoeuvre is about $\frac{1}{3}$ the total time required for overtaking. The remaining $\frac{2}{3}$ of the time is accounted for by the opposing vehicle that is travelling at design speed.
- OSD is the unobstructed distance between two points on the carriage way which are 1.22m above the road surface. One point represents the position of the drivers eye & the other point represents

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the height of the object.

Safe overtaking sight Distance on 2 lane highway for various Design speed.

Speed km/hr	Time component in seconds			Safe overtaking sight Distance (OSD) m
	For overtaking manoeuvre	For opposing vehicle	Total	
40	9	6	15	165
50	10	7	17	235
60	10.8	7.2	18	300
65	11.5	7.5	19	340
80	12.5	8.5	21	470
100	14	9	23	640

Curves

The geometrical arcs provided at the change in alignment or gradient of a road are known as curves.
curves play an important role in the geometrical design of a road. Therefore, they should be so designed as to provide safety and convenience to the traffic.

Necessity of providing curves

The necessity of providing curves at the change in alignment or gradient of a road arises due to the following reasons:-

(i) To lay the road according to topography of the country
(ii) To provide access to the particular place.
(iii) To avoid costly land
(iv) To avoid excessive cutting or filling
(v) To avoid certain religious, monumental or such other important structures.
(vi) To make use of the existing bridge.

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(vii) To keep the driver alert by making change in the direction of road.

(viii) To make use of existing right of way

(ix) To avoid mental strain caused by the monotony of continuous journey along a straight route.

(x) To check the tendency of the driver to increase the speed of his vehicle beyond the safe limits on straight routes.

Advantages of curves

(i) They provide gradual change in direction

(ii) They provide gradual change in gradient

(iii) They provide easy turning of vehicles.

(iv) They provide comfort to the passengers

(v) They increase the life of vehicles.

(vi) They help to keep the speed of vehicles within limits

(vii) The help to keep the drivers alert due to change in the direction of road.

(viii) They help to avoid mental strain caused by the monotony of continuous journey along straight routes.

(ix) They help in providing adequate visibility to the traffic.

(x) They help in providing safe & economical alignment of a road.

Factors affecting the design of curves

The following are the factors which play an important role in the design of a road curve:—

(a) The design speed for the road

(b) The safe allowable coefficient of friction in the lateral direction between the tyres & the road surface.

(c) Max^m allowable superelevation.

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(d) permissible centrifugal ratio

(a) The Design speed for the road

The relation between superelevation, co-efficient of lateral friction, the design speed & radius of curvature of the road is given by the following equation :-

$$\text{i.e} \qquad e + f = \frac{v^2}{gR}$$

where

e = The superelevation in m/m

f = co-efficient of lateral friction between the tyres & the road surface.

v = The design speed for the road in $\frac{m}{sec}$

R = The radius of the curve in mtr.

The design speed for the road affects the curvature & hence the design of curves.

(b) Safe allowable co-efficient of friction in lateral direction

The safe allowable co-efficient of lateral friction between the tyres & the road surface generally depends upon the speed of the vehicle; type & condition of tyres, condition of the road surface & the effect of the weather.

(*) For design of curve, the co-efficient of friction 0.2, with a factor of safety of 1.33 may be considered so that the working value of co-efficient of lateral friction be taken as 0.15 in above eqn to design the curve.

(c) Maxm allowable superelevation

The maximum rate of superelevation on Indian highways as recommended by I.R.C is 7%.

This value is considered for finding the radius of curvature in the above eqn while designing the curve.

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(d) Permissible Centrifugal ratio

The ratio between centrifugal force & the weight of the vehicle is known as centrifugal ratio (C.R)

The expression $\frac{v^2}{gR}$ in the above mentioned eqn is known as centrifugal ratio $C.R = \frac{f}{W} = \frac{Wv^2}{gRW} = \frac{v^2}{gR}$

Allowable value of C.R in road = $\frac{1}{4}$, railways = $\frac{1}{8}$

→ The max^m permissible value of centrifugal ratio for comfortable travel on rural highway is taken as $\frac{1}{4}$ or 0.25

Thus, with the min^m allowable superelevation (7% / 0.07) and safe coefficient of lateral friction (0.15), the sum of 'e' and 'f' or the value of centrifugal ratio from the above mentioned eqn works out 0.22 which is less than the prescribed value of 0.25.

Hence, the value of 'e' and 'f', assumed for the design, ensure adequate comfort to the passengers.

Now, from the relation $\frac{v^2}{gR} = 0.22$, the required radius of curvature (R) can be calculated after fixing the value of the design speed, which shall be used in the design of curve.

Types of curves

Curves on highways have been divided into the following two main classes.

1. Horizontal curves
2. vertical curves

1. Horizontal curves

The curves provided at the turning points in the alignment (in the horizontal plane) of a road are known as horizontal curves.

These roads are provided to achieve gradual change in the direction of alignment of a road in the horizontal plane.

The min^m radius of a horizontal curve depends on the permissible design speed for the road or in other words, on the category of the road in that area. The values of min^m radii for various categories of roads in different areas, as recommended by IRC are given in table ... (1)

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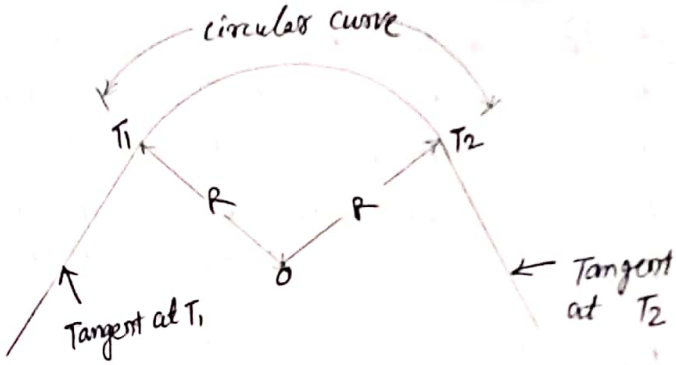
Horizontal curve used in the alignment of highways are further divided into following types:-

(a) Simple curve
(b) Compound curves
(c) Reverse curves
(d) Transition curves
(e) Lemniscate curves

(a) Simple curve

A circular curve consisting of a single arc of uniform radius connecting two tangents is called simple curve.



This curve is expressed in terms of degree of the curve, which is the angle in degree subtended at the centre by a chord of 30 m length.

This type of curve is suitable for slow moving traffic & for radius. Such a curve may lie within two tangents lengths or two transition curves.

Table - 1

Classification of Roads	plain terrain		Rolling Terrain		Mountainous terrain				steep terrain			
					Area not affected by snow		Snow bound areas		Area not affected by snow		Snow bound areas	
	Ruling Min^m	Abs Min^m	Abs	Min^m Ruling Min^m	Ruling Min^m	Abs Min^m	Ruling Min^m	Abs Min^m	Ruling Min^m	Abs Min^m	Ruling Min^m	Abs Min^m
NH & SH	360	230	230	155	80	50	90	60	50	30	60	33
MDR	230	155	155	90	50	30	60	33	30	14	33	15
ODR	155	90	90	60	30	20	33	23	20	14	23	15
VR	90	60	60	45	20	14	23	15	20	14	23	15

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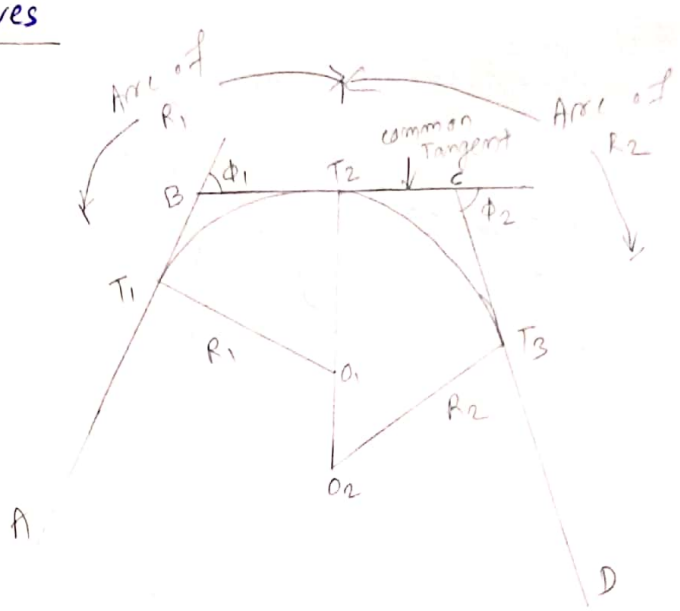
(b) Compound curves

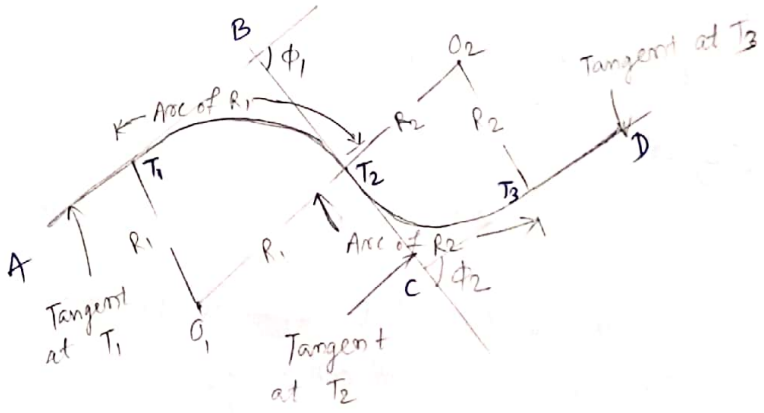


When a circular curve consisting of series of two or more simple curves of different radii, which turn in the same direction is called a compound curve.

In this curve, the adjacent simple curve (T_1T_2 & T_2T_3) have a common tangent (BC at T_2) & they lie on the same side of the curve.

This type of curve is used when compelled by topography of the area in ordered to avoid cutting through hard rock, heavy cutting or filling etc.

(c) Reverse curve/serpentine curve



A circular curve consisting of two simple curves of same or different radii in the opposite direction is called a reverse or serpentine curve

In this curve, the adjacent simple curves (T_1T_2 & T_2T_3) have a common tangent (BC at T_2) and their centres lie on opposite sides of the curve.

(*) This type of curve is oftenly used in the alignment of a hill road.

(d) Transition curve/spiral curve/easement curve

The curve having its radius varying gradually from an infinite to a finite value equal to that of the circular curve to be connected

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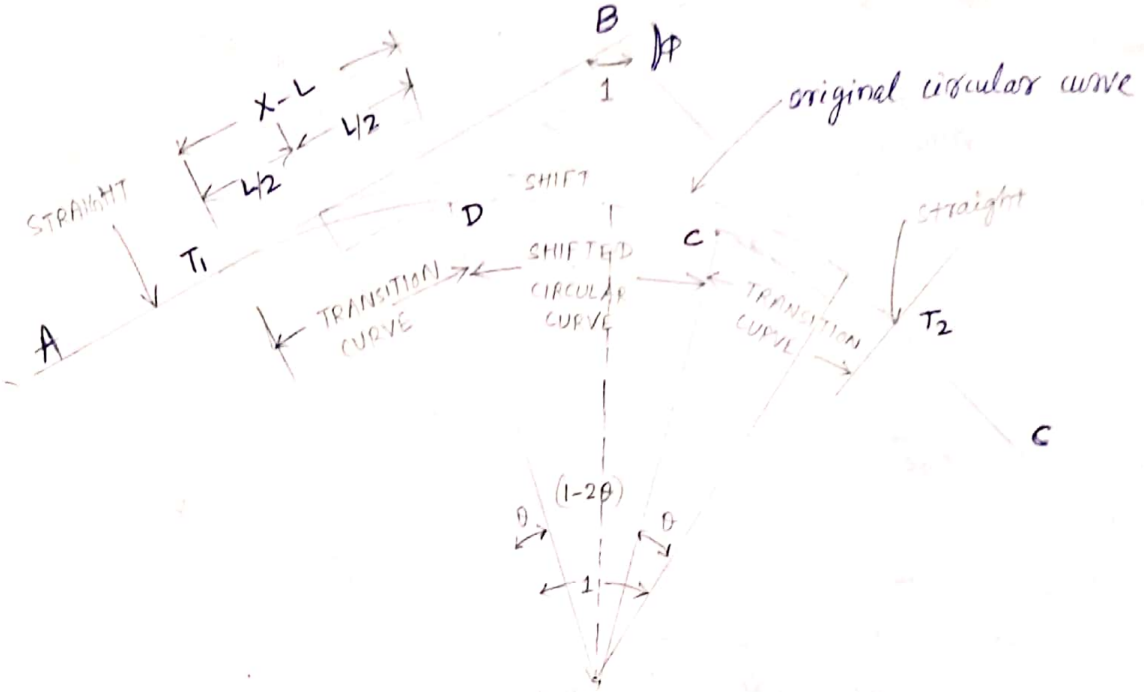

or vice versa is known as transition, spiral or easement curve. This type of curve is commonly introduced on highways between a tangent and a circular curve or between the circular curve & a tangent to provide ease and gradual change in direction of a road alignment.

Objects

(i) To provide gradual change in the radius of curvature from infinite value at the tangent to the radius of circular curve to be introduced & vice-versa.

(ii) To provide smooth entry of vehicle from a straight portion to a curved portion of the road so as to avoid discomfort to the passengers which may otherwise cause due to sudden increase of curvature from a straight to a circular path.

(iii) To permit gradual application of the superelevation & thus reducing shocks on the vehicle.

(iv) To permit gradual application of extra widening of carriageway needed at the horizontal curves.

(v) To minimise wear on pavements.

(vi) To provide safety to the vehicular traffic.

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Essential requirement of a transition curve

(i) Its radius of curvature should vary gradually from infinite value at the straight to a finite value at the circular curve.

(ii) It should meet the straight and the circular curve tangentially.

(iii) The length of transition curve should be such that the designed superelevation is attained at or before its junction with the circular curve.

(iv) The rate of increase of curvature and that of superelevation should be same.

(*) The IRC has recommended the use of spiral transition curve in the horizontal alignment of highways.

Length of transition curve

Min^m length of the transition curve should be determined from the following two considerations and the larger of the two values be adopted for design purposes :-

(i) The rate of change of centrifugal acceleration should not cause discomfort to drivers. From this consideration, the length of transition curve is given by the equation

$$L_s = \frac{0.0215\, V^3}{CR}$$

Where

L_s = Length of transition in mtrs

V = speed in km/h

R = Radius of circular curve in mtrs

$C = \frac{80}{75+V}$ (subject to a max^m of 0.8 and min^m of 0.5)

(ii) The rate of change of superelevation (i.e the longitudinal grade developed at the pavement edge compared to through grade along the centre line) should be such as not to cause discomfort to passengers or to make the road appear unsightly. Rate of change should not be steeper than 1 in 150 for roads in plain and rolling terrain, and 1 in 60 in mountainous and steep terrain. On this basis, min^m length of transition is found out from the relations...

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(a) For plain & Rolling Terrain

$$L_s = \frac{2.7V^2}{R}$$

(b) For mountainous and steep Terrain

$$L_s = \frac{1.0V^2}{R}$$

On the bases of above consideration, the min^m transition lengths for different speeds and curve radii as per recommendation of IRC are given below in table

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Minimum Transition Lengths for different Speeds and Curve Radii

Plain and Rolling Terrain							Mountainous and Steep Terrain					
Curve Radius (R) (metres)	Design Speed (km/h)						Curve Radius (R) (metres)	Design Speed km/h				
	100	80	65	50	40	35		50	40	30	25	20
45					NA	70	14				NA	30
60				NA	75	55	20				35	20
90				75	50	40	25			NA	25	20
100			NA	70	45	35	30			30	25	15
150			80	45	30	25	40		NA	25	20	15
170			70	40	25	25	50		40	20	15	15
200		NA	60	35	20	20	55		40	20	15	15
240		90	50	30	20	NR	70	NA	30	15	15	15
300	NA	75	40	25	NR		80	55	25	15	15	15
360	130	60	35	20			90	45	25	15	15	NR
400	115	55	30	20			100	45	20	15	15	
500	95	45	25	NR			125	35	15	15	NR	
600	80	35	20				150	30	15	15		
700	70	35	20				170	25	15	NR		
800	60	30	NR				200	20	15			
900	55	30					250	15	15			
1000	50	30					300	15	NR			
1200	40	NR					400	15				
1500	35						500	NR				
1800	30											
2000	NR											

Note: In the table, NA stands for not applicable and NR for Transition not required.

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Vertical curves

The curves provided at change of gradient (in vertical plane) of a road are called vertical curves.

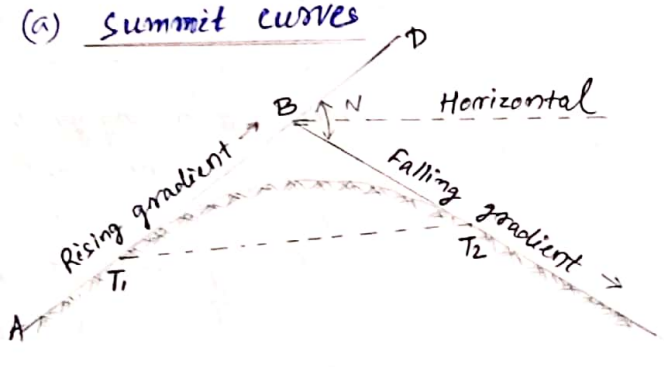
Vertical curves are provided to achieve the following objects:-

(i) To provide gradual change in grade
(ii) To provide safety and adequate visibility to the traffic
(iii) To provide comfort to the passengers

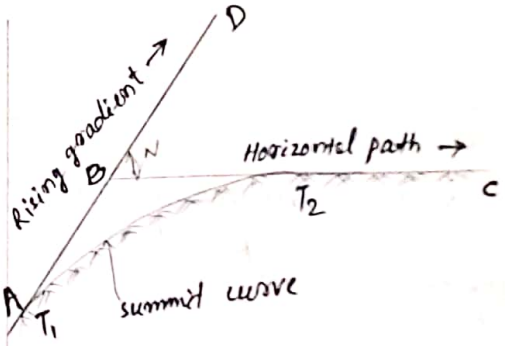
Vertical curves are further divided into following two types..

(a) Summit curves/spur curves
(b) Valley curves

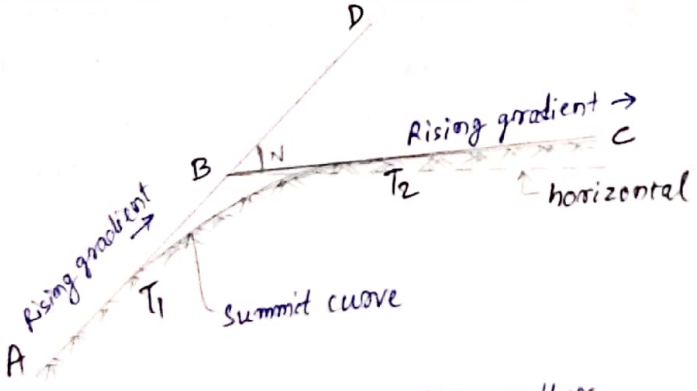
(a) Summit curves



A Rising gradient intersecting a falling gradient



A rising gradient meeting a horizontal path



A rising gradient meeting another rising gradient

The vertical curves having their convexity upward are known as **summit or spur curves**

⊛ This type of vertical curve is provided when a rising gradient intersects a falling gradient, a rising gradient meets another rising gradient or a rising gradient meets a horizontal path.... as shown in alongside figures respectively.

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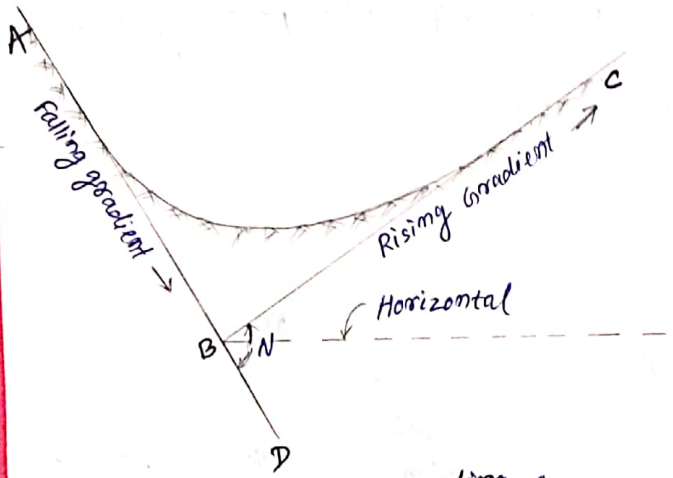
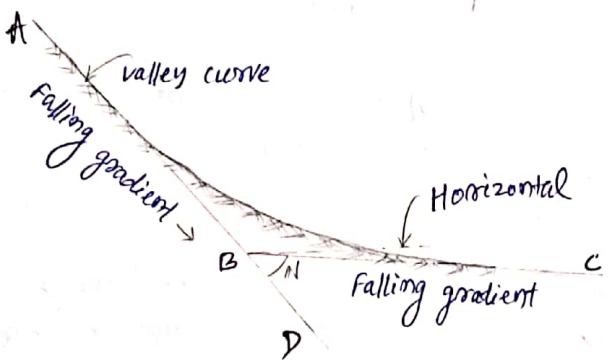
The length of summit curves is governed by the choice of sight distance whether stopping sight distance or the overtaking sight distance. On hill roads, application of the overtaking sight distance may not be practicable except at a very high cost. Hence, this criterion should not be enforced except in very easy terrain.

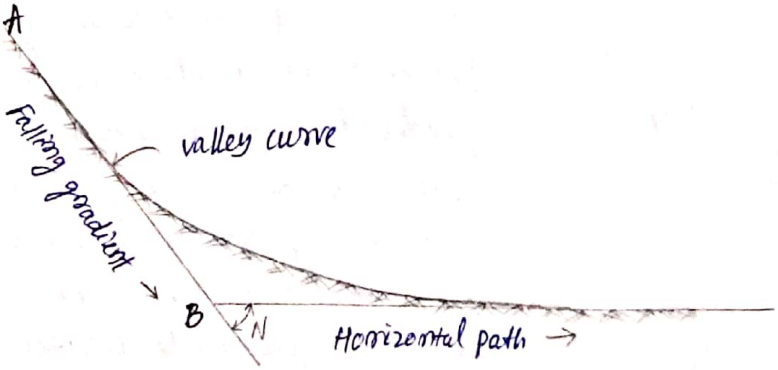
(b) valley curves / sag curves



A Falling Gradient intersecting a rising gradient



A falling Gradient meeting another falling Gradient.



A falling Gradient meeting a Horizontal path

The vertical curves having their convexity downward are called valley / sag curves

These curves are provided when a falling gradient intersects a rising gradient, a falling gradient meets another falling gradient & a falling gradient meets a horizontal path as shown alongside.

⊛ Valley curves should be designed as square parabolas. Their length should be such that for night travel the head light beam distance is the same as the stopping sight distance.

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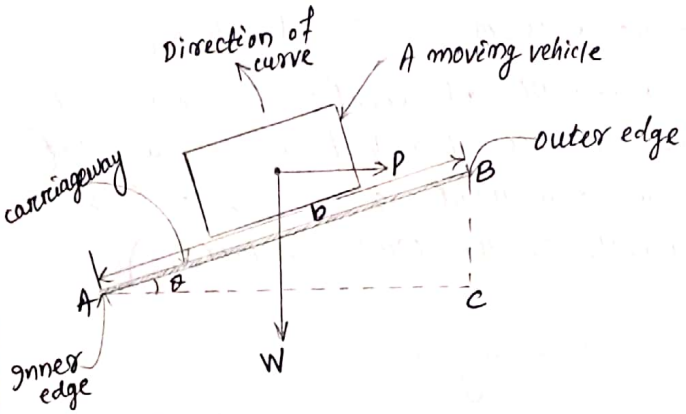
Super elevation

The inward transverse inclination provided to the cross-section of the carriageway at horizontal curved portion of a road is called **superelevation, cant or banking.**

It is expressed as the ratio of elevation of outer edge above the inner edge to the horizontal width of carriageway or as the tangent of the angle of slope of the road surface. It is generally denoted by 'e' or 'S.E'



Centrifugal Force

In physics, centrifugal force is the force that makes objects move outwards when they are spinning around something or travelling in a curve.

⊛ centrifugal force is the out-ward pushing force felt by bodies moving in a circular motion.

superelevation, $e = \frac{BC}{AC} = \tan\theta$

In practice, the value of 'e' is very small. The max^m superelevation allowed is 1 in 15 $\theta = \tan^{-1}\left(\frac{1}{15}\right) = 3^0 48' 50.67'' \simeq 4^0$

Therefore, the value of $\tan\theta$ would be practically equal to $\sin\theta$.

Hence, $e = \tan\theta = \sin\theta = \frac{BC}{AB}$

The ratio of elevation of the outer edge above the inner edge to the width of carriageway.

⊛ Thus superelevation of 1 in 20 means that the outer edge will be raised by $\frac{1}{20} \times 7 \times 100 = 35$ cm above the inner edge in 7m width of carriageway.

Objects of providing superelevation

(i) To counteract the effect of centrifugal force acting on the moving vehicle to pull the same outward on a horizontal curve.

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(ii) To help a fast moving vehicle to negotiate a curved path without overturning and skidding.

(**skidding**:- The linear motion of vehicle with its wheel locked on a road surface is known as skidding.

→ In skidding, the distance travelled along the road is more than the movement of wheels' circumference. It is due to application of brakes & provision of improper superelevation.

Slipping:- The revolving of wheels without any linear motion along the road surface is called slipping.

→ In slipping, the movement of wheel's circumference is more than the distance travelled along the road as in case of a muddy road. It is due to lack of friction between wheels & the road surface.)

(iii) To ensure safety to the fast moving traffic

(iv) To prevent damaging effect on the road surface due to improper distribution of load.

Advantages of providing Superelevation

(i) It permits running of vehicle at high speed on a curved path as on a straight path without any danger of overturning and thus results into increased volume of traffic.

(ii) It provides more or less even distribution of load on wheels & hence uniform stress is offered on the foundation which results into less wear on wheel tyres and springs as well as economy in maintenance cost of the road.

(iii) It also helps to keep the vehicles to their proper side on the pavement and thus prevents collision of vehicles moving in opposite directions on a curved portion of the road.

(iv) It provides drainage of the whole width of road towards the inner side. Thus, there is no necessity of providing side drain on the outer side of the road.

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3.0 ROAD MATERIALS

3.1 Subgrade Soil :-

→ Soil is an accumulation or deposit of earth material derived naturally from the disintegration of rocks or vegetation, that can be excavated readily with power equipment in field or disintegrated by gentle mechanical means in the laboratory.

→ The supporting soil beneath pavement & its special under courses is called subgrade.

→ Compacted subgrade is the soil compacted by controlled movement of heavy compaction.

Aggregate :-

→ It is collective term for mineral materials such as sand, gravel & crushed stone that are used with a binding medium (such as water, portland cement etc) to form compound materials (such as bituminous concrete & portland cement concrete).

→ By volume, aggregate generally accounts for 92 to 96% of bituminous concrete & 70 to 80% of portland cement concrete.

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Binder Materials

→ The bituminous binder used in pavement construction include both bitumen & tar.

→ Bitumen is a petroleum product obtained by the distillation of petroleum.

→ Tar is obtained by destructive distillation of coal or wood.

→ When bitumen contains some inert materials, it is known as asphalt.

3.2 Function of soil as highway Subgrade :-

→ Stability

→ Incompressibility

→ Permanency of strength

→ Good drainage

→ Ease of compaction

→ Minimum changes in volume & stability under adverse condition of weather & ground water

3.3 California Bearing Ratio :-

→ This method is developed as method of classifying & evaluating soil subgrade & base course material for flexible pavements.

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→ It is an empirical test to determine the material properties for pavement design.

→ It is a penetration test wherein a standard piston, having area 19.62 cm^2 (or 50 mm dia.) is used to penetrate the soil at a standard rate of 1.25 mm/min.

→ The pressure upto a penetration of 12.5 mm & its ratio to the bearing value of a standard crushed rock is termed as CBR.

→ In most cases, CBR decreases as penetration increases.

→ The ratio at 2.5 mm penetration is used as CBR.

→ In some cases, the ratio at 5mm may be greater than that at 2.5mm. If this occurs, the ratio at 5mm should be used if confirmed by repeating the test.

→ CBR test is measure of resistance of a material to penetration of standard plunger under controlled density & moisture condition. It can be performed in re-moulded or undisturbed specimen in laboratory. It is extensively used for field correlation of flexible pavement thickness requirement.

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Procedure :-

→ CBR apparatus consist of a mould 150mm dia with a base plate & collar, a loading frame & dial gauge for measuring the penetration value & expansion on soaking.

→ The specimen in the mould is soaked in water for 4 days & the swelling & water absorption values are noted.

→ Load is applied on the sample by standard plunger with dia. of 50mm at the rate of 1.25 mm/min. A load penetration curve is drawn.

→ The loads values on standard crushed stones are 1370 kg (70 kg/cm²) & 2055 kg (105 kg/cm²) at 2.5mm & 5mm penetration respectively.

→ CBR value is expressed as percentage of actual load causing the penetration of 2.5mm or 5mm to the standard loads mentioned above.

$$CBR = \frac{\text{Load carried by specimen}}{\text{load carried by standard specimen}} \times 100$$

→ Two values of CBR will be obtained. If the value of 2.5mm is greater than that of 5mm penetration, the former is adopted.

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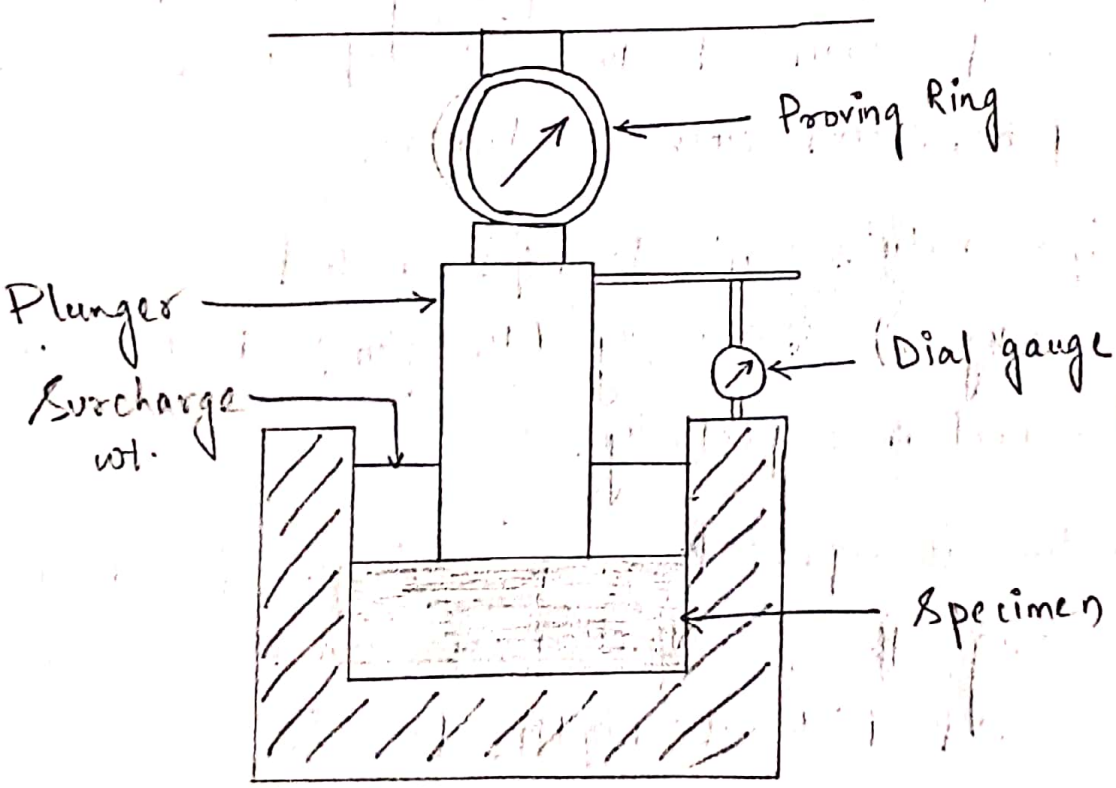
→ If the CBR value obtained from test at 5mm penetration is higher than that at 2.5mm, then the test is to be repeated for checking.

→ If the check test again gives similar results, then higher value obtained at 5mm penetration is reported as CBR value.

→ The avg. value of CBR of three test specimens is reported as CBR value of sample.



→ If the specimen has surface irregularities, the initial portion of curve may have concavity upwards. In that case, a tangent is drawn to the curve at the point of greatest slope. This tangent plus the convex portion of original curve is the corrected

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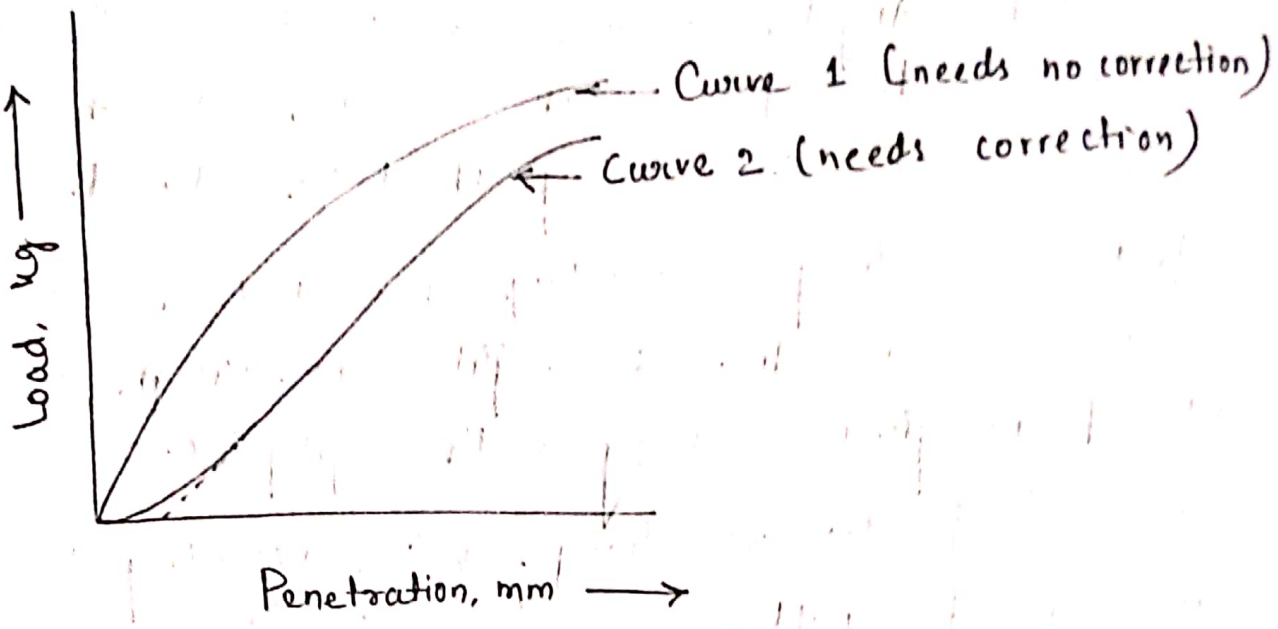
curve., with origin moved to the point where the tangent cuts the x-axis.



3.4 Testing Aggregates

1. Abrasion Test:

→ It is carried out to test the hardness property of aggregate & to decide whether they are suitable for different construction work.

This test are of following types:-

a) Los Angeles Abrasion test
b) Deval Abrasion test
c) Dorry Abrasion test

→ Los Angeles Abrasion test is preferred one for carrying out hardness property.

→ The principle of los Angeles abrasion test is to find the percentage wear due to relative rubbing

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action between the aggregates & steel balls used as abrasive charges.

→ The los Angeles machine consist of circular drum of internal dia. of 700mm & length 520mm mounted on a horizontal axis enabling it to be rotated.

→ An abrasive charge consisting of cast iron spherical balls of 48mm dia. & weight 340 - 445gm is placed in cylinder along with aggregates.

→ The no. of abrasive charges varies according to the grading of sample.

→ The quantity of agg. to be used depends upon the gradation & usually ranges from 5-10 kg.

→ The cylinder is then locked & rotated at the speed of 30-33 rpm for a total of 500-1000 revolutions depending upon gradation of aggregates.

→ After specified revolutions, the material is sieved through 1.7mm sieve & passed fraction is expressed as percentage total wt. of sample.

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$$\text{Abrasion Value} = \frac{\text{Wt. of aggregate retained on 1.7mm Sieve}}{\text{Total wt. of agg taken}} \times 100$$

→ This value is called Los Angeles Abrasion Value. A max. value of 40% is allowed for WBM base course in Indian conditions.

→ For bituminous concrete, a maximum value of 35% is specified.

2. Impact Test

→ This test is carried out to evaluate the resistance to impact of aggregate or toughness of aggregate.

→ Aggregate passing 12.5mm IS sieve & retained on 10mm IS sieve is filled in a cylindrical steel cup of internal diameter 10.2 cm & depth 5cm which is attached to a metal base of impact testing machine.

→ The material is filled in 3 layers where each layer is tamped for 25 no. of blows.

→ Metal hammer of weight 13.5 to 14 kg is arranged to drop with a free fall of 38 cm by vertical guides & the test specimen is subjected to 15 no of blows.

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- The crushed aggregate is allowed to pass through 2.36mm IS sieve. The impact value is measured as percentage of aggregates passing sieve (W_2) to the total weight of sample (W_1).

$$\text{Aggregate impact Value} = \frac{W_2}{W_1} \times 100$$

→ Aggregates to be used for wearing course, the impact value should not exceed 30%. For bituminous macadam the maximum permissible value is 40%.

3. Soundness Test

→ This test is carried out to study the resistance of aggregates to weathering action, by conducting accelerated weathering test cycles.

→ The porous aggregates subjected to freezing & thawing are likely to disintegrate prematurely. To ascertain the durability of such aggregates, they are subjected to an accelerated soundness test.

→ Aggregates of specified size are subjected to cycles of alternate wetting in a saturated solution of either sodium sulphate or magnesium sulphate for 16-18 hours & then dried in oven at 105°c - 110°c to a constant weight.

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→ After 5 cycles, the loss in wt. of aggregates is determined by sieving out all undersized particles & weighing. The loss in weight should not exceed 12% when tested with sodium sulphate & 18% with magnesium sulphate solution.

4. Crushing Strength Test

→ Aggregate crushing value provides a relative measure of resistance to crushing under gradually applied crushing load.

→ The test consist of subjecting the specimen of aggregate in standard mould to a compression test under standard load conditions.

→ Dry aggregates passing through 12.5mm sieve and retained on 10mm sieve are filled in a cylindrical measure of 11.5 cm dia. & 18cm ht. in three layers.

→ Each layer is tampered 25 times with standard tamping rod.

→ The test sample is weighed & placed in the test cylinder in three layers each layer being tampered again.

→ The specimen is subjected to a compressive load of 40 tonnes gradually applied at the rate of 4 tonnes

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per minute.

→ Crushed aggregates are then sieved through 2.36 mm sieve & wt. of passing material (W_2) is expressed as percentage of wt. of total sample (W_1) which is the aggregate crushing value.

$$\text{Aggregate Crushing Value} = \frac{W_2}{W_1} \times 100$$

→ A value less than 10 signifies an exceptionally strong aggregate ~~which~~ while above 35 would normally be regarded as weak aggregate.

5. Water Absorption :-

→ The difference between the apparent & bulk specifi gravities is nothing but the water - permeable voids of the aggregates.

→ We can measure the volume of such voids by weighing the aggregates dry & in saturated surface dry condition with all permeable voids filled with water.

→ The difference of above two is M_w.

~~→ M_w = wt. of dry agg~~

→ M_w = wt. of saturated surface dry aggregate − wt. of dry aggregate

$$\text{Water Absorption} = \frac{M_w}{M_D} \times 100.$$

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Requirement of Road Aggregates :-

→ They shall be clean, hard, durable & cubical in shape.
→ They must be free from dust, organic matter & other deleterious matter.
→ They shall not be flaky or elongated.
→ They must not consist of injurious or harmful materials such that they reduce the strength of structure.
→ They should resist wear due to abrasive action of traffic on the surface course.

Bitumen :-

→ It is petroleum product obtained by the distillation of petroleum crude.
→ It is black to brown in colour.
→ It is soluble in carbon disulphide & carbon tetra chloride.
→ Molecular wt. range for road bitumen is 400 to 5000.
→ Bitumen show resistance to coating road aggregate & also does not retain in presence of water.
→ It shows resistance to weathering action.
→ less temperature susceptibility.

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Tar :-

→ It is obtained by destructive distillation of coal or wood.

→ It is soluble in toluene.

→ Molecular wt. range for road tar is 150 to 3000.

→ Tar coats more easily & retain it better in presence of water.

→ It shows less resistance to weathering action.

→ More temperature susceptibility.

Cement :-

→ It is a binder, a substance used for construction that sets, hardens & adheres to other materials to bind them together.

→ Cement is seldom used on its own, but rather to bind sand & gravel together.

→ Cement mixed with fine aggregate produces mortar and with sand & gravel produces concrete.

Properties of Cement :-

→ Provides strength to masonry.

→ Stiffens or hardens early.

→ Possesses good plasticity.

→ An excellent building material.

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→ Easily workable.
→ Good moisture resistant.

Penetration Test :-

→ A method of measuring viscosity by penetration of standard needle under standard condition of load, time & temperature. The test measures the hardness or softness of bitumen in terms of penetration expressed in mm/10th of standard needle;

Temperature = 25°c.
Load on needle = 100 gm.
Time in which penetration is recorded = 5 sec

Penetration is measured by a graduated dial.

→ Bitumen softens to a poring consistency ~~to~~ a depth more than 15mm in the ~~diameter~~ container is poured. The expected penetration sample is cooled in 60 min. in air & 60 min. in water before testing. The standard needle is positioned to get a penetration value for 5 sec & is noted.

The penetration value obtained is represented in 80-100 or 80/100 grade bitumen at standard consistence & it range from 20-225 mm.

In cold region bitumen with high penetration value is used.

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In warm region. low penetration value is used i.e 30/40 grade.

The factor which affect the penetration test is -test temperature, needle size & weight & period of cooling.

Viscosity Test :-

Viscosity is the property of a fluid that determines the resistance offered by the fluid to a shearing force under laminar flow condition, it is thus the opposite of fluidity.

The liquid under test is poured to a specified level into a container surround by water or oil bath providing temperature control at the base of container is a small orifice with a simple valve control on opening valve, the time in seconds is recorded for a stated quantity of liquid to discharge into a measuring liquid below.

The bitumen is placed into a standard tar viscometer & its temperature is raised to test temperature specified. When the temp. reaches respective test temperature orifice valve is open. Time elapse is noted to collect the bitumen of 50ml. The time required to collect a bottom

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container of 50ml is give indirectly the viscosity of bitumen.

Cutback Bitumen :-

The viscosity of bitumen is reduced by a volatile diluents. Cutback bitumens are available in three types :-

i) Rapid Curing (RC)
ii) Medium Curing (MC)
iii) Slow Curing (SC)

→ Cutbacks are designated by numerals representing progressively thicker or viscous cutback. For example RC-2 is more thick than RC-1 but RC-2, MC-2 & SC-2 have same viscosity.

→ RC-0 & SC-0 may have 45% solvent & 55% bitumen where as RC-5 & MC-5 may contain 15% solvent & 85% bitumen.

→ RC-Cutback : They have penetration value of 80 to 120 e.g. petroleum such as naptha or gasoline.

→ MC-Cutback : They have good welting properties e.g. Kerosene & light diesel oil.

→ SC-Cutback : These can be obtained by blending bitumen with high boiling point gas oil or by controlling the rate of flow & temperature of crude during the first cycle of refining.

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ROAD PAVEMENTS

General

A natural earth track, under modern traffic load, can hardly be expected to perform functions of a road satisfactorily. It lacks two basic requirements of a good road, namely the strength and a good riding surface. Therefore, it becomes necessary to construct some structure in the form of pavement on the top of the natural earth track to enable it to support wheel load safely and to provide a good riding surface for a longer period.

ROAD PAVEMENTS

A layered structure supported by the subgrade soil to form the carriageway of a road is called a road pavement.

→ A pavement is essentially constructed in order to fulfill the two basic requirements of the road, namely the strength and a good riding surface. Thus, with the provision of pavement, the road can carry heavy wheel loads of vehicular traffic and provide a smooth riding surface.

→ The road pavement may be flexible or rigid according to their method of construction and their structural action.

Purposes of road pavements :-

(i) To carry heavy wheel loads of vehicular traffic and to distribute the same over a large area of the underlying subgrade soil.
(ii) To prevent the subgrade soil from the bad effects of weathering agencies
(iii) To provide a smooth riding surface.

Requirement of a good road pavement

(i) It should be cheap & easy in construction.
(ii) It should be strong & durable.
(iii) It should provide an impervious & sanitary surface.
(iv) It should be smooth enough to provide low tractive resistance but not so smooth which may become slippery.
(v) It should not develop corrugations
(vi) It should not cause glare in the sun
(vii) It should provide good visibility at night
(viii) It should be suitable for all types of traffic
(ix) It should provide a safe and comfortable riding surface under all weather conditions.
(x) It should have long life.
(xi) It should have low maintenance cost.

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Component parts of a Road Pavement structure

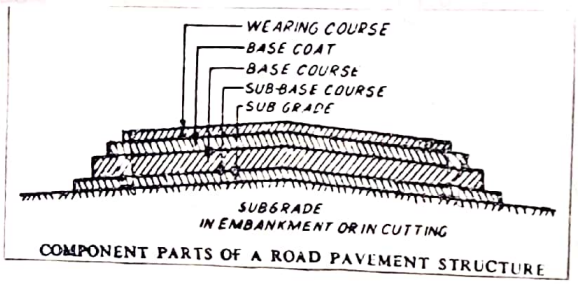
The following are the component parts of the pavement structure of a road starting from bottom :-

1. Subgrade or formation ;
2. sub-base ;
3. Base course or foundation course ;
4. Base coat or intermediate course ;
5. wearing course

All these component parts may occur in a typical flexible pavements, whereas a rigid pavement usually consists of subgrade, sub-base and a concrete slab, plain or reinforced.

1. Subgrade :-

The finished and compacted surface of earthwork on which a road pavement rests is called subgrade or formation.



COMPONENT PARTS OF A ROAD PAVEMENT STRUCTURE

The subgrade of a road pavement may be provided on an embankment, in cutting or at existing ground level depending upon the topography & the finalised formation level. It consists of well compacted natural soil brought to the required camber and gradient. The thickness and type of pavement structure depends upon the supporting power of the subgrade because the entire load of the pavement, including the load of traffic transmitted through the pavement, is ultimately taken up by the subgrade.

Functions of sub-grade :-

(i) To bear ultimately the entire load of pavement including the load of traffic transmitted through the pavement.

(ii) To provide an adequate and uniform support to the road pavement.

2. Sub-base :-

A layer of granular material provided in between the sub-grade & the base course in a road pavement is called sub-base.

→ It is provided as an additional layer when subgrade is of poor quality. It consists of a layer of comparatively cheaper material like burnt clinker, natural gravel or slag.

Functions of sub-base :-

(i) To improve the bearing capacity of the subgrade

(ii) To improve drainage and to check capillary rise of sub-soil water

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(iii) To eliminate frost heave in frost affected area

(iv) To prevent subgrade material from working up into the base cours

3. **Base course :-**

A layer of boulders or bricks (in single or double layers) provided over the sub-base or immediately over the sub-grade in the absence of sub-base in a road pavement is called base course, soling or foundation course.

This course is considered as the most important and a major component of road pavement structure because this course is to bear the impact of traffic transferred through the wearing course.

→ It consists of a stable material like boulders, gravel, one or two layers of well burnt bricks, etc.

→ In case of a rocky subgrade, this layer is not provided.

Functions of base course :-

(i) To withstand high shearing stresses imposed upon it due to impact of traffic on the wearing course.

(ii) To act as foundation for the road pavement and to transfer the wheel loads coming over the pavement surface safely to the sub-base and subgrade lying underneath.

4. **Base coat :-**

The layer of hard stones provided in between the base course and the wearing course in a road pavement is called base coat/intermediate coat/ bearing course.

This course may be provided in flexible pavements. It is usually omitted in case of a rocky subgrade, rigid pavements or when the base course consists of hard stones.

Functions of base coat :-

(i) To transmit wheel loads coming on the pavement surface over large area of the base course.

(ii) To act as a layer of transmission material since, otherwise, there is great difference in sizes of aggregates used in wearing course & base course.

5. **Wearing course :-**

The topmost layer of the road pavement directly exposed to traffic is called wearing course / surfacing.

It may consist of one or more number of layers in case of flexible pavements. A good wearing course should be impervious & weather resisting.

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It should be able to resist abrasive action of the traffic.

Functions of wearing course :-

(i) To distribute the traffic load safely to the base course.

(ii) To act as an impervious layer so that the surface water may find its access to the base course.

(iii) To prevent dust nuisance

(iv) To withstand abrasion caused due to movement of traffic.

(v) To provide a smooth riding surface.

Quality of materials required in different courses of a road pavement :-

The top layer of a road pavement is directly exposed to traffic. It is obvious that this layer is subjected to maxm stresses developed by the traffic load. Therefore, this layer should be made of dense material of the best quality. The stresses go on reducing with increase of depth. By the time, stresses are transmitted to the subgrade, their intensity is considerably reduced. So, we can generalise that quality of materials in different layers of a road pavement goes on improving, starting from its bottom to the top layer.

Types of Road Pavements

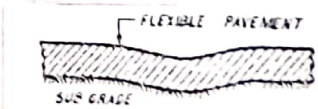
Road pavements are broadly classified into the following two basic types :-

1. Flexible pavements ;
2. Rigid pavements

1. Flexible pavements

The road pavements which can change their shape to some extent without rupture are known as flexible pavements.

→ Any change of shape occuring in the subgrade and subsequent layers provided on it, is reflected by the surface of these pavements as shown alongside.



A Flexible Pavement reflecting depression due to settlement of Subgrade

→ The common examples of flexible pavements are :- All bituminous pavements gravel pavements, water bound macadam (WBM) pavements, etc.

2. Rigid Pavements

The road pavements which cannot change their shape without rupture are known as rigid pavements.

→ Any change occuring in the shape of subgrade is not reflected by the surface of these pavements

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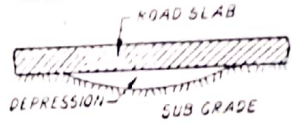
The best example of rigid pavements is :- cement concrete pavement.

In case of a depression in subgrade, occurring due to settlement, a rigid pavement acts as a beam or cantilever. It remains in its position unless bending stresses induced in the pavement are so great as to cause failure of the road pavement as shown in fig: alongside.



A Rigid Pavement acting as a Beam over a depression due to settlement of Subgrade

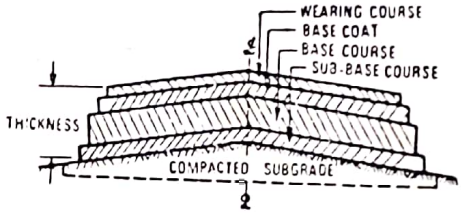


Failure of Rigid Pavement in case of depression due to settlement of Subgrade

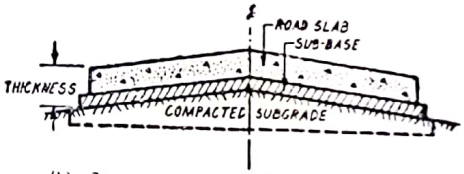
Typical cross-sections of flexible and rigid pavements

Flexible and rigid pavements consists of different structural components as shown in fig alongside ----

- A typical flexible pavement usually consists of the following structural components, starting from the bottom :-
  (i) subgrade ;
  (ii) sub-base ;
  (iii) Base course ;
  (iv) Base coat ;
  (v) wearing course.



(a) Structure of a Flexible Pavement



(b) Structure of a Rigid Pavement

TYPICAL CROSS-SECTIONS OF FLEXIBLE AND RIGID PAVEMENTS

- A typical rigid pavement usually consists of the following structural components, starting from the bottom :-
  (i) sub-grade ;
  (ii) sub-base ;
  (iii) A concrete slab, P.C.C or R.C.C

→ In case of a rocky subgrade, sub-base is omitted.

(*) Under similar subgrade and traffic conditions, the thickness of pavement required in case of a flexible pavement is considerably more than that in case of a rigid pavement.

Selection of type of road pavement

The factors which may affect the selection of flexible and rigid Road pavements are given below :-

(a) **Flexible pavement** This type of road pavement is selected under the following circumstances :-
  (i) when funds are not sufficiently available.

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(ii) Large number of roads are to be constructed.
(iii) Good subgrade is available.
(iv) Local materials are to be advantageously used.
(v) Traffic load is not very heavy.
(vi) Pavement is to be constructed in stages.

(b) **Rigid Pavement** :-

This type of road pavement is selected under the following circumstances

(i) When funds are sufficiently available
(ii) Long life of the road pavement is desired
(iii) Traffic load is very heavy.
(iv) Subgrade is poor or of varying nature
(v) Route is very important and frequent interruptions in the movements of traffic for repair, etc. are not desirable.

Merits & demerits of flexible and rigid road pavements :-

The comparative study of merits and demerits of flexible and rigid road pavements are as per follows..

Sl.No	Points of comparison	Flexible pavements	Rigid pavements
01.	Initial cost	Their initial cost is low.	Their initial cost is high.
02.	Life span	Their life span is short	Their life span is long.
03.	Thickness	Their thickness is more	Their thickness is less.
04.	Joints	Joints are not required	Joints are essentially required.
05.	Skill & supervision	Moderate skill and less supervision is required.	High degree of skill & more supervision is required.
06.	Repair work	Their repair work is easy	Their repair work is difficult.
07.	Subgrade	A reasonably good subgrade is required	A good subgrade is not necessary.
08.	Durability	They are less durable.	They are more durable
09.	Maintenance cost	Their maintenance cost is high.	Their maintenance cost is low.
10.	Resiliency	They are more resilient to traffic load.	They are less resilient to traffic load.
11.	corrugations	They develop corrugations	They do not develop corrugations.
12.	Glaring effect	They do not usually cause glare due to reflected sun light.	They often cause glare due to reflected sun light
13.	Traffic suitability	They are suitable for all types of traffic	They become noisy under iron wheeled traffic
14.	Tractive resistance	They offer more tractive resistance.	They offer less tractive resistance.

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15.	Temperature effects	Stresses are not induced in these pavements due to temperature variations.	Heavy stresses are induced in these pavements due to temperature variations.
16.	Behaviour with subgrade settlement	They adjust according to any deformation of subgrade without rupture.	They donot adjust according to any deformation of subgrade without rupture
17.	Feasibility of providing underground works	It is easy to lay, locate or repair underground pipes below these pavements.	It is very difficult to provide or repair underground pipes below these pavements.
18.	Opening to traffic after construction	They can be opened to traffic shortly after construction.	They require curing after construction and thus cause delay in opening to traffic
19.	Night visibility	Black top pavements provide poor visibility at night	They provide good visibility at night.
20.	Fisibility of stage development	Stage development is feasible in their case.	Stage development is not feasible in their case.
21.	Effect of loading.	They adjust themselves to normal loading by undergoing elastic deformation.	They tend to act as a beam or cantilever and resist deformation.

:- Subgrade Preparation :-

The art of providing a finished and compacted surface of earthwork according to the desired gradient and camber along the alignment of a proposed road is known as preparation of subgrade.

(*) This is the first step in the construction of a proposed road after setting out its alignment on the ground.

The subgrade may be provided on an embankment, in cutting or at existing ground level depending on the topography and the finalised level as per longitudinal section of the proposed road.

The preparation of a road subgrade involves the following operations :-

1. Site clearance & grubbing;
2. Earthwork;
3. compaction & consolidation;
4. checking of subgrade.

1. Site clearance & grubbing

These operations are performed in advance of earthwork operations in accordance with the required specifications.

The site clearance operation includes removal and disposal of all materials such as trees, shrubs, stumps, rubbish, etc. which are unsuitable for use in

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the work, from within the right of way and such other areas as may be specified on the drawings or by the Engineer-in-charge.

→ The grubbing operation includes removal and disposal of the roots and stumps of trees. The grubbing should be done to a depth of atleast 0.5m below final subgrade level because the roots remaining within the ground are liable to decay in due course of time and may create hollow pockets. Such hollow pockets may cause settlement and result into road failure.

2. Earthwork

After site clearance and grubbing, earthwork is to be done for preparing the subgrade of the proposed road. Before handling earthwork, control pegs should be established for guidance. These control pegs include clearing stake, batter pegs, reference pegs etc.

In almost all types of areas, road construction involves some amount of earthwork. In open country, a major portion of the road length is constructed in embankment due to drainage consideration.
In a rolling terrain (area with many ups & downs), the embankments and cutting are to be very frequently provided in a road length. Thus, the earthwork for constructing a road may be done in the following two forms:-

(a) Earthwork in embankment
(b) Earthwork in cutting

(a) Earthwork in embankment

The process of removing earth from borrow pits or cuttings, transporting and placing it as a fill in the form of an embankment is known as earthwork in embankment.

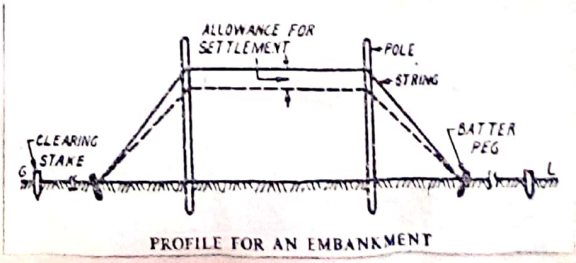
When the finelised formation level of a road is to be kept higher than the natural surface level, the road is to run in embankment. The side slopes of an embankment vary from $1\frac{1}{2}:1$ to $3:1$, depending upon the type of soil and drainage conditions.

The earthwork in an embankment is carried out in the following steps:-

(i) marking profiles of embankment
(ii) stripping and storing top soil
(iii) constructing embankments

(i) Marking profiles of embankment

Profiles of embankment are constructed with poles and strings as shown in fig- alongside. These are constructed at 30m intervals for guidance of labour.



PROFILE FOR AN EMBANKMENT

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For making profiles of an embankment, batter pegs are driven on both sides of the centre line to mark toes of the embankment. Bench marks consisting of masonry pillars are erected at about 100m interval to indicate edge of the embankment and formation level. For ordinary embankments, 10% allowance for settlement is kept while making profile as shown in figure.

Materials for embankments :-

The materials used in the construction of embankment is earth, moorum, gravel, a mixture of these or any other approved material obtained either from excavation for road construction borrow-pits and other sources. Such materials should be free of logs, stumps, roots, rubbish or any other material likely to affect the stability of the road embankment

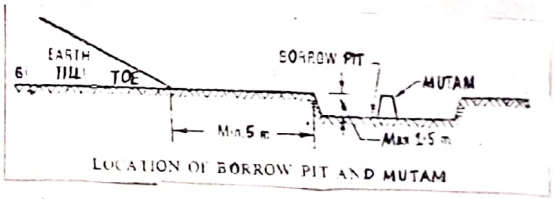
The size of the coarse material in the mixture of earth to be used in the construction of road embankment should not usually exceed 75mm. Ordinarily, only the materials satisfying the density requirement, as per give table below should be used in embankment construction.

Density Requirements of Embankment Materials

Sl. No	Types of work	Max^m Laboratory Dry Density as per IS: 2720 (part VII)
1.	Embankment upto 3m height	Not less than 1.44 gm/cc
2.	Embankment exceeding 3m height or embankments of any height subjected to constant weathering	Not less than 1.52 gm/cc
3.	Top 0.5m of the embankment below the subgrade level & earth shoulders	Not less than 1.65 gm/cc

Borrow pits :-

The pits dug along the alignment of a road for using their material in the construction of road embankment are known as borrow pits.



LOCATION OF BORROW PIT AND MUTAM

These pits are usually prismoidal or rectangular in shape and dug just outside the permant land width acquired for the proposed road.

Borrow pits should be dug at least 5m away from toe of embankment. These are dug to uniform depth which is limited to 1.5m. The longer sides of borrow pits are kept parallel to the centre line of the road. These pits should never be made continuous because in rainy season, they are likely to become a running stream and may endanger the embankment. Therefore, ridges of not less than 8mm width should be left at intervals not exceeding 300m.

Mutams :-

The small portion of earth left undug in a borrow pit to measure the quantity of excavation work is known as mutam/deadman.

→ It should be located at such a position that gives the average depth of excavation for the area to which it belongs. Mutams are maintained till the measurement of earthwork is completed.

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(ii) Stripping and storing top soil

After making profiles of embankment, the top soil existing over the embankment foundation is stripped to specified depth, not exceeding 150mm & stored for covering embankment slopes, cut slopes and other disturbed areas where re-vegetation is desired.

(iii) Constructing embankments

After stripping and storing top soil, the original ground is consolidated by rolling with a max^m of six passes of 8-10 tonne roller. Then filling is started from edges and worked towards centre of the embankment in layers of thickness not more than 250mm in loose state. These layers are kept slightly concave in shape with convexity downward for proper compaction of the previous layer. The successive layers of earth should be placed only when the previous layer has been thoroughly compacted.

(b) Earthwork in cutting

The process of cutting or loosening and removing earth including rock, from its original position, transporting and damping the same as a fill or in the form of a spoil bank is known as earthwork in cutting or excavation.

When the finalised formation level of a road is to be kept lower than the natural surface level, the road is to run in cutting. The side slopes of a cutting vary from 2:1 to even vertical, depending upon the strata through which cutting is to be done.

The earthwork in cutting is carried out in the following steps :-

(i) setting out
(ii) stripping and storing top soil
(iii) Excavation

(i) Setting out

After clearing the site, the limits of excavation are set out true to lines, curves, slopes, grades and sections as shown on the drawings or as directed by the Engineer-in-charge.

(ii) Stripping and storing top soil

After setting out the limits the top soil existing over the sites of excavation is stripped to specified depths and stored at suitable locations for re-use in covering cut slopes, berms and other disturbed areas where re-vegetation is desired.

(iii) Excavation

The excavation is done in lifts. The initial left or which the contractor is not made any additional payment varies from 0.5m to 1.5m, depending upon the soil condition. Paths & gangways, required for movement of labours,

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should be provided well within the cutting so that when these are removed, the section in cutting becomes true to drawing.

3. Compaction & Consolidation

The earthwork in embankment is compacted to increase the density & stability of the subgrade soil. It reduces settlement and decreases the adverse effects of moisture. Hence, proper compaction of embankment is considered essential for highway construction.

For better results, each layer of the material should be thoroughly compacted to the densities as per following values mentioned in tabular form.

Specification requirements for minimum subgrade soil compaction as recommended by the IRC --- are given

Compaction Requirements for Embankments

S. No.	Type of Work/Material	Field Dry Density as percentage of Maximum Laboratory Dry Density as per ISI : 2720 (Part VII)
1.	Top 0·5 m portion of embankment below subgrade level and shoulders.	Not less than 100
2.	Other portion of embankment.	Not less than 95
3.	Highly expansive clays.	85 to 90

The subsequent layers should be placed after the finished layer has been tested and accepted by the Engineer-in-charge. The moisture constant at the time of compaction should be optimum moisture content for obtaining maximum dry density of the soil.

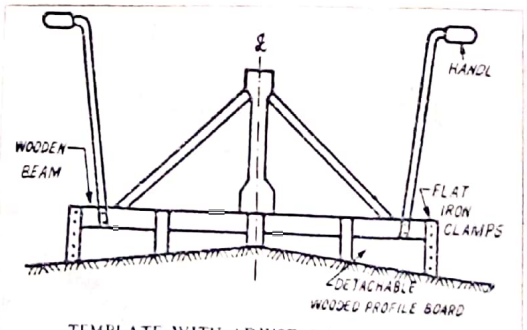
soil compaction is achieved in the field either by rolling, ramming vibration or by watering. Hence, the compacting equipment may be rollers, rammers or vibrators.

After completion of earthwork, a trench is usually dug for finishing the subgrade finally. The depth of this trench is kept equal to the designed thickness of the pavement. The bottom of the trench is provided with some cross-slope i.e camber prescribed for the proposed pavement.

4. Checking of Subgrade

After preparing subgrade, it is checked for its trueness. Surface levels to the subgrade along the road alignment can be checked by a levelling instrument. As per IRC specifications, actual levels should not differ from levels given on drawings by more than 25mm.

The transverse profile is checked by a template. The max^m variation permissible is 15mm



TEMPLATE WITH ADJUSTABLE PROFILE

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Quality control in preparation of subgrade :-

For adequate quality control in preparation of subgrade, it is essential to have proper field control in compaction.

The following two field control tests are performed to have the required quality control :-

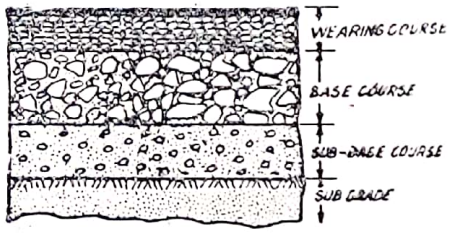
(i) Measurement of moisture content ;
(ii) measurement of dry density

The moisture content of the soil is found before compaction by suitable method at the site. The soil should have optimum moisture content (OMC) at the time of compaction. After controlling the moisture at OMC, the next control of density is achieved. Dry density may be found by any suitable method, the sand replacement method is considered quite satisfactory.

⇒ The top 50cm layer of soil is compacted thoroughly by rolling at optimum moisture content. (OMC)

Sub-bases & Bases for flexible Pavements :-

The comparison between sub-base and base course of a flexible pavement is given as below



TYPICAL SECTION THROUGH A FLEXIBLE PAVEMENT

Sl NO	Points of Comparison	Sub-base	Base course
01.	Construction	It consist of one or two layers of granular materials.	It consists of more stable material than that of sub-base or subgrade.
02.	Thickness	It is of less thickness.	It is of more thickness.
03.	Design accuracy	It may or may not be accurately designed.	It should be accurately designed according to the wheel loads expected over the pavement.
04.	Location as a structural component	It is provided on the subgrade when the later is of poor quality.	It is provided after sub-base or on subgrade in the absence of sub base.
05.	Materials to be used	The materials used in the construction of this layer are natural sand, gravel, laterite, kankar or other naturally occuring or artificial soft aggregate.	The materials used in the construction of this layer are first class bricks, stone aggregate etc.
06.	Function	Its function is to improve the bearing power of the subgrade and to prevent capillary rise of subsoil water in the foundation course.	Its function is to act as foundation to the pavement & distribute uniformly the stresses developed due to wheel load over larger area of the subgrade.

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TYPES OF BASE COURSES OF FLEXIBLE ROAD PAVEMENTS

The following are the various types of base courses or bases of flexible road pavements according to their method of construction :-

1. Stabilized soil base courses;
2. Brick or stone soling;
3. Macadam base courses.

1. Stabilized soil base courses

The base consisting of stabilized soil is known as stabilized soil base course.

(*) The process of improving the stability or bearing power of the ordinary soil by the use of controlled compaction, proportioning and adding of suitable stabilizers is known as soil stabilization.

⇒ Earth roads, constructed from ordinary soil available along their alignment, wear out very soon & become unserviceable within no time. Such roads require constant repair which is not practicable in villages. Hence, to keep these roads in serviceable condition for a longer period and to reduce their maintenance cost, soil stabilization techniques are being extensively utilized for their construction.

Thus, the principle of soil stabibed road construction is the effective utilization of local soil & other suitable stabilizing agents.

Objects of soil stabilization :-

(i) To increase the bearing power of the soil.

(ii) To increase shear strength i.e resistance to punching action of the soil.

(iii) To increase resistance to softening action (due to water) of the soil.

(iv) To increase flexibility in the soil to take the wheel load without deformation & cracking.

(v) To reduce the tendency of swelling or increase in volume of the soil due to wetting & shrinkage on account of withdrawl of moisture.

(vi) To increase the stability of earthwork in embankment as a whole.

Common soil stabilizers :-

Following stabilizers or admixtures are in common use for soil stabilizat

(a) cement; (b) Lime; (c) Bitumen; (d) organic compounds such as resinous materials.

(e) chemicals such as calcium chloride, sodium chloride, etc.

Stabilized soil has been satisfactorily used in base course construction in flexible pavements. Such bases are becoming very popular in India because of their low cost & good performance under moderate traffic.

Some important types of stabilized soil bases are

(a) Mechanically stabilized base courses
(b) Lime stabilized base course
(c) cement stabilized base course
(d) Fly-ash stabilized base course.

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(a) Mechanically stabilized base courses :-

The base consisting of soil stabilized simply by controlled compaction is called mechanically stabilized base course.

Mechanical stabilization is based on the principle that quantity of various size particles in a soil is so adjusted that the particles of one size are just sufficient to fill the voids existing in the particles of next higher size. Such a soil produces dense and stable material after compaction. It is called gradation concept of mechanical stabilization.

For constructing a mechanically stabilized base course, a suitable gradation is adopted. locally available soil and other materials are blended, mixed & compacted at optimum moisture content and thus the base of desired thickness is prepared.

(*) ⇒ Dr. Mehra has recommended the use of 7.5 cm thick mechanically stabilized soil base course for Indian conditions. According to him, the sand content in soil should be 50% and P.I between 5 to 7.

(*) Equipment used for blending and mixing includes disc harrow, spring tooth and cultivator's plough. Compaction is done by rollers.

(b) Lime stabilized base course :-

The base consisting of soil stabilization by mixing the required proportion of hydrated lime as a stabilizer is called lime stabilized base course.

Lime stabilization has gained popularity in improving subgrades and constructing base courses in areas rich in clayey soil. Black cotton soils responds very well to lime stabilization.

Lime reacts chemically with soil and affects it in the following ways :-

(i) It reduces plasticity and shrinkage of the soil.
(ii) It makes clayey soil easily workable.
(iii) It brings about flocculation of soil particles
(iv) It binds the soil particles together

For constructing a lime stabilized base course, the top of existing surface may be flattended or slightly trenched to receive the lime which may be mixed with the prepared material either in slurry form or dry state. The soil is excavated upto 15 cm depth, pulverised, sieved and mixed with 2 to 10% by weight of commercial dry lime slaked at site or pre-slaked lime delivered to the site in suitable packing. The lime used for this puspose should have purity not less than 70 percent when tested in accordance with IS : 1514. The mixing of lime and soil is done by rotary mixers, disc harrow or other suitable equipment. Immediately after spreading, grading & levelling of mixed material, compaction should be carried out with 8 to 10 tonne smooth wheel rollers. The soil is compacted by rolling at OMC and in this way the base course of desired thickness is prepared.

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(C) cement stabilized base course :-

The base consisting of soil stabilized by mixing the required proportion of cement as a stabilizer is called cement stabilized base course. The cement stabilized base courses are also known as soil-cement bases.

Almost all types of soils can be stabilized by cement. However, the quantity of cement required, and difficulty in mixing it with the soil, prohibits the use of cement for stabilizing organic and clayey soils. Due to shortage of cement, this type of stabilization is rarely used in India

Construction of a cement stabilized base course involves rigid control on moisture content, mixing & compaction. Damp curing over a period of 10 days is required for cement stabilized bases.

The method of construction of a cement stabilized base is same as in case of the lime stabilized base. The quantity of cement required varies from 5% to 20%. The exact quantity of cement for a soil is found out experimently.

(d) Fly-ash stabilized base course :-

The base consisting of soil stabilized by mixing fly-ash (cinder) in suitable proportion is called fly-ash stabilized base course.

Fly-ash is a residual material left after burning of coal. Big utility plants, using coal, are producing large amount of fly-ash as a waste material. In the past, fly-ash posed a problem for disposal. But in the recent times, use of fly-ash has been made in soil stabilization.

The mix of fly-ash, lime and Jhama brick aggregates, in suitable proportions, has been found to produce a concrete like material. This mix can be advantageously used for base course construction of flexible pavements subjected to light traffic.

2. Brick or Stone soling

A compacted layer of bricks or stones laid directly on subgrade of a road pavement is called soling.

In India, brick or stone solings are commonly used in important road pavements. These two types of soling are

(a) Brick soling

One or two layers of bricks laid directly on subgrade of a road pavement is called brick soling.

Bricks may be laid flat or on edge, in one layer or in two layers to get the desired thickness of soling. The thickness of soling depends upon traffic conditions. A row of bricks on end is provided on each side of subgrade trench to contain the running layers of pavement to be laid on the top of soling. Bricks are laid with their lengths at right angles to the centre line of

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road, breaking joints in adjacent rows.

After laying bricks, sandy soil is worked into joints by using brooms and water. Then a layer of sand of about 2.5 cm thickness is spread on bricks. The soil is sufficiently sprinkled over with water & rolled by light roller. Brick soling is allowed to dry before the next component layer is placed on it.

(b) Stone soling

A compacted layer of hammer-dressed stones laid directly on subgrade of a road pavement is called stone soling.

stones of proper size and shape, according to thickness of soling required, are selected and hand packed on the prepared subgrade. Stones are laid on their wider faces in such a way that their tops conform to the profile of pavement. Voids in stones are filled with smaller stones. Stone soling is laid to extend 15 cm on either side beyond the proposed width of pavement. After laying stones, the profile is checked with a template & corrected wherever found defective. Stones are then consolidated by rolling with a heavy roller. Dry rolling is followed by spreading of sandy soil of 2.5 cm thickness, watering & then rolling by a 6 to 8 tonne roller. Stone soling is then allowed to dry.

3. Macadam base courses

The bases consisting of broken stone aggregates bound together under controlled compaction with or without a binding material are known as macadam base courses.

The important types of macadam base courses are :-

(a) water-bound macadam base course
(b) Built-up spray grout base course
(c) Bituminous macadam base course

(a) water-bound macadam base course

The base consisting of clean, crushed aggregates, mechanically interlocked by rolling and bounded together with screening and water is called water-bound macadam base course.

Materials required :-

The materials required for a W.B.M. base course are coarse aggregates, screenings & filler materials.

The coarse aggregates used may be either crushed or broken stone, crushed slag or overburnt brick aggregate. The aggregate should conform to the physical requirement given in the following table --

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The coarse aggregate is of 90mm to 40mm size, 63mm to 40mm size or 50mm to 25mm size according to the type of aggregate available & the total consolidated thickness of base course required.

It should fulfil the specified gradation requirement as per following table

Physical requirements of Coarse Aggregates for Water Bound Macadam Base Course

S. No.	Test	Test Method	Requirements
1.	Los Angles Abrasion Value	IS : 2386 (Part N)	50 per cent max.
	OR		
	Aggregate Impact Value	IS : 2386 (Part N)	40 per cent max.
		OR	
		IS : 5640	
2.	Flakiness Index	IS : 2386 (Part I)	15 per cent max.

The Grading Requirements of Coarse Aggregates for W.B.M. Base Course

Grading No.	Size Range	Sieve Size mm	Per cent by Weight passing the Sieve
1.	90 mm to 40 mm	100	100
		80	65 to 85
		63	25 to 60
		40	0 to 15
		20	0 to 5
2.	63 mm to 40 mm	80	100
		63	90 to 100
		50	35 to 70
		40	0 to 10
		20	0 to 5
3.	50 mm to 20 mm	63	100
		50	95 to 100
		40	35 to 70
		20	0 to 15
		10	0 to 5

The screenings to fill voids in the coarse aggregate should be of the same material as the coarse aggregate. However, where permitted, moorum or gravel (other than rounded river born material) may be used for this purpose provided liquid limit & plasticity index of such materials is below 20 & 6 respectively and fraction passing 75 micron sieve does not exceed 10 percent.

The screening should conform to the grading given as per following table....

The Gradation Requirements of Screenings for W.B.M. Base Course

Grading Classification	Size of Screenings	Sieve Size mm	Per cent by Weight Passing the Sieve	Grading No. of C.A.
A	12·5 mm	12·5	100	1 and 2
		19·0	90 to 100	
		4·75	10 to 30	
		150 micron	0 to 8	
B	10 mm	10·0	100	2 and 3
		4·75	85 to 100	
		150 micon	10 to 30	

The binding material used for WBM course may be comprising of a suitable material approved by the Engineer-in-charge, having plasticity index value ~~&~~ less than 6. Its application may not be necessary when the screening used are of crushable type such as moorum or gravel.

Method of construction:-

The construction of a WBM base course is completed in the following steps:

(i) preparing the base
(ii) Spreading coarse aggregate
(iii) Rolling
(iv) Application of screening
(v) sprinkling and grouting
(vi) Application of binding material
(vii) setting & drying.

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(i) Preparing the base

The subgrade or sub-base to receive the WBM base course is prepared to the specified grade and camber. Any ruts or weak spots are corrected and rolled until firm. If the water bound macadam is to be laid directly over the subgrade, without any other intervening pavement course, a 25 mm course of screening (Grading B) shall be spread on the prepared subgrade before spreading of coarse aggregate. This layer is known as inverted choke.

(ii) Spreading coarse aggregate

After preparing the base, the coarse aggregates are spread uniformly and evenly upon it in such quantities that the thickness of the compacted layer does not exceed 75 mm in general and in no case, however, it should exceed 100 mm. Thus, upto 100 mm compacted thickness of the base course, its construction may be done in single layer and for greater thickness, its construction should be completed in two or more layers.

The stone aggregates should be laid carefully with the broad face downwards. The aggregates are then hand-packed to the desired camber of the top surface. After the hand packing has been completed, the irregularities in surface should be checked by template and carefully set right.

(iii) Rolling

Immediately after the spreading of coarse aggregate, rolling is started with three wheeled power roller of 6 to 10 tonne capacity or tandem or vibratory roller of approved type.

Except on superelevated portions of the road where the rolling should proceed from inner edge to the outer, rolling of the coarse aggregate should be started from the edges and gradually progressed towards the centre. Rolling is stopped when the aggregates are partially compacted with sufficient void space in them to permit application of screenings. The rolled surface is then again checked transversely and longitudinally with templates and irregularities, if any are corrected and consolidated.

(iv) Application of screenings

After dry rolling, screenings are applied gradually over the surface. Dry rolling should be done while the screenings are being spread so that vibrations of the roller may cause them to settle into the voids of the coarse aggregate.

The screenings should be applied at a slow & uniform rate accompanied by dry rolling and brooming so as to ensure filling of all voids. This operation should be continued until no more screenings can be forced into the voids of the coarse aggregate.

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(v) Sprinkling & grouting

After the screenings have been applied, the surface is sprinkled over with sufficient quantity of water, swept & rolled.

Hand brooms should be used for sweeping the screenings into voids & to distribute them evenly. The sprinkling, sweeping and rolling operations should be continued with additional screenings applied as necessary, until the coarse aggregate has been thoroughly keyed, well bonded and firmly set in its full depth.

(vi) Application of binding material

After sprinkling and grouting, the binding material consisting of sandy soil (containing about 75% sand & remaining clay or moorum) is applied successively in two or more thin layers. After each application, the surface is sprinkled over with sufficient quantity of water. The resulting slurry is swept in with hand brooms or mechanical brooms to fill the voids properly and then the surface is rolled. During rolling, the water is sprinkled over wheel of the roller to wash down the binding material sticking to them.

These operations should be continued until the resulting slurry after filling of voids, forms a wave ahead of the wheels of the moving roller.

(vii) setting & drying

After the final compaction of the WBM base course, the surface is allowed to dry overnight. Next morning, the depressions, if any, are filled with screenings of binding material, lightly sprinkled with water & rolled. The W.B.M base is then allowed to dry completely.

(b) Built-up spray grout base course

The base course consisting of two layers of crushed coarse aggregates with application of bituminous binder after each layer and key aggregate on the top of the second layer is known as built-up spray grout base course.

In this type of base course, the aggregate are laid in accordance with the desired specifications and, in conformity with the lines, grade and cross-sections shown on the drawings or as directed by the E-I-C. This type of base course is always constructed during dry weather and laid on a dry base when the atmospheric temperature in shade is above 16°C

Materials required :-

The materials required for the construction of a built-up spray grout base course are binder, coarse aggregate and key aggregates.

The binder should be straight-run bitumen of a suitable grade, as directed by the Engineer-in-charge complying with IS 73, or as approved cut-back.

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The aggregates should consist of coushed stone, crushed gravel (shingle) or other stones. They should be clean, strong, durable, cubical in shape & free from disintegrated pieces, organic and other harmful matter & adherent coatings.

* The aggregates to be used in built-up spray grout base course should fulfil the gradation and physical requirements as per IRC specifications given in following tables

IRC Specifications for Coarse and Key Aggregates to be used in Built-up Spray Grout Base Course

(A) *Gradation Requirements*

Sieve Size mm	Per cent by Weight passing the Sieve	
	coarse aggregate	key aggregate
50	100	—
25	35—70	—
20	--	100
12·5	0—15	35-70
4·75	—	0-15
2·36	0—5	0-5

(B) *Physical Requirements*

S. No.	Test	Test Method	Requirements
1.	Los Angeles abrasion value*	IS : 2386 (Part IV)	50% Max
2.	Aggregate impact value*	—do—	40% Max
3.	Flakiness index	IS : 2386 (Part I)	25% Max
4.	Water absorption	IS: 2386 (Part III)	1% Max
5.	Stripping value	—	25% Max

Method of Construction :-

The construction of a built-up spray grout base course is completed in the following steps :-

(i) preparing the base
(ii) Applying prime coat
(iii) spreading the coarse aggregates
(iv) Rolling the first layer of coarse aggregates
(v) spraying the First layer of binder
(vi) Spreading the second layer of coarse aggregate
(vii) Rolling the second layer of coarse aggregate
(viii) spraying the second layer of binder
(ix) Spreading the key aggregate
(x) Rolling the surface.

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(i) **Preparing the base**

The subgrade or sub-base to receive the built-up spray grout base course is prepared, shaped and conditioned to the specified grade and camber. Any ruts or weak spots are corrected and rolled until firm.

(ii) **Applying prime coat**

After preparing the base, prime coat, consisting of a single coat of low viscosity liquid bituminous material, is applied to surface of prepared base.

(iii) **Spreading the coarse aggregates**

Immediately after the application of prime coat, the coarse aggregates in dry and clean form are spread uniformly and evenly at the rate of 0.5 m^2 per 10 m^2 area. After this, the surface of the aggregate layer is checked carefully with templates and the deficiency, if any should be rectified.

(iv) **Rolling the 1st layer of coarse aggregates**

Immediately after spreading of the coarse aggregates, the entire surface is rolled with 8-10 tonne smooth wheeled roller, taking all the precautions during rolling of the road surface.

After initial rolling, the entire surface is checked transversely and longitudinally with templates and corrected followed by re-rolling the surface. Rolling should be stopped before voids in the aggregate layer are closed to such as extent as to prevent free and non-uniform penetration of the cinder.

(v) **Spraying the 1st layer of binder**

The binder is heated to the temperature appropriate to the grade of bitumen and sprayed on the aggregate layer at the rate of 12.5 kg/10 m^2 in a uniform manner with the help of a mechanical sprayer. Excessive deposits of binder, if any; should be promptly corrected.

(vi) **Spreading the 2nd layer of coarse aggregate**

Immediately after spraying the first layer of binder, the second layer of coarse aggregate is spread in a similar manner as done in case of 1st layer.

(vii) **Rolling the 2nd layer of coarse aggregate**

After spreading the second layer of coarse aggregate, the entire surface is again rolled in a similar manner as done in case of First layer.

(viii) **Spraying the second layer of binder**

After rolling the second layer of coarse aggregate, the binder is sprayed on the aggregate layer at the rate of 12.5 kg/10 m^2 in a similar manner as done in case of first layer.

(ix) **spreading the key aggregate**

Immediately after spraying the second layer of binder, key aggregate in a

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clean and dry state is spread uniformly at the rate of $0.13\ m^3/10\ m^2$ so as to cover the entire surface. The surface is then broomed, if necessary to ensure uniform application of the key aggregate.

(X) Rolling the surface

After spreading the key aggregate uniformly, the entire surface is rolled with 8-10 tonne smooth wheeled roller. While rolling is in progress, additional key aggregate is spread, where required. Rolling should be continued until the entire base course is properly compacted & the aggregate is firmly in position.

(C) Bituminous macadam base course

The base course consisting of a single layer of crushed aggregates premixed with a bituminous binder is called bituminous macadam base course.

In this type of base course, the crushed aggregates premixed with a bituminous binder is laid, immediately after mixing, on a base prepared previously in accordance with the desired specifications, and in conformity with the lines, grade and cross sections shown on drawings or as directed by the Engineer-in-charge. This type of base course usually consists of 50mm / 75mm thick compacted layer of crushed aggregates premixed with a bituminous binder. It should not be laid during rainy weather or when the base is damp or wet.

Materials required :-

The materials required for the construction of a bituminous macadam base course are binder and aggregates which should fulfill the requirements mentioned in the case of built-up spray grout base course.

The binder content for premixing should be 3.5 and 4 percent by weight of the total mix for aggregate grading No. A and B respectively. The quantities of aggregates should be sufficient to provide the specified thickness of the bituminous macadam base course after compaction.

The aggregates for different compacted thickness of bituminous macadam base course should fulfil the gradation and physical requirements as per IRC specifications given in following tables (A and B) respectively.

(B) *Physical Requirements*

S. No.	Test	Test Method	Requirements
1.	*Los Angeles Abrasion Value**	IS : 2386 (Part IV)	35% Max.
2.	*Aggregate Impact Value**	—do—	30% Max.
3.	Flakiness Index	IS : 2386 (Part I)	35% Max.
4.	Water Absorption	IS : 2386 (Part III)	2% Max.
5.	Stripping Value	—	25% Max.

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Method of construction :-

The construction of a bituminous macadam base course is completed in the following steps :-

(i) Preparing the base
(ii) Applying the prime coat
(iii) preparing & transporting the mix
(iv) Spreading the mix
(v) Rolling the surface

IRC Specifications for Aggregates to be used is different Thicknesses of Bituminous Macadam Base, Binder Course

(A) *Gradation Requirements*

Sieve Size mm	Per cent by Weight passing the Sieve for different Compacted Thicknesses			
	75 mm compacted thickness		50 mm compacted thickness	
	Grading A	Grading B	Grading A	Grading B
63	100	—	—	—
50	90—100	—	100	—
40	35—65	100	90—100	—
25	20—40	70—100	50—80	100
20	—	50—80	—	70—100
12·5	5—20	—	10—30	—
10	—	—	—	35—60
4·75	—	10—30	—	15—35
2·36	—	5—20	—	5—20
75 micron	0—5	0—4	0—5	0—4

(i) Preparing the base

The subgrade or sub-base to receive the bituminous macadam base course is prepared, shaped and conditioned to the specified lines, grade & cross sections in accordance with the desired specifications or as directed by the Engineer-in-charge. The surface is then thoroughly swept and scraped clean so as to make it free from dust and foreign matter.

(ii) Applying the prime coat

After preparing the base, the prime coat of a suitable bituminous material is applied to the surface of prepared base except when the laying of bituminous macadam is being preceded by a bituminous levelling course.

(iii) preparing and transporting the mix

For preparing the mix, hot mix plant of adequate capacity should be used. The temperature of binder at the time of mixing should be in the range of 150° — 177°c and that of aggregates in the range 155° — 163°c. The difference in temperature between the aggregates and binder should not be allowed to exceed 14°c while preparing the mix.

The aggregate and binder both are mixed thoroughly in the hot mixing plant to ensure that a homogeneous mixture is obtained. The mix is then transported from the mixing plant to the point of use in suitable vehicles. Dump trucks are efficiently used for this purpose.

(iv) Spreading the mix

Immediately after mixing, the mix is transported and spread by means of a self propelled mechanical paver with suitable screeds capable of spreading, tamping and finishing the surface to the specified lines, grade and cross-sections. The spreading of the mix can also be done manually, especially in restricted locations and in narrow widths where the available

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plants cannot operate.

The temperature of the mix at the time of spreading should be in the range of 121° – 63°C

(v) Rolling the surface

Immediately after the spreading of mix, rolling is done by 8-10 tonne smooth wheeled power roller. After initial rolling, the entire surface is checked transversely and longitudinally with templates and corrected. The rolling is then continued until the entire surface is rolled to compaction, there is no crushing of aggregates and all roller marks are eliminated.

Comparative Merits/De-merits of Three Types of Base courses (W.B.M, Built-up spray Grout and Bituminous Macadam)

Sl.NO	Points of comparison	W.B.M	Built-up spray grout	Bituminous macadam
01.	Structural strength	low	medium	high
02.	Traffic intensity suitable for base course	low	medium	high
03.	Thickness of base course for same traffic intensity	Maximum (75mm)	medium (60mm)	minimum (45mm)
04.	Durability	low	medium	high
05.	Susceptibility to manual construction	high	medium	low
06.	Initial cost of construction	lowest (100)	highest (200)	high (190)
07.	Superiority equivalency factor	1.00	1.25	1.75

:- PAVEMENT SURFACES :-

The topmost component of a road pavement structure directly exposed to traffic is called pavement surface or wearing surface of the road.

In a flexible pavement, the wearing surface is supported by a base course. whereas in a rigid pavement, the road slab serves the function of wearing surface as well as of base course

Roads are usually named according to the type of their pavement surfaces. Various types of pavement surfaces are classified into the following groups:-

1. Low cost surfaces
2. High cost surfaces

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1. Low cost surfaces

The pavement surfaces which can be constructed as well as maintained at low cost are known as low cost surfaces, and the roads with such surfaces are known as low cost roads.

These roads are also known as fair weather roads since such roads remain in serviceable condition only during the period when the weather is fair. These roads become almost unserviceable or present great difficulty to the free movement of traffic during rainy season.

The various types of low cost surfaces/roads are given below :-

(a) Earth roads (ordinary or natural soil roads as well as stabilized soil roads)
(b) Kanker roads ;
(c) Gravel roads ;
(d) Traffic bound macadam roads ;
(e) water bound macadam (W.B.M) roads.

2. High cost surfaces

The pavement surfaces constructed as well as maintained at high cost are known as high cost surfaces, and the roads with such surfaces are known as high cost roads.

These roads are also known as all weather roads since such roads remain in serviceable condition in all weathers throughout the year. These roads present no difficulty to the free movement of traffic even during rainy season.

The various types of high cost surfaces roads are given below :-

(a) Bituminous roads ;
(b) concrete roads.

Comparative Merits & De-merits of low cost and High cost surfaces/Roads

Sl. NO	points of comparison	Low cost surfaces/ Roads	High cost surfaces/Roads
01.	Initial cost	Their initial cost is less	Their initial cost is more
02.	construction & Repair	Their construction as well as repair is easy	Their construction as well as repair is difficult.
03.	Labour & supervision	These road surfaces require ordinary labour & supervision for their construction.	These road surfaces require skilled labour & supervision for their construction.
04.	Riding surface	These road surfaces provide inferior riding surface	These road surfaces provide superior riding surface
05.	Impervicuness of surface	Their surface is pervious.	Their surface is impervious.
06.	sanitary surface	Their road surfaces do not provide sanitary surface since they cannot be washed and cleaned easily.	These road surfaces provide sanitary surface since they can be washed and cleaned easely.

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07.	Dust nuisance	These road surfaces cause dust nuisance under pneumatic tyred traffic	These roads surfaces do not cause dust nuisance under pneumatic tyred traffic
08.	Suitability to weather	These road surfaces remain serviceable during the period when the weather is fair.	These road surfaces remain serviceable during all weathers even during monsoon.
09.	Traffic capacity	Their traffic capacity is less	Their traffic capacity is more.
10.	Life	Their life is less (1 to 4 years)	Their life is more (12 to 30 years)
11.	Maintenance cost	Their maintenance cost is low	Their maintenance cost is high

WATER BOUND MACADAM ROADS (W.B.M ROAD)

The road having its wearing surface consisting of clean, crushed aggregates, mechanically interlocked by rolling and bound together with filler material (screening) and water, laid on a prepared base course is called water bound macadam (W.B.M) road

In W.B.M roads, the aggregate used for their construction is known as macadam. These are superior type of low cost roads. These roads are constructed in thickness ranging between 8 to 30 cm, depending upon design requirements. The thickness of each layer of W.B.M is kept 12 to 15 cm and thus thickness upto 30 cm is composed of two or three layers. A camber of 1 in 36 to 48 is recommended for these roads.

These roads get deteriorated rapidly under the mixed traffic conditions & therefore, W.B.M is frequently used as base course for important high cost roads such as bituminous macadam, premix carpet and asphalt concrete roads.

Thus, a W.B.M road is considered as mother road for all types of modern road construction.

Construction of W.B.M roads

Materials required

The materials required for construction of a W.B.M road are coarse aggregates, screenings and binding materials, where necessary.

The coarse aggregates should be either crushed or broken stone, crushed slag, overburnt brick aggregate or one of the naturally occuring aggregates such as kankar or laterite of suitable quality. It should conform to one of the gradings mentioned in page 17 (The grading Requirement of coarse Aggregate for W.B.M Base course)

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The screenings, to be used for filling voids in the coarse aggregate should generally consist of same material as the coarse aggregate. However, where permitted, moorum or gravel (other than rounded river borne material) may be used for this purpose. The gradation requirements of screenings have already mention in page 17 (The gradation requirement of screening for WBM base course) Screenings of type A or B should be used with coarse aggregate of grading 2 and type B screenings should only be used with coarse aggregate of grading 3.

Binding material, to be used for WBM construction, should comprise a suitable material approved by the Engineer-in-charge, having plasticity index value of less than 6. Its application may not be necessary when the screenings used are of crushable type such as moorum or gravel.

All these materials required in the construction of a W.B.M road are stacked on the road side.

It is recommended to provide 20% extra broken stones on volume basis. Thus, the quantity of aggregate required for constructing one kilometre length per metre width and one centimetre thickness of W.B.M road surface will be $= \frac{120}{100} \times 1000 \times 1 \times \frac{1}{100} = 12\ m^3$

The quantity of screenings required is about $30\ m^3$ per $100\ m^2$ surface area of the road. Thus, the quantity of screenings for constructing 1 km length, per metre width of W.B.M road surface will be $\frac{30}{100} \times 1000 \times 1 = 300\ m^3$

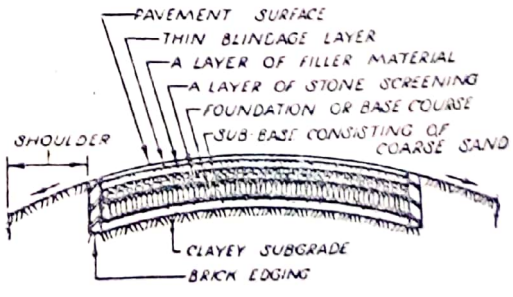
Method of construction

The construction of a W.B.M road is completed in the following stages :-

(i) preparation of sub-grade
(ii) preparation of the base course
(iii) preparation of intermediate and wearing course
(iv) preparation of shoulders
(v) opening to traffic

(i) preparation of subgrade

Unless otherwise specified, subgrade is in the form of a trench. The bottom level of the trench is fixed after deducting the pavement thickness from the finished formation level. Thus, for preparing the subgrade, a trench is dug in conformity with the lines, grade and cross-section after completion of earth-work as shown on the drawings.



A WATER BOUND MACADAM ROAD

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No trenching is necessary in rocky-cutting areas, where the rock excavation is done upto subgrade level and the haunch support to soling is provided by stone spall edging.

After making the trench, the subgrade is thoroughly compacted by rolling with a road roller of weight not less than 8 tonnes. Water is sprinkled uniformly on the subgrade on evening prior to rolling. Any low spot, that develop in the subgrade during rolling, is rectified and the surface is brought to grade as required.

On clayey subgrade, a layer of granular material such as natural sand, moorum, gravel, laterite or kankar is spread to a thickness varying from 100 mm to 150 mm.

(ii) Preparation of the base course

The base of foundation course consists of 12 to 18 cm size boulders or broken pieces of stones, long size kankar, overburnt bricks or brick soling. The width of this course should be 60cm wider than the pavement width, projecting 30 cm on each side.

After preparing the subgrade or sub-base, the required type of base course is constructed with specified materials in conformity with lines, grade, thickness and cross-sections.

(iii) Preparation of intermediate & wearing course

It is laid in one or two layers, depending upon the total designed thickness. The thickness of loose metal in each layer should not exceed 10 cm.

Preparation of intermediate & wearing course of a W.B.M road is done in the following steps:-

(a) preparing the surface
(b) providing edging or earthen kerbs
(c) spreading of coarse aggregate
(d) Dry rolling
(e) spreading of screenings
(f) Wet rolling
(g) Application of binding material, watering & rolling
(h) Finishing the surface
(i) setting & drying

(a) Preparing the surface

The surface of the newly laid base course i.e soling, on which some traffic has been allowed, is checked and the defective portions are rectified. In case of an existing metalled road, the surface is sacrified & brought to the required camber. The sacrifying may not be done in case when the thickness

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of renewal coat is 6.5 cm or above and stones of old course have not lost their angular faces or when the old surface consists of kankar.

(b) Providing edging or earthen kerbs

After preparing the surface brick on end edging is provided along the outer edges of the carriageway of the road. Where the edging is not possible as in the case of stone or kankar soling or remetalling, earthen kerbs, consisting of two parallel bounds of clay puddle 25cm x 15cm in section, are made. These earthen kerbs should be strong enough to prevent the new road metal from spreading as well as to retain the water used in consolidation of the wearing course.

(c) Spreading of coarse aggregate

The road metal is spread evenly over the prepared base to the specified thickness. After spreading the aggregate, it is hand packed with bigger pieces placed below and smaller pieces in the interstices and the surface is thus brought to the required camber.

(d) Dry rolling

After spreading the coarse aggregate, dry rolling is done by means of a suitable roller. This dry rolling provides interlocking of the aggregates. The rolling should be started from edges and gradually shifted towards the centre after properly rolling each strip.

The rolling should be continued until the aggregates are partially compacted with sufficient voids space in them to permit application of screenings.

(e) Spreading of screenings

After dry rolling, a bindage layer, consisting of stone screenings (12 mm grits) is spread at a slow and uniform rate so as to ensure filling of all voids. This is accompanied by dry rolling and brooming. These operations should be continued until no more screenings can be forced into the voids of the coarse aggregates.

(f) Wet rolling

After spreading the screenings, the surface is sprinkled over with sufficient quantity of water, swept & rolled.

(g) Application of binding material, watering & rolling

After the application of screening and wet rolling, the binding material is applied successively in two or more thin layers at a slow and uniform rate. After each application, the surface is freely sprinkled with water. The resulting slurry is swept in with brooms to fill the voids properly and rolled with 6 to 10 tonne roller. water is poured on the wheels of the roller to wash down the clayey material sticking to them. This layer of binding material is also known as blindage layer. The surface is then sprinkled over with heavy dose of water (40 to 55 litres per m^2 of surface area). The slurry is allowed to fill the voids and the

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surface is rolled with 6 to 10 tonne rollers.

The spreading of binder, sprinkling of water, sweeping with brooms & rolling should be continued until the slurry, that is formed after filling all voids, forms a wave before the wheels of the moving rollers.

(h) Finishing the surface

After the final compaction, road surface is allowed to dry overnight. Next morning, a layer of sand or earth, about 6mm thick, is spread on the surface. The surface is then lightly sprinkled with water & rolled.

(i) Setting and drying

The surface is then allowed to cure for 7 to 9 days. In dry season, the surface should be highly sprinkled with water during the curing period.

(*) The object of curing is to enable the road surface to become hard which remains otherwise soft due to heavy sprinkling of water.

(iv) Opening to traffic

After drying, the road is opened to traffic. The traffic should be well distributed over the full width of the road by placing obstacles longitudinally in the form of drums, branches of trees, etc. on the road surface. The process of placing such obstacles longitudinally on the newly prepared road surface for distribution of traffic is known as lik-katai

(v) Preparation of shoulders

During curing, the shoulders are prepared by filling earth to the specified cross slope. These are then properly compacted by rolling or tamping.

Defects of water bound macadam roads

In W.B.M roads, the aggregates are keyed together and prevented from moving out simply by means of slurry of screenings, sand and clay, since no other binding material is used. Binding effect of this slurry mostly depends upon the presence of moisture. When a fast moving vehicle passes over a W.B.M road, the binding material, in the form of slurry, is easily sucked out by pneumatic wheel tyres of vehicle and thus the surface is disintegrated. Under the very high contact pressure of iron tyred bullock carts and due to attraction effects, the aggregates are crushed to powder. Thus, mixed traffic is likely to destroy the W.B.M road in a very short time. This can be improved by surface dressing or painting.

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BITUMINOUS ROADS

The roads having their surface consisting of bituminous materials are known as bituminous roads or black-top roads.

These roads are constructed of different thickness varying from a thin layer of bituminous surface dressing to about 22 cm thick layers of bituminous materials, according to importance of the road. The bituminous binders used in the construction of bituminous roads are either straight-run bitumen, road tar, cut-back or emulsion.

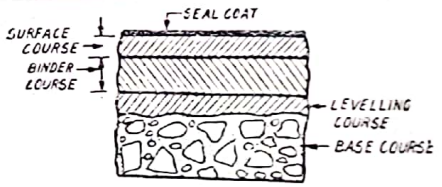
The choice of a particular binder depends upon the type of construction, availability of material, equipments, climatic conditions, etc.

Bitumen and tar require heating to bring them to proper viscosity before their use. The construction technique using these materials is known as hot mix technique - Since the bituminous materials are applied cold.

(*) The bituminous roads are considered as high cost roads. Since bituminous roads are economical in construction and can be immediately opened to traffic after their construction as compared to concrete road, these roads are in extensive use in developing nations like our country. These roads also facilitate stage development which help in improving their surface according to traffic demands in future.

Components of a bituminous Surface and their functions :-

A bituminous surface rests on a base course. If the base course is wavy or it has developed excessive irregularies, a layer of aggregates of varying sizes may be provided which is known as levelling course.



SECTION SHOWING COMPONENTS OF A BITUMINOUS SURFACE

The various components of a typical bituminous surface road shown above and are described below :-

(a) Binder course (b) surface course (c) seal coat (d) prime coat (e) Tack coat

(a) Binder course

The layer of bituminous material, provided in between the base course & surface course of a bituminous surface, is called binder course.

It serves as a stress distribution medium and provides an adequate bed for surface course.

(b) Surface course

The layer of bituminous material provided on the top of binder course of a bituminous surface, is called surface course or wearing course.

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This course is directly exposed to traffic. The thickness of this course is less than that of binder course but it is made of superior material.

(R) The function of this course is to provide a good riding surface and to bear safely the wheel pressure due to traffic safely.

(c) seal coat

A final coat of bituminous material provided on the top of surface course for sealing the voids against entry of moisture is known as seal coat.

The main functions of providing seal coat on a bituminous surface are given below:-

(i) To make the surface water-tight
(ii) To provide a more desirable surface texture.
(iii) To reduce slipperiness of the surface
(iv) To enliven an existing dry or weathered surface.

Seal coat may be either of the following two types :-

Type A

This type of seal coat is a liquid seal coat which consists of a layer of bituminous binder followed by a cover of stone chippings.

method of construction of Type A seal coat

For constructing Type A seal coat, the binder is heated in a boiler of suitable design and to the temperature appropriate to the grade of bitumen approved by the Engineer-in-charge. Then the seal coat is applied in accordance with the construction operations.

Type B

This type of seal coat is a premixed seal coat which consists of a thin application of fine aggregate premixed with bituminous binder.

method of construction of Type B seal coat

For constructing type B seal coat, the binder is heated in a boiler of suitable design and to the temperature appropriate to the grade of bitumen approved by the Engineer-in-charge. Then fine aggregates, in dry state, are also heated to specified temperature before the same are placed in a mixer. After this, mixing of binder with aggregates to the specified proportions is done in the mixer. Mixing should be continued till the aggregates are thoroughly coated with the binder. Then the mix is immediately transported from the mixing plant to the point of use and spread uniformly on the bituminous surface to be sealed. As soon as sufficient length of the bituminous surface has been covered with the premixed mix, the surface is rolled with 6-9 tonne smooth wheeled power roller. Rolling should be continued until the premixed material completely seals the voids existing in the bituminous course and a smooth uniform surface is obtained.

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Opening to traffic

In case of Type A seal coat, traffic should not be permitted to run on any newly laid surface till the following day. In Special circumstances, the Engineer-in-charge may open the road to traffic immediately after rolling, but in such cases, its speed should be limited to 16 km/hr till the Following day

Opening to traffic

In case of Type B seal coat, traffic may be allowed soon after final rolling when the premixed material has cooled down to the surrounding temperature.

→ In addition to the components of a bituminous surface described above, the following two coats are also recommended to achieve proper bond i.e adhesion of the bituminous surface with the existing base at the interface.

(d) Prime coat

A single coat of low viscosity bituminous binder applied to an existing untreated base of pervious nature like W.B.M, in order to promote adhesion between the base and the bituminous surface is called prime coat.

The bituminous binder used for prime coat is the low viscosity cut-back.

The functions of providing prime coat to an existing untreated road surface preparatory to any bituminous construction are given below:-

(i) To promote adhesion or bond between the existing pervious base & the wearing surface.
(ii) To bind together any loose aggregate which may be present on the base.
(iii) To plug the capillary rise of moisture.
(iv) To provide a temporary seal against the infiltration of surface water.

(e) Tack coat

A single coat of low viscosity bituminous binder applied to an existing treated base of impervious nature like bituminous or cement concrete base, in order to promote adhesion between the treated base and the bituminous surface is known as tack coat.

The bituminous binder used for tack coat is a bitumen of suitable grade as directed by the Engineer-in-charge and conforming to IS: 73, 217 or 454, as applicable or any other approved cut-back.

The function of providing tack coat is to ensure adequate bond between the existing impervious base and the wearing surface.

For applying a prime or tack coat, the surface should be thoroughly swept and scraped clean of dust and any other objectionable material. The binder is then heated to the temperature appropriate to the grade of bitumen used, and the rate of spread in terms of straight-run bitumen

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should be 5 kg per 10 m^2 area for an existing bitumen treated surface and 10 kg per 10 m^2 area for an untreated water bound macadam (W.B.M) surface. The binder should be applied uniformly with the aid of sprayers. This coat should be applied just before the next bituminous construction.

Types of Bituminous roads :-

The various bituminous roads are generally classified on the basis of their methods of construction. The following techniques are being used for constructing bituminous roads and they are named accordingly.

1. Inverted penetration type construction;
   Example : surface dressing
2. Penetration type construction;
   Example : penetration macadam
3. Premix type construction;
   Example : (i) premix macadam or bitumen bound macadam,
   (ii) Premix carpet
   (iii) Asphaltic concrete
   (iv) sheet asphalt or rolled asphalt
   (v) Mastic asphalt

(1) Inverted penetration type construction :-

In this type of bituminous road construction, the binder is sprayed first which is followed by application of aggregates. Light rolling is then done which sets the aggregates into the binder due to upward penetration of the later into the former. This upward penetration of binder into the aggregates is considered as inverted penetration type construction technique.

The best method of constructing a bituminous road by this technique is surface dressing or surface painting which is described below :-

Surface dressing or surface painting :-

The method of applying one or two coats of bituminous material, each consisting of a layer of bituminous binder sprayed on the prepared base, followed by a cover of stone chipping properly rolled to form a wearing course is known as surface dressing or surface painting.

When one layer of bituminous material is applied, the process is called single-coat surface dressing, and when two layers are applied, it is known as two-coat surface dressing. The thickness of this type of surfacing varies from 20 to 30 mm. The bituminous roads having their surface course prepared by surface dressing or surface painting are known as

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: Surface painted roads.

Surface dressing thus provides a thin, water proof and dustless wearing surface. Though the film does not reinforce the pavement structure, yet it provides a better surface than a W.B.M road. Surface painted roads can take light pneumatic traffic very well and can withstand only some portion of bullock cart traffic because too much of iron wheeled traffic will cause depressions and pot holes very soon and thus the road will become unserviceable and uncomfortable. Hence, such roads are recommended for light pneumatic traffic such as buses, trucks cars, etc. mixed with some iron wheeled traffic.

Advantages :-

The following are the advantages of surface dressing or surface painting on water bound macadam and other low cost roads :-

(i) It prolongs the life of the road by providing a water-proof surface.
(ii) It eliminates the dust nuisance.
(iii) It provides a sanitary surface since the road can be easily cleaned and washed.
(iv) It yields a non-skid surface and thus provides comfort to the traffic
(v) It provides a smooth surface & thus the wear & tear of vehicle tyres is reduced.
(vi) It provides less frictional resistance and thus the petrol consumption by vehicles is reduced.

Limitations / Disadvantages :-

The following are the limitations of surface dressing or surface painting :-

(i) It does not strengthen the pavement
(ii) It cannot rectify the irregularities in the existing base.
(iii) It is unsuitable under heavy traffic

(2) Penetration type construction :-

In this type of bituminous road construction, the aggregates are spread on the prepared base & compacted by a roller. Then the bituminous binder is sprayed on the surface which penetrates to full or part depth of the compacted aggregates & thus binds them together.

The best method of constructing a bituminous road by this techniques is grouted or penetration macadam as stated below/next page...

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Grouted or penetration macadam :-

The method of bituminous road construction in which the aggregates are bound together by grouting bitumen into the voids to full or part depth of the compacted aggregate is called grouted or penetration macadam. It is also known as built-up spray grout macadam.

When the bitumen is allowed to penetrate to full depth of compacted aggregate, the process is known as full grout macadam, and when the penetration of bitumen is upto about half the depth of the compacted aggregate, the process is called semi-grout macadam.

This type of bituminous road construction is usually adopted for preparing base course of important bituminous surfaces, subjected to heavy and mixed traffic. Full grout macadam construction is adopted in regions of heavy rainfall and the semi-grout in regions of average rainfall. The recommended thickness for full grout is 5 to 8 cm and that for semi-grout macadam construction, it is usually kept 5 cm.

(3) Premix type Construction :-

The bituminous road construction in which the aggregates and the binder are mixed together in a mixing plant prior to their placing & spreading is known as premix type construction.

Advantages :- The following are the advantages of premix type construction over grouted or penetration macadam construction :-

(i) It facilitates to coat each and every individual aggregate with uniform thickness of binder film whereas it is not possible in grouted macadam construction.

(ii) It requires comparatively less quantity of bitumen to achieve the same strength.

(iii) It provides increased stability of the mix as compared to grouted macadam construction.

The various bituminous roads constructed according to premix type construction are
a. premix macadam
b. premix carpet
c. Asphaltic concrete
d. sheet asphalt
e. Mastic asphalt

a. Premix macadam

The premix type construction in which a mix of dense graded macadam and bitumen is used in hot state for constructing the base course or wearing course of a bituminous road is known as premix macadam or bitumen bound macadam construction.

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Thus, the bituminous roads having their wearing surface prepared by premix macadam are known as premix macadam bituminous roads or bitumen bound macadam roads. The finished thickness of premixed macadam varies from 5 to 7.5cm.

In this method, bitumen and dense graded macadam are heated separately to specified temperature and mixed in a bitumen mixer. The quantity of bitumen varies from 3.5 to 4 percent by weight of the total mix for aggregate of grading No A and B. This hot mixture is then laid on the prepared base in between the kerbs and adequately rolled.

It has been observed in advanced countries that bitumen bound macadam for base course construction results in stronger and economical pavement section than other conventional type of bituminous pavements. These days, the use of bitumen bound macadam is also recommended for constructing bases as they can last longer under adverse climatic and poor subgrade situations.

b. Premix Carpet

The premix type construction in which a single layer of mix, composed of stone chippings, sand and binder is laid in hot state for constructing the wearing course of a bituminous road is called premix carpet construction.

Thus, the bituminous roads having their wearing surface constructed of premix carpet are known as premix carpet roads.

The thickness of premix carpet varies between 20 to 25 mm. General practice in our country is to provide 20 mm thick carpet for surface course laid on a previously prepared bituminous or non-bituminous base course.

This type of bituminous road construction provides smooth surface and pleasant looking finish. It is quite suitable for light and moderate traffic. It is a very popular bituminous surface provided on roads.

c. Asphaltic concrete

The premix type construction in which a single layer of 25 to 50mm thick asphaltic concrete is laid in hot state for constructing the wearing course of a bituminous road is called asphaltic concrete construction, and the mix used is called asphaltic concrete.

Thus, the bituminous roads having their wearing surface constructed of asphaltic concrete are known as asphaltic concrete roads.

The term asphaltic concrete denotes the mix of high cost specifications. Such mixes should, therefore, be properly prepared to satisfy the design requirements of the stability and durability of road surface. Asphaltic concrete consists of a well proportioned mixture of coarse aggregates,

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fine aggregates, mineral filler and a bitumen binder. The mix is prepared in a hot mix plant. The asphaltic concrete, thus prepared is then laid & compacted to the desired thickness under rigidly controlled conditions. Asphaltic concrete used in the construction of superior type of bituminous surface is of hot-mix hot-laid type. The other varieties such as hot-mix and cold-mix cold-laid are used under exceptional circumstances and for repair work. Hot-mix hot-laid is the best type of asphaltic concrete, and the road surface, so prepared, can be opened to traffic as soon as it cools down to the surrounding temperature.

Asphaltic concrete road surfaces are provided on heavy duty highways. In India, 40 mm thick surface courses of asphaltic concrete are generally provided. The outstanding merits of asphaltic concrete roads are their high stability and durability under heavy loads and adverse climatic conditions.

d. Sheet asphalt

The premix type construction in which a carpet of sand and bitumen mix is used for preparing the wearing course of a bituminous road is known as sheet asphalt or rolled asphalt construction, and the mix used is called sheet asphalt.

Thus, the bituminous roads having their wearing surface constructed of sheet asphalt or rolled asphalt are known as sheet asphalt roads or rolled asphalt roads

Sheet asphalt provides a better type of wearing surface. It consists of 60 mm thick layer of asphaltic concrete over which 20 to 40 mm thick carpet of sand-bitumen mix is laid. This type of construction is generally used as wearing course on existing W.B.M or cement concrete slab base.

e. Mastic asphalt

The premix type construction in which a well proportioned mixture of bitumen, fine aggregate and filler is used for preparing the wearing course of a bituminous road is known as mastic asphalt-construction, and the mix used is known as mastic asphalt.

Thus, the bituminous roads having its wearing course constructed of mastic asphalt are known as mastic asphalt roads.

Mastic asphalt provides a hard, stable and durable wearing course which is capable of withstanding heavy traffic. It can also withstand vibrations and has got property of self healing of cracks without bleeding. Thus, this type of surfacing is very suitable for road pavements over bridges

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Construction procedure for surface dressing :-

The surface dressing is done either in a single coat or in two coats for providing a renewal or wearing course over an existing or new water bound macadam base course.

(a) Materials required

The materials required for surface dressing include bituminous binder and stone chippings.

The binder used for surface dressing should be straight-run bitumen of a suitable grade as directed by the Engineer-in-charge and conforming to IS : 73

The selection of binder is done keeping in view the climatic conditions, availability of materials, equipments etc. The grade of the bitumen is decided according to the climatic condition. However, 80/100, 180/200 penetration grade bitumen are commonly used. The quantity requirements of binder required for surface dressing, as per IRC specifications are given in table

IRC Specifications for Qauntity Requirements of Binder Required for Surface Dressing

S. No.	Type of Base Course	Recommended Quantity of Binder in kg per 10 m² of Road Surface					
		First or single coat			Second coat		
		Bitumen	Tar	Cut-back	Bitumen	Tar	Cut-back
1.	*Water bound macadam*	17 to 19 5	17 to 22	19 to 22	10 to 12	12 to 15	12 to 15
2.	*Renewal of black top surfacing*	10 to 12	12 to 17	10 to 12	10 to 12	12 to 15	12 to 15

Note :—The quantity of binder required for 10 m² of road surface dressing is it usually taken as 18 kg for single coat or first coat of *two coat surface dressing* and 11 kg for second or renewal coat on all types of base courses.

The stone chippings, used for surface dressing, should consist of fairly cubical fragments of clean, hard, tough and durable rock of uniform quality throughout. These are obtained by crushing stone, river gravel or other approved materials.

The size and quantity of stone chippings should be in accordance with the IRC specification given as per following table. The stone chippings should satisfy the physical requirements already mentioned in page-22 (B) physical requirement

IRC Specifications for Gradation and Quantity Requirements of Stone Chippings for Surface Dressing

S. No.	Type of Construction	Normal size of Stone Chippings	Recommended Quantity for 10 m² of Road Surfaces	Specifications
1.	*Single coat surface dressing or the first coat of two coat surface dressing*	12 mm	0 15 m³	100 per cent passing through 20mm IS sieve and retained on 10 mm IS sieve.
2.	*Renewal coat or second coat of two coat surface dressing*	10 mm	0 10 m³	100 per cent passing through 12 5 mm IS sieve and retained on 6 3 mm IS sieve.

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(b) Method of construction

Surface dressing is completed in the following steps :-

(i) preparing the base
(ii) Application of binder
(iii) Application of stone chippings
(iv) Rolling
(v) Application of second coat of surface dressing
(vi) Finishing the surface & opening to traffic

(i) Preparing the base :-

Usually, surface dressing is done on an existing or new W.B.M road surface. In case of an existing W.B.M road, the base is prepared, shaped and conditioned to the specified lines, grade and cross-sections in accordance with the desired specifications, as directed by the Engineer-in-charge. In the case of a new W.B.M road, the bindage and blindage layers are not applied. In that case, the aggregates are properly rolled by sprinkling water & when road metal is sufficiently interlocked & the surface becomes dry, the coat of surface dressing is applied.

The surface is then thoroughly swept and scraped clean of dust and any other harmful matter before spraying the binder. As necessary, the cleaning of surface should be done first with hard brushes, then with soft brushes and finally by blowing with sacks or gunny bags.

(ii) Application of binder :-

The binder is heated to 160° - 177°c and then sprayed on the prepared base in a uniform manner preferably with the help of a mechanical sprayer. Excessive deposits of binder, if any, should be suitably corrected before the stone chippings are spread.

(iii) Application of stone chippings :-

Immediately after the application of binder, stone chippings in a dry & clean state are spread uniformly on the layer of binder. A twisting motion is given to baskets while throwing chippings to avoid segregation. The spreading of stone chippings is preferably by means of a mechanical gritter. If necessary, the surface may be broomed to ensure uniform spreading of stone chippings.

(iv) Rolling :-

Immediately after the application of stone chippings, rolling is done by 8 to 10 tonne smooth wheeled roller. Rolling is done longitudinally, starting from edges and working gradually towards the centre except in superelevated portions where it should proceed from the inner edge to the outer. Each pass of the roller should uniformly overlap not less than one-third of the strip rolled in

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the preceeding pass.

While rolling is in progress, additional chippings, in required quantity, may be spread by hand to make up irregularities. Rolling should be continued until all aggregate particles are firmly bedded in the binder and present a uniform closed surface.

This is the final rolling in case the surface dressing is to be done in single coat.

(v) Application of second coat of surface dressing :-

In the case of two-coat surface dressing, application of second coat is done in steps in a similar manner as described/written in the case of single coat surface dressing.

(vi) Finishing the surface & opening to traffic :-

After final rolling, the surface is checked for its profile. A tolerance of 6mm unevenness for every 3m length is permissible. After finishing the surface, the road is opened to traffic after 24 hours. In special circumstances, however, the Engineer-in-charge may open the road to traffic immediately after rolling, but in such cases, its speed should be limited to 16 km per hour till the following day.

(c) Quality control for surface dressing

control on the quality of materials and works can be exercised by taking the following precautions :-

1. Materials should be checked for specification requirements. For this purpose, one check per 50 cum of aggregate has been recommended by the IRC.
2. Surface dressing should be permitted after the underlying course has been repaired to correct profile and grade.
3. Surface dressing should not be done under the following situations :-
   (i) when the temperature in the area is less than 16°C
   (ii) when aggregates are damp
   (iii) when weather is foggy, rainy or dusty
   (iv) when underlying surface is wet.
4. Temperature of binder should be checked regularly
5. Rate of application of materials should be checked once in every 500 sq.m of surfacing. Rate of spraying of binder should be within 2.5% of the specified rate.
6. Aggregates should be spread uniformly without causing segregation.
7. Excessive rolling should not be permitted.
8. second coat, if specified, should be applied immediately after applying the first coat.

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9. Road should be kept closed to traffic for 24 hours after completion, on special circumstances, however, the Engineer-in-charge may open the road to traffic immediately after rolling, but in such cases, its speed should be limited to 16 km per hour till the following day.

Construction of premix carpet

The construction of a premix carpet road is carried out under the following stages:-

(a) Materials required
(b) Method of construction
(c) Quality control for premix carpet

(a) Materials required

The materials required for premix carpet construction include aggregate & the bituminous binder.

The aggregates should consist of angular fragments of clean, hard, tough & durable rock of uniform quality throughout. The gradation and quality requirements for 20mm thick premix carpet for surface course are given in following table

JRC Specifications for Gradation and Quantity Requirements of Aggregates for 20 mm Thick Premix Carpet

S. No.	Aggregate Size	Sieve Size, (mm)		Quantity in m^3 for 10 m^2 of Road Surface
		passing	retained	
1.	*Stone chippings (12 mm)*	20	10	0·18
2	*Stone chippings (10 mm)*	12·5	6·3	0·09

Note: Stone chippings of both the sizes are mixed in the ratio of 2 : 1.

The binder used for premix carpet should be of a suitable grade as directed by the Engineer-in-charge, and satisfying the requirements of IS: 73, 217, 454 or other approved cut-back. Bitumen of 80/100 grade or road tar of grade RT-3 is used as binder for premix carpet construction.

The quantities in terms of straight-run bitumen for 20mm thick premix carpet construction are given in following table...

JRC Specifications for Quantity Requirements of Binder for 20 mm thick Premix Carpet Construction

S. No.	Detail of Construction	Recommended Quantities of Binder (in terms of Straight run Bitumen)
1.	*For 0·18 m^3 of 12 mm size stone chippings at 52 kg per m^3*	9·5 kg
2.	*For 0·09 m^3 of 10 mm size stone chippings at 56 kg per m^3*	5·1 kg
	Total quantity of binder for 1 m^2 of road surface	14·6 kg

(b) Method of construction

The bituminous premix carpet construction for wearing course of the road is completed in the following steps:-

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(i) preparing the base
(ii) Applying prime coat or tack coat
(iii) Preparation of mix
(iv) Transporting and spreading the mix
(v) Rolling
(vi) Applying seal coat
(vii) Opening to traffic.

(i) Preparing the base :-

The underlying base on which the premix carpet is to be laid is prepared, shaped and conditioned to the specified lines, grade and cross-sections.

(ii) Applying prime coat or tack coat :-

After preparing the underlying base, a prime coat is applied when there is untreated W.B.M surface. The tack coat is applied when there is an existing black top or cement concrete slab surface. This coat is applied by spraying bitumen at specified rate & temperature by means of a spraying device fitted to the tar-boiler. The tack coat is, however, not necessary when the laying of carpet follows soon after the provisions of a bituminous base / levelling course.

(iii) Preparation of mix :-

The mix of aggregates and binder in specified ratio is prepared in a hot-mix plant or hot mix unit. Aggregates are heated to the specified temperature before these are placed in the mixer. After about 15 seconds of dry mixing, the binder is distributed over the aggregates at the specific rate. These are then thoroughly mixed to ensure uniform coating. Excessive mixing is avoided & the temperature during mixing is rigidly controlled.

(iv) Transporting & spreading the mix :-

The mix is immediately transported from the mixter to the site of use in dump trucks or wheel barrows. The mix is then spread on the prepared base with rakes to the required thickness and camber or distributed evenly with the help of a drag spreader or paver finisher immediately after applying the prime or tack coat. The profile and thickness is adjusted with the help of screeds.

(v) Rolling :-

As soon as sufficient length (about 15m) of premix materials has been laid rolling is started with 6-9 tonne smooth wheeled power roller. All the necessary precautions should be taken during rolling. Rolling is continued until the entire surface has been rolled to compaction & all the roller marks are eliminated. The roller wheels should be kept damp to prevent the mix from sticking to the wheels and being picked up.

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(vi) Applying seal coat :-

After rolling, the seal coat is provided. The most popular seal coat is type B which consists of a layer of sand-bitumen mixture, laid hot on the carpet and rolled.

(vii) Opening to traffic :-

In case Type B seal coat, traffic may be allowed soon after final rolling when the premixed material has cooled down to the surrounding temperature. However, as regards Type A seal coat, the Engineer-in-charge may open the road to traffic immediately after rolling, but in such a case, its speed should be limited to 16 km per hour till the following day.

(c) Quality control for premix carpet

Control on the quality of materials and works can be exercised by taking the following precautions :-

1. Materials should be checked for specification requirements. For this purpose, one test per 50 cum of aggregates has been recommended by IRC
2. Base binder course should conform to the specified grade and camber before laying the premix.
3. premix construction should not be permitted under the following situations :-
   (i) Temperature in the area is less than 16°C
   (ii) Underlying surface of materials are wet or damp.
4. Binder content in the mix should be within 2.5% of specified rate. It should be checked twice in a day.
5. Mixing of material should be thoroughly done at specified temperature but excessive mixing should be avoided as it hardens the bitumen.
6. Spreading and rolling should be done at proper temperatures. Rate of spread of the mix should be controlled through checks.
7. Seal coat, if specified, should be provided evenly and the surface is then finally rolled.
8. The finished surface should be checked for profile and grade. Deviations beyond permissible tolerance should not be allowed.
9. Traffic should not be allowed until the surface cools down to the surrounding temperature when type B seal coat is provided. whereas in the case of Type A seal coat, the speed of traffic should be regulated to 16 km per hour for one day after opening the road to traffic

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construction of asphaltic concrete surfaces

The construction of an asphaltic concrete surface is carried out under the following stages :-

(a) Materials required
(b) Methods of construction
(c) Quality control for asphaltic concrete construction

(a) Materials required :-

The materials required for constructing an asphaltic concrete surface include coarse aggregate, fine aggregates, material filler and the bitumen binder.

The coarse aggregates to be used for asphaltic concrete construction are crushed stone, crushed gravel (shingle) or other suitable stones. These aggregates should be clean, strong, durable and of fairly cubical shape. They should satisfy the physical requirements already given in page-22 table (B) physical requirements.

The fine aggregate to be used for asphaltic concrete construction is the fraction passing 2.36 mm IS sieve and retained on 75 micron IS sieve. These aggregates should be consisting of crusher run screenings, natural sand or mixture of both. They should be clean, hard, durable, uncoated & dry.

The filler to be used for asphaltic concrete construction is an inert material, the whole of which passes 600 micron IS sieve, atleast 90 percent passing 150 micron IS sieve and not less than 70 percent passing 75 micron IS sieve. The filler should be consisting of stone dust, cement, hydrated lime, fly ash or other non-plastic mineral matter approved by the Engineer-in-charge.

The aggregates including mineral filler to be used for asphaltic concrete construction should be so graded or combined as to conform to the requirement as per following table.

IRC Specifications for Gradation Requirements of Aggregates for Asphaltic Concrete Construction

Sieve Size	Percentage by Weight	
	Grading 1	Grading 2
20 mm	—	100
12.5 mm	100	80—100
10 mm	80—100	70—90
4·75 mm	55—75	50—70
2·36 mm	35—50	35—50
600 micron	18—29	18—29
300 micron	13—23	13—23
150 micron	8—16	8—16
75 micron	4—10	4—10

Note :—For compacted layer thickness of 25—40 mm of asphaltic concrete construction, any of the two gradings can be used but for layer thickness of 40—50 mm, only grading 2 should be used.

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The binder to be used for asphaltic concrete construction is straight-run bitumen of a suitable grade satisfying the requirements of IS : 73. However, bitumen of grades 30/40, 60/70 and 80/100 can be used for asphaltic concrete construction. The actual grade of the binder to be used is decided by the Engineer-in-charge, keeping in view the climatic conditions of the area.

The quantity of binder to be used for asphaltic concrete construction varies from 5 to 7.5 percent by weight of the total mix.

(b) Method of construction :-

Asphaltic concrete construction for wearing surface of a road is completed in the following steps :-

(i) Preparing the base
(ii) Applying tack coat
(iii) Preparation of mix
(iv) Transporting and spreading the mix
(v) Rolling
(vi) Opening to traffic

(i) Preparing the base :-

Asphaltic concrete surface is generally laid on a bituminous base. Permanent side supports such as kerbs of stone, precast cement concrete blocks or first class bricks are provided well in advance of laying asphaltic concrete surface course. For laying an asphaltic concrete wearing surface, the underlying base is prepared, shaped and conditioned to the specified levels, grade and camber. The base is then thoroughly swept and scraped clean to make the surface free of dust and other foreign matter.

(ii) Applying tack coat :-

After preparing the underlying base, the tack coat of suitable binder (at the rate of 5 to 10 kg per 10 sqm) is applied on the prepared surface.

Application of tack coat is, however, not necessary when the laying of asphaltic concrete follows soon after the provision of a bituminous base / levelling course.

(iii) Preparation of mix :-

Asphaltic concrete mix is made in a central hot mix plant under strictly controlled conditions. Aggregates conforming to specified grading are heated to 155° — 163°C and the bitumen is is heated to 150° - 177°C. Materials,

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in pre-determined proportions, are then thoroughly mixed to ensure that a homogeneous mixture is obtained in which all particles of the mineral aggregates are coated with the binder uniformly.

(iv) Transporting and spreading the mix :-

The prepared mix is then immediately transported from the mixing plant to the site of use in suitable vehicles such as dump trucks, wheel barrows, etc. After transporting the mix to the site of use, it is immediately spread by means of a self-propelled mechanical paver provided with suitable screeds capable of spreading tamping and finishing the mixture to the specified lines, grade and cross-sections. The temperature of the mix at the time of laying should be in the range of 121° – 163 °C.

(v) Rolling :-

Immediately after spreading the mix, it is thoroughly compacted by a set of rollers, moving at a speed not exceeding 5 km per hour. The initial rolling is done with 8 – 12 tonne three wheeled roller and the surface is finished by final rolling with 8 – 10 tandem rollers. The wheels of the roller should be kept moist to prevent the mix from sticking to them. Usual precautions should be taken during rolling the surface. Rolling should be continued until the density achieved is at-least 95% of that of the laboratory specimen and all roller marks are eliminated.

(vi) Opening to traffic :-

After final rolling, the traffic is permitted as soon as the surface cools down to the surrounding temperature.

(c) Quality control for asphaltic concrete construction

Asphaltic concrete is a very extensive mix. Therefore, a rigid control on quality is exercised on every stage of asphaltic concrete construction. A field laboratory is set up for this purpose. Quality control recommendations for asphaltic concrete construction as per IRC : 29 – 1968 are given below :

1. Periodic sieve analysis of each type of aggregate at the cold feeder end of the mixing plant should be done to see that the gradations of aggregates reasonably follow the original gradations as per design. The number of samples per day would depend upon the number of bulk supply of aggregates made in a day at the plant site.

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2. Periodic checks on penetration and softening point of the binder should also be done in the manner specified by ISI.

3. Aggregates should not be wet. The temperature of aggregates in dryer should not exceed 163 °C. A tolerance of 8 °C on lower side is permitted.

4. The binder should be heated to a temperature between 150° to 177 °C. Temperature of binder is checked regularly.

5. At no time, the difference between temperature of aggregates and that of binder should exceed 14 °C

6. Gradation of aggregates should be checked periodically.

7. At least one sample for every 100 tonnes of asphaltic mix, discharged at the exit of pug-mill or minimum one sample per plant per day should be collected and tested for :-
   (i) stability, flow, void ratio and density requirements.
   (ii) Binder content
   (iii) Gradation requirements.

8. Temperature of mix while laying should not exceed 163 °C and it should not be less than 121 °C.

9. Rolling should be done when the mix is neither too hot nor too cold so that the development of hair cracks can be eliminated.

10. Density of compacted surface in the field should be determined once for every 1000 sq.m of the road surface. It should not be less than 95% of the density obtained in the laboratory.

11. Finished surface is tested with a straight edge 4.5 m long and irregularities greater than 6 mm should be corrected.

-: CEMENT CONCRETE ROADS :-

The roads having their wearing surface consisting of cement concrete slab (plain or reinforced) are called cement concrete roads or simply concrete roads.

These roads fall under the category of rigid pavements. These are high cost roads which remain in serviceable condition under all weather conditions. Thus, concrete roads are also considered as all weather roads. Due to their excellent riding surface, pleasing appearance and long life under most severe traffic conditions, the cement concrete roads are much preferred. But due to their high initial cost and inadaptability to stage construction,

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concrete roads are not popular in India.

These roads are very popular as high cost pavements in developed countries like America, Russia, Japan, etc. Considerable research has been done towards improvements in the design and construction of concrete road pavements.

⊛ These roads are superior to most of other types of roads including bituminous roads.

Advantages and dis-advantages of cement concrete roads :-

Advantages :-

(i) concrete roads have got long life.
(ii) They are quite durable and are practically unaffected by weathering agencies.
(iii) They provide an excellent riding surface under all weather conditions.
(iv) They provide an impervious surface.
(v) They provide a dustless and sanitary surface.
(vi) They do not develop corrugations.
(vii) They can be laid on any subgrade.
(viii) They can be easily reinforced when they are to resist high stresses due to heavy wheel loads of the traffic.
(ix) They are practically non-slippery.
(x) They offer less tractive resistance.
(xi) They provide good visibility for traffic during night hours.
(xii) Their maintenance cost is very low,

Disadvantages :-

(i) The initial cost of concrete roads is high.
(ii) They require skilled supervision and labour for their construction.
(iii) They are liable to crack and warp due to temperature variations.
(iv) They become noisy under iron-tyred traffic
(v) They are less resilient than bituminous or W.B.M roads.
(vi) They may cause glare due to reflected sun-light.
(vii) They require long time (upto 28 days) for their curing and thus they cannot be opened to traffic earlier.
(viii) It is very difficult to locate & repair sewers and water mains lying under the pavement in their case.

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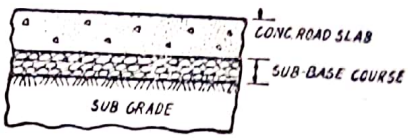
Structural components of concrete pavements :-

The following are the structural components of concrete pavements, starting from the bottom :-

(a) Subgrade ;
(b) sub-base ;
(c) Concrete slab



STRUCTURAL COMPONENTS OF CONCRETE PAVEMENTS

The cement concrete road pavements may be constructed with or without sub-base, depending upon the type of subgrade. The various functions of the sub-base course beneath the concrete slab are :-

(i) to provide a strong supporting layer.
(ii) to provide a capillary cut-off, preventing damages due to mud pumping.
(iii) To reduce the thickness of concrete slab.

The concrete road slab performs the functions of base course as well as of wearing surface. The thickness of road slab varies from 15 to 20 cm, depending upon the traffic load. In case of heavy traffic and poor subgrade, reinforcement may be provided in the concrete road slab.

Materials required for concrete road slab & their requirements :-

The different materials required for constructing a road slab are given below :-

(a) cement
(b) coarse aggregate
(c) Fine aggregate
(d) water
(e) Reinforcement

(a) **cement** :- Ordinary portland cement is mostly used for concrete road slab construction. Rapid hardening cement may also be used in emergency when the construction is to be completed in short time.

The cement to be used in the construction of a road slab should fulfil all the requirements

(b) **Coarse aggregate** :- Crushed stone, crushed gravel (shingle), other suitable stone is used as coarse aggregate for concrete road slab construction. It should be clean, strong, durable and free from dirt, dust etc.

The desirable values of important properties of coarse aggregates for cement concrete road construction are stated ....

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(i) Los Angeles abrasion value ——— 16% max

(ii) Aggregate impact value ——— 30% max

(iii) Aggregate crushing value ——— 30% max

(iv) Soundness (average loss in weight after 10 cycles) ——— (a) 12% max in sodium sulphate
(b) 18% max in magnesium sulphate.

(c) **Fine aggregate** :- Natural sand is commonly used as fine aggregate. when natural sand is not available, crushed stone sand may be used. It should be sharp and clean.

(d) **Water** :- Water is used for mixing and curing of concrete. It should be free from harmful substances. The water fit for drinking purposes is recommended for preparing the concrete mix to be used for constructing a road slab and also for its curing.

(e) **Reinforcement** :- It consists of mild steel bars or steel wire fabrics. It is to be provided when the subgrade is of poor strength and the road is subjected to heavy traffic. The usual practice of providing reinforcement in a concrete road slab is to place the steel at 3 to 7 cm depth below the surface of the slab.

The basic object of providing reinforcement is to prevent widening of cracks developed in the concrete road slab.

The following are the advantages and disadvantages of providing reinforcement in a concrete road slab :-

Advantages :-

(i) It prevents the cracks from widening and thus entry of dirt and water in the road structure is prevented.

(ii) It strengthens the structure of concrete pavement in case of weak sub-grade and thus prevents the cracking of a road slab in its early age.

(iii) It facilitates provision of thinner sections of the road slab.

(iv) It controls warping, prevents cracking and increase the flexural strength of the road slab and thus ultimately prolongs the life of the concrete road.

Disadvantages :-

(i) It increases the cost of construction of the concrete road.

(ii) It interferes with the smooth progress of the construction work & thus delays the construction of the road pavement.

(iii) The construction of reinforced concrete road slab requires more skilled labour and supervision than that of a plain concrete slab.

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Methods of laying cement concrete road slab :-

The cement concrete slabs are laid after taking the convenient length of the road at a time and then dividing it into bays or strips of suitable length. The length of these bays is kept 4 to 5 m and their width varies according to the width of pavement

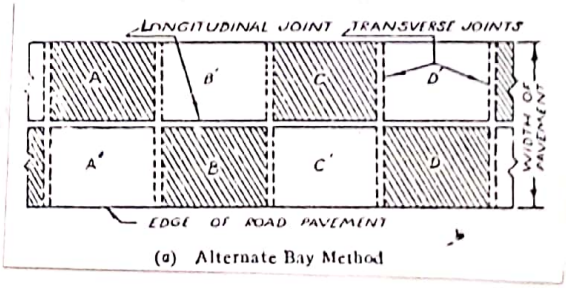
When the proposed width of pavement is less than 4.5m, the width of each bay is kept equal to the pavement width. In case, the pavement width is more than 4.5m, the width of each bay is kept 2.5m to 3.5m so as to divide the whole width into a number of strips. Thus, a 7m width pavement is divided into two strips with a central longitudinal joint. In this case, the width of each bay will be 3.5m and length varying from 4 to 5m.

There are the following two methods of laying cement concrete road slabs:-

(a) Alternate bay method;

(b) continuous construction method.

(a) Alternate bay method :-



(a) Alternate Bay Method

In this method of construction, bays or slabs are constructed in alternate succession (ABCD), leaving the intermediate bays (A'B'C'D') as shown in figure alongside. These intermediate bays are constructed after a gap of atleast one week if ordinary portland cement is used and two days in the case when rapid hardening cement is used.

The main object of this method is to allow the concrete placed in the previous bays to set and take up their initial contraction before the intermediate bays are constructed.

This method is practicable and found useful when the proposed width of pavement is more than 4.5m. This method provides additional working convenience for laying of slabs. In this method, the construction of joints is also easier but this method has got ~~many~~ many drawbacks.

The main draw backs of alternate bay method are given below:-

(i) Large number of transverse joints are to be provided.

(ii) There is risk of sub-soil drying out when laying the intermediate bays.

(iii) During rains, the surface water collects on the subgrade and there is danger of uneven settlement of the subgrade.

(iv) The construction is spread over full width of the road pavement and the traffic will have to be completely diverted.

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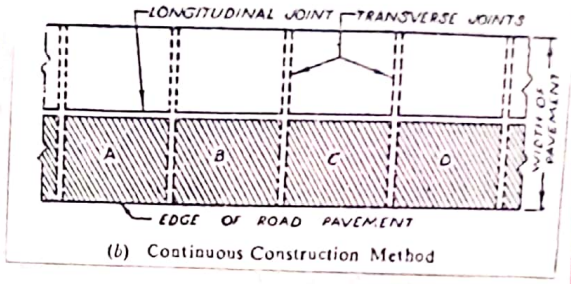
(b) Continuous construction method :-

In this method of construction, all the bays or slabs (ABCD) of a strip are constructed continuously without any break as shown in figure alongside. In this case construction joints are, however, provided when the day's work is not ended at the specified joint.



(b) Continuous Construction Method

In addition to these construction joints, dummy joints are also provided at 5m intervals in the transverse direction to check the planes of weakness & to control cracking. In this case, expansion joints are constructed at about 16 to 20 m intervals i.e after every fourth slab.

This method is generally preferred as compared to alternate bay method because of its main advantage of construction of half the pavement width at a time. Thus, the essential traffic can be diverted on the other half of the road. This method is also very rapid. The only drawback of this method is that the construction of joints is difficult in its case.

Construction of cement concrete roads :-

The construction of cement concrete roads involves the following operations:-

(a) preparation of subgrade;
(b) provision of sub-base;
(c) placing of forms;
(d) Batching of materials and mixing;
(e) Transporting and placing of concrete;
(f) Compaction;
(g) Floating;
(h) Belting;
(i) Brooming;
(j) Edging;
(k) curing;
(l) Filling of joints and edging;
(m) opening to traffic.

(a) Preparation of subgrade

The subgrade to receive the road slab is prepared to the required grade and profile in the usual manner. The profile is checked by scratch templates. Uneveness greater than 12 mm in 3m, if any should be rectified. When concrete is to be directly placed on subgrade, the surface should be saturated with water 6 to 20 hours in advance of placing the concrete. This is done

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to ensure that the subgrade does not absorb water from the concrete. If water proof paper is to be laid between subgrade and concrete, moistening of subgrade may not be done.

(b) Provision of sub-base

When natural subgrade is not very firm, a sub-base over the subgrade is desirable. It may consist of any one of the following layers :-

(i) A layer of well graded soil - gravel mixture of maximum thickness 15 cm.

(ii) Brick soling with one layer of W.B.M of maximum total thickness 10 cm

(iii) Two layers of W.B.M of maximum total thickness 15 cm.

(iv) A layer of lean cement concrete of maximum thickness 10 cm.

When the subgrade contains harmful salts, a capilbary cut-off of any of the following materials may be provided below the sub-base :-

(i) 4 to 6 cm coarse sand

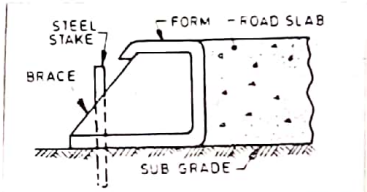
(ii) 30 to 45 cm fine sand

(iii) A suitable cut off of bituminous material

When subgrade soil is very poor, the sub-base should be placed over a blanket of some granular material or stabilized soil.

(c) placing of forms

concrete road slab is constructed between side forms. These forms are made of mild steel channel section in 3m length. The depth of form is equal to the thickness of concrete pavement. Forms are properly braced and fixed to the ground by means of stakes. Three stakes for every 3m length are provided. Forms lengths are connected to each other by suitable arrangement.



SECTION OF A TYPICAL METAL FORM FOR ROAD SLAB CONSTRUCTION

A typical side form is shown alongside.

When the forms have been fixed, they are checked for their trueness. Maximum deviations permissible in the vertical plane is 3 mm and in horizontal plane, it is 5 mm in 3m length of the form. The forms are oiled before placing concrete in them.

(d) Batching of materials and mixing

After determining the proportions of ingredients for the concrete mix, the fine and the coarse aggregates are properly proportioned by weight in a weight-batching plant. They are then fed into the hopper alongwith necessary quantity of cement which is also measured by weight. Generally, quantities of various ingredients in a concrete batch are determined for one or more number of cement bags.

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Mixing of concrete mix is done in a batch mixer, cement, sand and coarse aggregates of a batch are fed into the mixer simultaneously. water is poured into the mixer within the first 16 seconds of mixing. Mixing of each batch is then done for one and a half minute after adding water.

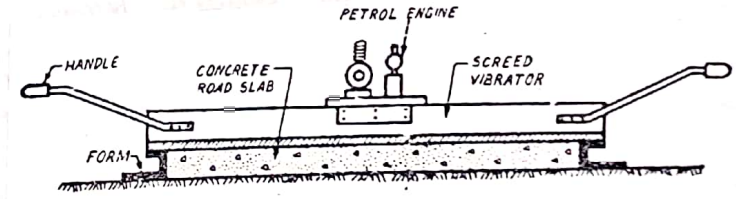
(e) Transporting & placing of concrete

After mixing, the concrete is transported to the site in wheel barrows or in pans which are carried manually. It is deposited on the subgrade or sub-base to the required depth and width of pavement section. While placing the concrete, it is roded with suitable tool to eliminate voids. segregation of concrete is avoided during transportation and placing.

When reinforcement has been specified in road slab, concrete is placed in two stages. In the first stage, concrete is placed and compacted to the depth corresponding to the level of reinforcement shown on the drawings. Reinforcement is then placed on the top of compacted concrete and remaining thickness of slab is then completed in second stage.

(f) Compaction

After placing the concrete in position, compaction of road slab is achieved by means of a power driven finishing machine or by vibrating hard screed. Upto 12·5 cm thickness of slabs, screed vibrators alone can be used for compaction. For greater thickness, screed vibrator alongwith immerssion vibrator is used. Edges and corners of concrete pavement are best compacted by immersion vibrators. screed vibrators rest on the forms and vibrate the surface of road slab as shown in figure alongside.



COMPACTION OF ROAD SLAB BY A SCREED VIBRATOR

In case of hand consolidation, the tamper is placed on the side forms and is drawn ahead in combination with a series of lifts and drops to compact the road slab.

(g) Floating

After compaction, the entire slab surface is floated longitudinally with a wooden float board. The longitudinal float is held in position parallel to the centre line of the carriageway and is passed from one edge of the pavement to the other.

The purpose of floating is to provide an even surface free from corrugations. After floating, unevenness of the surface is checked with a wooden edge 3m long which should not be more than 3 mm.

(h) Belting

After floating, the surface is further finished by belting just before the concrete becomes hard. Belting consists in drawing 20 cm wide belt of canvas

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across the pavement in short strokes.

The purpose of belting is to make the road surface non slippery and skid resistant. This operation is sometimes committed.

(i) Brooming

After belting, brooming is done by drawing brushes at right angles to the centre line of road from edge to edge. It is done with 45 cm wide steel or fibre brooms with long handles. Corrugations produced by brooming should be uniform and not greater than 1.5 mm deep. Brooming is done just before the concrete becomes non-plastic. This operation is also sometimes omitted.

(j) Edging

After brooming, the edges of the slab are carefully finished with an edging tool before the concrete is finally set.

(k) Curing

Curing consists in checking the loss of water from the concrete slab and keeping the fresh concrete slab moist during hardening period.

It is started as soon as the concrete has been finished and it has become non plastic. Initial curing consists of spreading burlap or jute mats saturated with water on the surface of fresh concrete slab.

Spreading of mats is done from foot bridges. Initial curing is done for 24 hours. By this time, the concrete becomes hard enough to walk upon and then wet mats are removed and final curing is done for 2 to 3 weeks.

Final curing is done by any one of the following methods:-

(i) By ponding
(ii) By covering the slab with 4 to 8 cm thick layer of wet sand or earth.
(iii) By using a suitable chemical such as sodium or calcium chloride.
(iv) By using a water-proof paper.

(l) Filling of joints and edging

After curing, the surface is cleaned and washed. Then the joints are properly filled-in with a suitable sealing compound.

After this, the concrete road slab is protected by brick edging. Then the shoulders are prepared in level with the top of brick edging.

(m) opening to traffic

After filling of joints and edging, the concrete road is opened to traffic when the concrete attains the required strength or after 28 days of curing.

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JOINTS IN CONCRETE ROADS

Joints are essential in concrete roads to allow for expansion, contraction and warping of the road slab due to temperature variations. They are also sometimes required when there is abrupt break in the construction or the day's work is not ended at the specified joints in order to make proper bond between the old and new construction work of the road slab.

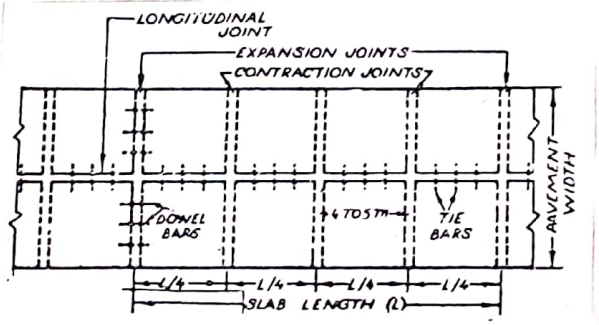
The various joints in concrete roads are classified in to the following two categories :-

1. Longitudinal joints ;
2. Transverse joints.

1. Longitudinal joints

The joints provided in the longitudinal direction between two strips of the road slab when the pavement width exceeds 4.5 m are known as longitudinal joints.

These construction joints are usually provided at the centre of road slab of two lane width (7 m) and can be placed at 1/3 or 1/4 width if it is 9m or more. Such joints are also recommended at the extreme ends betn the kerbs and the road slab. The space between longitudinal joints should not exceed 4.5 m.



JOINTS IN CONCRETE ROADS

Objects :-

The following are the objects of providing longitudinal joints in a road slab :-

(i) To control development of longitudinal cracks due to transverse contraction, warping and uneven settlement of subgrade.

(ii) To facilitate construction of road slab (more than 4.5 m in width) in convenient widths with hand tampers.

(iii) To help to maintain the two slabs together at the same level.

The various types of longitudinal joints are described as below....

(a) plain butt joint (b) Butt joint with tie bars (c) Tongue and groove warping joint.

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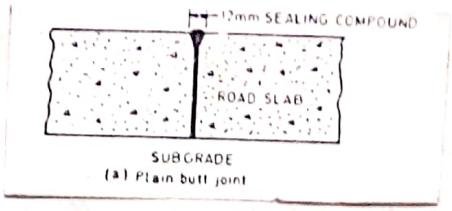
(a) Plain butt joint :-

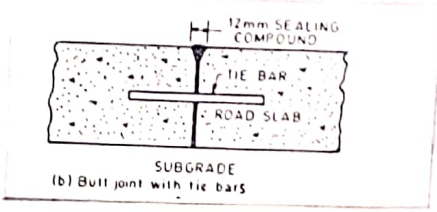
It is the simplest type of longitudinal joint. It is formed by simply painting the joints faces with a sealing compound as shown in figure alongside.



(a) Plain butt joint

(b) Butt joint with tie bars :-

In this type of longitudinal joint, tie bars of 12 to 15 mm diameter are provided as shown in figure alongside. These tie bars are about 1m in length and are placed at 600mm centre to centre distance. The top of the joint is then sealed with a sealing compound.



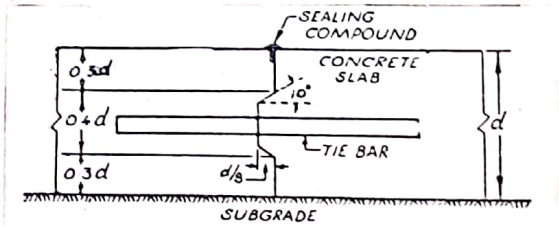
(b) Butt joint with tie bars

The function of providing tie bars is to hold the adjacent strips of the road slab together.

Butt joint with or without tie bars have been recommended by IRC for their use in concrete road slabs.

(c) Tongue & groove warping joint :-

In this type of longitudinal joint, a tie bar is inserted between the two strips with a key as shown in figure alongside. The top of the joint is then sealed with a sealing compound.



(c) Tongue and Groove Warping Joint

2. Transverse joints

The joints provided in the transverse direction of the road slab, mainly at 5m intervals are known as transverse joints.

The function of transverse joints is to allow for expansion, contraction and warping of road slab and thus to prevent development of cracks in the transverse direction.

The various types of transverse joints are described below :-

(a) Expansion joints
(b) contraction joints
(c) warping joints
(d) construction joints

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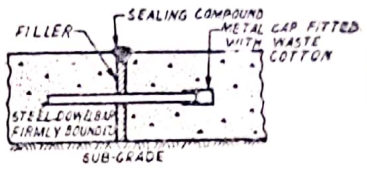
(a) Expansion joints :-



(a) Expansion Joint

The transverse joints constructed to allow for expansion of the road slab due to increase in temperature are known as expansion joints.

These transverse joints are provided at right angles to the centre line of the road at 18 to 20 m intervals. These joints extend to the full width & thickness of the road slab.

In these transverse joints, dowel bars of 15 to 18 mm dia are provided to transfer the wheel load from one slab to the other. These bars are 300 mm to 600mm in length and are placed at 300 to 500mm centre to centre distance as shown in figure above.

One end of each dowel bar is fixed in the concrete slab on one side of the joint. The other end lying in the adjoining slab is placed in a 100mm long metal sleeve or this end is oiled. Thus, the end of the dowel bar which is placed in metal sleeve or oiled, remains free to move whenever there is increase in temperature.

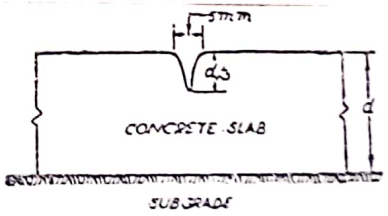
(b) Contraction joints :-

The transverse joints constructed to allow for contraction (shrinkage) of the road slab due to decrease in temperature are known as contraction joints.

These transverse joints are provided at 4 to 5 metres intervals usually at right angles to the centre line of the road. These joints are either or plain butt type or dummy type joints.

plain butt type contraction joints are vertical and extend to the full thickness of the road slab as already written in case of longitudinal joints (a & b)

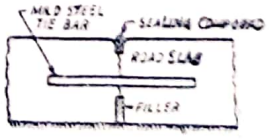
⇒ Dummy type contraction joints are also vertical but extend to about $\frac{1}{3}$rd of thickness from top of the slab as shown in figure alongside. This type of joint is formed by cutting into the surface after finishing the slab by means of wedge shaped plate and then this cut is sealed by a suitable sealing compound.



(b) Dummy Type Contraction Joint

(c) Warping joints :-

The transverse joints constructed to control the bending or warping of a road slab due to difference in moisture content or temperature at its top and bottom are known as warping joints.



(c) Warping Joint

These joints are in the form of butt joints with tie bars - as shown in figure.

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(d) construction joints :-

The transverse joint constructed when the construction work of the road slab is to be ended at a place other than a specified joint due the any reason is called a construction joint.

Such transverse joints are to be constructed under the following circumstances :-

(i) when the day's work of constructing the road slab is not ended at a specified joint due to any mishappening.

(ii) when some mechanical defect occurs in the mixing plant & full length of the slab is not completed

(iii) when there is rain & the work is to be stopped without completing the full length of the slab.

In such circumstances, construction joints are provided by embeding tie bars of 10 mm to 12 mm dia, at a depth of 3 cm from the top of road slab. The tie bars, about 1m long, are provided at 600 mm centres to centre distance.

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Hill Roads

Introduction

In India, about 2,59,000 sq km of area is covered with hills. In some parts like Himalayan region, vegetation and human habitation extends upto altitudes as high as 4900m above mean sea level.

In such an area, navigation and railway routes are not feasible. Even air traffic is difficult in hilly areas due to heavy cost of construction of air ports, control towers, etc. Thus, the only source of communicatn left in these areas is through roads which are known as ghat roads or hill roads

In consideration of strategic need of this country, roads have now been constructed even upto altitude of 5500m in order to provide communication facilities for the military. To accelerate the development of roads in hilly areas, the Border Roads Development Board was set up by the Govt. of India in March 1960.

* The present expenditure on hill roads construction is about 30 to 40 percent of the total central budget for roads.

2nd para
Right upto 1949 hilly region of the country which occupies about 10% of total area of the country was neglected. The British developed a few hill stations as their summer resorts and connected them by rail and road. Stations joined by road were Mussorie (UK), Shimla (Himachal Pradesh), Darjelling (West Bengal), Ooty (Udhagamandalam (Tamil Nadu)), Dalhousie (Himachal Pradesh), Lansdone (UK), Nainital (UK).

* where possible some of the stations such as Darjelling, Ooty and Shimla were connected by Railways.

→ Almost all these stations were cantonments made to serve the regime of Britishers.

Chinese attack in 1962 drew the attention of the Govt. of India for development of the hills.

* The extension of the Rail from Pathankot to Jammu, though not necessarily a hilly region, however, lays the base for extension of the Railway line well inside J&K. The project of constructing railway line well inside the valley is in a very advanced stage and preliminary work has already been started. This project is the latest in the development of Railways in the Hills.

Hill Roads

The road constructed in mountainous regions of a country are known as hill roads or ghat roads.

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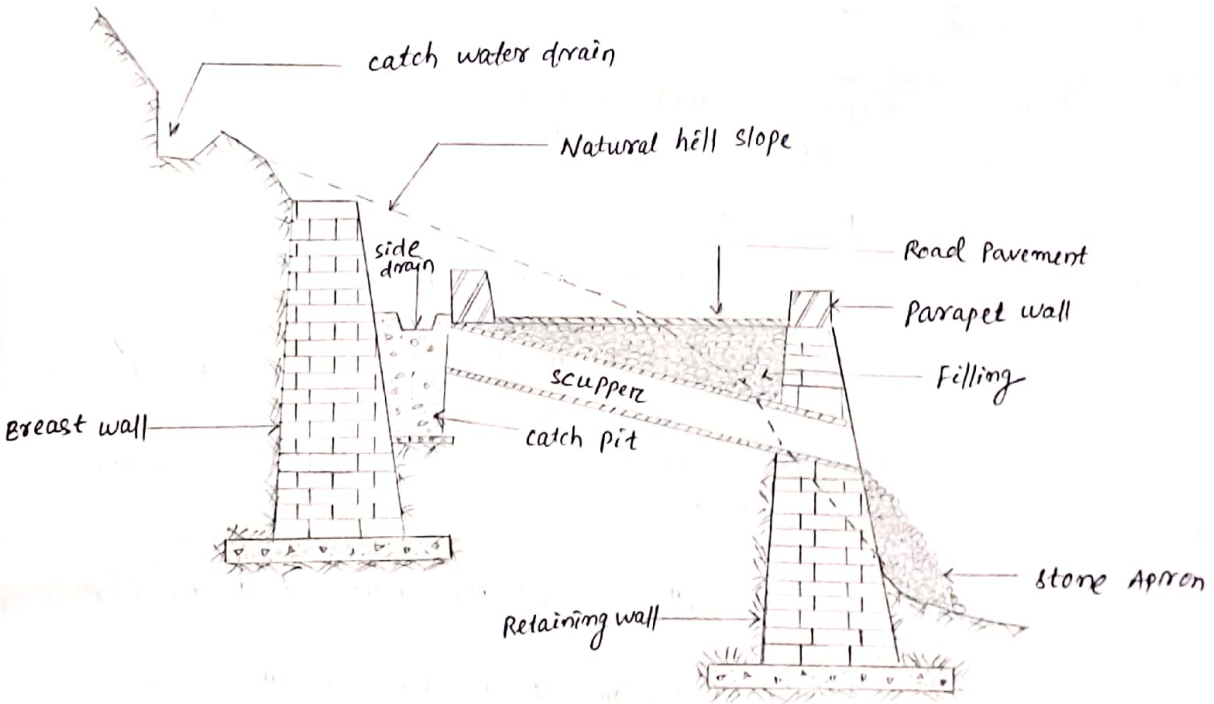
These roads present great difficulty in their alignment, design, construction and maintenance. Curves, sharp bends, steep gradient & limited width of roadway make hill roads more liable to accidents. Also the effects of heavy rainfall on the construction and maintenance of a hill road are serious.

→ At many locations on the hill roads, the landslides and slips may occur during heavy rainfall. Therefore, much care is needed during their layout and construction so as to provide a stable & safe road. Moreover, a large number of streams cross the road and hence a suitable facility for cross drainage is needed.

(*) The overall cost as well as cost per km length of a hill road is higher than that of a similar road in plains. The reason for this is that hill road construction involves difficult rock cutting, construction of more number of drainage crossings and costly protective works like retaining walls, breast walls, parapet walls, drains, etc. which are specially needed for hill roads.

Parts of a hill road & their functions



C/s of a Typical Hill Road showing its component parts

The following are the component parts of a typical hill road :-

(a) Road bed;
(b) side drains;
(c) Parapet walls;
(d) catch water drains;
(e) Breast walls;
(f) Retaining walls;
(g) cross drains

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(a) **Road bed** — The pavement portion of a hill road is known as road bed.

→ The function of a road bed is to resist safely the stresses caused due to the moving wheel loads of traffic

(b) **Side drains** — The drain provided on the road side, usually at the foot of hill slope is known as side drain.

→ The function of side drains is to collect & drain off rain water, falling over the road surface as well as on hill slope, into any cross drainage works.

(c) **Parapet walls** — The wall provided above the formation level of a hill road, usually towards the down slope side is known as parapet wall.

→ This type of wall is not constructed as continuous wall but with suitable gaps in between, for economy in its construction.

→ The function of such a wall is to provide protection to the traffic against falling down the hill slope.

(d) **Catch water drains** — The drains provided higher up on the hill slope side, parallel to the road are known as catch water drains.

→ These drains should not be provided less than 4.5m distance from the edge of the road.

→ The function of these drains is to intercept the run off from the hill slope, which would otherwise rush on to the road & wash it away, and then to divert the same into a nearby cross drainage work.

(e) **Breast wall** — The wall constructed towards upslope side of the road to prevent the hill side from sliding down is called breast wall.

→ This type of wall is constructed when hill side is steep & there is tendency of its sliding down towards the road. Breast walls are constructed of stone masonry, brick masonry or cement concrete.

→ The top width of breast walls as well as of retaining walls should be 60 cm minimum. Their front face should have a batter of 1 in 4 to 1 in 3. The stones used in their construction should be of large size and roughly hammer dressed.

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(f) **Retaining walls**

The wall constructed towards down slope side of the road to resist the pressure of earth falling & traffic load coming on the road is known as retaining wall

→ This type of wall is commonly constructed in case of a hill road when the later is to be constructed partly in cutting & partly in embankment or wholly in embankment. Retaining wall can be constructed in stone masonry, brick masonry or cement concrete Dry stone masonry retaining walls are the simplest and easy, therefore, commonly constructed along a hill road.

⇒ The height of a dry stone masonry retaining wall is restricted to 6m.

→ When the height of the wall is more than 6m, three courses are provided in suitable mortar after every 3m height

(g) **Cross drains**

The drain constructed to drain off the rain water across the road is known as cross drain

→ The cross drains may be in the forms of small under drains, scuppers, etc.

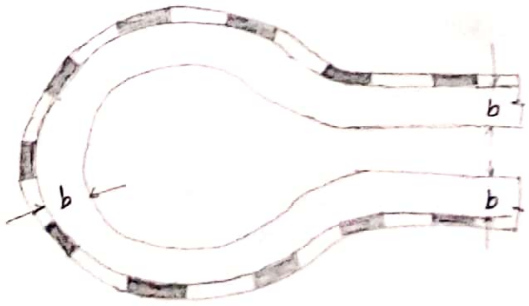
Types of curves on hill roads

The following are the important types of curves provided on hill roads:-

(a) Hair pin curves/bends
(b) salient curves
(c) Re-entrant curves

(a) Hair pin curves/bends

The curve in a hill road which changes its direction through an angle of 180° or so, down the hill on the same side is known as hair-pin curve/bend.



This curve is so called because it conforms to the shape of a hair-pin
The bend so formed at the hair-pin curve in a hill road is known as

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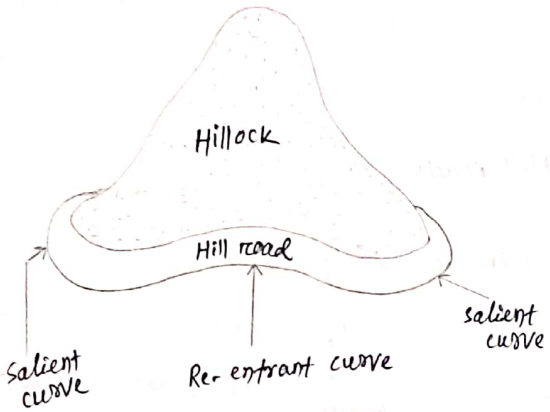
hair-pin bend.

This type of curve should be located on a hill side having the min^m slope and max^m stability. It must also be safe from view point of land slides and ground water.

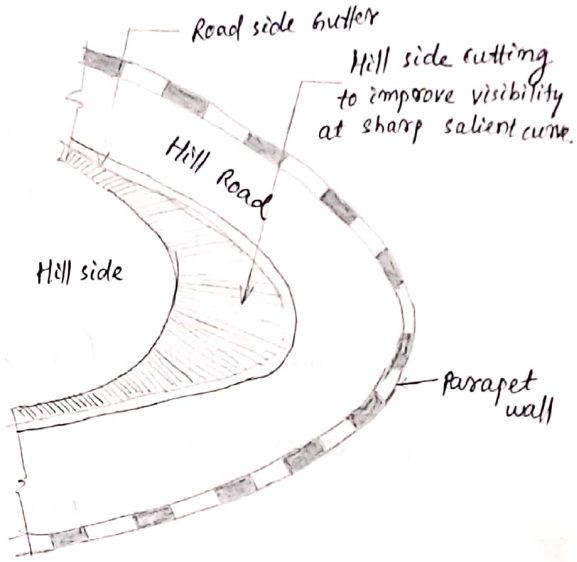
→ Hair pin bends with long arms and farther spacing are always preferred. They reduce construction problems and expensive protective works.

→ Hair pin curves or bends of serpentine nature are difficult to negotiate and should, therefore be avoided as far as possible.

(b) Salient curves



A hill road showing salient & Re-entrant curves



A corner bend at the salient curve

The curves having their convexity on the outer edges of a hill road are called salient curves.

→ The centre of curvature of a salient curve lies towards the hill side. This type of curve occurs in the road length constructed on the ridge of a hill. The bend so formed at the salient curve in a hill road is known as corner bend.

→ Salient curves are very dangerous for fast moving traffic. At such a curve or at corner bend, the portion of projecting hill side is usually cut down to improve the visibility.

→ The outer edge of the road at such a curve is essentially provided with a parapet wall for protection of the vehicles from falling down the hill slope.

(c) Re-entrant curves

The curves having their convexity on the inner edge of a hill road are called re-entrant curves.

→ The centre of curvature of a re-entrant curve lies away from the hill side. This type of curve occurs in the road length constructed in the

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valley of a hill.

→ These curves are less dangerous as they provide adequate visibility to the fast moving traffic. At such curves, the parapet wall is provided only for safety of fast moving traffic.

Alignment of a Hill Road

The success & utility of a hill road depends much upon its proper alignment. In the alignment of a hill road, a number of sharp curves such as hair-pin bends, corner bends, etc are to be provided. The min^m radius of such sharp curves should be 30m but not less than 14m in any case. The min^m radii of curves recommended for different roads in hilly area are given below

Minimum Radii of curves in Hill Roads

Sl. No	Category of Road	Min^m Radii, (mtrs)			
		Mountaneous Terrain		Steep Terrain	
		not snow bound	snow bound	not snow bound	snow bound
1.	NH & SH	50	60	30	33
2.	MDRs	30	33	14	15
3.	ODRs	20	23	14	15
4.	VRs	14	15	14	15

During the alignment of a hill road, much care should be taken to see that the road should be as short as possible so as to achieve economy in the cost of construction, transportation and maintenance. It should have easy gradients so that all types of vehicles can traverse long distance without any difficulty.

The alignment of a hill road should be fixed in the following three stages

1. Reconnaissance
2. Trace cut
3. Detailed survey

Trace Cut: This operation consists in constructing 1 to 1.2m wide track along the selected alignment to facilitate access to the area for inspection & detailed surveys.

→ During reconnaissance, a general route for the alignment is selected. A trace is cut thereafter to translate this route on the ground,

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So as to provide an access for the subsequent detailed surveys.

→ The final alignment according to desired geometrics is fixed by detailed survey in the last phase.

Classification of Hill Roads

Normally hill roads are also classified, as in the case of plain roads into the following five categories.

1. NHs
2. SHs
3. MDRs
4. ODRs
5. VRs

Hill roads may also be classified into the following three categories :—

1. Motor Roads
2. Bridle paths
3. Village paths or tracks

1. Motor Roads

The hill roads meant for motors, trucks and other such traffic are known as motor roads.

→ These roads form main communication system in hilly areas. They may have one or more lanes width. The ruling gradient recommended for such roads is 1 in 20 & the limiting gradient is 1 in 15. The exceptional gradient recommended is 1 in 12 but it should only be provided in 75m per km length of the road. These roads are constructed with any suitable pavement according to their importance.

2. Bridle Paths

The hill roads meant for pedestrians and other pack transport such as laden ponies, mules, camels, etc are known as bridle paths.

These paths serve as feeder roads to motor roads, thus connecting interior places with the main communication system. They are generally 2 to 2.5m in width but may be kept 2.45 to 3.65m wide when they are to be used as jeepable roads. The ruling gradient recommended for such paths is 1 in 10 and the limiting gradient is 1 in 7.5. The surface of these paths is generally paved with stone blocks.

3. Village paths or tracks

The hill roads meant for pedestrians, cattle, etc are called village paths or tracks.

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These tracks connect small villages and other working places with bridle paths or motor roads. They are generally 0.9 to 1.2m in width. The ruling gradient recommended for such paths is 1 in 7.5 and the limiting gradient is 1 in 5. The surface of these paths generally consists of natural rock, dressed to the required gradient & camber.

The Board of Road organisation has classified hill roads as given below:-

1. National Highways;
2. class 9 (6m wide for 3 tonnes vehicles);
3. class 5 (4.9m wide for 1 tonne vehicles);
4. class 3 (2.45 to 3.65m wide for jeeps);

:- Formation of Hill Roads :-

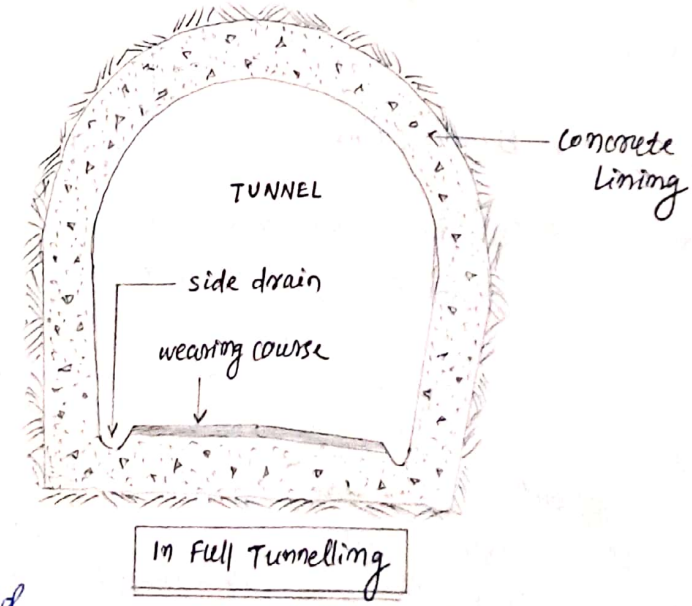
The formation of a hill road may be done in any one of the following three situations:-

1. wholly in cutting
2. wholly in embankment/Filling
3. partly in cutting and partly in embankment

1. Road wholly in cutting

The hill road having its formation width constructed fully in cutting is called the road wholly in cutting.

A hill road, wholly in cutting, is constructed when the side slope of the hill is very steep. The road in cutting may be either in half tunnelling or in full tunnelling. The road in half tunnelling is provided when the rock strata dips away from the road side and when the rock is quite hard and stable. The road in full tunnelling is only provided when there is no other alternative, for the economy of the road project.



In Full Tunnelling

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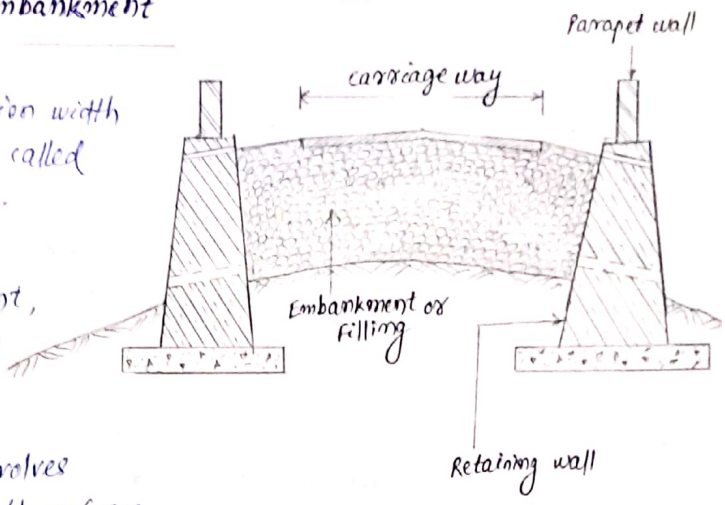
2. Road wholly in embankment

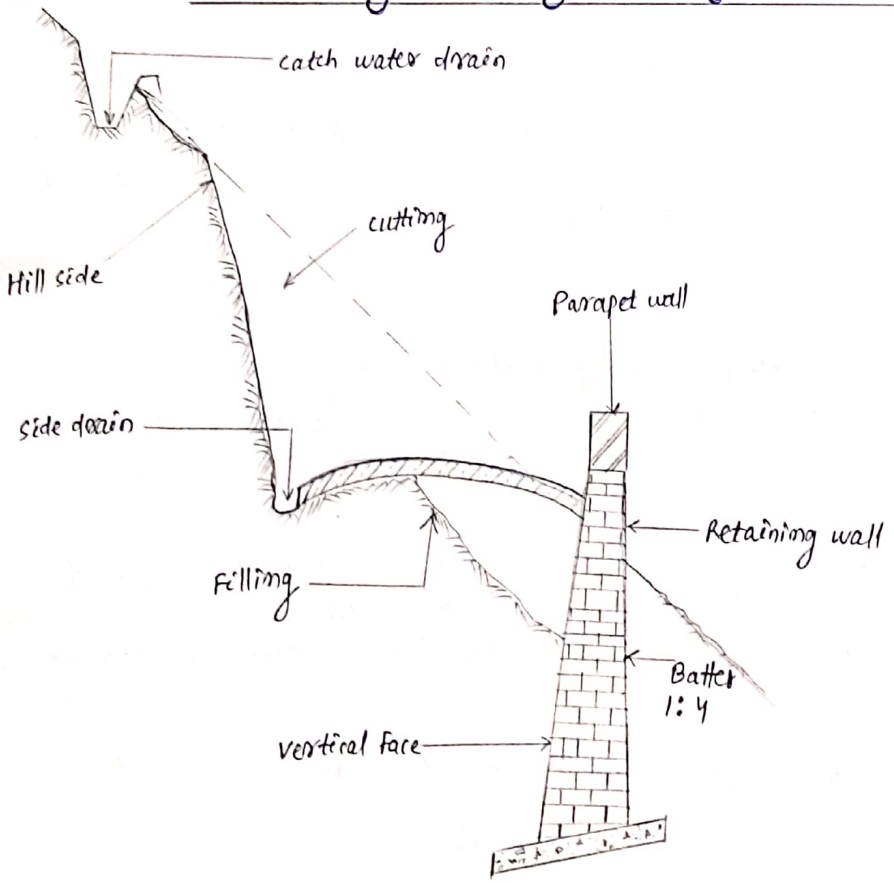


The hill road having its formation width constructed fully in filling is called the road wholly in embankment.

A hill road, wholly in embankment, is constructed when a depression comes in the alignment of the road.

(X) This type of construction involves heavy earthwork and should, therefore, be avoided as far as possible.

3. Road partly in cutting & partly in embankment



The hill road having its formation width constructed partly in cutting and partly in filling is called the road partly in cutting & partly in embankment/filling.

→ Hill roads are generally constructed partly in cutting and partly in embankment. Usually the road section $\frac{2}{3}$ in cutting & $\frac{1}{3}$ in filling is preferred for more stability of the road.

→ This type of road construction is done when the side slope of the hill is not very steep & cost of cutting through rocks is too much.

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LAND SLIDES

The downward and outward movement of slope-forming materials such as natural rocks, soils, artificial fills, etc is known as land slide or land slip.

Land slide or land slip is the most important problem in the maintenance of hill roads. In rainy season, land slides or slips are frequent and the road often get blocked. Thus, land slides interfere with the free movement of traffic on hill roads.

Hence, it becomes essential to adopt suitable measures to prevent land slides so as to provide free movement of traffic on the hill roads throughout the year.

Types of Land slides

Land slides occur along surface of separation by falling, sliding, flowing or by their combined effect. Thus, land slides may occur in any one of the following forms :-

(a) Fall;
(b) slide;
(c) flow;
(d) complex slide.

(a) Fall

It includes free fall and rolling of rocks and debris (accumulation of loose rocks) down the hill slope.

(b) Slide

It is the movement of slope forming materials along one or several surfaces down the hill slope. It is caused due to finite shear failure of rocks.

(c) Flow

It is the movement of the slope forming materials within the displaced mass. The form taken by the moving materials resembles (have a similarity to) to that of viscous fluid. In its case, the slip surface cannot be located.

(d) Complex land slide

It includes movement due to combined effect of two or more types of land slides.

Causes of land slides

The following are the causes of land slides....

(i) Increase in water content during rainy season.
(ii) Due to failure of a retaining wall or breast wall
iii) Due to seepage pressure of percolating ground water
iv) undermining caused by erosion.
v) Shocks and vibrations caused by earthquakes and nearby blasting of rocks.

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(vi) Increase of load due to traffic or accumulation of snow on the road surface.

(vii) Formation of faults in bedding planes of the strata due to vibration.
The fracture along which the movement of one block with respect to the other takes place is known as faults.

(viii) Removal of part of the mass by excavation and increase in slope angle.

(ix) Hair-cracking due to alternate swelling & shrinkage of the soil mass.

(x) Fissuring of pre-consolidated mass due to release of lateral pressure while doing cutting of rocks.

Prevention & control of Land slides

Study of causes of land slides gives the clues to preventive measures against these slides. Though we are helpless to prevent land slides caused due to earthquakes, yet we can surely prevent and control land slides due to other causes by taking the following measures.

(a) By efficient surface and cross drainage

(b) By providing sub-surface drains at foot of the hill slope to control seepage flow.

(c) By benching of soil slope.
The art of cutting hill sides in steps is called benching.

(d) By reducing the angle of slope or providing breast walls and retaining walls

(e) By constructing buttresses at toe of hill slopes.

(f) By slope treatment to minimise the erosion and to improve the stability of hill sides. This is done by turfing, stone pitching, cement grouting, etc.

Drainage structures for Preventing land slides

The various drainage structures for preventing land slides are given below:

(a) catch water drains

(b) side drains

(c) sub-surface drains

(d) cross drainage structures in the form of under drains, scuppers, causeways, culverts, minor or minor bridges, etc.

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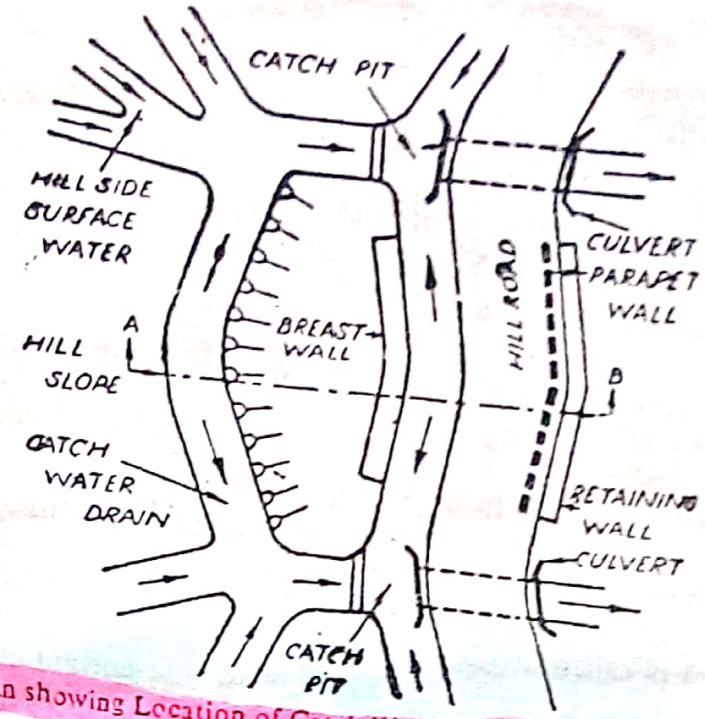
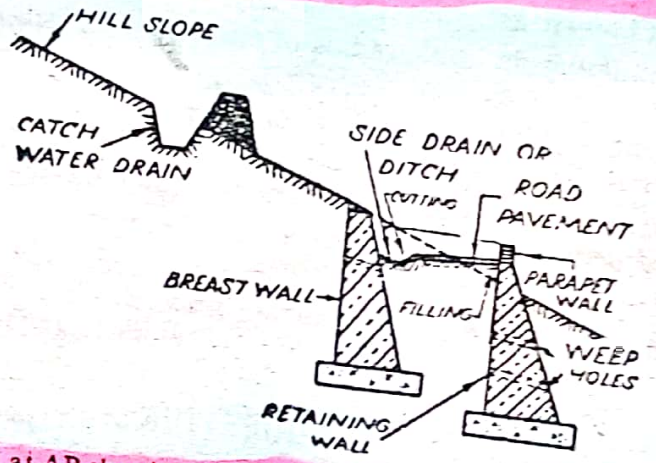


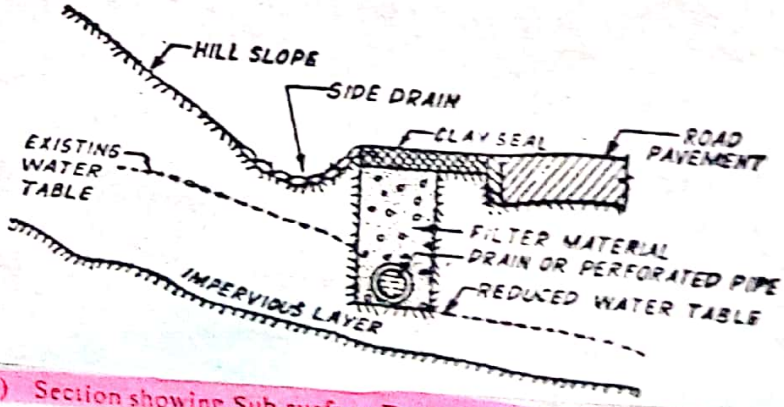
(a) Plan showing Location of Catch Water Drain and Side Drain



(b) Section at AB showing Location of Catch Water Drain and Side Drain

Fig. 7-8

(Contd.)



(c) Section showing Sub-surface Drain to intercept Seepage Flow

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Unit-6 : Road Drainage

Introduction :

The stability of road pavements can only be maintained if their surface and foundation bed remain in dry condition. The entrance of water in the subgrade or any other layer of the road pavement, even for short intervals is undesirable and dangerous because it is considered as one of the major causes of the road pavement failure. Thus, efficient collection, removal and disposal of surface and sub-surface water known as road drainage, is very essential for proper design & adequate maintenance of roads.

Road Drainage

The process of removing & controlling the access of surface & sub-surface water within the right-of-way of a road is called road drainage.

It also includes interception (collection) & diversion of water from the road surface and the subgrade.

(*) The process of interception and diversion of surface water through suitable side drains is called surface drainage, and the process of interception and removal of sub-soil water through suitable sub-surface drains is called sub-surface drainage.

(*) The main object of road drainage is to keep the road surface and its foundation as dry as possible so as to maintain its stability.

Thus, a good drainage system is essential for efficient highway transportation with minimum maintenance cost.

Modes ~~Method~~ of water penetration into road structure

The following are the different modes of water penetration into the road structure

(i) surface water from the top of pavement by percolation through cracks and poor pavement surface.

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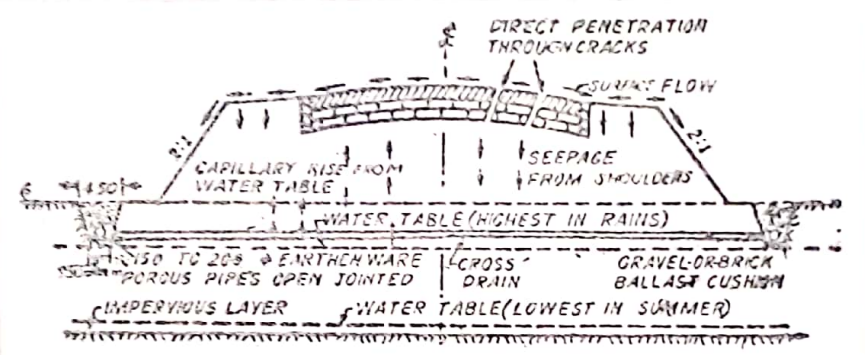
(ii) surface water from sides of the pavement

(iii) sub-soil water from underside of the pavement by capillary rise.

(iv) sub-soil water from sides of the pavement

(v) intercepted water due to overflooding of coass drainage works.



MODES OF WATER PENETRATION INTO ROAD STRUCTURE

ill effects of water entering the pavement structure on sub-grade materials

The following are the ill effects of water entering the pavement structure on sub-grade materials:-

(i) The water entering the road pavement causes softening & reduction in bearing strength of sub-grade materials.

(ii) It causes mud pumping in rigid pavements

(iii) It causes the clayey masses of the road bed to expand.

(iv) It causes the black cotton soil forming the sub grade material to swell

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(v) It causes the sand loam to retain moisture by capillary action.

(vi) It causes the peat loam to loose its cohesion which finally disintegrates with increase of moisture.

Necessity of Road drainage

The surface water may enter the pavement structure from top, bottom or sides due to percolation. The subsoil water may find its access to the pavement subgrade from bottom due to capillary rise.

The water entering the pavement structure causes the clayey mass of the road bed to expand. Expansion in the soil mass of the bed may also take place due to freezing or salt action. It also causes softening of the subgrade soil and decrease in its bearing capacity. All these ill effects of water entering the pavement result into consequent breaking up & shattering of the road pavement. Water also causes erosion of bank slopes whether in embankment or in cutting and may also result into land slides.

The following are the factors which necessiate the provision of road drainage :—

(i) The entrance of the water in the soil subgrade of the pavement causes considerable decrease in its bearing strength and thus the pavement is likely to fail due to poor strength of the subgrade.

(ii) Excess of moisture content causes reduction in bearing strength of base course bed materials like stabilized soil, water bound macadam etc.

(iii) The increase in moisture content causes the clayey masses of the road bed to expand considerably which may sometimes result in pavement failure.

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(iv) Due to poor drainage, waves and corrugations are formed in flexible pavements. This also causes mud-pumping in rigid pavements. All these defects lead to pavement failure.

(v) The increase in moisture contents causes increase in weight & simultaneously reduction in strength of the soil mass of side slopes. This is considered as the main reason of failure of side slopes.

(vi) At places where temperature often reaches to freezing point, the frost action of water entering the pavement structure may cause damage to the road pavement.

(vii) Poor drainage of road also causes washing out unprotected top surface of roads, erosion of bank slopes, land slides etc.

Considering all these factors, the provision of an adequate & efficient road drainage for quick removal of surface and sub-soil water has become very essential in the design and proper maintenance of roads. With an adequate & efficient drainage system, the efficiency and life of a road can be considerably increased.

Requirement of a good road drainage system

The following are the requirement of a good road drainage system:—

(i) For a good drainage system, the road pavement should have an impervious top surface provided with sufficient camber so as to drain off the surface water without allowing it to percolate into the road pavement

(ii) The shoulders of road pavements should also have an impervious top with proper cross slopes so as to drain off the surface water

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effectively towards the road sides.

(iii) The side slopes of road embankment or cutting should be sufficiently flat and well compacted so as to allow the surface water to flow sideways without causing cross ruts or erosion.

(iv) The side drains should have sufficient capacity & longitudinal slope to carry away the collected water without overflowing.

(v) Where the topography of the area is such that the water flows towards the roadways, intercepting drains of adequate capacity should be constructed parallel to the road but outside the road limit to intercept water of the outer area.

(vi) There should be adequate arrangement for sub-surface drainage so that the height level of ground water table should remain preferably 1·2 m below the level of subgrade.

(vii) Seepage & other sources of underground water, if any, should be tapped and the water should be drained off by sub-surface drains.

(viii) In water-logged areas, special measures should be taken especially, if salts are present or if flooding of water is likely to occur.

(ix) All drainage structures should be of adequate design to drain off water immediately without overflooding.

Road drainage systems

Road drainage systems, adopted for removing & controlling the surface and sub-soil water, entering the pavement structure, are grouped into the following heads: -

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1. Surface drainage
2. Sub-surface drainage
3. Cross drainage

1. Surface drainage

The system of collection & disposal of surface water within right-of-way of a road is called surface drainage.

This system consists in allowing the surface water to flow from the pavement surface without percolation into the shoulders & then down the embankment slope when the road is in embankment or in side drains when the road is in cutting or when it is on ground line.

The surface water is first collected in longitudinal suitable side drains which is then disposed off in the nearest stream, valley, etc.

Method of providing surface drainage

The following steps are taken to provide effective surface drainage on roads:-

(i) Providing an impervious type of road surfacing.

(ii) providing sufficient camber or superelevation as desired & adequate gradient to the road surface.

(iii) Making shoulders of rural highways impervious and providing them proper side slope towards side drains.

(iv) Protecting side slopes from erosion by properly rounding off edges and also by turfing.

(v) Keeping height of the road embankment atleast 1.2m above the highest flood level of the area.

(vi) Providing side drains on one or both the sides according to the alignment of the road.

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(viii) Providing intercepting drains when the slope of the surrounding area is towards the road. Such drains should be constructed parallel to the road but outside the right-of-way

2. Sub-surface drainage

The system of collection & removal of sub-soil water from underside of a road pavement is called sub-surface drainage or sub-soil drainage.

⊛ The function of sub-surface drainage is to control the moisture content of the road subgrade.

This system consists in first controlling the **free water**.
The water which originates under the pavement due to spring action or has penetrated the pavement structure due to perviousness of road surface is called free water or gravity water.

This can be done by intercepting surface water before it enters the road subgrade. The second step is to reduce the **capillary moisture**. This is achieved by providing sub-surface drains to lower the ground water table or by providing a granular sub-grade.
The particles of water drawn due to capillary action of soil particles from free water or saturated strata, lying below the road subgrade is known as capillary moisture.

Sub-surface drainage system is recommended under the following conditions:

(i) When the road is through a flat country and the ground water table is considerably high (less than 1-2m below the road subgrade).

(ii) When there is danger of rise of moisture to the pavement structure due to capillary rise even if the underground water table is sufficiently low.

(iii) When the road is in cutting & there is considerable seepage through the side slopes.

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(iv) when the pavement structure is subjected to the action of spring.

(v) when the road is at the foot of a hill and the water there from may seep into the road subgrade.

Methods of providing sub-surface drainage

The following the methods of sub-surface drainage adopted under different situations :-

(a) lowering the water table;
(b) control of seepage flow;
(c) control of capillary rise.

(a) Lowering the water table

The height level of ground water table should be atleast 1.2m below the level of road subgrade so that the subgrade & pavement layers are not subjected to excessive moisture. In plains, where the sub-soil water table is high, the best remedy is to provide the road formation in embankment of height not less than 1.2m. when the road formation is to be provided at ground level or in cutting, then it becomes essential to lower the sub-soil water table.

The sub soil water table can be lowered by any one of the following three methods.

(i) By merely constructing longitudinal side drains (Jelly type) with drain pipe & filter bed

(ii) By providing transverse drains in addition to longitudinal drainage trenches

(iii) By providing perforated pipes or open-jointed drains along side slopes of cutting in addition to longitudinal side drains

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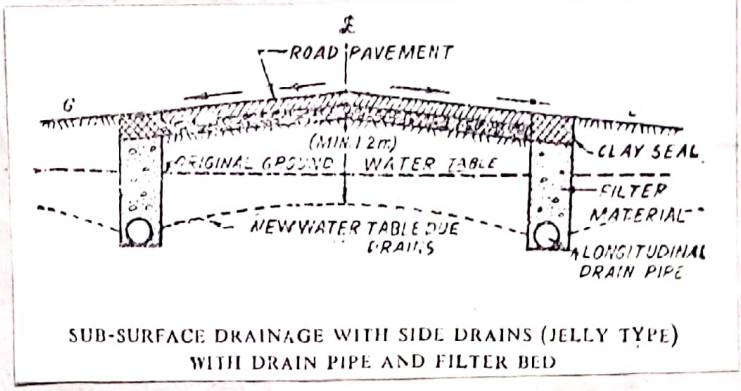
(i) By merely constructing Longitudinal side drains (Jelly type) with drain pipe and filter bed :-



SUB-SURFACE DRAINAGE WITH SIDE DRAINS (JELLY TYPE) WITH DRAIN PIPE AND FILTER BED

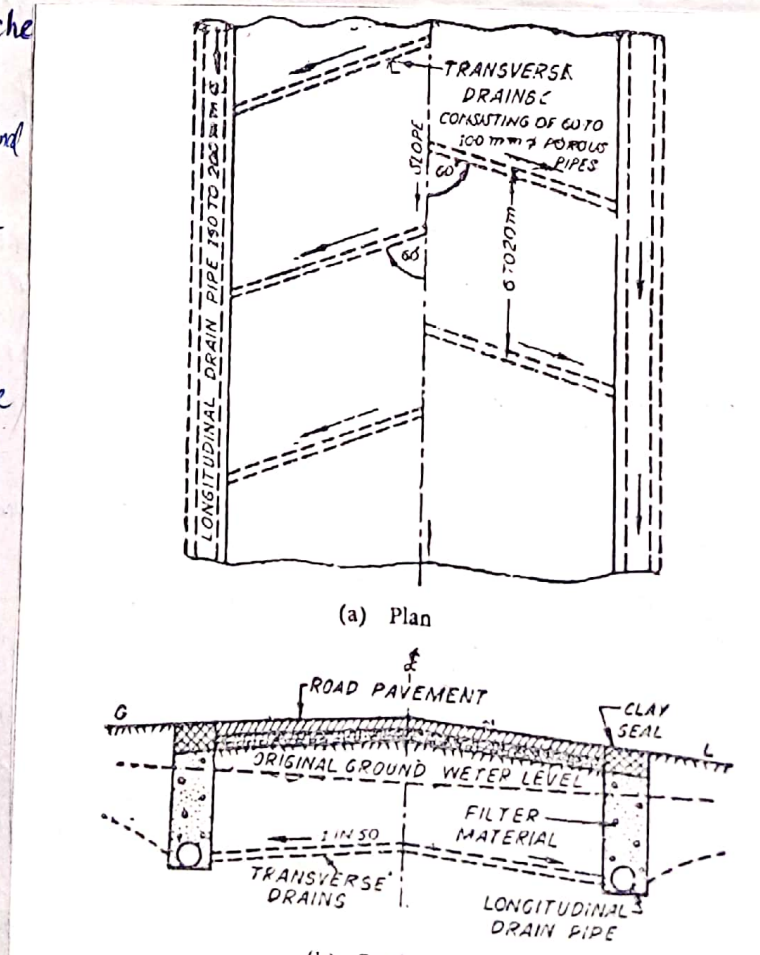
In this method, Jelly type side drains are constructed. These drains are provided with drain pipe and filter bed as shown in fig. The depth of drainage trenches depends on the required lowering of the sub-soil water table.

Suitability:- This method is only suitable when the soil is more permeable.

(ii) By Providing transverse drains in addition to longitudinal drainage trenche

In this method, longitudinal drain pipe are provided in trenches below the pavement structure at the specified depth.

The pipes are usually made of vitrified clay and are placed with open joints butting against each other. They are laid on a bed of sand, crushed stone or clay bed 150mm thick. The longitudinal drain pipes are 150 mm to 200mm in diameter. Cross or transverse drains, known as 'Mitre drains' or 'French drains' are 60 mm to 100mm in diameter & are laid



SUB-SURFACE DRAINAGE WITH TRANSVERSE DRAINS ALONGWITH LONGITUDINAL SIDE DRAINS

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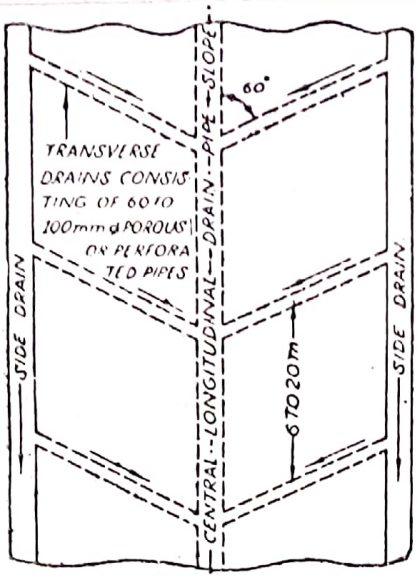
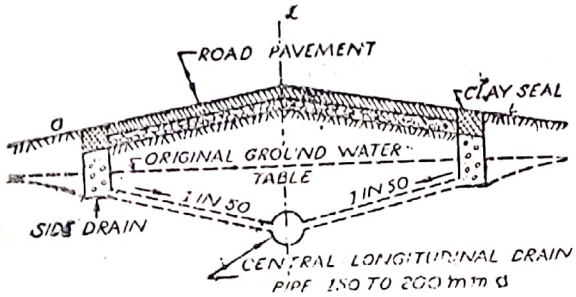


(a) Plan



(b) Section

SUB-SURFACE DRAINAGE WITH TRANSVERSE DRAINS AND A LONGITUDINAL CENTRAL DRAIN

cross wise from the centre of the road, sloping 1 in 50 diagonally towards the flow. The distance between adjacent cross drains is kept 6m 20m depending upon the nature of the subgrade soil.

The main longitudinal drain pipes (150 to 200 mm dia) may be provided on both sides or at centre of the road. When the main longitudinal drain pipes are provided on both the sides of a road, the slope of transverse or ~~cts~~ cross drains is kept 1 in 50 from centre towards sides as shown in fig along side. This method of sub-surface drainage is comparatively costly but can be easily constructed and maintained.

When the main longitudinal drain pipe is provided at the centre, sufficiently below the road pavement, the side drains may be open or jelly type without perforated pipes. In this case, the slope of the transverse or cross drains is 1 in 50 from side drains towards the main longitudinal drain pipe provided at the centre as shown above in fig. This method of sub-surface drainage is comparatively economical but cannot be easily constructed and maintained.

The cross pipes thus collect, remove and dispose off sub-surface water into the central longitudinal drain pipe. The longitudinal drain pipe in turn disposes of the sub soil water, received from cross drains or pipes, to a suitable cross drainage structure.

Suitability The above two methods of lowering the water table are adopted when the soil is relatively less permeable and where the road runs in a flat country with low embankment or in cutting

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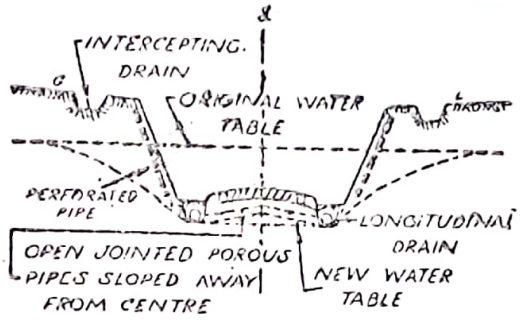
where maximum sub-soil water table lies just below the road subgrade.

(iii) By Providing perforated pipes or open-jointed drains along side slopes of cutting in addition to longitudinal side drains



DRAINAGE OF SIDE SLOPES IN CUTTING BY PROVIDING PERFORATED PIPES ALONG SIDE SLOPES OF THE CUTTING IN ADDITION TO LONGITUDINAL SIDE DRAINS

In this method, perforated pipes or open-jointed drains are provided along the side slopes of cutting to prevent water seeping through the sides of slopes and thus the erosion of side slopes is prevented. The detail of construction of perforated pipes or open jointed drains are drawn alongside's figure.

Suitability This method is suitable when the road is in cutting and where the sub-soil water table is higher than the top surface of the pavement.

(b) Control of the seepage flow

Seepage flow is likely to exist when the ground as well as the impervious strata, lying below, are sloping. This system of sub-surface drainage is required particularly in case of hill roads.

(c) control of capillary rise

When there is danger of rise of moisture to the road subgrade due to capillary rise, the same may be checked by any one of the following two methods.

(i) By providing a capillary cut-off, consisting of a thick layer of coarse material like gravel between the sub-soil water table and the road subgrade.

(ii) By providing an impermeable membrane between the sub-soil water table and the road subgrade.

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Cross drainage :

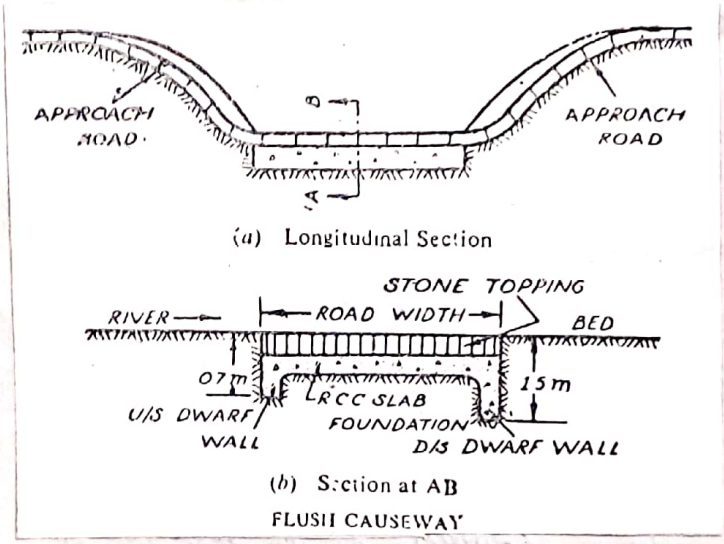
The system of disposing the water collected in side drains or that of the natural streams across a road is known as cross drainage.

The system consists in providing a suitable cross drainage structure in the form of causeways, scuppers, culverts, minor bridges, major bridges etc.

Where streams have to cross the road or when the water from side drains is to be taken across the road in order to divert the same away from the road to a water course or valley.

(i) Causeways :-

These are submersible bridges which are generally constructed with their floor either flush or little above the bed of the stream. In their case, the high flood discharge always passes over the surface of road pavement. They are provided on roads of less importance for passing a large quantity of water across the road for short intervals in a year.



A causeway may consist of a concrete slab with its top level flush with the bed of stream without any opening underneath or in combination with under drains or scuppers. If no opening is provided, it is termed as flush causeway or metal dip and if there is any opening provided below the road slab, it is known as low level causeway or Irish bridge. To guard against scour and to protect the flooring, dwarf curtain walls are constructed, both on upstream sides and downstream sides as shown in fig above

⊗ When a small stream crosses a road with linear waterway (Horizontal width) less than 6m, the cross drainage structure provided is known as culvert.

→ The common types of culverts in use are:- slab culvert, arch culvert, pipe culvert & box culvert.

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Side Drains

The drains provided parallel to the road for collecting & disposing the surface water are known as side drains.

They are usually trapezoidal in section, cut into ordinary soil at a suitable distance parallel to the road. When the road is in embankment, the distance of side drains should not be less than 1.85m from the toe of embankment.

⇒ In case of cutting, these drains are provided just after the edges of shoulders

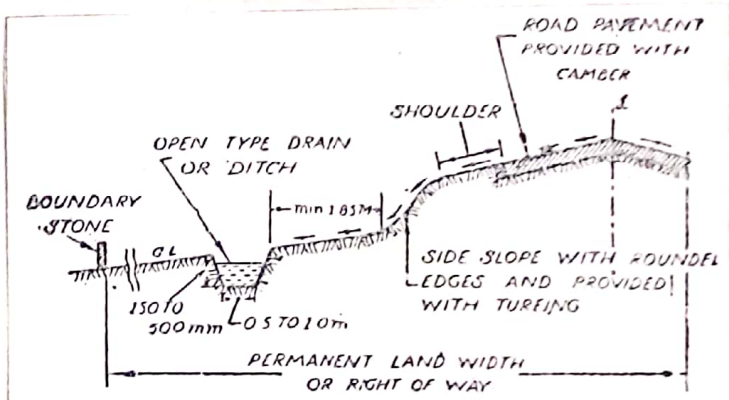
The various side drains, usually provided for surface drainage of roads, are mainly classified into following two groups...

1. Open drains;
2. closed drains

1. Open drains

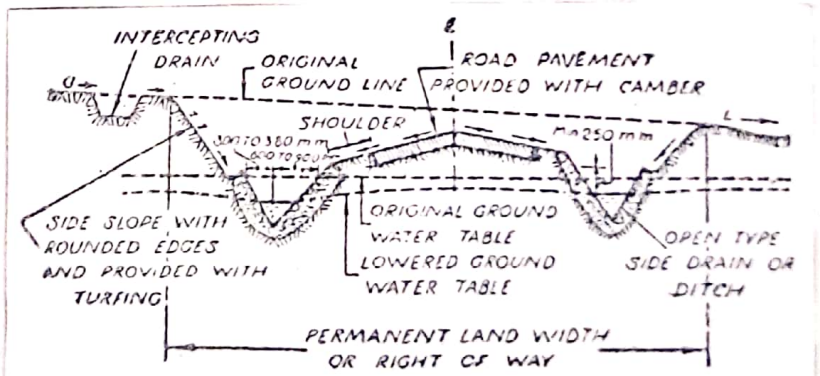
The side drains constructed without any filter material & remain open to view are known as open drains.

These drains are usually provided parallel to roads in rural areas when the designed depth of side drains is less & the road is subjected to light traffic.



(a) A Road Section in Embankment showing an Open Type Side Drain (Trapezoidal in Section)

(*) The function of open side drains is to collect the surface water. The open drains in plain areas are generally trapezoidal or triangular in section as shown in fig alongside.



(b) A Road Section in Cutting showing an Open Type Side Drain (Triangular in Section)

OPEN TYPE SIDE DRAINS

They are having sufficient depth, so that the sub-soil water, if any, from the road bed can be drained into them. The bed width of such a drain varies from 0.5 to 1m and depth 150mm to 500mm, depending upon topography of the area. These drains should be so designed

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that the max^m water level in them remains atleast 250mm below the road bed in cutting

(*) These drains can be cheaply and easily constructed & maintained. But these drains provide an ugly look & prove very dangerous because of having deep excavations. Open drains are, therefore, undesirable where the traffic is heavy.

2. Closed drains

The side drains covered at their top are known as closed drains.

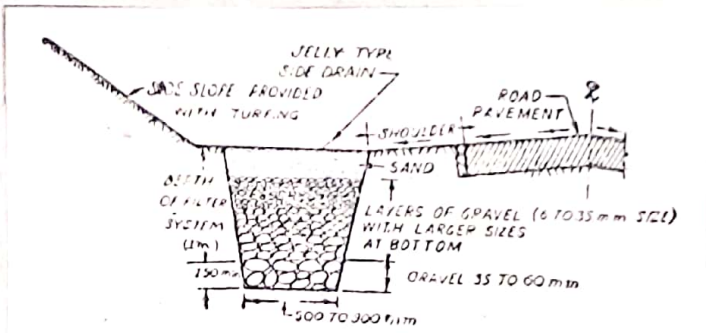
⇒ These drains are provided parallel to the road subjected to heavy traffic in rural area as well as in urban area.

⇒ These drains are costly & difficult to construct and maintain. But these are neither dangerous nor provide an ugly look.

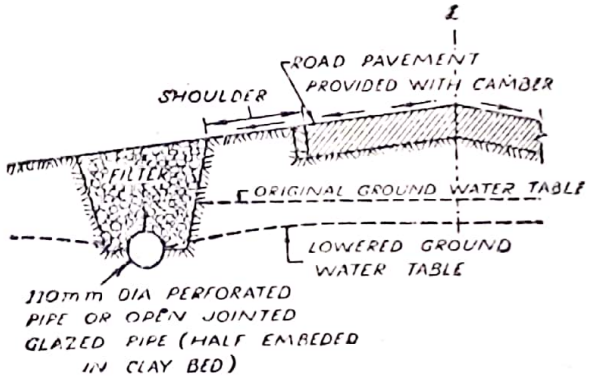
⇒ closed drains are further divided into the following two types:-

(a) Jelly drains;

(b) Drain provided with C.I gratings



(a) Road Section showing a Jelly Drain without perforated Pipe



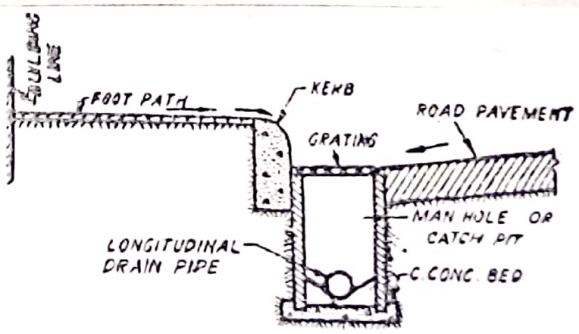
(b) Road Section showing a Jelly Drain with Perforated Pipe

JELLY TYPE SIDE DRAINS

(a) Jelly drains

The closed drains filled with filter material are known as Jelly drains.

These drains are usually provided along a road in rural area when the depth of drains required is more and the road is subjected to heavy traffic.



DRAIN PROVIDED WITH GRATING

Jelly drains are usually trapezoidal in section & may be deeper than open type side drains. The filter material is filled in these drains in such a manner that the particle size increases towards the bottom of the drain. These drains may be having 150mm

thick layer of gravel (35 mm to 60 mm size) at the base or provided with a 110mm perforated or open jointed glazed pipe at the bottom. The pipe is generally half embeded in the clay bed of the drain.

Suitability: These drains are usually preferred when more depth of side drains is required in order to lower down the sub-soil water level in rural areas.

(b) Drains provided with Gratings

These are under ground longitudinal drains which are usually constructed along a road in urban areas. These drains collect the surface water & then convey the same to an underground sewer.

Water, drained from the pavement surface, is carried forward in longitudinal direction between the kerb and the road pavement for short distances. The water then flows down into catch pits through gratings provided over them. The water thus collected in catch pits is lead through an under-ground longitudinal drain pipe into a storm sewer.

Drainage of Hill Roads :-

An adequate and effective drainage is very essential for better service & less maintenance cost of hill roads. The drainage of hill roads consists of the following systems :-

1. surface drainage;
2. Controlling seepage flow;
3. Cross drainage

1. Surface drainage

In the case of hill roads, surface water causes erosion to the road surface & hill sides and may result in land slides or slips. A proper arrangement for drainage of surface water is, therefore, of utmost importance to prevent erosion & land slides. An efficient network of surface drainage system of a hill road consists of the following works,

(i) providing side drains
(ii) providing catch water drains or intercepting drains
(iii) stabilizing the hill slopes by any suitable method such as

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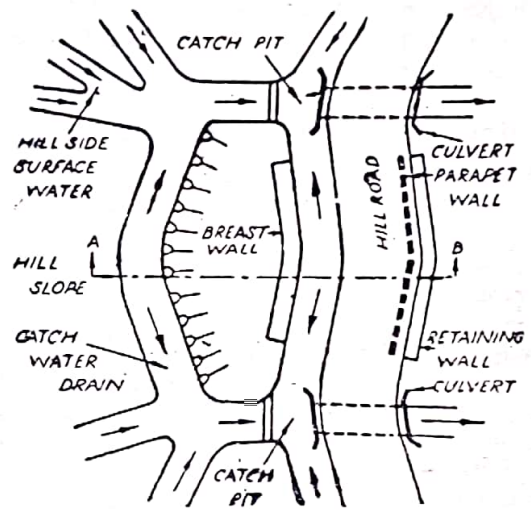
providing adequate side slope, benching, gravel pitching, cement grouting or by constructing breast walls.

The following drainage structures are required for surface drainage of hill roads:—

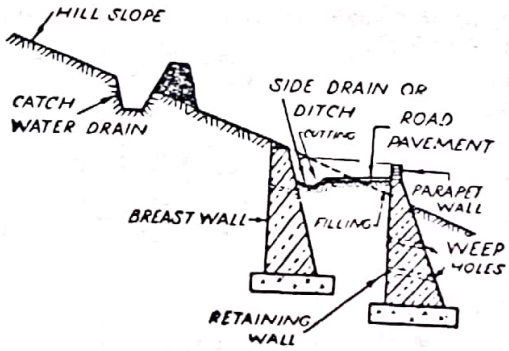
(a) Side drains ;
(b) catch water drains or intercepting drains.

(a) Side drains ;—

These drains are provided to carry rain water from the surface of road and also from sloping side of the hill. These drains may be provided on both sides of the roadway only where the road runs in through cutting. In case of side cutting, such drains are recommended only on one side, usually on the hill side of the road.



(a) Plan showing Location of Catch Water Drain and Side Drain



(b) Section at AB showing Location of Catch Water Drain and Side Drain

The side drains, provided in case of hill roads, should be of such a form that even if wheels of a moving vehicle get into any drain in emergency, the same should be able to come out of that drain easily.

The various forms of side drains being used in case of hill roads are given below

(i) Angle drains
(ii) saucer drains
(iii) Kerb and channel drains

The min^{m} depth of these drains should be 30cm. where there is soft formation, the side drains should be suitably lined or provided

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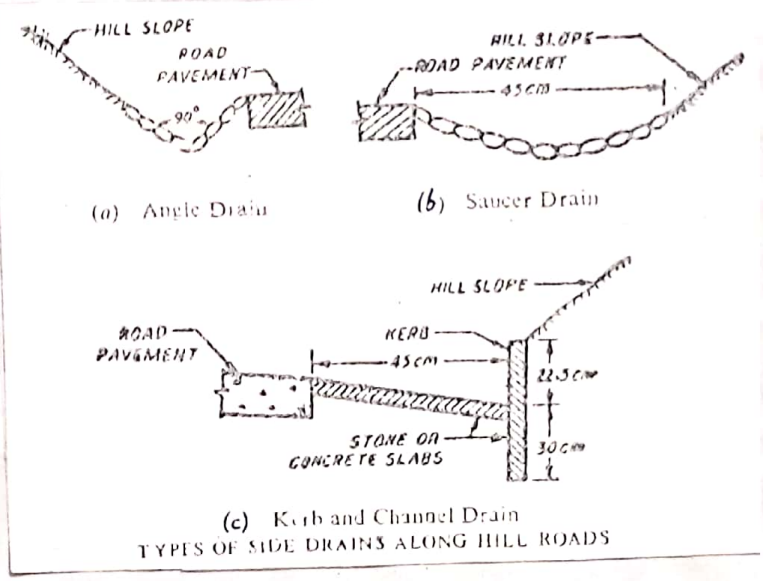
with stone pitching. The water from these side drains is collected in catch pits and diverted across the road through suitable cross drainage works, provided at convenient intervals.



TYPES OF SIDE DRAINS ALONG HILL ROADS

(b) Catch water drains :-

These drains are also known as intercepting drains. One or more catch water drains are provided higher up the hill side, parallel to the road — as shown in fig (Page-16)

The function of providing catch water drains is to intercept the large quantity of surface water which is likely to come on to the road from the hill side. These drains thus prevent heavy rush of water on to the road which may otherwise cause land slides or slips. They also help in reducing the size of side drains.

These drains are generally 0.9m × 0.9m in section and should not be located closer than 4.5m from the road edge.

The flow in catch water drains is very rapid and hence outlets must be provided at suitable intervals. The slope of these drains should be kept min^m to control high velocity with which water usually flows through them.

2. Controlling seepage flow

When the general ground as well as the impervious strata lying underneath are sloping, the seepage flow is likely to exist. If the seepage zone is at a depth less than 0.6m to 0.9m from the surface of pavement, it is desirable to intercept the seepage flow. For controlling seepage flow,

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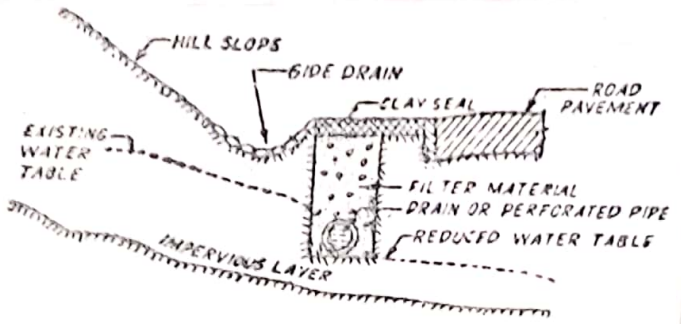
Sub-surface drains are provided at foot of the hill slope as ~~shown~~ per fig shown alongside.



CONTROL OF SEEPAGE FLOW BY A SUB-SOIL DRAIN

When retaining walls or breast walls are to be constructed along a hill road, weep holes should also be provided in these walls to allow drainage of seepage flow.

3. Cross drainage

An efficient cross drainage system is essential for disposing off the surface water collected by catch water drains & side drains across the hill road. It consists in providing cross drainage structures at frequent intervals. An effective cross drainage system prevents side drains from overflowing and flooding the road surface. The provision of cross drainage structures at frequent intervals also helps in reducing the size of side drains as well as of catch water drains.

Cross drainage is provided by constructing the following structures, according to their suitability :-

(a) Small under drains;
(b) Scuppers;
(c) Causeways;
(d) Culverts;
(e) Minor or Major bridges

(a) Small under drains

These drains are in the form of small openings of 0.3m to 0.5m span, provided across hill roads below their bed level. These drains are usually built of random rubble masonry or in the form of small pipes drains. A minimum cross slope of 1 in 6 is provided to the bed of these drains for proper drainage.

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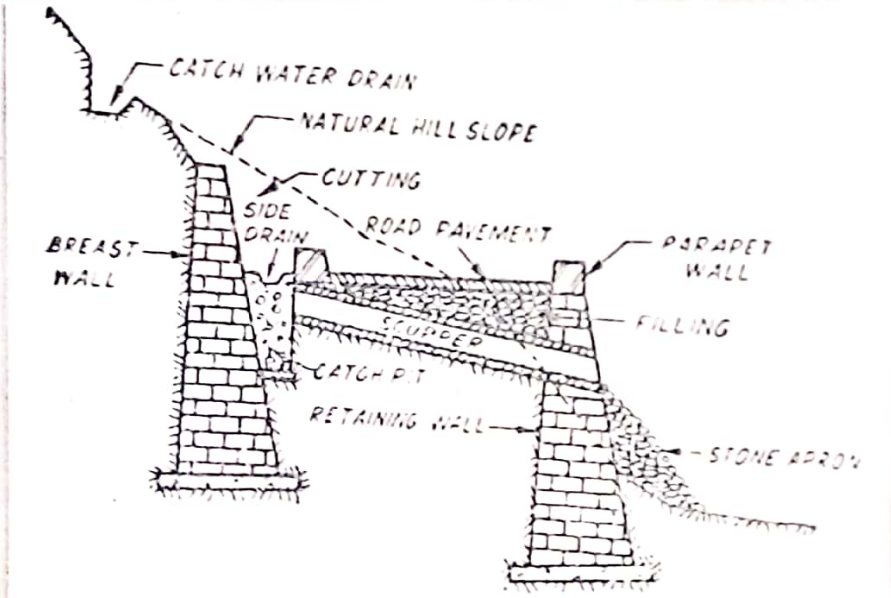
(b) Scuppers

These are small culverts which are usually constructed of random rubble masonry. These are of 0.6m to 0.9m span and min^{m} cross slope of 1 in 6 is provided to the bed of this type of cross drainage structure.



A SCUPPER

A catch pit of 0.3m square in plan with its floor 0.3m below the bed of the under drain or scupper is provided.

⊛ The function of a catch pit is to hold boulders, sand particles, etc. and thus to prevent choking of under drains or scuppers as shown above in fig!-

weep hole :- The holes provided in a retaining structure (retaining wall, breast wall, abutment, etc) to drain off the seepage flow are known as weep holes.
These holes are 8 cm to 12 cm in diameter or square in section and are provided at 1.2m centres in a staggered fashion.

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Road Maintenance

Def^n:- The art of preserving and upkeeping the road pavements, shoulders, sides and other facilities provided on roads in the best possible condition to enable the traffic to move smoothly and safely is known as road maintenance or maintenance of roads.

Object :- The object of road maintenance is to provide safe and convenient movements of passengers and goods at all the times.

General

After any road is constructed and opened to traffic, its pavement, shoulders, sides, etc are subjected to wear and tear. Unless constant and proper upkeep of these road elements is not made, they get deteriorated and may lead to unsafe, uneconomical and inconvenient movement of traffic. Hence, maintenance i.e upkeeping of road elements & other facilities provided on roads in their best cond^n is essential to provide safe & smooth movements of traffic.

The possible defects and cause of failures should be considered before hand for properly designing and constructing the road pavements. Poorly designed and ill-constructed roads pose constant maintenance problems. Even, if the roads are well designed and constructed, periodic repairs are essential to keep them in best serviceable condition. However, the extent of maintenance depends upon the type of road pavement.

Need for Highway Maintenance

Highway maintenance is needed firstly to ensure smooth functioning of the highway so that the flow of traffic is not obstructed and secondly to maintain the various components of the highway so that they remain in satisfactory working conditions. Maintenance is an important activity which helps in providing better service facilities, longer life and better appearance. Highway location, design and construction have a bearing on the maintenance cost. Highway maintenance is the function of preserving, repairing and restoring a roadway

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and keeping it in condition for safe, convenient and economical use. Maintenance includes both physical maintenance activities such as patching, filling joints etc. and traffic service, activities including painting pavement marking. Maintenance works are planned to offset the effect of weather, organic growth, deterioration due to the effects of aging, material failures design & construction faults. The failures may be due to any one or a combination of several causes.

General causes of pavement Failures

Following are the general causes of the failure of road surfaces

(a) Poor quality of material used either in surfacing or base course

(b) Construction defects and lack of quality control

(c) Inadequate surface or sub-surface drainage in the vicinity may result stagnation of water in the subgrade or in any of the pavement layers

(d) Increase in the wheel load beyond the designed capacity.

(e) settlement of base, sub-grade or embankment

(f) Natural calamities such as heavy snow fall, frost, heavy rains and floods, earthquake etc.

Road maintenance Jobs

The following are the road maintenance jobs :-

(a) Maintenance of road pavement

(b) maintenance of shoulders

(c) Maintenance of road sides

(d) maintenance of roadway drainage

(e) maintenance of bridges & other structures

(f) maintenance of traffic control devices such as road signs, traffic marking, traffic signals, traffic islands etc.

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(g) maintenance of miscellaneous items such as railway crossings, boundary stones, kilometre stones, etc.

Pavement Failures

The development of pot holes, ruts, waves & corrugations resulting excessive unevenness in flexible pavements or structural cracks resulting in progressive settlement of some portions of rigid pavements is known as pavement failure.

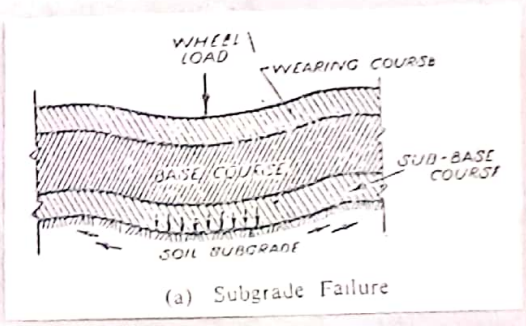
The possible failure and their causes should be considered before hand for properly designing & constructing the road pavements.

Types of failures in Flexible Pavements :-

The following are the different types of failures in flexible pavements :-

(a) Subgrade Failure
(b) Base course failure or Base failure
(c) wearing course failure or surface failure

(a) Subgrade failure :- Excessive deformation in subgrade soil of a flexible pavement is known as subgrade failure.



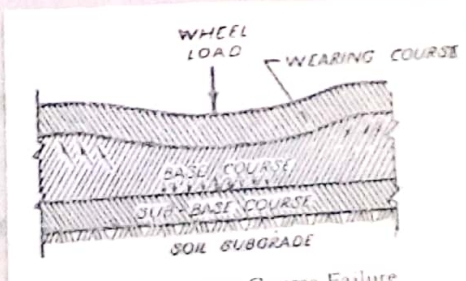
(a) Subgrade Failure

This is considered as one of the main causes of failure of flexible pavements. This type of failure causes excessive undulations (ups & downs) and corrugations in the pavement surface.

The following are the basic causes of subgrade failure
→ Inadequate stability
→ Inadequate road drainage
→ Excessive stress application

(b) Base Course Failure :-

Excessive deformation in the base or foundation course of a flexible pavement is known as base course failure or base failure.



(b) Base Course Failure

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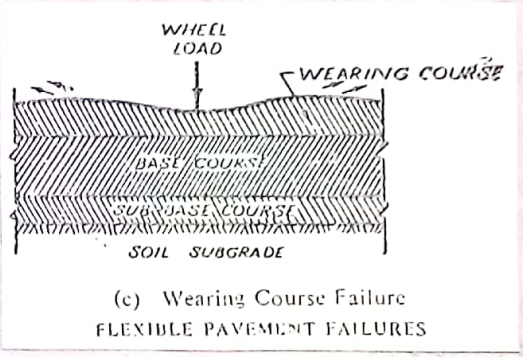
This type of failure causes pot holes, waves and corrugations on the pavement surface.

The following are various causes of base course failure:-

(i) Insufficient strength
(ii) Loss of binding action
(iii) Crushing of base course material
(iv) Lack of lateral confinement of the granular base course.
(v) Insufficient wearing course
(vi) Inadequate quality control
(vii) Inadequate road drainage

(c) Wearing course failure

Excessive deformation or disintegration of the wearing course of a flexible pavement is known as wearing course failure or surface failure.



(c) Wearing Course Failure
FLEXIBLE PAVEMENT FAILURES

This type of failure causes ruts, pot holes, cracks, etc. in the pavement

The following are the causes of wearing course failure:-

(i) Lack of proper mix design
(ii) Use of inferior type of binder
(iii) Inadequate quality control
(iv) Volatization and oxidation of binder

Important Flexible pavement Failures

Some of the important flexible pavement failures are

(a) Ruts
(b) Pot holes
(c) Frost heaving
(d) Shear failure cracking
(e) Longitudinal cracking
(f) Map cracking

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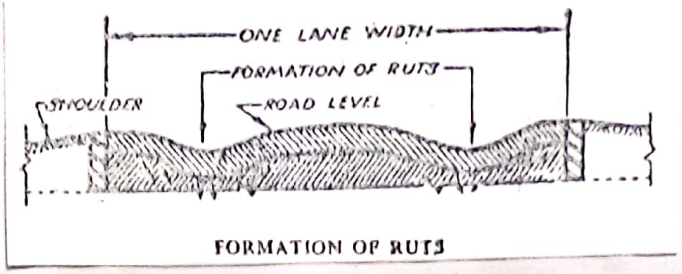
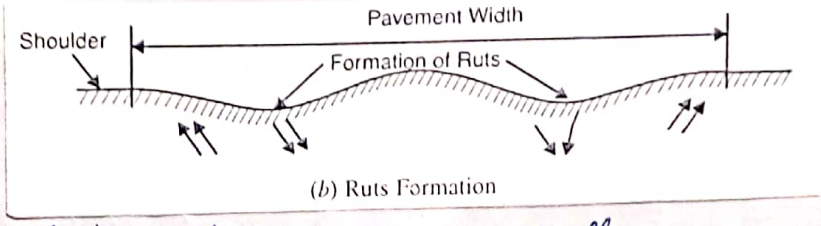
(a) Formation of Ruts

The longitudinal depression or cuts formed in flexible pavements are known as ruts.



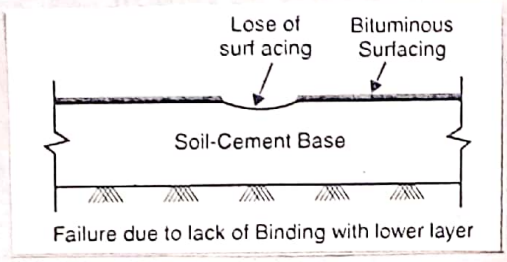
FORMATION OF RUTS

→ These are usually formed in earth or W.B.M road pavements of one lane width due to repeated application of traffic wheel loads on the same location, particularly under iron wheeled traffic.

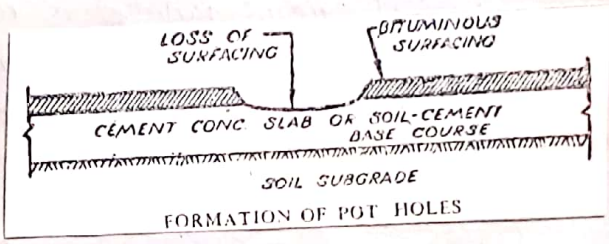


(b) Ruts Formation

(b) Formation of pot holes



Failure due to lack of Binding with lower layer

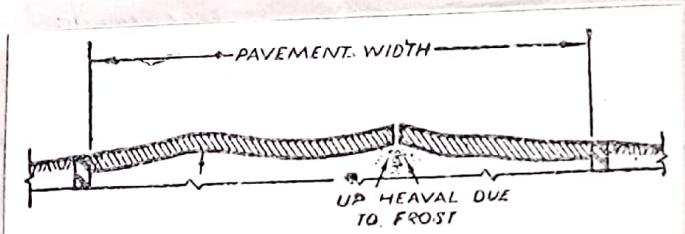


FORMATION OF POT HOLES

The isolated depressions, more or less circular in plan, formed in flexible road pavement are called pot holes or patches. These are usually formed in all types of flexible pavements either due to disintegration of road metal or due to lack of binding of surface course with the underlying base.

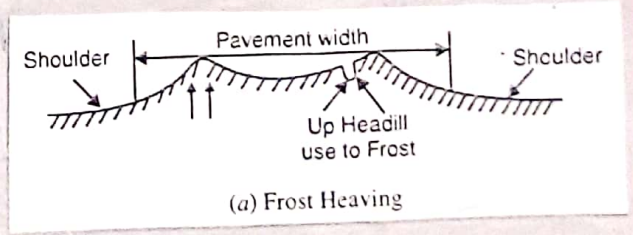
(c) Frost heaving

A localized heaving-up of pavement portion is called frost heaving.



FROST HEAVING

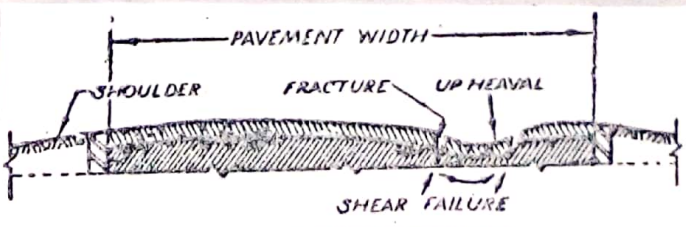
The water which may find access to pavement structure, freezes in cold climates. The expansion accompanying this frost action causes upheaval which in turn may crack the pavement surface.



(a) Frost Heaving

(d) Shear Failure Cracking

The formation of a fracture or cracking due to upheaval of pavement portion followed with



SHEAR FAILURE CRACKING

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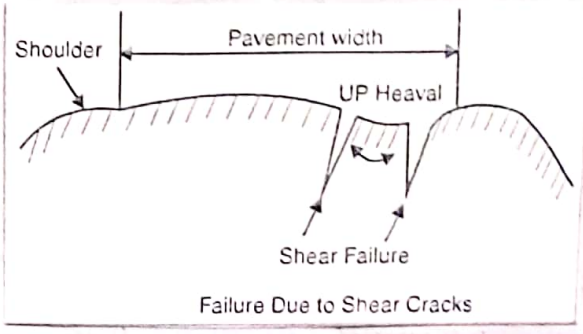
a depression called shear failure cracking.

⊛ This type of failure occurs due to localized weakness in the pavement.



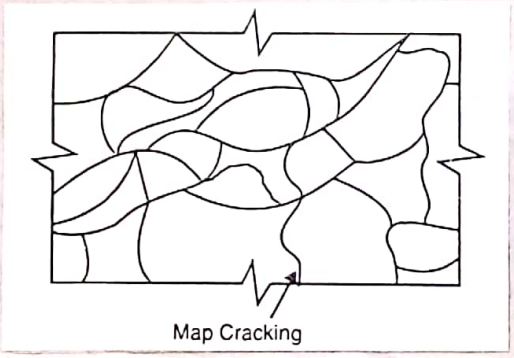
Failure Due to Shear Cracks

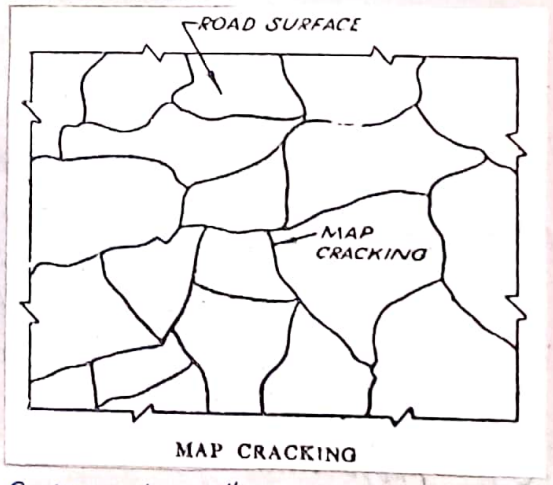
(e) **Longitudinal cracking**

The formation of cracks in the longitudinal direction of a road pavement is called longitudinal cracking.

⊛ This type of road failure is caused due to frost action, different volume changes in subgrade, settlement of filling material or due to sliding of side slopes

(f) **Map cracking**


Map Cracking



MAP CRACKING

The development of irregular cracks, usually formed on bituminous surfacing is called map cracking.

⊛ This type of flexible road failure is due to excessive wear of the road surface or due to localized weakness in the underlying base course.

Types of failures in rigid Pavements (cement concrete Pavement)

Failures in cement concrete pavements are mainly in the form of development of structural cracks. However, the different failures in cement concrete pavements are divided into the following two types:-

(a) Subgrade failure

(b) slab failure

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(a) **Subgrade failure :-**

The development of excessive deformations in the subgrade of a rigid pavement either in the form of general or differential settlement is known as subgrade failure.

The following are the causes of sub grade failure :

(i) Insufficient thickness of the road slab
(ii) Non-uniform compaction
(iii) Inadequate compaction
(iv) Inadequate road drainage
(v) Poor soil forming the subgrade

(b) **Slab failure :-**

The development of structural cracks which may further result in progressive subsidence of some portion of the road slab is known as slab failure.

The following are the basic causes of pavement slab failure :

(i) Faulty design
(ii) Inferior material and bad workmanship
(iii) Due to weather and abnormal traffic

Various Causes of Failures and their Effects in Concrete Pavements

S. No.	Category of Cause	Causes	Common Failure
1.	Faulty Design	(a) Inadequate foundations.	(i) General settlement of the slab. (ii) Differential settlement of slab (iii) Cracking of slab corners and edges (iv) Widening of longitudinal joints.
		(b) Unsuitable foundations with poorly sealed joints.	(i) *Mud pumping* at joints. (ii) Cracking of road slab.
		(c) Incorrect slab thickness.	General cracking and disintegration.
		(d) Incorrect spacing of joints.	(i) Transverse and longitudinal cracking (ii) Compression failures (blow-ups).
2.	Faulty Material and Workmanship	(a) Soft aggregate.	Disintegration of slab.
		(b) Inferior quality of concrete mix.	Servere map cracking of slab.
		(c) Inadequate compaction.	Honey-combing of concrete.
		(d) Poor workmanship in joint construction.	(i) Poor riding surface by coming out filling material and bad joints finishing. (ii) Cracking due to incorrect placing of *tie* or *dowel bars* in joints (iii) Spalling due to incorrect positioning of filling and sealing compounds.
		(e) Poor surface finish.	(i) Slippery surface caused by smooth finishing (ii) Poor riding surface caused by surface irregularity
		(f) Inadequate curing.	Shrinkage cracks.
		(g) Poor maintenance of joints.	Entrance of water causing softening of subgrade, mud pumping and cracking.
3.	Weather and abnormal Traffic	(a) Frost action on weak subgrade.	Heaving up of slab.
		(b) Frost action on weak concrete	(i) Scaling of slab surface. (ii) Disintegration of kerbs.
		(c) Wet weather during construction of slab.	Cracking and settlement of slab.
		(d) Tracked vehicles.	Excessive wear and abrasion of surface.

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Important Failures in rigid pavements

There are various types of failures in rigid pavements Some of the important failures are

(a) scaling of concrete slab
(b) Shrinkage cracks
(c) Warping cracks
(d) Mud pumping

(a) scaling of concrete slab

The defect showing overall deterioration of the concrete slab is known as scaling of concrete slab.

⇒ This defect is due to deficiency in the concrete mix, presence of some chemical impurities or due to excessive vibrations given to the mix during compaction of the road slab.

This makes the slab surface rough & shabby in appearance.

(b) Shrinkage cracks

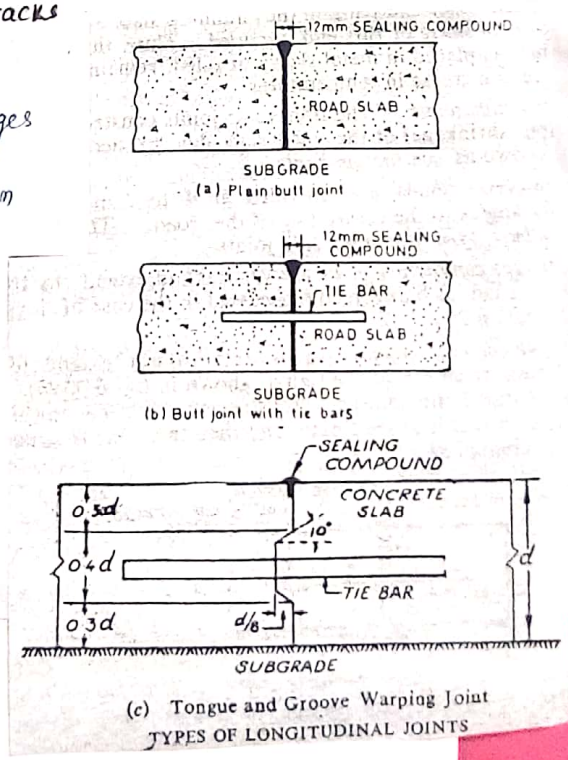
The cracks developed due to inadequate curing after construction of road slab are known as shrinkage cracks.

⇒ These cracks develop in longitudinal as well as in transverse directions.

(c) Warping cracks

The cracks developed at the edges of road slab due to excessive warping stresses are known as warping cracks.

⇒ These cracks do not cause any structural defect if proper reinforcement in the form of tie bars and dowel bars is provided at the longitudinal & transverse joints



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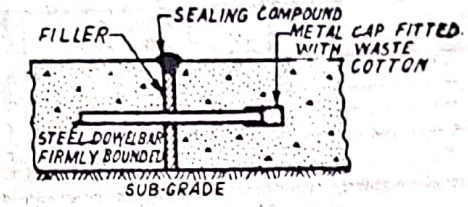
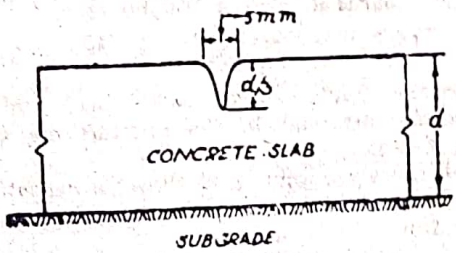
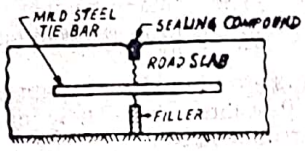


(a) Expansion Joint



(b) Dummy Type Contraction Joint



(c) Warping Joint

TYPES OF TRANSVERSE JOINTS

(d) Mud pumping

The ejecting out of the soil slurry through the joints and cracks of road slab when depressed due to traffic wheel loads is known as mud pumping.

The following are the factors which result in mud pumping in cement concrete pavements: —

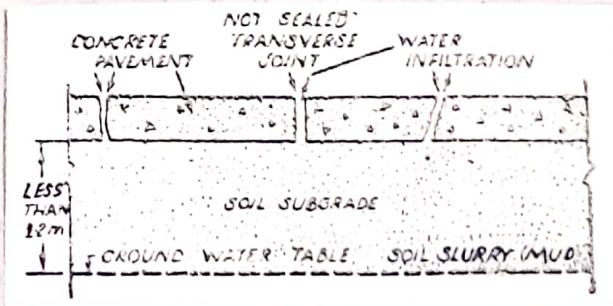
(i) when the subgrade is consisting of cohesive soil like silt or clay.

(ii) when the joints are defective and there is development of structural cracks in the road slab.

(iii) when there is access of water to the soil subgrade of road slab either from top (through not sealed or defective joints and structural cracks) or from bottom due to capillary rise.

In case, the water find its access to the soil sub-grade of road slab, it forms soil slurry or mud and thus weakens the subgrade.

Due to application of repeated wheel loads, the soil under the joints gets consolidated & thus a



(a) Entry of Water through Joints and Cracks

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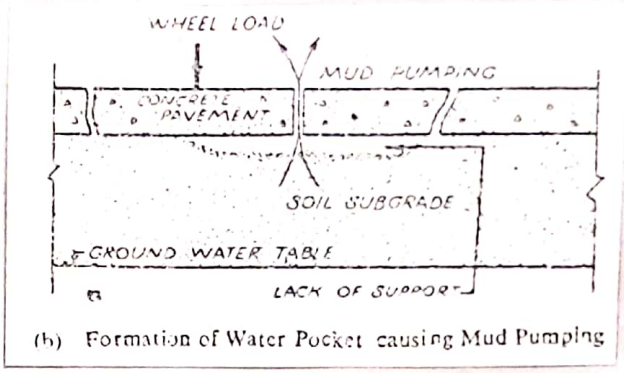
water pocket is formed under the joint.

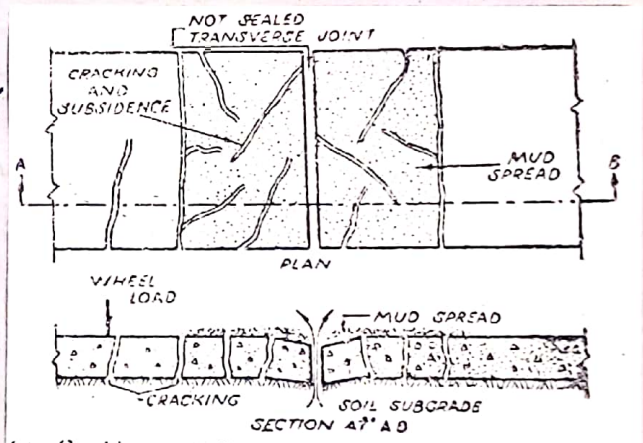
⇒ When the wheel load comes on one side of the joint, the road slab deflects, ejecting the water out of the joint. Since this water also carries with it small particles of soil, it is called soil slurry or mud, and this ejecting out of the soil slurry or mud is known as mud pumping.



(b) Formation of Water Pocket causing Mud Pumping

On due course of time, more & more subgrade soil in the form of mud is ejected out, which goes on depositing over the road surface. In this way, size of the cavity increases resulting considerable loss of subgrade support at such locations.

With continued traffic movements, there is progressive increase in the wheel load stresses in the road slab & thus cracks are developed which ultimately results in failure of the road slab.



(c) Cracking and Failure of Road Slab due to Mud Pumping

MUD PUMPING AND ITS EFFECTS

In order to prevent failure of road slab due to mud pumping, the subgrade should be consisting of good quality material and all the joints should be properly located and sealed.

CLASSIFICATION OF ROAD MAINTENANCE JOBS

The various road maintenance jobs are classified into the following 3 groups :-

1. Routine or periodic repairs;
2. Special repairs;
3. Re-surfacing.

1. Routine or periodic repairs

The day to day or through repairs done to road pavements are known as routine or periodic repairs.

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Periodic repairs are usually carried out for low cost roads. The day to day routine or periodic repairs are generally carried out by the departmental labour, whereas through repairs are generally carried out at certain intervals by contract system.

2. Special repairs

The repairs carried out to overcome special problems which may otherwise lead to the pavement failure are known as special repairs.

Special repairs are usually carried out for high class road. These repairs are very difficult & require special attention for their carrying out.

3. Re-surfacing

The process of renewal or surface dressing of the pavement which is sevely damaged (or after its useful service life) is known as re-surfacing

* This is usually done in case of bituminous roads.

MAINTENANCE OF ROAD PAVEMENTS

The art of upkeeping the road surfaces in the best possible condition for their excellent traffic functioning is known as maintenance of road pavements.

The maintenance techniques of various types of road pavements are stated below :-

1. Maintenance of earth roads.
2. Maintenance of gravel roads
3. Maintenance of water bound macadam roads
4. Maintenance of bituminous roads
5. Maintenance of cement concrete roads

4. Maintenance of bituminous roads

Maintenance jobs of bituminous roads can be divided into the following 3 categories

(a) Day to day repair;
(b) Through maintenance;
(c) Re-surfacing

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(a) Day to day repairs

Day to day repairs of bituminous roads include the following operations :-

→ patching pot holes
→ patching ruts
→ patching corrugations

→ Patching pot holes :-

For patching pot holes (over 35mm) depth, these should be cut out square or rectangular in shape upto the affected depth. The holes are then cleaned of all loose aggregate, dust, foreign matter etc. The internal portion of the holes is then painted with tar or bitumen. After this, usually premixed patching mix is placed in the holes and the surface is rammed or rolled according to the size of the patch. When the pot hole is more than 75mm deep, the patch should be made in two or three layers and each layer is rammed before placing the next layer. The finished level of the patches is kept slightly above the original level to allow for further compaction under traffic.

When the pot holes are only 12 to 25mm deep, their patching is done simply by cleaning, painting and gritting with dry chipping of 3 to 6 mm size. Their surfacing is then finished according to the adjacent road surface.

→ Patching ruts :-

For patching ruts or ravelled areas, continuous trenches are excavated to enclose the ruts or ravelled areas to establish a solid foundation for the patch. These ruts are then patched as per patching pot holes.

→ Patching corrugations :-

For patching corrugations, the high ridges, existing in the road surface, are cut off and the depressions are cleaned, painted and filled up with premix material. This is done to remove inequalities and waviness of the bituminous surface.

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(b) Through maintenance

Through maintenance of a bituminous road is done annually to prevent formation of pot holes, ruts, etc. This includes the following operations :-

(i) Base repairs
(ii) Surface treatment

→ Base repairs :-

Before any repair is carried out to the base of a bituminous road; its cause of failure should be determined & the necessary correction should be decided. when it is established that the failure is due to inadequate thickness of the base course, the same may be corrected by providing additional surface thickness. Thus, base repairs usually consist of increasing thickness of the base as decided by the Engineer-in-charge.

For base repairs, old surface of the base is loosened by scarifying into the full depth. The sub-grade may also be removed to the depth necessary to provide the decided additional thickness of the new base.

The old road metal is screened and may be utilized in the bottom of the base. The new base is then prepared in layers, not more than 75 mm in thickness. The base, thus prepared, should be covered with a suitable surface finish.

→ Surface treatment :-

Surface treatment of a bituminous road consists of correcting bleeding and applying a renewal coat or seal coat.

Bleeding of bituminous surface is corrected as soon as it occurs. For correcting bleeding, a layer of blotting material such as aggregate chipping (max^m size 15 mm) or coarse sand is applied and the rolling is done if necessary.

A renewal coat or seal coat is applied when structural cracks are developed in the road surface.

(c) Re-surfacing

Resurfacing of a bituminous road consists in applying a new layer of bituminous material over the existing

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wearing surface. This is done when the existing wearing course is totally worn out & provides a poor riding surface.

For re-surfacing, the existing surface is repaired by suitable patching. Then a light tack coat is applied over the surface.

5. Maintenance of cement concrete roads

Maintenance work of cement concrete roads includes the following operations :-

(a) Repair of joints;
(b) Repair of cracks;
(c) Patch repair;
(d) Repair of blow-ups;
(e) Repair of settlement due to mud pumping or sub-grade failure;
(f) Repair of scaling & other such surface defects

(a) Repair of joints

Repair of joints in cement concrete roads consists of refilling or resealing the squeezed out joints periodically. This is done to prevent entry of surface water through joints into the road subgrade and to maintain a good riding surface.

For repair of joints, it is necessary to do routine inspection of all the joints atleast twice a year & to refill or reseal the joints if the sealing compound has lost its adhesion or it has become too soft in any joint for refilling or resealing the joint, the old sealing compound is removed by means of a sharp pointed tool & the joint is properly cleaned by a stiff wire brush and then by a camel brush. The joint is then resealed with a suitable sealing compound. The joint should be dry at the time of resealing. After resealing, the top surface of the joint should be dusted with lime dust filler or sand to prevent the

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sealing compound sticking to the wheels of moving vehicles.

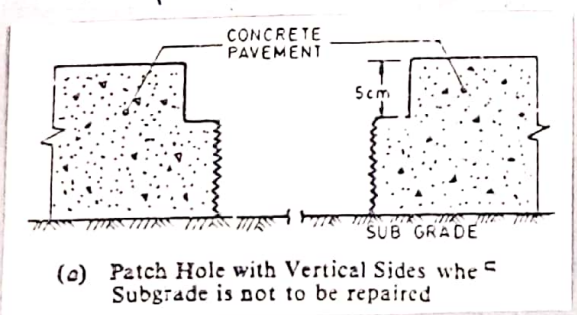
(b) Repair of cracks

Repair work of cracks in a cement concrete road consists of treating the cracks which are wide enough to admit water and grits into the road subgrade. Such cracks should be treated as soon as they occur. Fine cracks, usually known as hair cracks or shrinkage cracks, do not require any repair as they do not admit surface water to penetrate into the subgrade.

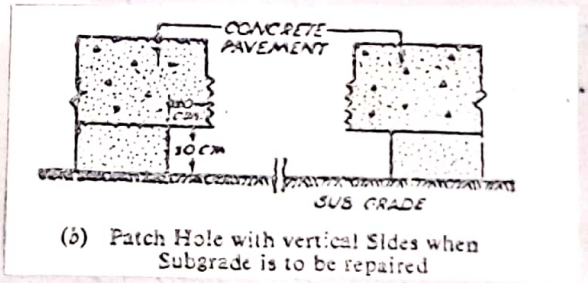
For repairing any cracks, the same is thoroughly cleaned by means of suitable sharp pointed tool and then dirt is blown off by a blower. The surface of the crack is then coated with kerosene oil to facilitate the adhesion of the sealing material with the old concrete. The crack is then filled with a molten sealing compound. The repair work of cracks should be done during summer season before monsoon.

(c) Patch repair

Patch work of cement concrete roads consists of patching any hole, depression or sharp break as soon as it occurs in the slab. Such irregularities or local failures, when less in number, can be successfully & economically patched with bituminous materials. However, when such irregularities have extended to full depth of road slabs over considerable area, concrete patching is done, since bituminous patching is considered inadequate under such circumstances.



(a) Patch Hole with Vertical Sides when Subgrade is not to be repaired



(b) Patch Hole with vertical Sides when Subgrade is to be repaired

For concrete patching, the patch hole should be cut out square or rectangular in shape and to full depth of the slab with edges parallel or normal to the centre line of the road.

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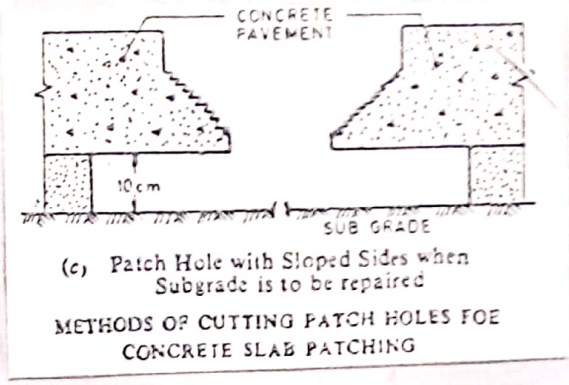


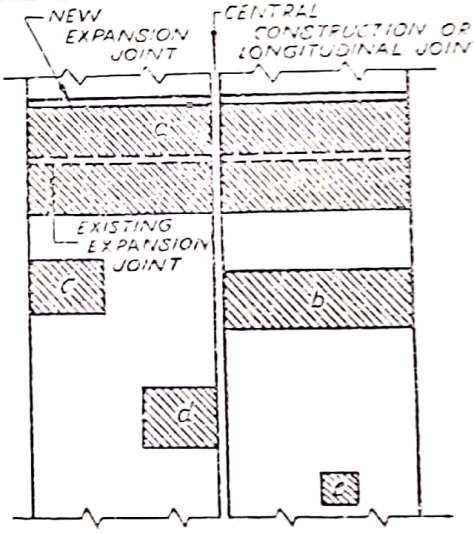
(c) Patch Hole with Sloped Sides when Subgrade is to be repaired

METHODS OF CUTTING PATCH HOLES FOR CONCRETE SLAB PATCHING

The different methods of forming holes for patch repair are shown in fig alongside (previous page & in this page). The subgrade should be carefully examined for its stability; if the subgrade also requires improvements, the same should be excavated atleast upto 10 cm depth & equally more in dimensions than the size of the patch hole.

After patch holes have been cut, their edges are thoroughly cleaned and wetted with water. A thick coat of cement grout is then applied on the sides of each patch holes. After this, the cement concrete is placed into the patch hole & compacted.

The proportions of the concrete mix should be same as in the original slab except that quick setting cement may be used in the concrete mix prepared for concrete patching. The surface of patches is then allowed to cure. During concrete patching continuity of joints should be maintained.



(a) Full Width Patch on Both Sides of the Existing Expansion Joint
(b) Half Width Patch (c) Exterior Edge Patch
(d) Interior Edge Patch (e) Plug Patch

LOCATIONS AND TYPES OF CONCRETE SLAB PATCHES

(d) Repair of blow ups

Repair work of blow ups in concrete roads consists of breaking the blow ups and correcting the surface of road slab. The blow ups should be corrected immediately after their occurance to provide smooth riding surface.

For repairing blow-ups, 15 cm depth of pavement is removed & replaced with a patch of concrete or bituminous material.

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Sometimes, repair of blow-ups is done temporarily with road side soil to allow the pavement to settle down to its original position under traffic & then permanent patch work is done.

(e) Repair to settlement due to mud pumping or sub-grade failure :-

This type of repair work of a concrete road consists of mudjacking or bituminous undersealing the depressions caused due to mud-pumping or sub-grade failure and raising the road slab to the desired elevation.

Mudjacking consists of pumping slurry, prepared by mixing soil, cement and water, through drilled holes in the pavement with a mudjacking machine. The slurry should have consistency of thick cream so as to fill majority of the voids under the pavement.

Bituminous undersealing consists of pumping highly viscous asphalt, heated to a temperature of about 200°c, through a drilled hole in the concrete pavement. This will also control mud-pumping in future. It is a good technique to underseal the road slab with asphalt during earlier stages of mud-pumping and thus preventing damage due to this cause to a large extent.

(f) Repair of other defects

Apart from repair work of defects mentioned above, the other defects in concrete roads needing repair are "scaling, spalling at joints, disintegration & wear of pavement.

Repair work of scaling consists of cleaning the scaled area by means of wire brushes. The cleaned surface is then given one or more applications of bituminous material. The surface is then immediately covered with coarse sand, grit or any other suitable material. In case of excessive scaling (more than 2.5 cm), it is repaired by providing a layer of premixed asphalt concrete on the primed surface of the scaled area. This type of repair produces bad appearance & hence not commonly recommended.

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. Repair work of spalling of joints consists of cutting the defective edge to a depth not less than 50mm & to a width 80 mm to 150 mm normal to joint. The surface of cutting is then cleaned with wire brushes and wetted with water. Then a thick coat (3 to 6 mm) of neat cement grout is applied over the surface and a rich mix of concrete (1:1.5:3) with low water cement ratio (16 to 18 litres per bag of cement) is laid & compacted.

Repair work of disintegration of concrete slab consists of cleaning the affected area of all dust and foreign matter. The surface is then sealed with one or more application of surface treatment.

Repair work of wear consists of treating the isolated spots such as steep areas or sharp curves which are subjected to excessive wear. This is done by bituminous surface treatment.

Maintenance of Shoulders

The art of upkeeping the shoulders of road pavements in their best serviceable condition is known as maintenance of shoulders.

The importance of maintenance of shoulders lies in the fact that they act as a part of the roadway & provide lateral strength to the road pavement. The necessity of maintenance of shoulders can be well realised by considering their various functions. shoulders are properly maintained to keep them stable, smooth, with proper cross slope and in level with the edges of road pavement.

Shoulders are mostly constructed of earth in our country. Such shoulders present great difficulty in their maintenance since they become dusty during dry weather, slippery during wet weather and are subjected to erosion during heavy rains.

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The following are the common defects in shoulders

→ Depression along the edges of pavement;

→ Deep ruts;

→ Abrupt holes or pot holes, etc

Maintenance jobs of shoulders

Maintenance jobs of shoulders can be divided into the following two categories :-

(a) Daily maintenance;

(b) Periodic maintenance

→ **Daily maintenance :-**

The daily maintenance of shoulders is done to remove the common defects as soon as they are noticed. It includes the following operations..

(i) Repair of depressions along the edge of pavement

(ii) Repair of ruts and pot holes

(iii) Repair of cross slope

→ **Periodic maintenance :-**

The periodic maintenance of shoulders is done before the start of the monsoon and after the periodic maintenance of pavements because the shoulders are used as traffic lanes during such times and get badly damaged.

periodic maintenance of shoulders includes the following operations :-

(i) Levelling the shoulders

(ii) Providing the required cross slope

(iii) Covering the pavement edges

(iv) Treating the shoulders, i.e turfing, stabilizing, etc according to availability of funds.

The periodic repairs of shoulders should be done in August and September so that the new earth is not washed away by rains.

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Maintenance of Traffic control Devices

Traffic control devices are signs, signals and marking provided for direction, warning and regulation of traffic stream for the safety of traffic. Therefore, such devices must be maintained. The maintenance work includes installation, repair, painting, or signs and care of markings. The marking should be renewed every year and for this purpose mechanised marking equipments are available in the market. Thermoplastic compounds are also used for markings. Traffic signal should be periodically checked and if found defective, then, they should be repaired & readjusted to the traffic cond^{n}. Warning signs should also be given due attention. These should be checked and if found defective, then these should be required on a high priority basis.

Blow ups :- The raised portions of the road slab at transverse joints of cracks, developed due to longitudinal expansions of the slab are known as blow ups.

Scaling :- The chipping of thin pieces from the surface of pavement due to excessive application of salt during curing & ice removal etc is called scaling.

Spalling :- The chipping of pavement at joints or cracks due to entry of grits or defective construction is known as spalling.

Disintegration of Pavement :- The breaking up of concrete pavement into small pieces due to unsound aggregate, freezing etc is called disintegration of concrete pavements.

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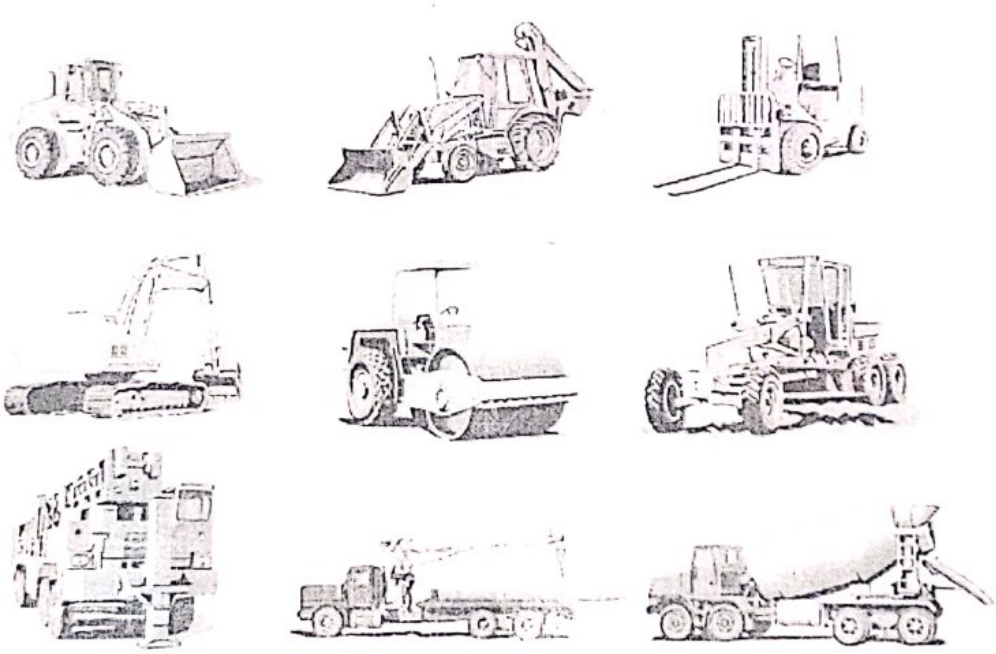
ROAD CONSTRUCTION EQUIPMENTS



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INTRODUCTION

- Modern age is the age of science and technology . All the crude and slow methods of construction of roads have been replaced by refined & quick methods by the use of modern plants and machinery. How ever in India, modern machinery is so far used for big road projects because the financial condition of this country do not permit the use of such a machinery for ordinary road projects. But highway engineer must be familiar with the use of modern road construction equipment such as plant and machinery.
- Here we will discuss the output and use of common types of plants and machinery so as to have basic knowledge about the use of all construction equipments used in road construction.

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TRACTOR

- Tractor is a self propelled machine which work with diesel engine having HP varying from 20-200 .
- Tractors are generally provided with various attachment such as dozer,scraper,..etc

Types of Tractors

(i) Crawler Tractor

(ii) Wheel Tractor

(a)Two wheel

(b)Four wheel

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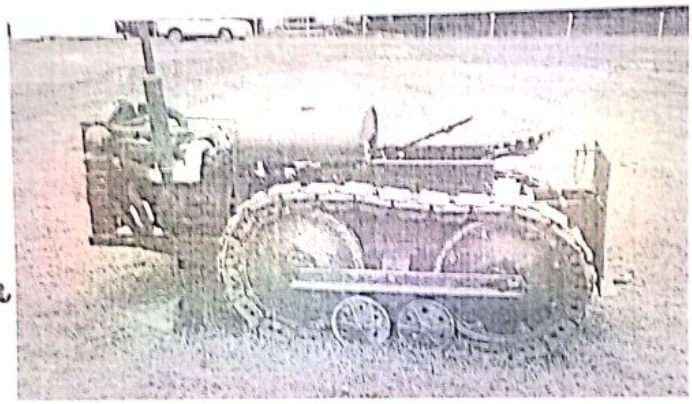
CRAWLER TRACTORS





These tractors move on an endless chain of plates. A crawler tractor is consider as the most basic and versatile machine among the road construction equipment.

USE

Crawler tractor are used when the area is uneven and rough.

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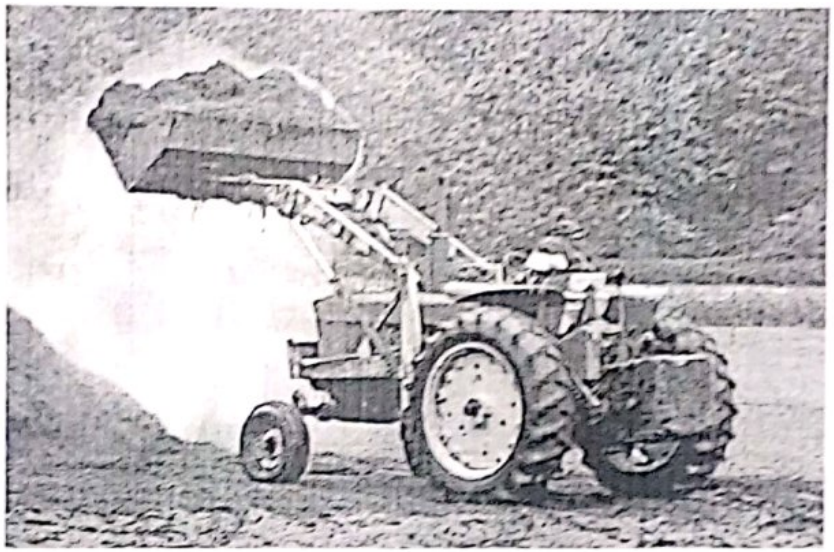

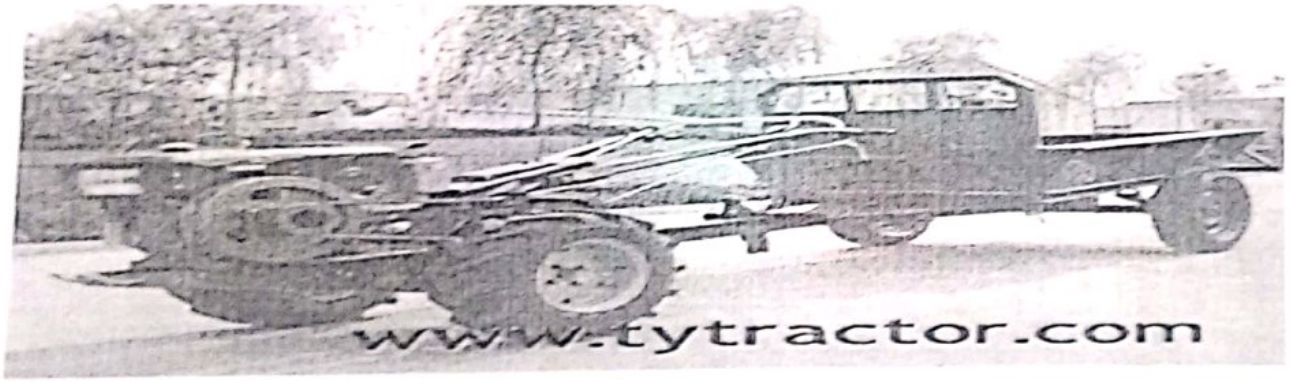

These tractor move on pneumatic tyres . A wheel tractor moves faster than crawler tractor. Some these tractors have maximum speed more than 50 kmph.

USE

Wheel tractors are used when the area is even and smooth.

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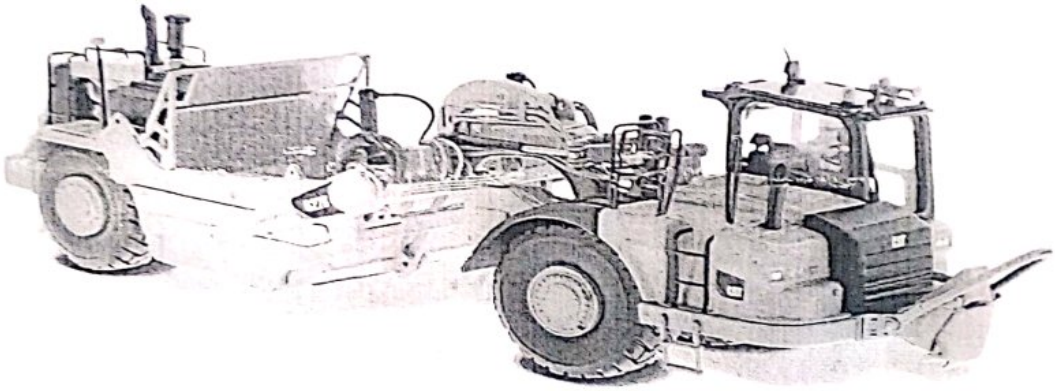
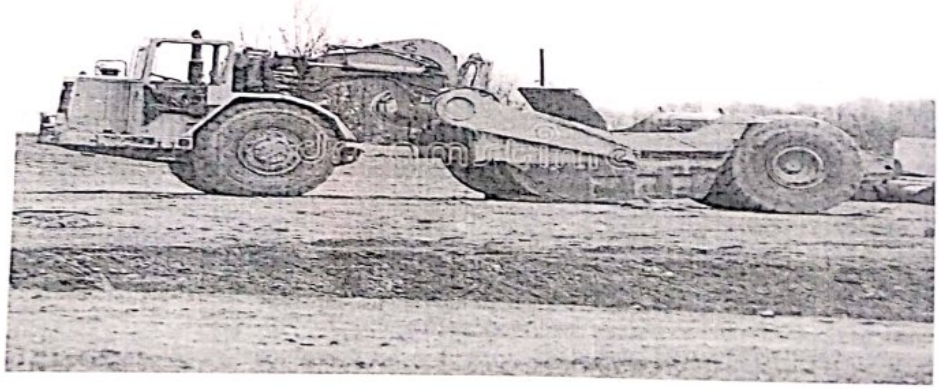
SCRAPER





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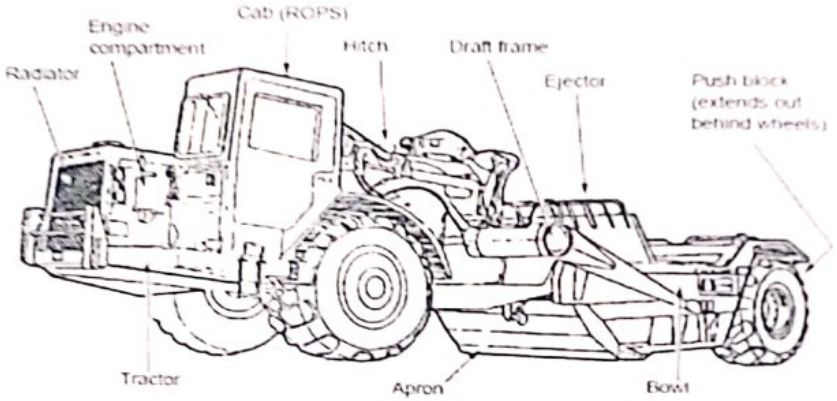
SCRAPER



A Scraper is a combination of the best loading and best hauling machines. It essentially consist of a large scoop provided with a cutting edge. The scoop is in the form of a bucket or a shallow container which is known as bowl or body. The scoop can excavate , transport and dump the material where required.

It is mounted on a four or two pneumatic wheel and is pulled by a tractor . Thus a scraper is consider as a sufficient machine, as it can do all the operation necessary for light soil ,viz it can excavate ,load, transport and dump the soil at the required site.

USE

(i)To excavate soft soils

(ii) To load the excavate soils

(iii) To Transport the excavate soils

(iv) To dump the excavate soil at required site

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DOZERS

These are attachment of tractor which are used for various works. Dozers are very versatile machines for various operation in road construction viz to clear the site ,moving earth ,levelling earth fills and to clear the floor of borrow pits, ets

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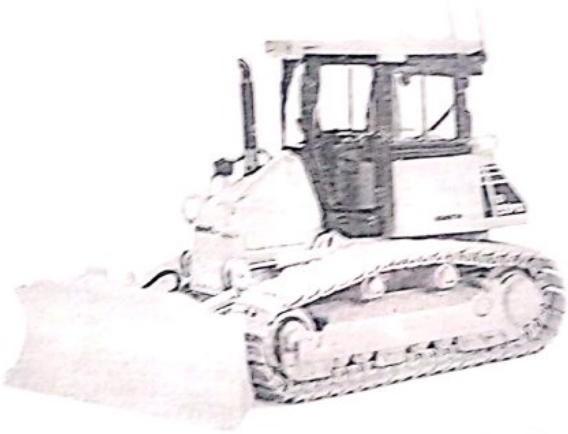
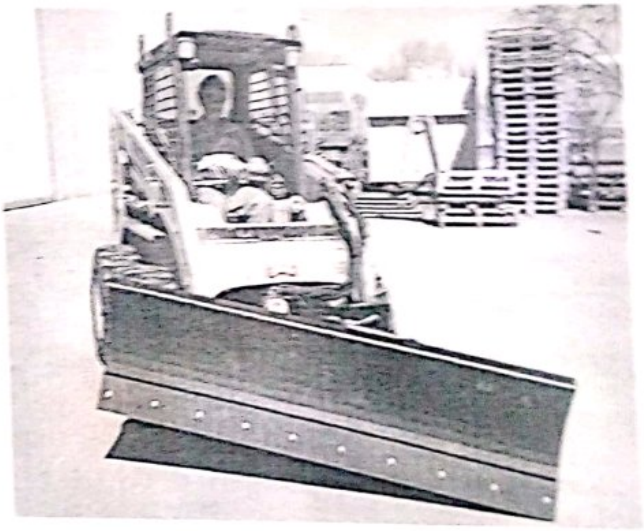
TYPE OF DOZER





1. Bull Dozer;
2. Angle Dozer;
3. Tree Doze



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BULL DOZER

This type of dozer consist of straight and wide blade of steel attached to the front side of a tractor, which is kept at right angle to the direction of movement of the machines.

The blade can be raised or lowered as desired.

USE

(i)Bull Dozer is commonly used for excavating the material and pushing the same in the forward direction.

(ii)It is mainly used for pushing and leveling a heap of exacted material.

ANGLE DOZER

This type of dozer consist of suitable blade attachment to the front side of a tractor, which can be set obliquely to the direction of movement of the machine. The maximum possible angle of the blade to the front face of the tractor is 30°.

USE

Angle dozer is mainly used for pushing the material side ways , to the right or to the left .

TREE DOZER

This is also sometimes called a *stumper* .This type of dozer consists of a slightly curved blade attached to a tractor with its concavity in the forward direction of motion of the machine. The blade is specially design for felling tree and for uprooting the stump.

USE

Tree dozer is mainly used for feeling trees and for uprooting stumps ,shrubs ,etc.

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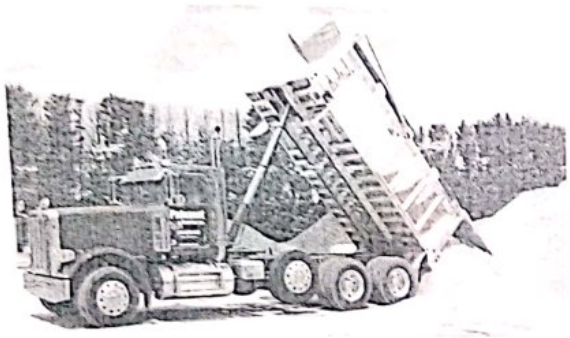
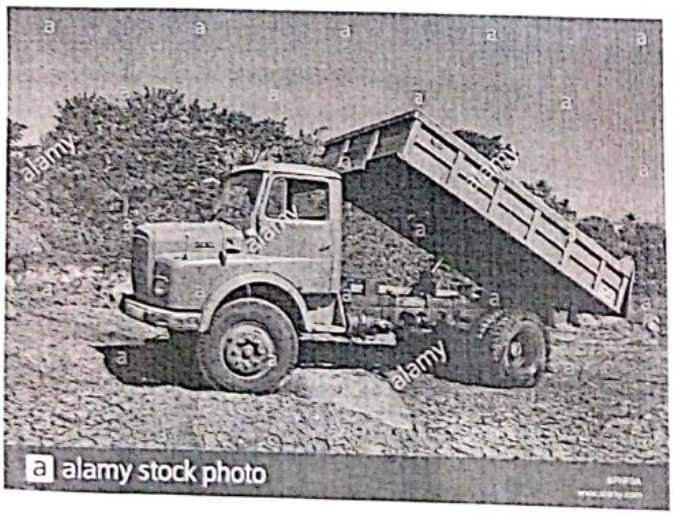
DUMPERS









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A dump truck is also called a dumper. It is a self propelled machine which works with a diesel engine. It consists of a trolley or container fitted on a truck, which can be quickly tilted in one or the other direction.

In dump truck one or two hydraulic operated pistons are provided to raise or lower the container carrying the material.

USE

Dump truck are used for loading, conveying and dumping the material at the required site quickly and conveniently. These truck may also be used for transporting bituminous concrete and air-entrained cement concrete to the construction site.

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DIAGRAMS OF POWER SHOVEL(2D & 3D)

Shovel

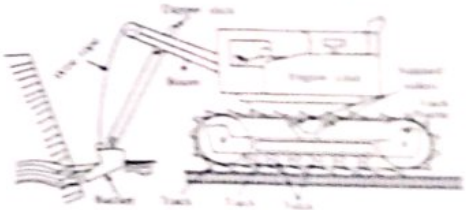
- Shovel is the most powerful and giant excavating machinery includes power shovels. It is one of the most versatile equipment used in the construction of highways especially for earth work.





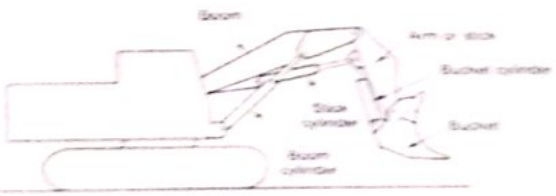
Working

Power Shovel





SHOVELS



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POWER SHOVELS

A power shovel essentially consist of mounting,cabin,boom,dipper stick,dipper or lid and hoist or cable line . The dipper or lid,also known as digging bucket,of a power shovel is capable of being forced upwards through the earth and away from the machine.

- Power shovels are available in different sizes ranging from0.375 cum to 5 cum. They can be mounted on crawler tractor or on wheel tractors as required.

USES

1. To excavate soils of all types except solid soils.
2. To collect and dump the material at required place within the reach of dipper stick.
3. To load the excavated soil into dump truck, conveyor belt or other hauling equipment.

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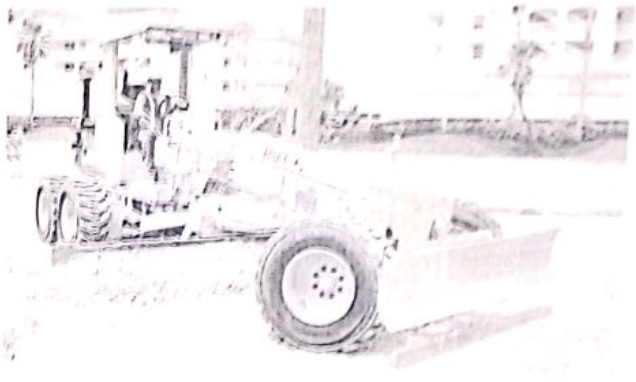
GRADER/MOTOR GRADER

A grader is an earth-moving machine which is either self propelled or towed to a tractor. It mainly consist of an angled blade 3 to 4 metres long supported on a frame work which is mounted on wheels. In case of a motor grader which is self propelled, the blade is supported.





USES

1. To give the proper shape to the road subgrade.
2. To construct earth roads quickly.
3. To spread the loose material evenly.
4. This is also useful for general maintenance of roads during land slides and snow clearance.

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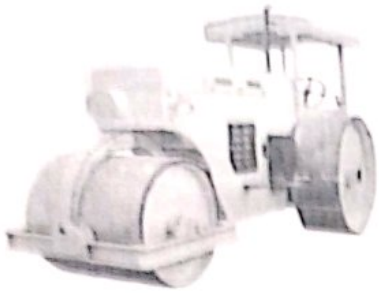
ROLLER



SMOOTH WHEEL ROLLER



PNEUMATIC TYRED ROLLER



SHEEP FOOT ROLLER

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ROLLERS

A roller is considered as an essential road making machinery. It usually consist of a number of wheels or rolls for rolling and compacting the road construction material. It generally self propelled but some of the rollers are towed to tractors also.

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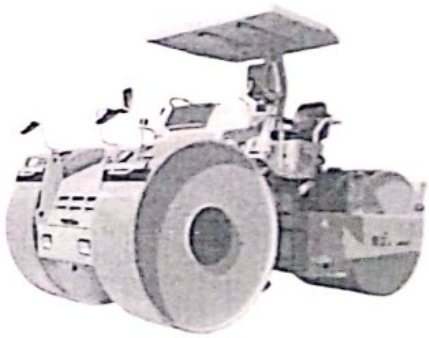
TYPES OF ROLLERS

1. Smooth wheeled roller
2. Pneumatic tyred roller
3. Sheep foot roller

Smooth Wheeled Roller

These rollers consist of one to three smooth wheels and may be hand or animal driven or power driven. A hand or animal driven smooth wheeled roller usually consist of one roll of stone or iron,about 0.9 m dia and 1.2m to 1.5m long and of two tonnes weight. The consolidation with such a roller is not good and moreover, it takes more time than a power driven roller to complete the compaction of road construction material.

The weight of tandem(two wheeled) roller is 6 to 8 tonnes and that of three wheeled types 8 to 10 tonnes.



SUITABILITY

These rollers are preferred when the soil is granular and are useful in compacting soils and other materials where crushing action is advantageous.

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PNEUMATIC TYRED ROLLER

These roller consist of a number of pneumatic wheels mounted on two or more axles under a loading platform. This type of roller is pulled by a tractors.

SUITABILITY

These roller are considered to be most suitable for compacting non-plastic silts and fine sands



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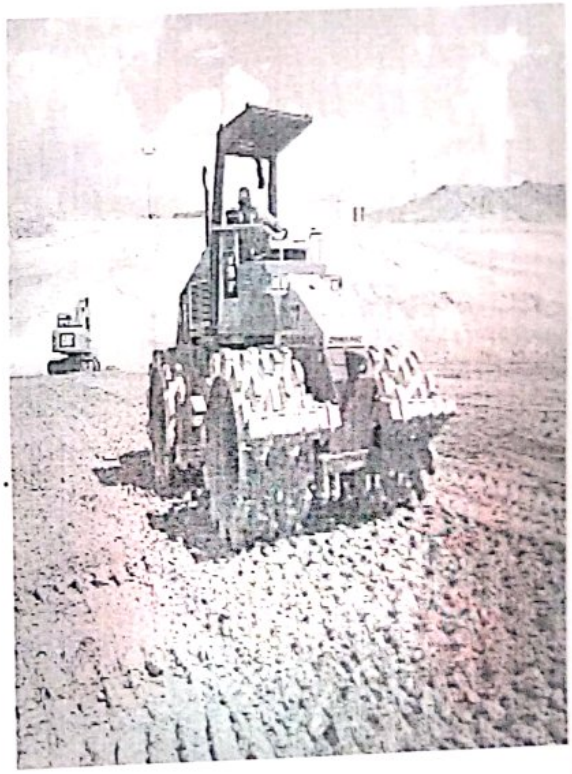
SHEEP FOOT ROLLER

- This type of roller consist of a hollow steel cylinder with raised projections each resembling sheep's foot. Its weight can be increased by filling the drum with wet soils. This type of roller is pulled by tractor.
- Since, these rollers do not provide a proper finish to the surface, their use is recommended to compact the lower layers except the top layer of earthwork for which smooth wheeled roller is used.



SUITABILITY

Sheep foot roller is considered to be most suitable for compacting clayey soils. The thickness of each compacting layer is kept 5 cm more than the legth of projections on the roller.

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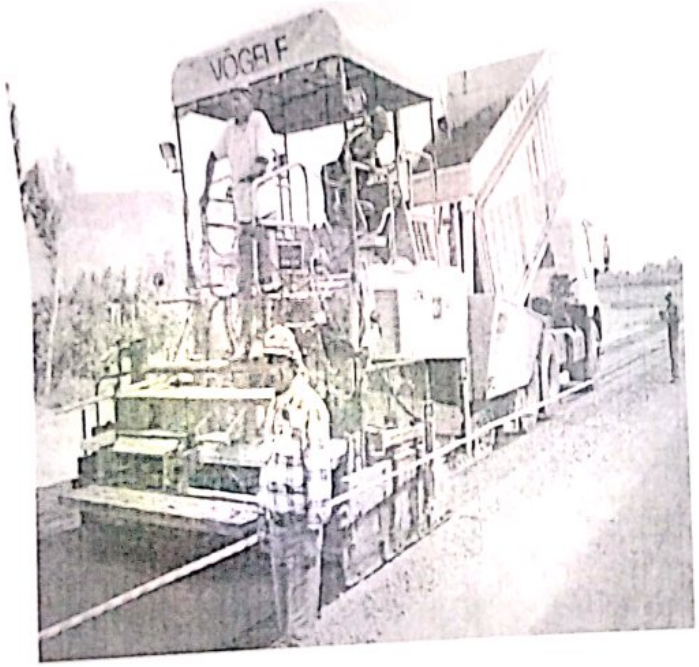
ROAD PAVERS





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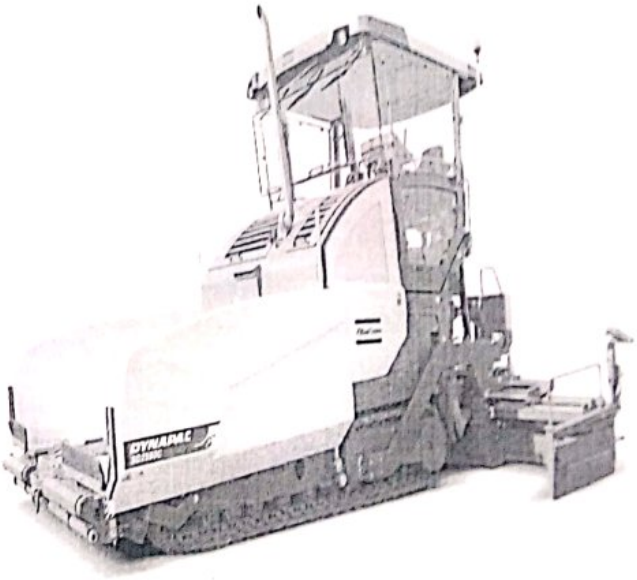
ROAD PAVERS





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ROAD PAVER/PAVER FINISHER

A machine which can lay the bituminous mix to a compact and uniform layer of desired thickness and profile is known as paver finisher/road paver.

A paver finisher can lay bituminous mix in a layer of 1.25 cm to 15 cm thickness upto 4.25m width at a time.

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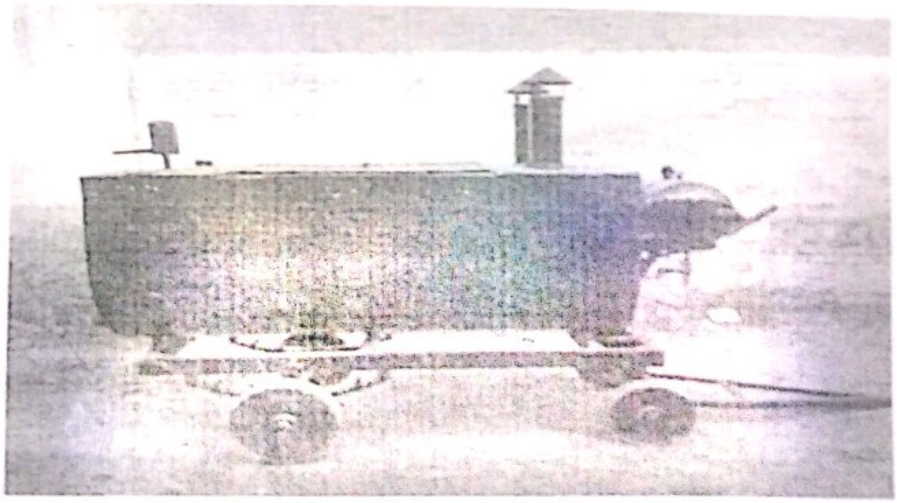
TAR BOILERS

- The tank mounted on wheels in which bitumen is heated by burning coal underneath and the same is sprayed by means of spraying device is called tar boiler with spraying device.

The spraying device is worked by a hand pump. Tar boiler of 300 litre capacity are commonly used.



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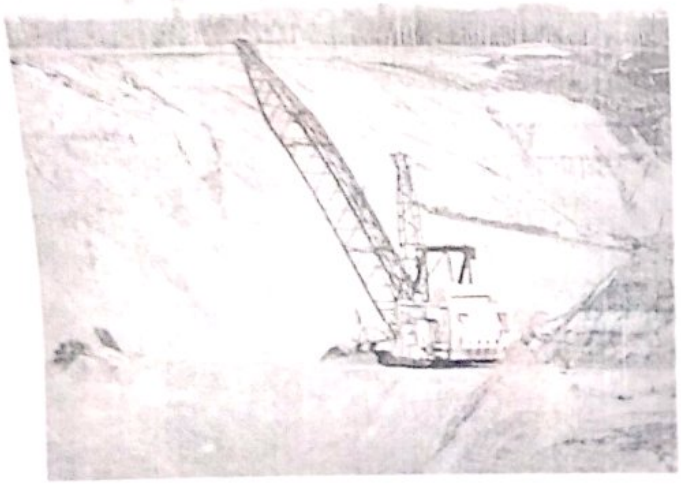
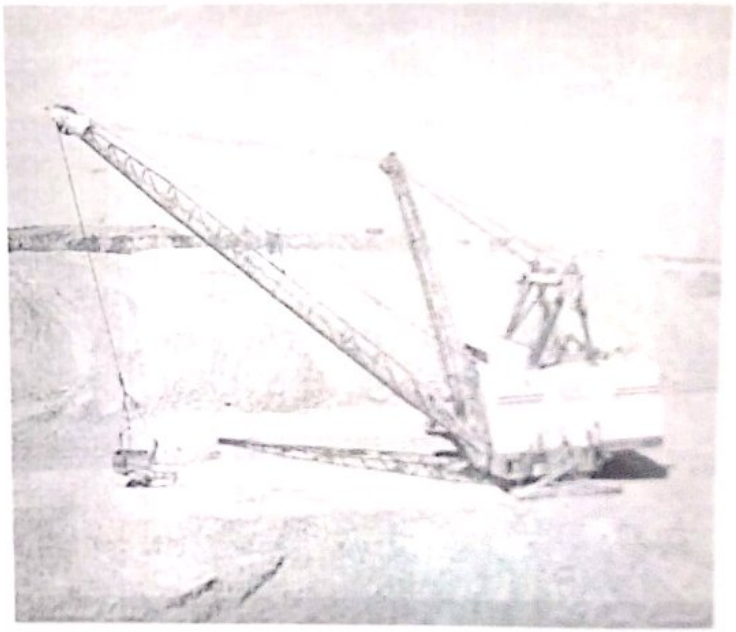
DRAGLINE





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